Load and Resistance Factor Design Specification for Steel Hollow Structural Sections

November 10, 2000

Supersedes *Specification for the Design of Steel Hollow Structural Sections* dated
April 15, 1997 and all previous versions.

Prepared by the
American Institute of Steel Construction, Inc.
under the direction of the
AISC Committee on Specifications and approved by
the AISC Board of Directors.

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PREFACE

The AISC *Load and Resistance Factor Design (LRFD) Specification for Structural Steel Buildings* is intended to cover the common design criteria in routine office practice. Accordingly, it is not feasible to also cover the many special and unique problems encountered within the full range of structural design practice. This AISC *Load and Resistance Factor Design Specification for Steel Hollow Structural Sections* is a separate document that addresses one such topic: the design and construction of building systems that utilize steel hollow structural sections (HSS). A list of Symbols and a non-mandatory Commentary with background information are provided.

The AISC Committee on Specifications, Task Committee 13—Hollow Structural Sections is responsible for its ongoing development. Additionally, the AISC Committee on Specifications has enhanced these provisions through careful scrutiny, discussion, suggestion for improvements, and endorsement.

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CROSS REFERENCE TO THE AISC LRFD SPECIFICATION

This table provides a cross reference from Sections in this Specification to the relevant Sections and related Appendices in the *LRFD Specification for Structural Steel Buildings* dated December 27, 1999.

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SYMBOLS

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Symbol	Definition	Section
A	Area used to calculate A_e, in.^{2} (mm^2)	2.1
A_g	Gross area of cross-section, in.^{2} (mm^2)	2.1
	Chord gross area, in.^{2} (mm^2)	9.4
A_e	Effective net area for tension member, in.^{2} (mm^2)	2.1
A_n	Net area, in.^{2} (mm^2)	2.1
A_w	Web area, in.^{2} (mm^2)	5.3
B	Overall width of rectangular HSS, in. (mm)	1.3
B_b	Overall width of rectangular HSS branch member in a truss connection, in. (mm)	9.2
C	HSS torsional constant	6.
D	Outside diameter of round HSS, in. (mm)	2.1
D_b	Outside diameter of round HSS branch member in a truss connection, in. (mm)	9.4
E	Modulus of elasticity, ksi (MPa)	2.2.1
F_{cr}	Critical stress for column buckling, ksi (MPa)	4.2
F_n	Nominal stress for rectangular HSS shear resistance, ksi (MPa)	5.2
F_u	Specified minimum tensile strength of the HSS, ksi (MPa)	3.1
F_y	Specified minimum yield strength of the HSS, ksi (MPa)	2.2.1
F_{y1}	Specified minimum yield strength of plate or connecting element that is welded to an HSS, ksi (MPa)	8.1
F_{yb}	Specified minimum yield strength of HSS branch member in a truss connection, ksi (MPa)	9.4
H	Overall height of rectangular HSS, in. (mm)	1.3
H_b	Overall height of rectangular HSS branch member in a truss connection, in. (mm)	9.2
K	Compression member effective length factor	4.1
L_b	Unbraced length, in. (mm)	5.1
L_{pd}	Maximum unbraced length for plastic moment M_p in plastic analysis, in. (mm)	5.3
M_n	Nominal flexural strength, kip-in. (N-mm)	5.1
M_p	Plastic moment of section, kip-in. (N-mm)	5.1
M_r	Yield moment of section, kip-in. (N-mm)	5.1
M_u	Required flexural strength, kip-in. (N-mm)	7.1
M_{ur}	Resultant required flexural strength for round HSS, kip-in. (N-mm)	7.1
N	Bearing length of concentrated load along length of HSS, in. (mm)	8.1
P_n	Nominal axial strength, kips (N)	2.2.1
P_u	Required axial strength, kips (N)	4.2
P_y	Axial yield load, kips (N)	2.2.1
Q	Effective area factor	4.2

Q_f Connection resistance reduction factor for compression in HSS, parameter used for truss connections . . . 9.4
Q_q Parameter used for truss connections . . . 9.4
Q_β Parameter used for truss connections . . . 9.4
R_f Reduction factor for wind forces on exposed HSS . . . 1.3
R_n Nominal resistance of connections to HSS, kips (N) . . . 8.
S Elastic section modulus, in.^{3} (mm^3) . . . 5.1
S_{eff} Effective elastic section modulus for thin-walled rectangular HSS, in.^{3} (mm^3) . . . 5.1
T_n Nominal torsional strength, kips (N) . . . 6.1
T_u Required torsional strength, kips (N) . . . 7.2
U Shear lag factor, parameter used for truss connections . . . 2.1
V_n Nominal shear strength, kips (N) . . . 5.2
V_u Required shear strength, kips (N) . . . 7.2
Z Plastic section modulus, in.^{3} (mm^3) . . . 5.1
a Length of essentially constant shear in a beam, in. (mm) . . . 5.2
b Flat width of rectangular HSS flange or side, which is permitted to be taken as $B - 3t$, in. (mm) . . . 2.2.1
b_1 Width of plate or connecting element that is welded to an HSS, in. (mm) . . . 8.1
b_{eoi} Parameter used for truss connections . . . 9.4
b_{gap} Parameter used for truss connections . . . 9.4
c Constant for bending in rectangular HSS branches of truss connections . . . 9.4
d Bolt diameter, in. (mm) . . . 9.1
f Stress, ksi (MPa) . . . 4.2
g Gap between branch members in a gapped K-connection, in. (mm) . . . 9.4
h Flat width of rectangular HSS web or side, which is permitted to be taken as $H - 3t$, in. (mm) . . . 2.2.1
k Distance from point of application of concentrated force to critical section of HSS, in. (mm) . . . 8.1
l Member length, in. (mm) . . . 2.3
l Connection length, in. (mm) . . . 2.1
r Radius of gyration, in. (mm) . . . 2.3
r_y Radius of gyration about the y-axis, in. (mm) . . . 5.1
t Design HSS wall thickness as given in Section 1.2, in. (mm) . . . 2.2.1
t_1 Thickness of plate or connecting element that is welded to an HSS, in. (mm) . . . 8.1
t_b Thickness of branch member in an HSS truss connection, in. (mm) . . . 9.4
$\bar{x}$ Eccentricity for shear lag, in. (mm) . . . 2.1
α Parameter used for truss connections . . . 9.4
β Parameter used for truss connections . . . 9.4
β_{eff} Parameter used for truss connections . . . 9.4
β_{eop} Parameter used for truss connections . . . 9.4
β_{gap} Parameter used for truss connections . . . 9.4
γ Parameter used for truss connections . . . 9.4
ϕ Resistance factor
η Parameter used for truss connections . . . 9.4
λ Wall slenderness . . . 2.2.1

λ_c	Column slenderness	4.2
λ_p	Maximum wall slenderness for compact section	2.2.1
λ_r	Maximum wall slenderness for non-compact section	2.2.1
ζ	Parameter used for truss connections	9.4

Load and Resistance Factor Design Specification for Steel Hollow Structural Sections

November 10, 2000

1. GENERAL PROVISIONS

1.1. Scope

This Specification is intended for the design of round and rectangular hollow structural sections (HSS) that are used as structural members in buildings and for the design of connections to HSS. HSS are: (1) prismatic structural shapes; and, (2) products of a pipe or tubing mill that meet the geometric tolerances, tensile requirements, and chemical requirements of a standard specification. Rectangular HSS include square and rectangular cross-sections that have rounded corners within the tolerances of an appropriate product specification. Only unstiffened non-composite HSS in non-fatigue applications are considered in this Specification.

This Specification includes the list of symbols.

This Specification is based on the AISC *Load and Resistance Factor Design Specification for Structural Steel Buildings* (AISC, 1999), hereinafter referred to as the LRFD Specification. In some cases, criteria taken from the LRFD Specification have been modified to appear in non-dimensional form and to apply directly to rectangular HSS, which have two webs. For situations that are not covered in this Specification, the criteria in the LRFD Specification shall apply. In seismic applications, HSS shall be designed to meet the requirements of the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 1997) and the *Seismic Provisions Supplement No. 2* (AISC, 2000).

1.2. Material

1. Structural Steel

HSS material that meets the requirements in one of the following ASTM specifications is approved for use under this Specification:

Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc Coated, Welded and Seamless, ASTM A53/A53M-99b Gr. B

Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes, ASTM A500-99

Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing, ASTM A501-99

Standard Specification for Hot-Formed Welded and Seamless High-Strength Low-Alloy Structural Tubing, ASTM A618-99

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

Certified mill test reports or certified reports of tests made by the fabricator or a qualified testing laboratory that meet the requirements in ASTM A370, Test Methods and Definitions for Mechanical Testing of Steel Products, and the governing specification shall constitute sufficient evidence of conformity with one of the above ASTM standards. If requested, the fabricator shall provide an affidavit stating that the structural steel furnished meets the requirements of the grade specified.

2. Design Wall Thickness

The design wall thickness t shall be used in calculations involving the HSS wall thickness. When the design wall thickness is not known, it is permitted to be taken as 0.93 times the nominal wall thickness.

1.3. Loads and Load Combinations

The nominal loads and factored load combinations shall be as stipulated by the applicable code under which the structure is designed or dictated by the condition involved. In the absence of a code, the loads and factored load combinations, including impact and crane loads, shall be those stipulated in ASCE 7. For design purposes, the loads stipulated by the applicable code or ASCE 7 shall be taken as nominal loads.

If permitted by the applicable building code, wind forces on the projected areas of exposed HSS are permitted to be reduced by the factor R_f from the forces on frameworks with similar configurations but using sections or shapes with flat elements. R_f shall be taken as follows:

(a) For round HSS, $R_f = 2/3$.
(b) For rectangular HSS with outside corner radii that are greater than or equal to 0.05 times the width B and wind force acting on the short side (B), $R_f = 0.4 + 0.6\,B/H \leq 2/3$, where H is the depth of the HSS. For rectangular HSS under other conditions, $R_f = 1.0$.

2. DESIGN REQUIREMENTS

2.1. Effective Area of Tension Members

The effective area A_e of tension members shall be determined as follows:

$$A_e = AU \tag{2.1-1}$$

(a) For a welded connection that is continuous around the perimeter, $A = A_g$, where A_g is the gross area and $U = 1$.
(b) For connections with concentric gusset plates and slotted HSS, $A = A_n$, where the net area A_n at the end of the gusset plate is the gross area minus the product of the thickness and total width of material that is removed to form the slots and

$$U = 1 - (\bar{x}/l) \leq 0.9 \tag{2.1-2}$$

In the above equation, $\bar{x}$ is the perpendicular distance from the weld to the centroid of the cross-sectional area that is tributary to the weld.

For round HSS with a single concentric gusset plate

$$\bar{x} = \frac{D}{\pi} \tag{2.1-3}$$

For rectangular HSS with a single concentric gusset plate

$$\bar{x} = \frac{B^2 + 2BH}{4(B + H)} \tag{2.1-4}$$

(c) For connections with rectangular HSS and a pair of side gusset plates, $A = A_g$, where A_g is the gross area and U shall be calculated using Equation 2.1-2 with

$$\bar{x} = \frac{B^2}{4(B + H)} \tag{2.1-5}$$

where

l = length of the connection in the direction of loading, in. (mm)
D = outside diameter of round HSS, in. (mm)
B = overall width of rectangular HSS, in. (mm)
H = overall height of rectangular HSS, in. (mm)

Larger values of U are permitted to be used in the foregoing cases when justified by tests or other rational criteria. For other end-connection configurations, U shall be determined by tests or other rational criteria.

2.2. Local Buckling

1. Classification of Steel Sections

HSS are classified for local buckling of the wall in compression as compact, noncompact, or slender-element cross-sections according to the limiting wall slenderness ratios λ_p and λ_r in Table 2.2-1. For an HSS to qualify as compact, the wall slenderness ratio λ must be less than or equal to λ_p. If λ exceeds λ_p but is less than or equal to λ_r, the HSS is noncompact. If λ exceeds λ_r, the HSS is a slender-element cross-section. The wall slenderness ratio λ shall be calculated as follows:

(a) For round HSS, $\lambda = D/t$, where D is the outside diameter and t is the wall thickness. This Specification is applicable only to round HSS with λ less than or equal to $0.448E/F_y$, where E is the modulus of elasticity and F_y is the specified minimum yield stress.
(b) For flanges of rectangular HSS, $\lambda = b/t$, where b is the clear distance between webs less the inside corner radius at each web and t is the wall thickness. If the corner radius is not known, b is permitted to be taken as the overall flange width B minus three times the wall thickness t.
(c) For webs of rectangular HSS, $\lambda = h/t$, where h is the clear distance between flanges less the inside corner radius at each flange and t is the wall thickness. If the corner radius is not known, h is permitted to be taken as the overall web depth H minus three times the wall thickness t.

2. Design by Plastic Analysis

Design by plastic analysis is permitted when λ is less than or equal to λ_p for plastic analysis in Table 2.2-1.

TABLE 2.2-1
Limiting Wall Slenderness for Compression Elements

Element	Wall Slenderness Ratio, λ	Limiting Wall Slenderness	
		λ_p (compact)	λ_r (noncompact)
Round HSS	D/t [a]		
for axial compression		n.a.	$0.114E/F_y$
for flexure		$0.0714E/F_y$	$0.309E/F_y$
for plastic analysis		$0.0448E/F_y$	n.a.
Rectangular HSS wall	b/t or h/t		
for uniform compression		$1.12\sqrt{E/F_y}$	$1.40\sqrt{E/F_y}$
for plastic analysis		$0.939\sqrt{E/F_y}$	n.a.
Rectangular HSS wall as a web in flexural compression	h/t	$3.76\sqrt{E/F_y}$	$5.70\sqrt{E/F_y}$
Rectangular HSS wall as a web in combined flexure and axial compression	h/t	[b]	$5.70\sqrt{E/F_y}\left(1-\dfrac{0.74P_u}{\phi_b P_y}\right)$

[a] D/t must be less than or equal to $0.448E/F_y$
[b] For $P_u/\phi_b P_y \le 0.125$

$$3.76\sqrt{E/F_y}\left(1-\frac{2.75P_u}{\phi_b P_y}\right)$$

For $P_u/\phi_b P_y > 0.125$

$$1.12\sqrt{E/F_y}\left(2.33-\frac{P_u}{\phi_b P_y}\right) \ge 1.49\sqrt{E/F_y}$$

3. Design in Seismic Applications

In seismic applications, λ shall also meet the requirements in the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 1997) and the *Seismic Provisions Supplement No. 2* (AISC, 2000).

