Tentative Criteria for Structural Applications of Steel Tubing and Pipe

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SECTION 7.2.3

(The square root signs have been added.)

$$\text{where } g = wt\left[1 - \frac{327}{\frac{w}{t}\sqrt{F_y}}\left(1 - \frac{64.9}{\frac{w}{t}\sqrt{F_y}}\right)\right]$$

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SECTION 7.3.2.
LAST EQUATION
COLUMN ONE

(The square root sign has been added.)

$$\text{or} \quad C_v = \frac{440}{h/t\sqrt{F_y}}, \quad \text{when } C_v \text{ is more than } 0.8$$

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EQUATION 8.2

(Lower case w has been capitalized.)

$$f_v = \frac{T}{2(W-t)(