2.3. Limiting Slenderness Ratios

For compression members, the slenderness ratio Kl/r preferably should not exceed 200.

For tension members, the slenderness ratio l/r preferably should not exceed 300. Members that are primarily tension members but that are subject to some compression under other load conditions need not satisfy the compression slenderness limit.

For bracing members in seismic applications, l/r shall meet the requirements in AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 1997) and the *Seismic Provisions Supplement No. 2* (AISC, 2000).

3. TENSION MEMBERS

3.1. Design Tensile Strength

The design tensile strength $\phi_t P_n$ shall be the lower value obtained according to the limit states of yielding in the gross section and fracture in the net section.

(a) For yielding on the gross area:

$$\phi_t = 0.9$$
$$P_n = F_y A_g \tag{3.1-1}$$

(b) For rupture on the net effective area:

$$\phi_t = 0.75$$
$$P_n = F_u A_e \tag{3.1-2}$$

where

A_e = effective net area, in.^{2} (mm^2)
A_g = gross area of HSS, in.^{2} (mm^2)
F_y = specified minimum yield strength, ksi (MPa)
F_u = specified minimum tensile strength, ksi (MPa)
P_n = nominal axial strength, ksi (MPa)

4. COLUMNS AND OTHER COMPRESSION MEMBERS

4.1. Effective Length and Slenderness Limitations

1. Effective Length

The effective length factor K for compression members shall be taken as follows or as determined by rational analysis:

(a) In trusses that are made with HSS branch (web) members that are welded around their full perimeter to continuous HSS chord members, the effective length factor K that is used to modify the length between panel points for in-plane buckling, or between locations of lateral bracing for out-of-plane buckling, shall be not less than:

$$K = 0.75 \text{ for branch members}$$
$$K = 0.9 \text{ for chord members}$$

(b) In trusses that are made with HSS branch members that do not meet the requirements in Section 4.1.1(a) or with non-HSS branch members connected to continuous HSS chord members, the effective length factor K that is used to modify the length between panel points for in-plane buckling shall be not less than:

$$K = 1.0 \text{ for branch members}$$
$$K = 0.9 \text{ for chord members}$$

(c) In frames for which lateral stability is provided by diagonal bracing, shear walls or equivalent means, K shall be taken as unity, unless a lesser value can be justified by rational analysis.

(d) In frames for which lateral stability is dependent upon the flexural stiffness of rigidly connected beams and columns, K shall be determined by rational analysis.

2. Design by Plastic Analysis

Design by plastic analysis is permitted if the column slenderness parameter λ_c is less than or equal to $1.5K$ and the axial force in columns of unbraced frames due to factored gravity loads plus factored lateral loads does not exceed ϕ_c times $0.75F_y A_g$.

4.2. Design Compressive Strength

The design strength for flexural buckling of compression members is $\phi_c P_n$.

$$\phi_c = 0.85$$
$$P_n = F_{cr} A_g \qquad (4.2\text{-}1)$$

F_{cr} shall be determined as follows:

(a) For $\lambda_c \sqrt{Q} \leq 1.5$,

$$F_{cr} = Q\left(0.658^{Q\lambda_c^2}\right) F_y \qquad (4.2\text{-}2)$$

(b) For $\lambda_c \sqrt{Q} > 1.5$,

$$F_{cr} = \left[\frac{0.877}{\lambda_c^2}\right] F_y \qquad (4.2\text{-}3)$$

where

$$\lambda_c = \frac{Kl}{r\pi}\sqrt{\frac{F_y}{E}} \qquad (4.2\text{-}4)$$

Q shall be determined as follows:

(a) For $\lambda \leq \lambda_r$ in Section 2.2, $Q = 1$

(b) For $\lambda > \lambda_r$ in Section 2.2,

(i) For round HSS with $\lambda < 0.448E/F_y$,

$$Q = \frac{0.0379E}{F_y(D/t)} + \frac{2}{3} \qquad (4.2\text{-}5)$$

(ii) For rectangular HSS,

$$Q = \frac{\text{effective area}}{A_g} \qquad (4.2\text{-}6)$$

where the effective area is equal to the summation of the effective areas of the sides using

$$b_e = 1.91t\sqrt{\frac{E}{f}}\left[1 - \frac{0.381}{(b/t)}\sqrt{\frac{E}{f}}\right] \leq b \qquad (4.2\text{-}7)$$

with $f = P_u/A_g$

5. BEAMS AND OTHER FLEXURAL MEMBERS

5.1. Design Flexural Strength

The design flexural strength $\phi_b M_n$ shall be determined as follows:

$$\phi_b = 0.90$$

(a) For round HSS, for $\lambda \leq \lambda_p$ in Section 2.2,

$$M_n = M_p = F_y Z \qquad (5.1\text{-}1)$$

(i) For $\lambda_p < \lambda \leq \lambda_r$,

$$M_n = \left(\frac{0.0207}{D/t}\frac{E}{F_y} + 1\right) F_y S \qquad (5.1\text{-}2)$$

(ii) For $\lambda_r < \lambda \leq 0.448E/F_y$,

$$M_n = \frac{0.330E}{D/t} S \tag{5.1-3}$$

(b) For rectangular HSS, for $\lambda \leq \lambda_p$ in Section 2.2,

$$M_n = M_p = F_y Z \tag{5.1-4}$$

(i) For $\lambda_p < \lambda \leq \lambda_r$,

$$M_n = \left[M_p - (M_p - M_r) \left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right) \right] \tag{5.1-5}$$

where

$$M_r = F_y S$$

(ii) For $\lambda > \lambda_r$,

$$M_n = F_y S_{eff} \tag{5.1-6}$$

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

$$b_e = 1.91t \sqrt{\frac{E}{F_y}} \left[1 - \frac{0.381}{(b/t)} \sqrt{\frac{E}{F_y}} \right] \leq b \tag{5.1-7}$$

L_b is not limited for HSS structures designed by elastic analysis.

5.2. Design Shear Strength

The design shear strength of unstiffened HSS $\phi_v V_n$ shall be determined as follows:

$$\phi_v = 0.9$$

(a) For round HSS,

$$V_n = F_{cr} A_g / 2 \tag{5.2-1}$$

where F_{cr} shall be the larger of

$$\frac{1.60E}{\sqrt{a/D}(D/t)^{5/4}} \quad \text{and} \quad \frac{0.78E}{(D/t)^{3/2}} \tag{5.2-2}$$

but shall not exceed $0.6F_y$ and a is the distance from maximum to zero shear force.

(b) For rectangular HSS,

$$V_n = F_n A_w \tag{5.2-3}$$

where

$$A_w = 2Ht \tag{5.2-4}$$

F_n shall be determined as follows:

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

$$F_n = 0.6F_y \tag{5.2-5}$$

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

$$0.6F_y(2.45\sqrt{E/F_y})/(h/t) \qquad (5.2\text{-}6)$$

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

$$F_n = 0.458\pi^2 E/(h/t)^2 \qquad (5.2\text{-}7)$$

5.3. Design by Plastic Analysis

Design by plastic analysis is permitted for compact round HSS with λ less than or equal to $0.0448E/F_y$ and for rectangular HSS with λ less than or equal to λ_p in Section 2.2.

For rectangular HSS bent about the major axis, the laterally unbraced length L_b of the compression flange adjacent to plastic hinge locations that are associated with the failure mechanism shall not exceed L_{pd}, where

$$L_{pd} = \frac{5000 + 3000(M_1/M_2)}{F_y} r_y \geq 3000r_y/F_y \qquad (5.3\text{-}1)$$

and

F_y = specified minimum yield stress, ksi (MPa)
M_1 = smaller moment at the end of the unbraced length, kip-in. (N-mm)
M_2 = larger moment at the end of the unbraced length, kip-in. (N-mm)
r_y = radius of gyration about the minor axis, in. (mm)
M_1/M_2 is positive when moments cause reverse curvature and negative for single curvature

5.4. Design in Seismic Applications

For seismic applications, refer to AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 1997) and *Seismic Provisions Supplement No. 2* (AISC, 2000).

6. TORSION MEMBERS

6.1. Design Torsional Strength

The design torsional strength $\phi_T T_n$ shall be determined as follows:

$$\phi_T = 0.90$$
$$T_n = F_{cr}C \qquad (6.1\text{-}1)$$

where C is the HSS torsional constant.

F_{cr} shall be determined as follows:

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

$$\frac{1.23E}{\sqrt{L/D}(D/t)^{5/4}} \quad \text{and} \quad \frac{0.6E}{(D/t)^{3/2}} \qquad (6.1\text{-}2)$$

but shall not exceed $0.6F_y$.

(b) For rectangular HSS,

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

$$F_{cr} = 0.6F_y \qquad (6.1\text{-}3)$$

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

$$F_{cr} = 0.6F_y(2.45\sqrt{E/F_y})/(h/t) \tag{6.1-4}$$

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

$$F_{cr} = 0.458\pi^2 E/(h/t)^2 \tag{6.1-5}$$

7. MEMBERS UNDER COMBINED FORCES

7.1. Design for Combined Flexure and Axial Force

The interaction of flexure and axial force shall be limited by Equations 7.1-1 and 7.1-2.

(a) For $P_u/\phi P_n \geq 0.2$,

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \tag{7.1-1}$$

(b) For $P_u/\phi P_n < 0.2$,

$$\frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \tag{7.1-2}$$

where

P_u = required axial tensile or compressive strength, kips (N)
P_n = nominal tensile or compressive strength determined in accordance with Sections 3.1 or 4.2, kips (N)
M_u = required flexural strength determined in accordance with LRFD Specification Section C1, kip-in. (N-mm)
M_n = nominal flexural strength determined in accordance with Section 5.1, kip-in. (N-mm)
x = subscript relating symbol to strong-axis bending
y = subscript relating symbol to weak-axis bending
ϕ = ϕ_t from Section 3.1 for tension
= 0.85 for compression
ϕ_b = 0.90

For biaxial flexure of round HSS that are laterally unbraced along their length and with end conditions such that the effective length factor K is the same for any direction of bending, the design is permitted to be based upon a single resultant moment M_{ur}, where:

$$M_{ur} = \sqrt{M_{ux}^2 + M_{uy}^2} \tag{7.1-3}$$

Alternatively, use of the provisions in LRFD Specification Appendix H3 (b) is permitted.

7.2. Design for Combined Torsion, Shear, Flexure, and/or Axial Force

When the required torsional strength is significant, the interaction of torsion, shear, flexure, and/or axial force shall be limited by Equation 7.2-1:

$$\left(\frac{P_u}{\phi P_n} + \frac{M_u}{\phi_b M_n}\right) + \left(\frac{V_u}{\phi_v V_n} + \frac{T_u}{\phi_T T_n}\right)^2 \leq 1.0 \tag{7.2-1}$$

where

P_u = required axial tensile or compressive strength, kips (N)
P_n = nominal tensile or compressive strength determined in accordance with Sections 3.1 or 4.2, kips (N)
M_u = required flexural strength determined in accordance with LRFD Specification Section C1, kip-in. (N-mm)
M_n = lesser of $F_y S$ and M_n determined in accordance with Section 5.1, kip-in. (N-mm)
S = elastic section modulus
V_u = required shear strength at the section corresponding to M_u, kips (N)
V_n = nominal shear strength determined in accordance with Section 5.2, kips (N)
T_u = required torsional strength, kip-in. (N-mm)
T_n = nominal torsional strength determined in accordance with Section 6, kip-in. (N-mm)
ϕ = ϕ_t from Section 3.1 for tension
= 0.85 for compression
$\phi_b = \phi_v = \phi_T = 0.90$

8. CONCENTRATED FORCES ON HSS

The design strength ϕR_n at locations of concentrated forces on unstiffened HSS shall be determined from the applicable criteria in Sections 8.1 through 8.3.

8.1. Concentrated Force Distributed Transversely

When a concentrated force is distributed transversely to the axis of the HSS, the design strength ϕR_n shall be determined as follows:

(a) For round HSS,

$$\phi = 1.0$$

$$R_n = \frac{5F_y t^2}{1 - 0.81 b_1/D} Q_f \tag{8.1-1}$$

where

b_1 = the width of the load, in. (mm)
Q_f = 1 for tension in the HSS
= $1 - 0.3f/F_y - 0.3(f/F_y)^2 \leq 1$ for compression in the HSS
f = the magnitude of the maximum compressive stress in the HSS due to axial force and bending at the location of the concentrated force, ksi (MPa)

(b) For rectangular HSS,

$$\phi = 1.0$$

$$R_n = \frac{10F_y t}{B/t} b_1 \le F_{y1} t_1 b_1 \quad (8.1\text{-}2)$$

where

b_1 = the width of the loaded plate, in. (mm)
t_1 = the thickness of the loaded plate, in. (mm)
F_{y1} = specified minimum yield strength of the loaded plate, ksi (MPa)

(i) When the force is distributed across the full width of the rectangular HSS, the limit state of local web yielding shall be checked for both tensile and compressive forces and the limit state of web crippling shall be checked for compressive forces.

For local web yielding,

$$\phi = 1.0$$
$$R_n = 2F_y t(5k + N) \quad (8.1\text{-}3)$$

For web crippling,

$$\phi = 0.75$$
$$R_n = 1.6t^2[1 + 3N/h]\sqrt{EF_y} \quad (8.1\text{-}4)$$

where

k = outside corner radius of the HSS, which if not known is permitted to be taken as $1.5t$, in. (mm)
N = bearing length of the load along the length of the HSS, in. (mm)
h = flat width of side wall of the HSS as defined in Section 2.2.1, in. (mm)

(ii) When the force is distributed across a width of the rectangular HSS that is greater than $0.85B$ but less than $B - 2t$, the design strength shall not exceed ϕR_n, where

$$\phi = 1.0$$
$$R_n = 0.6F_y t(2t_1 + 2b_{ep}) \quad (8.1\text{-}5)$$

where $b_{ep} = 10b_1/(B/t) \le b_1$

(iii) When compressive forces coincide on opposite faces of the rectangular HSS, the limit state of compression buckling of the webs shall be checked and the design strength shall not exceed ϕR_n, where

$$\phi = 0.90$$
$$R_n = \frac{48t^3\sqrt{EF_y}}{h} \quad (8.1\text{-}6)$$

8.2. Concentrated Force Distributed Longitudinally at the Center of the HSS Face

When a concentrated force is distributed longitudinally along the axis of the HSS at the center of the HSS face, the design strength ϕR_n shall be determined as follows:

(a) For round HSS,

$$\phi = 1.0$$

$$R_n = 5F_y t^2(1 + 0.25N/D)Q_f \tag{8.2-1}$$

(b) For rectangular HSS,

$$\phi = 1.0$$

$$R_n = \frac{F_y t^2}{1 - t_1/B}\left[\frac{2N}{B} + 4\sqrt{1 - t_1/B}\right]Q_f \tag{8.2-2}$$

where t_1 is the thickness of the loaded plate.

8.3. Concentrated Axial Force on the End of a Rectangular HSS with a Cap Plate

When a concentrated force acts on the end of an HSS with a cap plate and along the axis of the HSS, the design strength ϕR_n shall be determined for each loaded wall as follows. The limit state of local wall yielding shall be checked for both tensile and compressive forces and the limit state of wall crippling shall be checked for compressive forces.

For local wall yielding,

$$\phi = 1.0$$

$$R_n = (5t_1 + N)F_y t \leq BF_y t \tag{8.3-1}$$

For wall crippling,

$$\phi = 0.75$$

$$R_n = 0.80t^2\left[1 + 3\left(\frac{N}{B/2}\right)\left(\frac{t}{t_1}\right)^{1.5}\right]\sqrt{EF_y(t_1/t)} \tag{8.3-2}$$

where

t_1 = thickness of cap plate, in. (mm)
N = bearing length of the load across the width of the HSS, in. (mm)

9. CONNECTIONS AND FASTENERS

9.1. General Provisions for Connections and Fasteners

The provisions of LRFD Specification Section J1.1 through J1.11 and the provisions for bolts and threaded parts in LRFD Specification Sections J3.1 through J3.11 shall apply with the following additions and modifications.

1. Through Bolts

When connections are made using bolts that pass completely through an unstiffened HSS, the bolts shall be installed to only the snug-tight condition and the connection shall be considered to be a bearing-type connection. The bearing strength per loaded wall is ϕR_n, where

$$\phi = 0.75$$
$$R_n = 1.8 F_y dt \tag{9.1-1}$$

where

F_y = specified minimum yield strength of the HSS, ksi (MPa)
d = bolt diameter, in. (mm)
t = HSS wall thickness, in. (mm)

2. Special Connectors

The design strength of special connectors other than the bolts considered in LRFD Specification Table J3.2 shall be verified by tests.

3. Tension Connectors

When bolts or other connectors in tension are attached to an HSS wall, the strength of the HSS wall shall be determined by rational analysis.

9.2. Welds

The non-uniformity of load transfer along the line of weld due to differences in relative flexibility of HSS walls in HSS-to-HSS and similar connections shall be considered in proportioning such connections. In such cases, the strength of fillet welds shall be determined from LRFD Specification Section J2.4, excluding the alternative in Appendix J2.4, and the effective weld length L_e of groove and fillet welds shall be limited as follows:

(a) In T-, Y-, and Cross-connections with rectangular HSS as defined in Section 9.4,

$$L_e = 2H_b + B_b \text{ for } \theta \leq 50 \text{ degrees} \tag{9.2-1}$$
$$L_e = 2H_b \text{ for } \theta \geq 60 \text{ degrees} \tag{9.2-2}$$

Linear interpolation shall be used to determine L_e for values of θ between 50 and 60 degrees.

(b) In gapped K-connections with rectangular HSS as defined in Section 9.4,

$$L_e = 2H_b + 2B_b \text{ for } \theta \leq 50 \text{ degrees} \tag{9.2-3}$$
$$L_e = 2H_b + B_b \text{ for } \theta \geq 60 \text{ degrees} \tag{9.2-4}$$

Linear interpolation shall be used to determine L_e for values of θ between 50 and 60 degrees.

(c) When a transverse plate is welded to the face of an HSS member,

$$L_e = 2\frac{10}{B/t}\frac{F_y t}{F_{y1} t_1} b_1 \leq 2b_1 \tag{9.2-5}$$

where

H_b = width of branch member wall that is parallel to the axis of the chord member, in. (mm)
B_b = width of branch member wall that is transverse to the axis of the chord member, in. (mm)
θ = least angle between branch member and chord member
B = width of chord member wall to which plate is attached, in. (mm)
b_1 = width of attached plate, in. (mm)
t = thickness of chord member wall, in. (mm)
t_1 = thickness of attached plate, in. (mm)
F_y = yield strength of HSS, ksi (MPa)
F_{y1} = yield strength of plate, ksi (MPa)

In lieu of the above, other rational criteria are permitted.

9.3. Other Connection Requirements

The provisions of LRFD Specification Sections J4 through J10 shall apply with the following additions and modifications.

1. Shear Rupture Strength

The design shear rupture strength along a path adjacent to a fillet weld on the HSS wall shall be taken as ϕR_n, where

$$\phi = 0.75$$
$$R_n = 0.6F_u t L \tag{9.3-1}$$

where

t = HSS wall thickness, in. (mm)
L = length of weld, in. (mm)

2. Tension Rupture Strength

The design tension rupture strength along a path adjacent to a fillet weld on the HSS wall shall be taken as ϕR_n, where

$$\phi = 0.75$$
$$R_n = F_u t L \tag{9.3-2}$$

where

t = HSS wall thickness, in. (mm)
L = length of weld, in. (mm)

3. Punching Shear Rupture Strength

When a plate that is parallel to the longitudinal axis of an HSS and projects from the wall is subjected to a load that is parallel but eccentric or has a component perpendicular to the HSS wall,

$$\phi_t f t_p \leq 1.2\phi_v F_u t \tag{9.3-3}$$

where

$\phi_v = 0.75$
$\phi_t = 0.90$
f = maximum stress in the plate perpendicular to the HSS wall, ksi (MPa)
t_p = plate thickness, in. (mm)
F_u = specified minimum tensile strength of the HSS, ksi (MPa)
t = HSS wall thickness, in. (mm)

4. Eccentric Connections

For trusses that are made with HSS that are connected by welding branch members to chord members, eccentricities within the limits of applicability in Section 9.4 are permitted without consideration of the resulting moments for the design of the connection, except in fatigue applications. In fatigue applications, refer to AWS D1.1.

9.4. HSS-to-HSS Truss Connections

HSS-to-HSS truss connections are defined as connections that consist of one or more branch members that are directly welded to a continuous chord that passes through the connection and shall be classified as follows:

(a) When the punching load in a branch member is equilibrated by beam shear in the chord member, the connection shall be classified as a T-connection when the branch is perpendicular to the chord and a Y-connection otherwise.
(b) When the punching load in a branch member is essentially equilibrated by loads in other branch member(s) on the same side of the joint, the connection shall be classified as a K-connection.
(c) When the punching load is transmitted through the chord member and is equilibrated by branch member(s) on the opposite side, the connection shall be classified as a Cross-connection.

When branch members transmit part of their load as K-connections and part of their load as T-, Y-, or Cross-connections, the design strength shall be determined by interpolation on the proportion of each in total.

For the purposes of this Specification, the centerlines of the branch member(s) and the chord members shall lie in a single plane and K-connections shall be used in the gapped configuration. For other configurations such as a multi-planar connection, a connection with a branch member that is offset so that its centerline does not intersect with the centerline of the chord, or when an overlapped K-connection is used, the provisions of AWS D1.1, other verified design procedures, tests, or rational analysis shall be used.

1. Definitions of Parameters

β = the width ratio; the ratio of branch diameter to chord diameter $= D_b/D$ for round HSS; the ratio of overall branch width to chord width $= B_b/B$ for rectangular HSS.
β_{eff} = the effective width ratio; the sum of the perimeters of the two branch members in a K-connection divided by eight times the chord width.
γ = the chord slenderness ratio; the ratio of one-half the diameter to the wall thickness $= D/2t$ for round HSS; the ratio of one-half the width to wall thickness $= B/2t$ for rectangular HSS.

η = the load length parameter, applicable only to rectangular HSS; the ratio of the length of contact of the branch with the chord in the plane of the connection to the chord width $= N/B$, where $N = H_b/\sin\theta$ and θ is the angle between the branch and chord.

ζ = the gap ratio; the ratio of the gap between the branches of a gapped K-connection to the width of the chord $= g/D$ for round HSS $= g/B$ for rectangular HSS.

2. Criteria for Round HSS

The design strength of the branch ϕP_n and/or ϕM_n for axial loads in the branch and for flexure in the branch, respectively, shall be determined from the limit states of chord wall plastification, punching shear rupture, and general collapse as applicable below.

The interaction of stress due to chord member forces and local branch connection forces shall be considered. The chord-stress interaction parameter Q_f shall be determined as

$$Q_f = 1.0 - \lambda\gamma U^2 \tag{9.4-1}$$

where U is the utilization ratio given by

$$U^2 = \left(\frac{P_u}{A_g F_y}\right)^2 + \left(\frac{M_u}{S F_y}\right)^2 \tag{9.4-2}$$

and

λ = 0.030 for axial load in the branch
= 0.044 for in-plane bending in the branch
= 0.018 for out-of-plane bending in the branch

P_u = required axial strength in chord, kips (N)

A_g = chord gross area, in.^{2} (mm^2)

F_y = chord yield strength, ksi (MPa)

M_u = larger required flexural strength in chord and connection, kip-ft (N-mm)

S = chord elastic section modulus, in.^{3} (mm^3)

2a. Limits of Applicability

The criteria herein are applicable only when the connection configuration is within the following limits of applicability:

(1) joint eccentricity: $-0.55D \le e \le 0.25D$, where D is the chord diameter and e is positive away from the branches
(2) branch angle: $\theta \ge 30°$
(3) wall stiffness: ratio of diameter to wall thickness less than or equal to 50 for chords and branches in T-, Y- and K-connections and less than or equal to 40 for chords of Cross-connections
(4) width ratio: $0.2 < D_b/D \le 1.0$
(5) gap: g greater than or equal to the sum of the branch wall thicknesses

2b. Branches with Axial Loads

For T-, Y-, and gapped K-connections, the design strength of the branch ϕP_n shall be the lower value obtained according to the limit states of chord wall plastification and punching shear rupture. For Cross-connections, the design strength of the branch

ϕP_n shall be the lowest value obtained according to the limit states of chord wall plastification, punching shear rupture, and general collapse.

(a) For the limit state of chord wall plastification,

$$\phi = 0.80$$
$$P_n \sin\theta = t^2 F_y [6\pi\beta Q_q] Q_f \tag{9.4-3}$$

where

$$Q_q = \left(\frac{1.7}{\alpha} + \frac{0.18}{\beta}\right) Q_\beta^{0.7(\alpha-1)}$$

(i) For $\beta \le 0.6$, $Q_\beta = 1.0$

(ii) For $\beta > 0.6$, $Q_\beta = \dfrac{0.3}{\beta(1 - 0.833\beta)}$

α = chord ovalization parameter
= 1.7 for T- and Y-connections
= $1.0 + 0.7g/D_b$, $1 \le \alpha < 1.7$ for gapped K-connections
= 2.4 for Cross-connections

(b) For the limit state of punching shear rupture,

$$\phi = 0.95$$
$$P_n \sin\theta = \pi D_b t (0.6 F_y) \tag{9.4-4}$$

(c) For the limit state of general collapse,

$$\phi = 0.80$$
$$P_n \sin\theta = 1.8 t^2 F_y (1.9 + 7.2\beta) Q_\beta Q_f \tag{9.4-5}$$

2c. Branches with Flexure

For T-, Y-, gapped K-, and Cross-connections, the design strength of the branch ϕM_n shall be the lower value obtained according to the limit states of chord wall plastification and punching shear rupture.

(a) For the limit state of chord wall plastification,

$$\phi = 0.80$$
$$M_n \sin\theta = t^2 F_y [D_b/4][6\pi\beta Q_q] Q_f \tag{9.4-6}$$

where

$$Q_q = \left(\frac{2.1}{\alpha} + \frac{0.6}{\beta}\right) Q_\beta^{1.2(\alpha-0.67)}$$

(i) For $\beta \le 0.6$, $Q_\beta = 1.0$

(ii) For $\beta > 0.6$, $Q_\beta = \dfrac{0.3}{\beta(1 - 0.833\beta)}$

α = chord ovalization parameter
= 0.67 for in-plane bending
= 1.5 for out-of-plane bending

For combinations of in-plane and out-of-plane bending, α shall be determined by interpolation and Q_f shall be determined with interpolated values of λ.

(b) For the limit state of punching shear rupture,

$$\phi = 0.95$$
$$M_n \sin\theta = D_b^2 t(0.6F_y) \qquad (9.4\text{-}7)$$

2d. Branches with Combined Axial Loads and Flexure

The interaction of combined axial loads and flexure in HSS-to-HSS truss connections shall meet the following requirement:

$$\left(\frac{P_u}{\phi P_n}\right)^{1.75} + \frac{M_u}{\phi M_n} \le 1.0 \qquad (9.4\text{-}8)$$

where

P_u = required axial strength of the branch, kips (N)
M_u = required flexural strength of the branch, kip-ft (N-mm)

3. Criteria For Rectangular HSS

The design axial strength of the branch ϕP_n and the design flexural strength ϕM_n of the branch shall be the lowest value obtained according to the limit states of chord wall plastification, punching shear rupture, sidewall strength, and uneven load distribution as applicable below.

The interaction of stress due to chord member forces and local branch connection forces shall be considered with the chord-stress interaction parameter Q_f, where

Q_f = 1 when the chord is in tension
$= 1.3 - 0.4U/\beta \le 1$ (9.4-9a)
in T-, Y-, and Cross-connections when the chord is in compression
$= 1.3 - 0.4U/\beta_{eff} \le 1$ (9.4-9b)
in K-connections when the chord is in compression

where

$$U = \left|\frac{P_u}{A_g F_y}\right| + \left|\frac{M_u}{SF_y}\right| \qquad (9.4\text{-}10)$$

where

P_u = required axial strength of the chord, kips (N)
M_u = required flexural strength of the chord, kip-ft (N-mm)

3a. Limits of Applicability

The criteria herein are applicable only when the connection configuration is within the following limits:

(1) joint eccentricity: $-0.55H \le e \le 0.25H$, where H is the chord depth and e is positive away from the branches
(2) branch angle: $\theta \ge 30°$
(3) wall stiffness: ratio of wall width to wall thickness less than or equal to 35 for chords and branches; also less than or equal to $1.25\sqrt{E/F_{yb}}$ for branches in compression

(4) strength: F_y less than or equal to 52 ksi (360 MPa) for chord and branches
(5) chord and branch aspect ratio: $0.5 \leq$ ratio of depth to width ≤ 2.0
(6) ductility: $F_y/F_u \leq 0.8$
(7) other limits apply for specific criteria

3b. Branches with Axial Loads in T-, Y- and Cross-connections

For T-, Y-, and Cross-connections, the design strength of the branch ϕP_n shall be the lowest value obtained according to the limit states of chord wall plastification, punching shear rupture, sidewall strength, and uneven load distribution. In addition to the limits of applicability in Section 9.4.3a, β shall not be less than 0.25.

(a) For the limit state of chord wall plastification,

$$\phi = 1.0$$

$$P_n \sin\theta = F_y t^2 \left[\frac{2\eta}{1-\beta} + \frac{4}{\sqrt{(1-\beta)}} \right] Q_f \tag{9.4-11}$$

This limit state need not be checked when $\beta > 0.85$.

(b) For the limit state of punching shear rupture,

$$\phi = 0.95$$

$$P_n \sin\theta = 0.6 F_y t B [2\eta + 2\beta_{eop}] \tag{9.4-12}$$

In the above equation, the effective outside punching parameter $\beta_{eop} = 5\beta/\gamma$ shall not exceed β.

This limit state need not be checked when $\beta > 1 - 1/\gamma$ nor when $\beta < 0.85$.

(c) For the limit state of sidewall strength, the design strength for branches in tension shall be taken as the design strength for local sidewall yielding. For the limit state of sidewall strength, the design strength for branches in compression shall be taken as the lesser of the design strengths for local sidewall yielding and sidewall crippling.

This limit state need not be checked unless the chord member and branch member have the same width ($\beta = 1.0$).

(i) For the limit state of local yielding,

$$\begin{aligned} \phi &= 1.0 \text{ for a branch in tension} \\ &= 0.8 \text{ for a branch in compression} \end{aligned}$$

$$P_n \sin\theta = 2t F_y (5k + N) \tag{9.4-13}$$

where

k = outside corner radius of the HSS, which is permitted to be taken as $1.5t$ if unknown, in. (mm)
N = bearing length of the load along the length of the HSS, $H_b/\sin\theta$, in. (mm)
H_b = height of the branch, in. (mm)

(ii) For the limit state of sidewall crippling, in T- and Y-connections,

$$\phi = 0.75$$
$$P_n \sin\theta = 1.6t^2[1 + 3N/H]\sqrt{EF_y}Q_f \tag{9.4-14}$$

(iii) For the limit state of sidewall crippling in Cross-connections,

$$\phi = 0.80$$
$$P_n \sin\theta = \left[\frac{48t^3}{H - 4t}\right]\sqrt{EF_y}Q_f \tag{9.4-15}$$

(d) For the limit state of uneven load distribution,

$$\phi = 0.95$$
$$P_n = F_{yb}t_b[2H_b + 2b_{eoi} - 4_{tb}] \tag{9.4-16}$$

where

$$b_{eoi} = \frac{10}{B/t}\frac{F_y t}{F_{yb}t_b}B_b \le B_b \tag{9.4-17}$$

F_{yb} = branch yield strength, ksi (MPa)
t_b = branch thickness, in. (mm)

This limit state need not be checked when $\beta < 0.85$.

3c. Branches with Axial Loads in Gapped K-connections

For gapped K-connections, the design strength of each branch ϕP_n shall be the lowest value obtained according to the limit states of chord wall plastification, punching shear rupture, shear yielding, and uneven load distribution. In addition to the limits of applicability in Section 9.4.3a, the following limits shall apply:

(1) $B_b/B \ge 0.1 + \gamma/50$
(2) $\beta_{eff} \ge 0.35$
(3) $\xi \ge 0.5(1 - \beta_{eff})$
(4) the smaller $B_b > 0.63$ times the larger B_b

(a) For the limit state of chord wall plastification,

$$\phi = 0.90$$
$$P_n \sin\theta = F_y t^2[9.8\beta_{eff}\sqrt{\gamma}\,]Q_f \tag{9.4-18}$$

(b) For the limit state of punching shear rupture,

$$\phi = 0.95$$
$$P_n \sin\theta = (0.6F_y)tB[2\eta + \beta + \beta_{eop}] \tag{9.4-19}$$

This limit state need only be checked if $B_b < B - 2t$ or the branch is not square.

(c) For the limit state of shear yielding of the chord in the gap, the design strength shall be checked in accordance with Section 5.2.

(d) For the limit state of uneven load distribution,

$$\phi = 0.95$$
$$P_n = F_{yb} t_b [2H_b + B_b + b_{eoi} - 4t_b] \tag{9.4-20}$$

where

$$b_{eoi} = \frac{10}{B/t} \frac{F_y t}{F_{yb} t_b} B_b \leq B_b \tag{9.4-21}$$

This limit need only be checked if the branch is not square.

3d. Branches with Bending

Primary bending moments M_u due to applied loads, cantilevered beams, sidesway of unbraced frames, and other sources, shall be considered in the design as an additional axial tension or compression load

$$P_{ua} = \frac{M_u}{c \sin\theta} \tag{9.4-22}$$

where

$c = N/4$ for in-plane bending
$c = B_b/4$ for out-of-plane bending

10. GENERAL REQUIREMENTS FOR HSS FABRICATION

The following requirements shall be met in addition to the requirements of LRFD Specification Chapter M.

(1) When water can collect inside an HSS, either during construction or during service, HSS shall be sealed, provided with a drain hole at the base, or protected by other suitable means.
(2) HSS shall be cleaned with a suitable solvent if paint is specified per LRFD Specification Section M3.1.
(3) HSS shall be cleaned with a suitable solvent at locations of welding.

COMMENTARY
on the Load and Resistance Factor Design Specification for Steel Hollow Structural Sections

November 10, 2000

1. GENERAL PROVISIONS

1.1. Scope

For the purposes of this Specification, HSS are defined as hollow structural sections with constant wall thickness and a round, square, or rectangular cross-section that is constant along the length of the member. HSS are manufactured by forming skelp (strip or plate) to the desired shape and joining the edges with a continuously welded seam. Although the term pipe is commonly associated with round members that are used for fluid transmission, only steel pipe products that are used for structural purposes are included in the HSS definition. Published information is available describing the details of the various methods used to manufacture HSS (STI, 1996; Graham, 1965).

Because the design requirements for guaranteed pressure containment systems are more stringent than those for structural members, this Specification does not apply to members for which pressure containment is essential. Several other potential applications are also excluded from the scope of this Specification: (1) buried cylindrical shapes for which soil interaction is an important factor in the required strength; (2) stiffened HSS; (3) composite HSS; and, (4) HSS in fatigue applications. However, it is not intended that HSS with connection elements that also stiffen the cross-section be excluded from the scope.

Non-HSS products such as fabricated pipes and stiffened shells are excluded from the scope of this Specification. These are defined as members that are formed by shaping plates and joining them with one or more longitudinal seam welds, but neither in a tubing mill nor in accordance with a product specification. Although it is certainly possible to fabricate large pipes and shells to the same quality level as is commonly obtained with manufactured HSS, such quality is not universally assured by standard product specifications. Because the buckling strength of cylindrical sections is greatly influenced by geometric imperfections, there is good justification for excluding such products from the scope of this Specification. Accordingly, it is left to the Structural Engineer of Record (SER) to determine the suitability of such products for use with this Specification.

HSS are efficient structural members for the resistance of compressive and torsional forces. Consequently, they are increasingly selected in structural applications such as columns and members in plane trusses or space frames. HSS generally have a lower ratio of exposed surface area to volume when compared with other shapes, which results in reduced painting, fireproofing, and maintenance expense. Additionally, their low resistance to external fluid flow provides a distinct advantage for frameworks that are exposed to wind or water currents. The use of HSS has been limited in the past by difficulty in joining, but modern fabricating technology has overcome this disadvantage.

This Specification combines design guidelines from several sources. The primary basis for the recommendations is the design philosophy and criteria contained in the LRFD Specification. Because much of the LRFD Specification reflects the behavior of wide flange members, modifications have been made where HSS have been shown to behave differently or when interpretation of LRFD criteria to HSS applications can be clarified or simplified. Such modifications are explained in the Commentary. The criteria have also been modified to appear in a non-dimensional form.

In areas where the LRFD Specification contains little direct guidance for the design of buildings with HSS, such as connections, basic research, and criteria from other sources have been used. Much of the basic research is taken from the Comite International pour le Developpement et l'Etude de la Construction Tubulaire (CIDECT) programs, which have sponsored many projects in Europe and Canada concerning HSS construction. These have been incorporated into this Specification with modifications to provide design strengths that are comparable with those in the LRFD Specification. Consequently, this Specification is intended for the design of structural members in building structures that are normally encountered in structural engineering practice and experience. When the general uncertainty in loading or quality of control is substantially different, this Specification may not apply.

When HSS are used in the seismic force resisting system of buildings in high-seismic regions, the requirements in the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 1997) and Supplement No. 2 to the AISC *Seismic Provisions* (AISC, 2000) are applicable.

1.2. Material

ASTM A53 Grade B is included as an approved HSS material specification because it is the most readily available round HSS product in the United States. Other North American HSS products that have properties and characteristics that are similar to the approved ASTM products are produced in Canada under CAN/CSA-G40.21-M, "Structural Quality Steels." In addition, steel pipe is produced to other specifications that meet the strength, ductility, and weldability requirements of the materials in Section 1.2, but may have additional requirements for notch toughness or pressure testing.

Specified minimum yield and tensile strengths are summarized in Table C1.2-1 for various HSS material specifications and grades.

Round HSS can be readily obtained in most of the material specifications and grades in Table C1.2-1, although atmospheric-corrosion-resistant material (ASTM A618 and A847) may require a special order. For rectangular HSS, ASTM A500 Grade B is the most commonly available material, and a special order would be required for any other

TABLE C1.2-1
Minimum Tensile Properties of HSS Steels

Specification	Grade	F_y, ksi (MPa)	F_u, ksi (MPa)
ASTM A53	B	35 (240)	60 (415)
ASTM A500 (round)	A B C	33 (228) 42 (290) 46 (317)	45 (311) 58 (400) 62 (428)
ASTM A500 (rectangular)	A B C	39 (269) 46 (317) 50 (345)	45 (311) 58 (400) 62 (428)
ASTM A501	–	36 (248)	58 (400)
ASTM A618 (round)	I and II III	50 (345) 50 (345)	70 (483) 65 (448)
ASTM A847	–	50 (345)	70 (483)
CAN/CSA G40.21	350W	51 (350)	65 (450)

material. Depending upon size, either welded or seamless round HSS can be obtained. In North America, however, all ASTM A500 rectangular HSS for structural purposes are welded. Rectangular HSS differ from box sections in that they have uniform thickness except for some thickening in the rounded corners.

ASTM A500 Grade A material does not meet the ductility "limit of applicability" for direct connections in Section 9.4.3a. This limit requires that $F_y/F_u \leq 0.8$. In determining that other materials meet the ductility limit, it is important to note that ASTM A500 permits the yield to be determined by either the 0.2% offset method or at 0.5% elongation under load (EUL). Since ASTM A500 materials are cold-formed and have rounded stress-strain curves with no yield plateau, the latter method indicates yield strengths greater than the 0.2% offset. The ductility limit is intended to apply to yield strengths determined by 0.2% offset. However, mill reports may indicate the EUL yield, raising concerns that the material does not have adequate ductility. Supplemental tension tests may be required to determine the 0.2% offset yield.

Even though ASTM A501 includes rectangular HSS, hot-formed rectangular HSS are not currently produced in the United States. CAN/CSA G40.21 includes hot-formed Class H and cold-formed Class C. However, Class H rectangular HSS are produced by hot-finishing HSS that were manufactured by cold-forming. Hot-formed HSS have relatively low levels of residual stress, which enhances their performance in compression and may provide better ductility in the corners of rectangular HSS.

ASTM A500 tolerances allow for a wall thickness that is not greater than plus/minus 10 percent of the nominal value. Because the plate and strip from which electric-resistance-welded (ERW) HSS are made are produced to a much smaller thickness tolerance, manufacturers in the United States consistently produce ERW HSS with a wall thickness that is near the lower-bound wall thickness limit. Consequently, AISC and the Steel Tube Institute of North America (STI) recommend that 0.93 times the nominal wall thickness should be used for calculations involving engineering design properties of ERW HSS. This results in a mass variation that is similar to that found in other structural shapes. Submerged-arc-welded (SAW) HSS are produced with a wall thickness that is near the nominal thickness and require no such reduction. The design wall thickness and section properties based upon this thickness have been tabulated in AISC and STI publications since 1997.

1.3. Loads and Load Combinations

In many instances, the members in a framing system have no influence on the type or magnitude of the loads that must be considered in design. This is certainly true for dead loads, live loads, and impact loads. Horizontal crane forces, when they are present, and wind forces on enclosed structures are also not influenced by the type of members used in the framing system. Consequently, reference is made in this Specification to the applicable building code or ASCE 7. There are, however, two situations in which the use of HSS may allow a reduction in the design forces that must be considered: wind forces on exposed frameworks and pressures created by the enclosed nature of the HSS.

Wind forces on exposed frameworks can occur either in the final structural configuration or during construction. The shape of a round HSS has a lower resistance to fluid flow than shapes with flat elements (e.g., W shapes) and therefore reduces the wind forces. The general determination of wind pressures is given in the applicable building code or ASCE 7 when building codes do not apply. The determination of wind forces on exposed frameworks is a complex problem involving the solidarity ratio, shielding, and wind angle. In the absence of other wind-force reduction provisions that consider member shape, the provisions in this Specification can be used.

Wind forces on an exposed profile are proportional to a drag coefficient C, which varies with the type of profile. The value of C for a square shape with sharp corners is 2.03. Research (Hayus, 1968) indicates that the rounded corners of a square HSS reduce this drag coefficient. As shown in Figure C1.3-1, further reductions occur when rectangular HSS are oriented with the short side perpendicular to the wind. However,

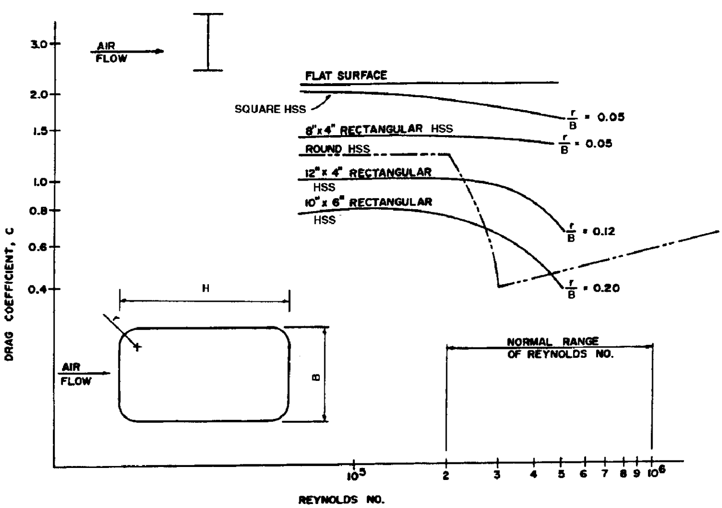

Fig. C1.3-1. Variation in drag coefficient.

TABLE C1.3-1
Drag Coefficients

Section	Corner Radius	C	$C/2.03$
Square HSS	$0.05B$	2.03	1.00
Round HSS	–	1.25	0.62
8-in. × 4-in. HSS	$0.05B$	1.4	0.69
12-in. × 4-in. HSS	$0.12B$	1.0	0.49
10-in. × 6-in. HSS	$0.20B$	0.8	0.39

C varies considerably with the orientation of the section relative to the wind and with the Reynolds number. The maximum values of C as indicated by the flat portions of the curves in Figure C1.3-1 provide conservative values for use in design. Table C1.3-1 lists these values along with their relative magnitude normalized by 2.03, which is the value of C for the square HSS. The Figure and Table also include the corresponding C for a round HSS. In this case, a one-third reduction (two-thirds factor) from the force on a flat surface is justified.

A similar reduction concept can be applied to wind acting on the short side of an HSS. Using the two data points for HSS with the sharpest corners (square and 8-in. × 4-in. rectangular HSS,) a linear reduction factor on the wind force is approximated by $0.4 + 0.6(B/H)$, in which the aspect ratio of the HSS is used as the variable. Conservatively, the reduction has been cut off at the one-third reduction for round HSS, even though the data indicates that with a larger corner radius, C may be less than that for a round HSS. There is no reduction for the wind force on the long side of a rectangular HSS.

2. DESIGN REQUIREMENTS

2.1. Effective Area of Tension Members

End connections for HSS in tension are commonly made by welding around the perimeter of the HSS. Alternatively, an end connection with gusset plates can be used. Single gusset plates are welded in longitudinal slots that are located at a centerline of the cross-section. Because welding around the end of the gusset plate is not recommended, the net area at the end of the slot in the HSS will be less than the gross area, as illustrated in Figure C2.1-1. A pair of gusset plates can be welded to opposite sides of a rectangular HSS with flare bevel groove welds.

For end connections of these three types, the general provisions of LRFD Specification Section B3 are simplified and the connection eccentricity $\bar{x}$ can be explicitly defined. These types of gusset-plate connections and the definitions of $\bar{x}$ and L are illustrated in Figure C2.1-2.

2.2. Local Buckling

The wall slenderness parameters and the slenderness limits λ_p and λ_r in Table 2.2-1 are taken from LRFD Specification Section B5, but have been presented in a non-dimensional form. The design wall thickness as defined in Section 1.2.2 is used to determine slenderness.

The limits for rectangular HSS walls in uniform compression have been used in AISC Specifications since 1969. They are based upon the work of Winter (1947) where adjacent stiffened compression elements in box sections of uniform thickness were observed to provide negligible torsional restraint for one another along their corner edges. The λ_p limit for plastic analysis is adopted from CSA (1994). The web slenderness limits are the same as those used for webs in wide-flange shapes.

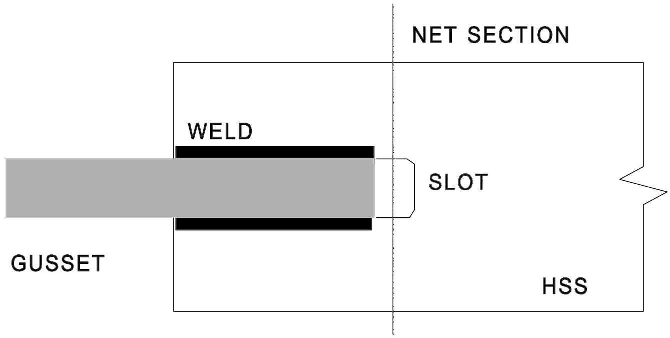

Fig. C2.1-1. Net area through slot for single gusset plate.

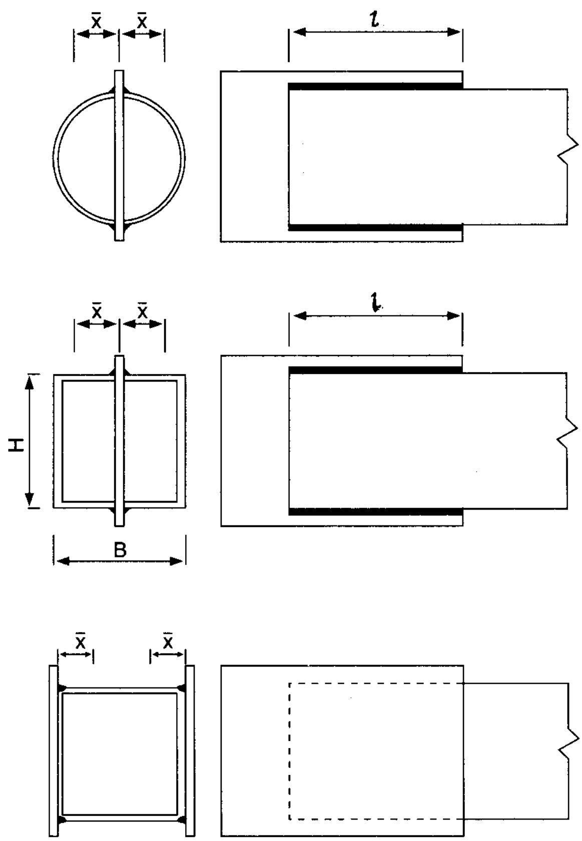

Fig. C2.1-2. Determination of $\bar{x}$.

The λ_r limit for round HSS in compression was first used by AISC in the 1978 AISC ASD Specification. It was recommended by Schilling (1965) based upon research at Cornell University that produced provisions in the 1968 AISI Cold-Formed Specification (Winter, 1968). The same limit was also used to define a compact shape in bending in the 1978 AISC ASD Specification. However, the limits for λ_p and λ_r were changed in the 1986 AISC LRFD Specification based upon experimental research on round HSS in bending (Sherman, 1985; Galambos, 1988). Excluding the use of round HSS with $D/t > 0.448E/F_y$ was also recommended by Schilling (1965).

Lower values of λ_r are specified for high-seismic design in the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 1997; AISC, 2000) based upon tests (Lui & Goel, 1987) that have shown that rectangular HSS braces subjected to reversed axial load fracture catastrophically under relatively few cycles if a local buckle forms. This was confirmed more recently in tests (Sherman, 1995) where rectangular HSS braces sustained over 500 cycles when a local buckle did not form, even though general column buckling had occurred, but failed in less than 40 cycles when a local buckle developed. The seismic λ_r is based upon tests (Lui & Goel, 1987) of HSS that had a small enough b/t so that braces performed satisfactorily for members with reasonable column slenderness. Filling the rectangular HSS with lean concrete (concrete mixed with a low proportion of cement) has been shown to effectively stiffen the HSS walls and improve cyclic performance. Because fracture at a low number of cycles is also possible with cold-formed round HSS braces that form local buckles, the limiting λ_r in compression in the 1992 AISC *Seismic Provisions* was chosen to be the same as for plastic analysis.

3. TENSION MEMBERS

3.1. Design Tensile Strength

Except for HSS that are subjected to cyclic load reversals, there is no information that the factors governing the strength of HSS in tension differ from those for other structural shapes. Therefore, the criteria in Section 3.1 are identical to those in LRFD Specification Section D1. However, because the number of different end connection types that are practical for HSS is limited, the determination of the net effective area A_e can be simplified as it has been in Section 2.1.

4. COLUMNS AND OTHER COMPRESSION MEMBERS

4.1. Effective Length and Slenderness Limitations

The high torsional strength and stiffness of an HSS provides increased restraint for members that frame into it when compared to that provided by other structural shapes. For example, the connection between an HSS branch member and a continuous HSS chord member is commonly made with a continuous weld around the perimeter of the branch member. In such a connection, the chord then provides considerable end restraint both in-plane and out-of-plane of the truss; the HSS branch member also provides a degree of lateral restraint against rotation of the chord. In both cases, advantage can be taken of the end restraint by using the effective lengths in Section 4.1.1(a). The use of K equal to 0.75 for the branch members and 0.9 for the chord between bracing points is based upon the recommendations of CIDECT research (Rondal, 1992).

It is important to note that even though end restraint is present, it is still reasonable to assume that the truss joints are pinned. Secondary moments due to end fixity may be neglected unless the joint eccentricity exceeds the limits of applicability in Section 9.4 or fatigue is a design consideration. For fatigue applications, refer to AWS D1.1.

The provisions for unbraced frames and braced frames are taken from the LRFD Specification. Values of K for compression members in frames can be determined from the alignment charts and equations in the corresponding LRFD Specification Commentary.

4.2. Design Compressive Strength

The axial compressive strength of an HSS is influenced by its method of production, shape, and dimensions and is further complicated by large differences between theoretical predictions and experimental results for local buckling, especially for round HSS. Rather than repeat the excellent discussions that can be found elsewhere concerning the behavior of various hollow cross-sections (Schilling, 1965; McGuire, 1968; Galambos, 1988; and Sherman, 1992), the results and basis for the design equations is explained herein. Some of the major considerations that must be included in comprehensive criteria for HSS are as follows:

1. As with any thin-walled member of constant cross-section, either overall flexural buckling or local buckling can be the controlling limit state. Flexural buckling strength under axial load is governed by the slenderness ratio KL/r, the yield strength F_y, residual stresses, and initial out-of-straightness. Local buckling of rectangular HSS is based upon the principles of plate buckling theory and is governed by the square of the width-thickness ratio. In very short round sections, local buckling is similar to that for an infinitely wide plate and the ratio of the length to the thickness is of prime importance. For longer round sections, the local buckling configuration consists of approximately square waves along the length and around the circumference and the strength is a function of the ratio of the diameter to the thickness. The local buckling strength of thin round sections is extremely sensitive to initial distortion from the perfect cylindrical surface. Because manufactured HSS generally have less initial distortion than a comparable fabricated cylinder, local buckling generally occurs at a higher load.

2. The strength of short compression members is governed by local buckling, whereas the strength of long compression members is governed by flexural buckling. In compression members of intermediate length, there is an interaction between local and flexural buckling.

3. Rectangular HSS that are cold-formed from round HSS have a rounded stress-strain curve due to through-thickness residual stresses that vary from about $0.4F_y$ to $0.8F_y$. This reduces the flexural buckling strength of intermediate-length members to below that of comparable hot-formed HSS with similar yield strength. Residual stresses are not as large in cold-formed round HSS and are negligible in hot-formed HSS.

4. Manufactured HSS tend to have small initial out-of-straightness, which increases the strength of intermediate-length HSS relative to other shapes.

Because many HSS have a wall slenderness ratio that exceeds λ_r, the equations for flexural buckling include the interaction with local buckling as given in LRFD Specification Appendix B. In effect, the local buckling reduction factor Q reduces the yield stress in both the inelastic flexural buckling equation and in the slenderness λ_c. Its influence is illustrated in Figure C4.2-1. Of course, if λ is less than or equal to λ_r, the local buckling reduction factor Q is equal to unity and the equations reduce to the flexural buckling equations of LRFD Specification Chapter E.

For round HSS, Q has its origins in the critical stress from local buckling test data with conservative adjustments in an early AISI Specification (Winter, 1968). The constants have been further adjusted for a non-dimensional form and a design strength format rather than an allowable stress format.

The effective width equation that is used to obtain Q for rectangular HSS is also from an early AISI Specification (Winter, 1968). The constants in this equation were established for closed cross-sections with uniform thickness. The flexural buckling equations and ϕ_c equal to 0.85 are the same as those used in the LRFD Specification and can be used conservatively for all HSS. However, for hot-formed HSS or ASTM A500 Grade D HSS, which have lower residual stresses but are generally unavailable in the United States, a higher resistance factor could be justified and would give factored resistances that are comparable to those in other specifications that use multiple column curves. This is based upon the extensive CIDECT column test program in Europe and several less extensive studies in North America, as summarized in Table C4.2-1. A large amount of CIDECT data for seamless HSS is not included. The test data for HSS in axial compression are illustrated in Figure C4.2-2 for round HSS and Figure C4.2-3 for rectangular HSS. The curve for the LRFD nominal column strength ($\phi = 1.0$) is superimposed on the Figures. The yield load used to non-dimensionalize the ordinates is the cross-sectional area times the tensile-test yield strength, which was determined by

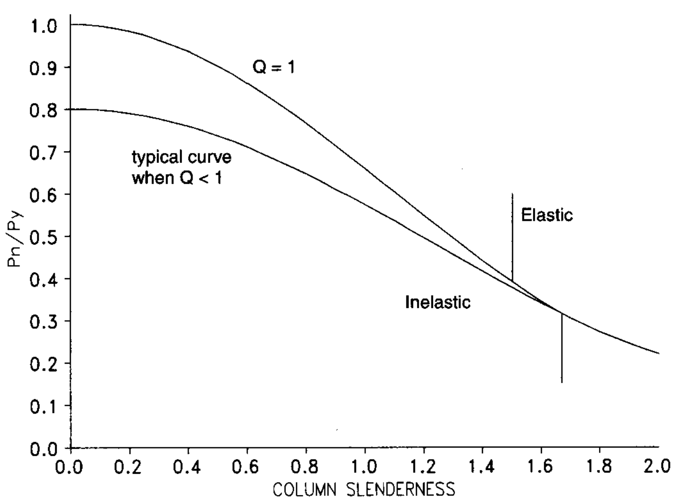

Fig. C4.2-1. Influence of Q on column strength.

Table C4.2-1
Summary of Test Programs on HSS Columns

Type	Symbol in Figure C4.2-2	Reference	No. of Tests
Round HSS			
Hot-Formed	□	CIDECT #	10
Cold-Formed	■	CIDECT #	65
		Sherman, 1980	4
Fabricated Pipe	✱	Chen, 1977	10
		Yang, 1987	6
Rectangular HSS			
Type	**Symbol in Figure C4.2-2**	**Reference**	**No. of Tests**
Hot-Formed	□	CIDECT #	88
		Estuar, 1965	10
Cold-Formed	■	CIDECT #	132
		Bjorhovde, 1979	1
		Key, 1985	11
Cold-Formed Stress Relieved	✱	Bjorhovde, 1979	19
		Sherman, 1969	2

No reference cited for CIDECT data.

the 0.2 percent offset method for the cold-formed HSS that exhibit rounded stress-strain curves. In the figures, solid squares represent data from tests on cold-formed HSS while open squares represent those on hot-formed HSS. Asterisks represent special cases of fabricated pipe in Figure C4.2-2 and stress-relieved rectangular HSS in Figure C4.2-3. Note that cold-formed ASTM A500 rectangular HSS that are stress relieved to 840°F (450°C) have strengths that are similar to those for hot-formed HSS.

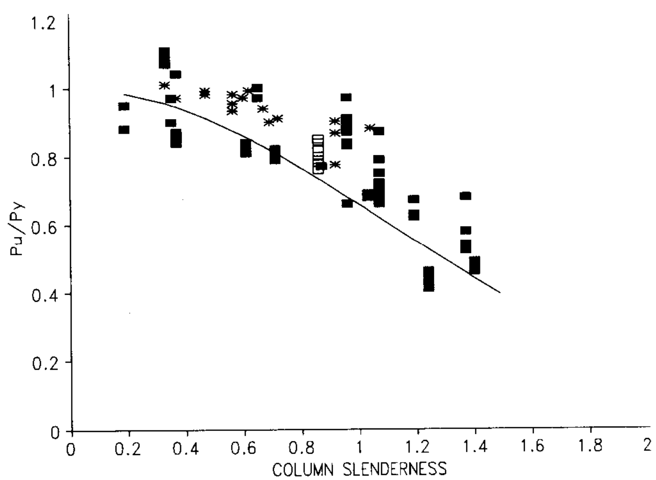

Fig. C4.2-2. Test Data for Round HSS.

5. BEAMS AND OTHER FLEXURAL MEMBERS

5.1. Design Flexural Strength

The provisions for the nominal flexural strength of HSS include the limit states of yielding, inelastic local buckling, and elastic local buckling. Round HSS, square HSS, and rectangular HSS bent about the minor axis are not subject to lateral-torsional buckling. Furthermore, for rectangular HSS bent about the major axis, the limit state of lateral-torsional buckling is not included in this Specification in spite of LRFD Specification provisions that reduce the flexural strength when the unbraced length exceeds limiting unbraced length values.

Because of the high torsional resistance of the closed cross-section, the critical unbraced lengths L_p and L_r that correspond to the development of the plastic moment and the yield moment, respectively, are very large. For example, as shown in Figure C5.1-1, an HSS $20\times4\times{}^5/_{16}$, which has one of the largest depth-width ratios among standard HSS, has L_p of 8.7 ft and L_r of 137 ft as determined in accordance with the LRFD Specification. An extreme deflection limit might correspond to a length-to-depth ratio of 24 or a length of 40 ft for this member. Using the specified linear reduction between the plastic moment and the yield moment for lateral-torsional buckling, the plastic moment is reduced by only 7 percent for the 40-ft length. In most practical designs where the moment gradient C_b is also a factor, the reduction will be nonexistent or insignificant.

The failure modes and post-buckling behavior of round HSS can be grouped into three categories (Galambos, 1998; Sherman, 1992).

(a) For low D/t, a long plastic plateau occurs in the moment-rotation curve. The cross-section gradually ovalizes, local wave buckles eventually form, and the moment

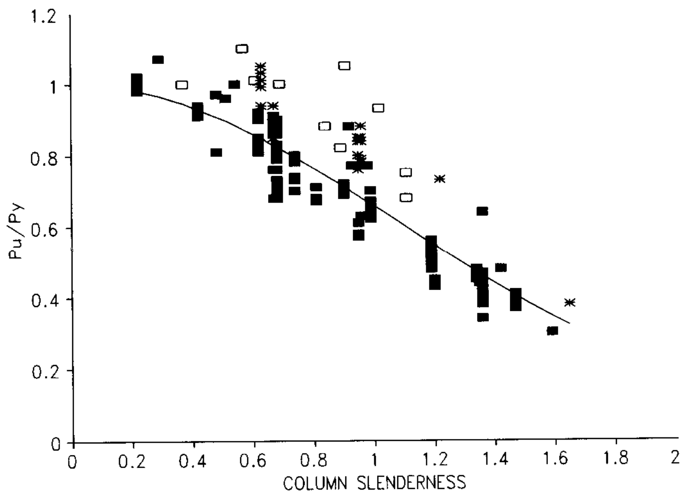

Fig. C4.2-3. Test Data for Rectangular HSS.

resistance subsequently decays slowly. Flexural strength may exceed the theoretical plastic moment due to strain hardening.

(b) For intermediate D/t, the plastic moment is nearly achieved but a single local buckle develops and the moment resistance decays slowly with little or no plastic plateau region.

(c) For high D/t HSS, multiple buckles form suddenly with very little ovalization and the flexural strength drops quickly.

The flexural strength criteria for round HSS reflect these three regions of behavior and are based upon five experimental programs involving hot-formed seamless pipe, electric-resistance-welded pipe, and fabricated tubing (Galambos, 1998). The criteria are the same as those in LRFD Specification Appendix F.

The criteria for local buckling of rectangular HSS are also the same as those in LRFD Specification Appendix F. The equation for the effective width of the compression flange when b/t exceeds λ_r is the same as that used for rectangular HSS in axial compression except that the stress is taken as the yield stress. This implies that the stress in the corners of the compression flange is at yield when the ultimate post-buckling strength of the flange is reached. When using the effective width, the nominal flexural strength is determined from the effective section modulus to the compression flange using the distance from the shifted neutral axis. A slightly conservative estimate of the nominal flexural strength can be obtained by using the effective width for both the compression and tension flange, thereby maintaining the symmetry of the cross-section and simplifying the calculation.

The shape factor (Z/S) for HSS is generally between 1.15 and 1.4. Hence, the maximum limit on M_p of $1.5M_y$ in LRFD Specification Chapter F is satisfied for HSS and need not be explicitly checked.

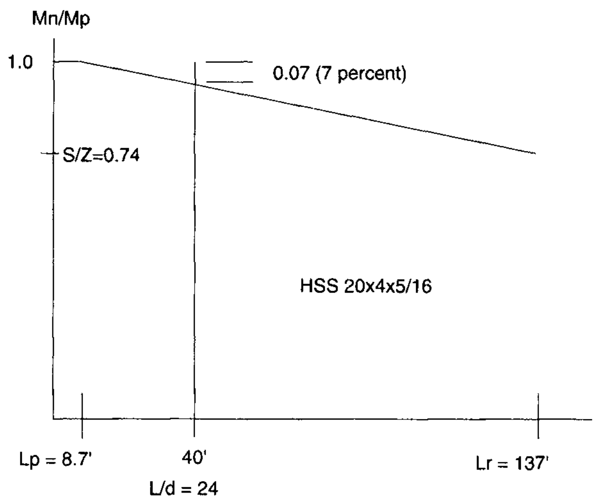

Fig. C5.1-1. Lateral-torsional buckling of rectangular HSS.

In order to use plastic analysis for design, the HSS must have sufficient rotational capacity at the plastic moment to develop the hinge mechanism. This requires a compact cross-section to prevent premature local buckling and lateral bracing for rectangular HSS that are bent about the major axis to prevent lateral-torsional buckling in the vicinity of hinges. The requirement for L_{pd} is taken from the LRFD Specification. The provisions for the wall slenderness defining the compact section are more restrictive than λ_p, which defines when M_p can be achieved without consideration of rotational capacity. This is due to the higher shape factors for HSS relative to wide-flange shapes where λ_p also applies to plastic analysis. The more restrictive D/t for round HSS is also used in the LRFD Specification, and the b/t limit for rectangular HSS is based upon test results for cold-formed sections (Korol, 1972), which are most frequently used in North America.

5.2. Design Shear Strength

Little information is available on round HSS subjected to transverse shear, and recommendations are based upon criteria for local buckling of cylinders subjected to torsion.

However, since torsion is generally constant along the member length and transverse shear usually has a gradient, it is recommended to take the critical stress for transverse shear as 1.3 times the critical stress for torsion (Galambos, 1998; Brockenbrough and Johnston, 1981). The torsion equations apply over the full length of the member, but for transverse shear it is reasonable to use a length to the location of zero shear. Only thin HSS may require a reduction in the shear strength based upon first shear yield. Even in this case, shear will only govern the design of round HSS beams for the case of thin sections with short spans.

In the equation for the nominal shear strength V_n of round HSS, it is assumed that the shear stress is at the neutral axis, which is calculated as VQ/Ib, is at F_{cr}. For a thin round section with radius R and thickness t, I is $\pi R^3 t$, the first moment of area Q is $2R^2 t$, and $b = 2t$. This gives the stress at the centroid as $V/(\pi Rt)$, in which the denominator is recognized as half the area of the round HSS.

The provisions for the nominal shear strength of rectangular HSS are the same as those for unstiffened webs in the LRFD Specification. For rectangular HSS, the nominal shear strength considers the two webs in the section.

6. TORSION MEMBERS

HSS are frequently used in space-frame construction and in other situations wherein significant torsional moments must be resisted by the members. Because of its closed cross-section, an HSS is far more efficient in resisting torsion than an open cross-section such as a W-shape or channel. While normal and shear stresses due to warping of the cross-section are usually significant in shapes of open cross-section, they are insignificant for closed cross-sections, and the total torsional moment can be assumed to be resisted by pure torsional shear stresses, which are sometimes called St. Venant torsional stresses.

In HSS, the pure torsional shear stress is assumed to be uniformly distributed and is equal to the torsional moment T_u divided by a torsional shear constant for the cross-section C. In a limit state format, the nominal torsional resisting moment is the shear constant times the critical shear stress F_{cr}.

For a round HSS, the torsional shear constant is equal to the polar moment of inertia divided by the radius:

$$C = \frac{\pi\left(D^4 - D_i^4\right)}{32D/2} = \frac{\pi(D-t)^2 t}{2} \tag{C6-1}$$

where D_i is the inside diameter.

For a rectangular HSS, the torsional shear constant is obtained as $2tA_o$ using the membrane analogy (Timoshenko, 1956), where A_o is the area bounded by the midline of the section. Conservatively assuming an outside corner radius that is equal to $2t$, the midline corner radius is $1.5t$, and

$$A_o = (B-t)(H-t)\frac{9(4-\pi)}{4}t^2 \tag{C6-2}$$

which yields

$$C = 2(B-t)(H-t)t - 4.5(4-\pi)t^3 \tag{C6-3}$$

The resistance factor and nominal strengths used for torsion are the same as those used for flexural shear.

When considering local buckling in round HSS subjected to torsion, most structural members will either be long or of moderate length and the criteria for short cylinders will not apply. The elastic local buckling strength of long cylinders is unaffected by end conditions, and the critical stress is as given by Galambos (1998) as:

$$F_{cr} = \frac{K_t E}{(D/t)^{3/2}} \tag{C6-4}$$

The theoretical value of K_t is 0.73, but a value of 0.6 is recommended to account for initial imperfections. An equation for the critical elastic local stress for round HSS of moderate length ($L > 5.1D^2/t$) where the edges at the ends are not fixed against rotation is given by Schilling (1965) and Galambos (1988) as:

$$F_{cr} = \frac{1.23E}{\sqrt{L/D}(D/t)^{5/4}} \tag{C6-5}$$

This equation includes a 15 percent reduction to account for initial imperfections.

The local buckling equations are plotted in Figure C6-1 with modulus of elasticity E equal to 29,000 ksi. Although there is some inconsistency concerning the division between long and moderately long HSS, it appears from Figure C6-1 that the expression for sections of moderate length is valid for most practical lengths. It is also evident that it would be uneconomical to neglect the increase in torsional strength for members of moderate length. Therefore, in this Specification, the length effect is included for the simple end conditions and the approximately 10 percent increase in buckling strength is neglected for edges fixed at the ends. A limitation is provided so that the shear yield strength is not exceeded.

The critical stress criteria for rectangular HSS are identical to the flexural shear criteria of Section 5.2. The shear distribution due to torsion is uniform in the longest sides of a rectangular HSS, which is the same distribution that is assumed to exist in a beam web. Therefore, it is reasonable that the criteria for buckling is the same in both cases.

7. MEMBERS UNDER COMBINED FORCES

7.1. Design for Combined Flexure and Axial Force

The provisions for interaction between flexure and axial force are taken directly from LRFD Specification Section H1. As stated in LRFD Specification Section C1, for structures designed on the basis of elastic analysis, the required flexural strength M_u shall be determined from a second-order analysis. Second-order effects must be considered not only for the beam-column member, but also for the other framing members that connect to it and the associated connections. In lieu of a second-order analysis, an approximate second-order procedure is permitted to determine M_u as detailed in LRFD Specification Section C1.

In the case of biaxial flexure on round HSS, which have the same section modulus about any axis, the LRFD Specification interaction equations lead to inconsistent results when compared with design for a resultant moment (Pillai and Ellis, 1971). The interaction equation adds the effects of M_{ux} and M_{uy} which can be conservative by as much as 40 percent when compared to combining them vectorially. Therefore, provision has been included in this Specification for the direct design based upon the resultant moment M_{ur}.

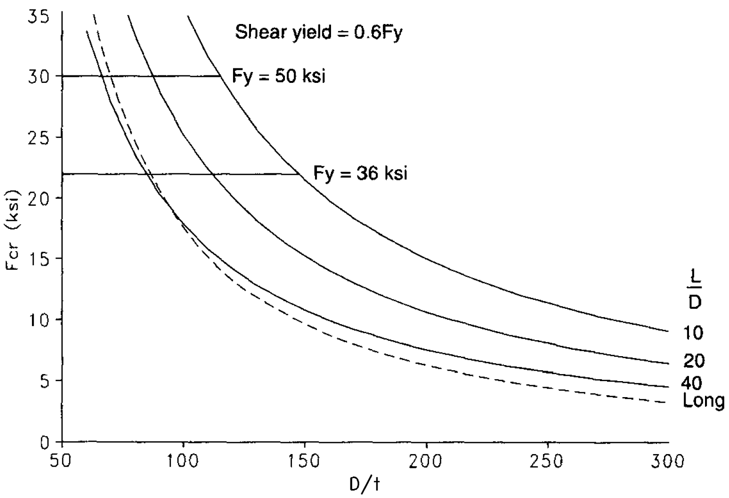

Fig. C6-1. Torsion F_{cr} for round HSS.

7.2. Design for Combined Torsion, Shear, Flexure, and/or Axial Force

Several interaction equation forms have been proposed for load combinations that produce both normal and shear stresses. In one common form, the normal and shear stresses are combined elliptically with the sum of the squares (Felton and Dobbs, 1967).

$$\left(\frac{f}{F_{cr}}\right)^2 + \left(\frac{f_v}{F_{vcr}}\right)^2 \leq 1 \qquad \text{(C7.2-1)}$$

In a second form, the first power of the normal stresses is used.

$$\left(\frac{f}{F_{cr}}\right) + \left(\frac{f_v}{F_{vcr}}\right)^2 \leq 1 \qquad \text{(C7.2-2)}$$

These equations are plotted in Figure C7.2-1. The latter form is more conservative but not overly conservative (Schilling, 1965) and is the basis for the interaction equation used in this Specification in a limit-states format. Flexure and axial force effects are combined linearly and then combined with the square of a linear combination of flexural shear and torsion effects. When an axial load is present, the required flexural strength M_u is obtained by second-order analysis. Because the interaction is based upon an elastic combination of stresses, the nominal flexural strength M_n is limited to the yield moment.

8. CONCENTRATED FORCES ON HSS

Concentrated forces result from line loads that are applied to the HSS through a plate, a connecting element that has a flange, or a similar element. The line load can be distributed either transversely or longitudinally as shown in Figure C8.1-1.

Round HSS with either transverse or longitudinal line loads fail by local plastic distortion of the section. The resistances for these loadings are given by Packer and Henderson (1997). Because the resistance equations given by Packer and Henderson

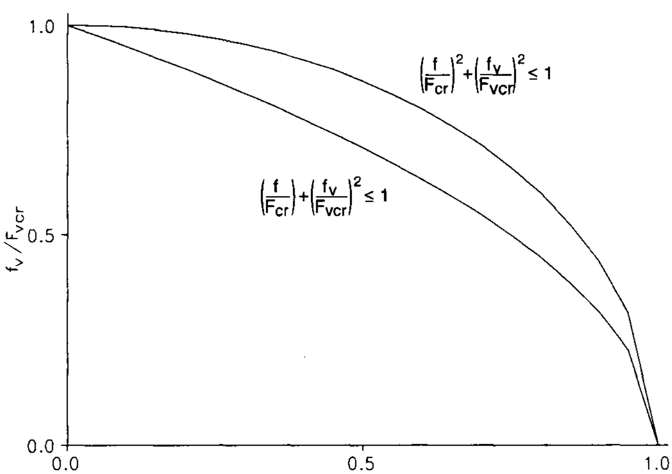

Fig. C7.2-1. Stress interaction of shear and normal effects.

account for variations between experimental strengths and theory, the appropriate resistance factor ϕ is 1.0. When there are compressive normal stresses in the HSS in the vicinity of the line load, the resistance to local plastic distortion is reduced and a reduction factor Q_f is applied. In Packer and Henderson, Q_f is determined as a function of f, which is the maximum compressive stress in the chord (a negative number). In this Specification, the Packer and Henderson equation for Q_f has been modified so that the magnitude of f is used (an absolute value).

The limit state for a rectangular HSS depends upon the width of the load relative to the width of the loaded face of the HSS. For transverse loads, the variable stiffness across the face of the HSS causes a non-uniform distribution of the force in the connecting element delivering the load. Failure may occur due to an excessive concentration of force on an effective width of the connecting element at the resistance given by

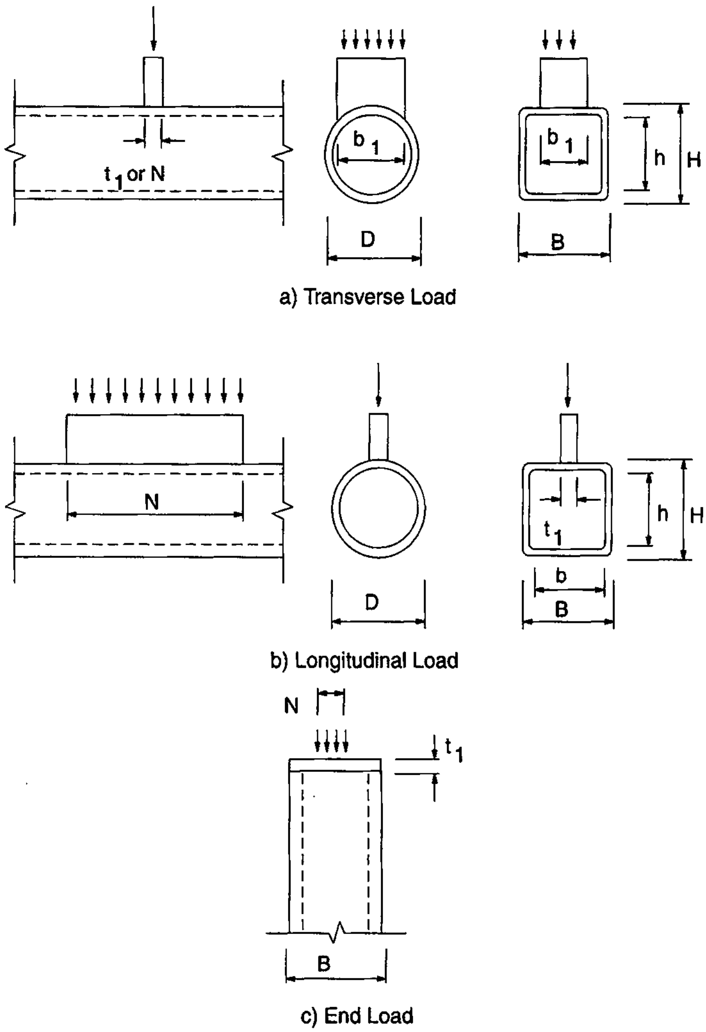

Fig. C8.1-1. Concentrated force configurations on HSS.

Packer and Henderson (1997). The limit of $F_{y1}t_1b_1$ does not represent the limit state for the axial strength of the loaded plate, but limits the effective width given by Packer and Henderson (1997) from exceeding the width of the HSS. When the force is across nearly the full width of the HSS, the concentration of force at the end of the connecting element can cause a punching shear rupture through the wall of the HSS. Again, the resistance is as given by Packer and Henderson (1997).

When the force is across the full width, failure of the sidewalls of the HSS is possible. The resistance for local sidewall yielding is taken from LRFD Specification Section K1.3, adjusted for the presence of two sidewalls acting as webs. The resistance for sidewall crippling is consistent with LRFD Specification Section K1.4 in a non-dimensional form and accounting for two sidewalls. The same is true for the compression buckling criteria from LRFD Specification Section K1.6 when compressive forces are applied on opposite faces of the HSS. For the limit states taken from the LRFD Specification, the identical resistance factors are used. When concentrated reactions are present in HSS members such as lintels, a stiffening end plate will eliminate concerns of failure of the sidewalls. In the absence of an end plate, the criteria for end forces in LRFD Sections K1.3, K1.4, and K1.6 adjusted for two webs should be used.

In the case of a longitudinal line load where the width is very narrow relative to the width of the HSS, the limit state is local plastic distortion of the face of the HSS and the resistance is as given by Packer and Henderson (1997). As in the case of round HSS where the limit state is also local plastic distortion, a reduction factor Q_f is applied when the HSS is in compression. Because the resistance equations given by Packer and Henderson include variability, the resistance factor ϕ is 1.0.

Another type of concentrated force on a rectangular HSS is on the end of a capped column supporting a beam or joist as in Figure C8.1-1c. Because the load is on only one side of the HSS, the resistance for local yielding is taken directly from LRFD K1.3. The criteria for crippling of the HSS wall is taken from LRFD Specification Equation K1-4 with the depth taken as one-half the full width of the HSS, $B/2$. This is based upon a model in which the force is assumed to spread out at 45 degrees in both directions from the center point of the load width. The force will be across the full width of the side at a depth no greater than $B/2$, below which crippling will not occur.

9. CONNECTIONS AND FASTENERS

9.1. General Provisions

Because HSS are frequently combined with other shapes in structures using standard simple shear or moment connections, the general provisions of LRFD Specification Sections J1.1 through J1.11 apply even though several of these provisions are not directly applicable to the HSS.

Although welding is the most frequently used method for making attachments to HSS, there are a variety of methods for using bolts or mechanical fasteners. When ASTM A325, A490, or A307 bolts are used to make an attachment to one wall of an HSS, using an open end or access hole to install the nut so that all plies of the connection are in contact within the grip of the bolt, the provision of LRFD Specification Section J3 apply. These provisions also apply to other portions of a structure that includes HSS. However, if a bolt passes completely through the HSS, an attempt to fully tension the bolt would distort the cross-section of the HSS wall and the specified minimum installed tension could not be achieved without damaging the member. Therefore, through-bolts should only be installed in the snug-tight condition. The bearing strength of such connections as given in Section 9.1.1 is consistent with that in LRFD Specification Section J8. The designer should note that some connection details do not distribute the load to both walls equally. Additionally, some details may induce bending in the through-bolt.

There are a variety of blind bolts, unique installation methods, welded studs, and special connectors that could be used with HSS. Many of these alternatives are still in a developmental stage and general criteria for their use are not available. In such cases, test data must be used to justify their suitability for a particular application.

With any connection configuration where the fasteners transmit a tensile force to the HSS wall, a rational analysis must be used to determine the appropriate limit states. These may include a yield-line mechanism in the HSS wall and/or pull-out through the HSS wall in addition to applicable limit states for the fasteners subject to tension.

9.2. Welds

These provisions are based upon similar provisions in AWS D1.1. The design strength of a connection, for a member welded to a wall of an HSS, is a function of the geometric parameters of the connected members. It is often less than the strength of the member and in many cases cannot be increased by increasing the weld strength. The weld, however, must be sized to provide for the uneven distribution of load along the weld line at the required strength.

The effective length provisions included in this Section are intended to provide for ductile joint behavior through prevention of progressive failure or “unzipping” of the weld. The variables are as illustrated for the Y-connection in Figure C9.4-1.

Other rational approaches are available in AWS D1.1, such as the use of fillet welds with an effective throat thickness of at least 1.1 times the thickness of the branch member or the use of prequalified fillet and partial-joint-penetration groove welded details that are sized to ensure ductile behavior.

9.3. Other Connection Requirements

The requirements of LRFD Specification Section J4 through J10 are applicable to connections to HSS as well as to other portions of a structure. The notation for shear rupture strength and tension rupture strength in LRFD Specification Section J4 has been modified when applied specifically to the HSS. An additional limit state for punching shear has been added based upon failures that were observed in tests of single plate connections to HSS (Sherman, 1996).

The tension force in a unit length of the plate is ft_p. Limiting this force to the through-thickness shear strength of the HSS for the two welded planes on the sides of the plate,

$$ft_p \leq 2(0.6F_u)t \tag{C9.3-1}$$

The punching shear rupture criterion includes the standard resistance factors of 0.9 for yielding and 0.75 for rupture. For a single plate connection, the stress f is a bending stress that results from the factored shear acting at an eccentricity (Sherman and Ales, 1991). A simple and conservative criterion is obtained by setting f equal to the yield strength of the plate F_{yp}, which is the maximum possible value. This reduces the above equation (with resistance factors) to

$$t_p < \frac{F_u}{F_{yp}}t \tag{C9.3-2}$$

An exception to LRFD Specification Section J5.1 on eccentric connections is that bending moments caused by eccentricities in HSS-to-HSS truss connections need not be considered for connection design as long as the eccentricities are within the limits of applicability in Section 9.4. These eccentricities have been included in the database upon which the connection strength criteria in Section 9.4 have been based.

9.4. HSS-to-HSS Truss Connections

A wide variety of connection configurations is possible where branch members are directly welded to a chord that is continuous through the connection. The criteria in this specification are limited to a few of the configurations that are typically used for planar HSS trusses in building applications: T-, Y-, gapped K-, and Cross-connections, as illustrated in Figure C9.4-1.

Overlapped K-connections are not covered in this Specification because they are more expensive to fabricate than connections with a gap, especially for round HSS. For this and other connection configurations that are not included in Figure C9.4-1, the design must be based upon the requirements in AWS D1.1, experimental test results, or other verified design procedures. Design procedures such as those contained in the recommendations of the International Institute of Welding (IIW, 1989) would generally be acceptable.

The criterion that is used for the design strength of a branch of a gapped K- connection is based upon the mechanism of load transfer rather than the configuration of the connection. The gapped-K-connection criteria apply only when the branch axial force is equilibrated by the forces in other branch members on the same side of the chord. If the force is equilibrated by shear in the chord, the criteria for a T- or Y-connection apply. If the load is transferred through the chord, the criteria for a Cross-connection must be used. Examples of these principles are shown in Figure C9.4-2. When the load is equilibrated by a combination of two of the mechanisms, the design strength is determined by interpolation between the two criteria based upon the percentage of the load transfer by each mechanism.

The symbols for key dimensions that are used in HSS-to-HSS truss connection design criteria are also defined in Figure C9.4-1. Parameters derived from ratios of these dimensions that are used in AWS D1.1 and other specifications are defined in Section 9.4.1.

The criteria in Section 9.4.2 for round HSS and Section 9.4.3 for rectangular HSS have been taken from AWS D1.1. The parameter Q_f, which is a strength reduction factor that accounts for the level of axial and bending stress in the chord at the connection, is defined in both sections. The criteria are partly empirical and are based upon numerous worldwide test programs involving hundreds of tests. Since the criteria are in some cases the result of empirical curve fitting, they may not be accurate outside the range of tested parameters. AWS D1.1 does not contain a set of limits of applicability for round HSS except for an upper limit on yield strength of 60 ksi. The HSS material specifications listed in Section 3.1 meet this limitation. However, since the database used by various international specifications and design recommendations is essentially the same, the limits of applicability given by Packer and Henderson (1997) for round HSS have been used in Section 9.4.2a.

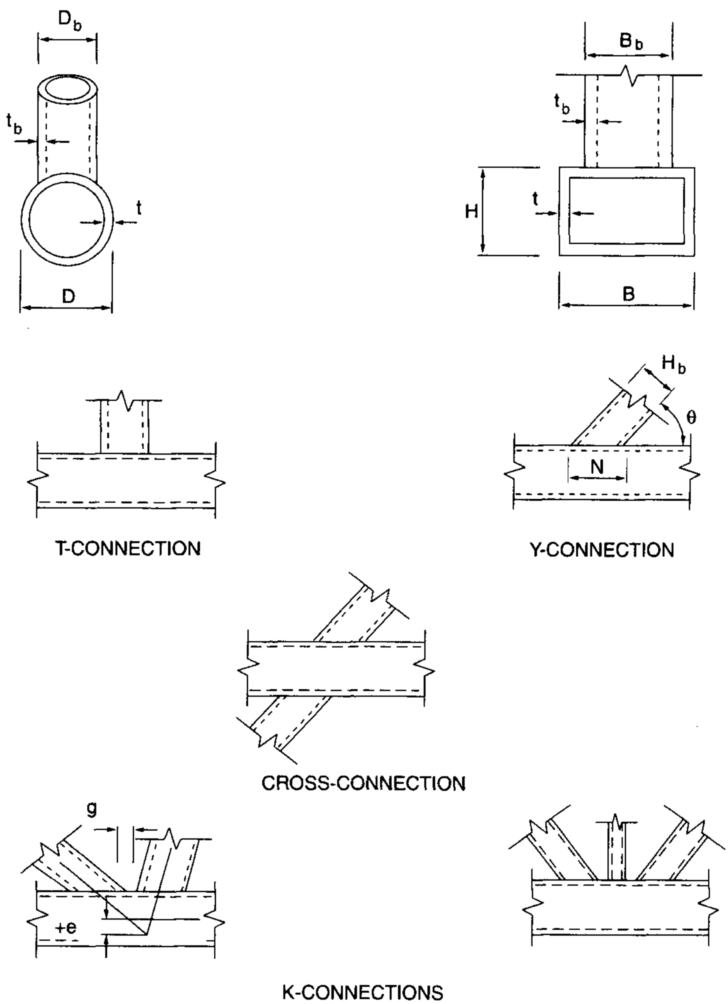

Fig. C9.4-1. HSS-to-HSS truss connection configurations.

The criteria in Section 9.4.3 for rectangular HSS are similar to those in other established HSS design specifications and recommendations throughout the world. They include the limits of applicability for rectangular HSS connections in Section 9.4.3a. One other limit has been specified for T-, Y-, and Cross-connections in Section 9.4.3b and several more for K-connections in Section 9.4.3c. In addition to the lower limit for the gap, the gap must have sufficient size to accommodate the welds to both branch members in the gap region. If the upper limit for the gap size is exceeded, as controlled by the limit on eccentricity, the branches should be considered separately as T- or Y-connections.

The criteria place limits on the magnitude of the component of the branch force that is perpendicular to the chord for various limit states. These limit states include:

(1) Chord wall plastification: High local stresses and distortions occur in the vicinity of the joint. These are the result of bending in the wall of the chord.
(2) Punching shear rupture: For chords with thin walls, shear stresses through the thickness of the chord around the perimeter of the branch result in a tearing out of material, which can be the controlling limit state.

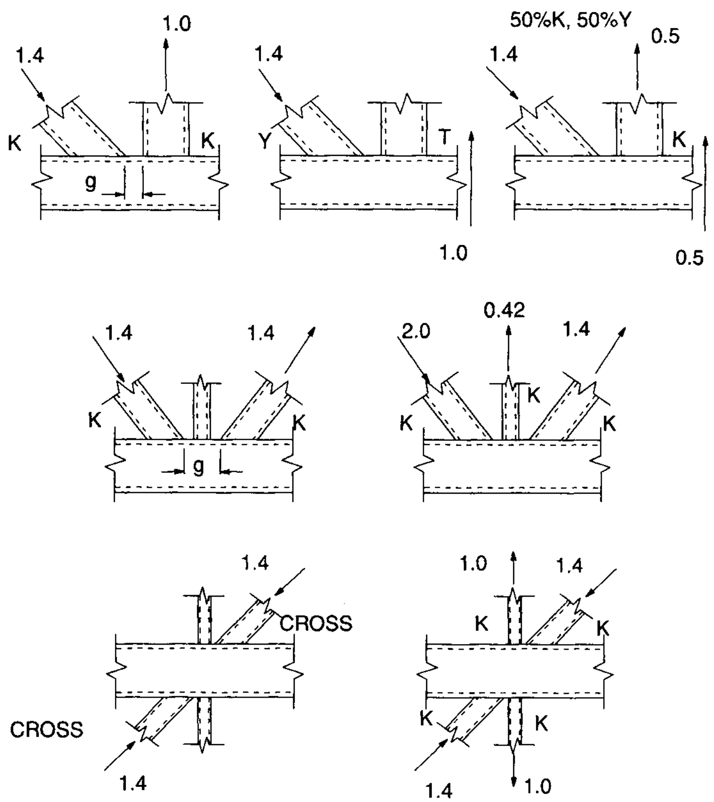

Fig. C9.4-2. Examples of connection criteria classifications (all inclined members at 45 degrees).

(3) General collapse: This is most common in Cross-connections where the branches are in compression or in other situations when the load is transferred through the chord. It results in a squashing of a round HSS chord or a buckling of the side wall of a rectangular HSS chord. The sidewall strength criteria in Section 9.4.3b, are based upon similar provisions in LRFD Specification Sections K1.3, K1.4, and K1.6. They have been modified to account for the presence of two sidewalls in the HSS and the buckling criteria have been expressed in a non-dimensional form.

(4) Uneven load distribution: In connections where the branch is equal or nearly equal in width to the chord, the increase in the stiffness of the chord wall from its center toward its sides concentrates the transfer of the branch force toward the sides of the chord and might cause a premature failure of the branch.

These limit states are illustrated in Figure C9.4-3. In some situations, it can be determined beforehand that a particular limit state will not apply. For example, chord wall plastification will not occur in rectangular HSS T-connections when the branch has the same or nearly the same width as the chord. Hence, ranges of parameters are indicated in Section 9.4.3b and Section 9.4.3c when a limit state need not be checked. In Section

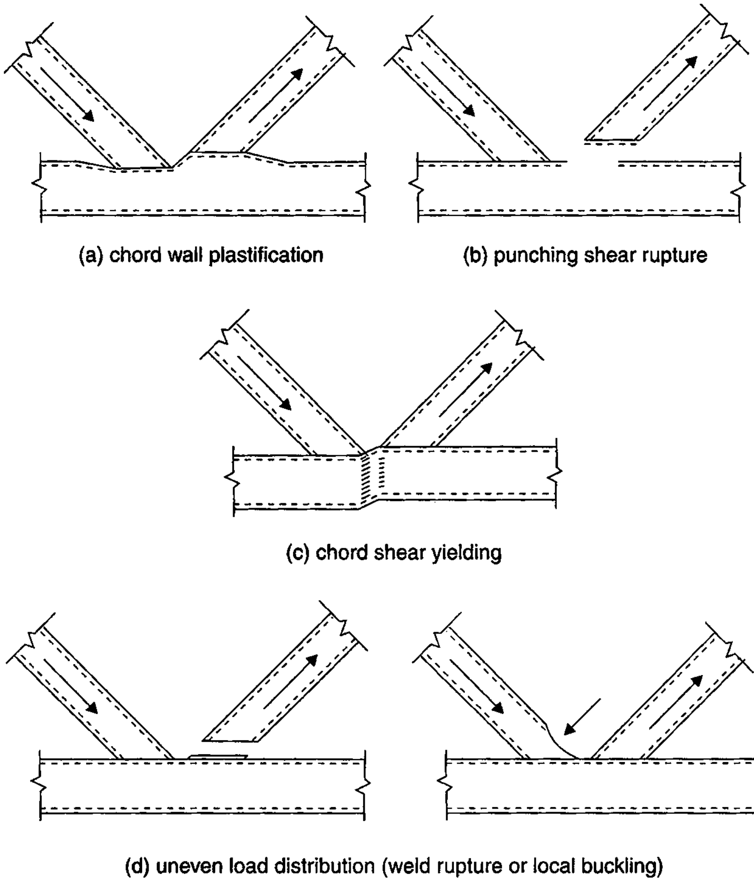

Fig. C9.4-3. Limit states in gapped K-connections.

9.4.3c, one such limit is given as $B_b/B \geq 0.1 + \gamma/50$, which is based upon Packer and Henderson (1997). This is a departure from AWS D1.1, which specifies a limit of $\beta_{eff} \geq 0.1 + \gamma/50$.

Since the criteria place a limit on the force that the branch can carry, it is not always possible to develop the full strength of the branch without reinforcing the chord with stiffeners. It should be noted that such reinforcement involves expensive fabrication. To minimize cost, it may be more desirable to maximize the efficiency of the connection with respect to the force that can be developed in the branch. Consider the following suggestions:

(1) Chord members should be relatively thick and branch members should be relatively thin. This is efficient for the strength of the connection, and thinner branches reduce the required weld size, if the branch member is joined to develop its wall strength. There may be some compromise required in the design of compression chord members because larger sizes with thinner walls make efficient compression members. This, however, may be detrimental to the strength of the connection.
(2) All members should have D/t or b/t values below the limit that would classify them as thin-walled sections. Tension-only branches are a possible exception.
(3) The angle between the branch member and chord member should be greater than 30 degrees. This is efficient for the truss layout and avoids difficult welding and inspection requirements and uncertainties in design criteria as well.
(4) Branch member width should be kept narrower than the chord member width to avoid difficult welding details and inspection at the edges.
(5) Gapped connections are preferred to overlapped connections. Although the latter are more efficient for connection strength, they increase the fabrication cost due to the increased difficulty in joint preparation, particularly with round HSS.

If the connection is inadequate to carry the branch required strength, the chord and branch members can be resized to improve the connection efficiency, the connection can be locally reinforced, or a joint can be used. External or internal local reinforcement is expensive to fabricate and the former may be unsatisfactory from a functional or aesthetic viewpoint. However, filling the chord with concrete in the vicinity of the connection is a satisfactory method of reinforcing the joint. Joint cans consist of a segment in the vicinity of the connection that has the same outer dimension as the rest of the chord but a larger wall thickness. This requires butt-welded splices to the rest of the chord, which can significantly increase fabrication cost. AWS D1.1 contains provisions for the design of joint cans including required length.

Section 9.4.3c is taken from AWS D1.1. The additional axial tension or compression load P_{ua} due to flexure in truss members is generally subject to a lower resistance factor than that used in beam bending. The definition of c, although given in a simplified form, is consistent with that given in AWS D1.1.

10. GENERAL REQUIREMENTS FOR HSS FABRICATION

The general provisions for fabrication, erection, and quality control as specified in LRFD Specification Chapter M are also applicable to HSS. In addition, the following HSS-specific concerns are addressed:

(1) Because the interior of an HSS is difficult to inspect, some concern has been expressed regarding internal corrosion. However, good design practice can eliminate the concern and the need for expensive protection.

Corrosion occurs in the presence of oxygen and water. In an enclosed building, it is improbable that there would be sufficient reintroduction of moisture to cause severe corrosion. Therefore, internal corrosion protection is a consideration only in HSS that are exposed to weather.

In a sealed HSS, internal corrosion cannot progress beyond the point where the oxygen or moisture necessary for chemical oxidation is consumed (AISI, 1970). The oxidation depth is insignificant when the corrosion process must stop, even when a corrosive atmosphere exists at the time of sealing. If fine openings exist at connections, moisture and air can enter the HSS through capillary action or by aspiration due to the partial vacuum that is created if the HSS is cooled rapidly (Blodgett, 1967). This can be prevented by providing pressure-equalizing holes in locations that make it impossible for water to flow into the HSS by gravity.

Situations where conservative practice would recommend an internal protective coating include: (1) open HSS where changes in the air volume by ventilation or direct flow of water is possible; and (2) open HSS subject to a temperature gradient that would cause condensation. In such instances it may also be prudent to use a minimum $^{5}/_{16}$-in. wall thickness.

HSS that are filled or partially filled with concrete should not be sealed. In the event of fire, water in the concrete will vaporize and may create pressure sufficient to burst a sealed HSS. Care should be taken to ensure that water does not remain in the HSS during or after construction, since the expansion caused by freezing can create pressure that is sufficient to burst an HSS.

Galvanized HSS assemblies should not be completely sealed because rapid pressure changes during the galvanizing process tend to burst sealed assemblies.

(2) As a result of manufacturing, a light oil coating is generally present on the outer surface of the HSS. If paint is specified, HSS must be cleaned of this oil coating with a suitable solvent; see the *Steel Structures Painting Manual*. (SSPC, 1991)

(3) To avoid weld contamination, the light oil coating that is generally present after manufacturing an HSS should be removed with a suitable solvent in locations where welding will be performed. In cases where an external coating has been applied at the mill, the coating should be removed at the location of welding or the manufacturer should be consulted regarding the suitability of welding in the presence of the coating.

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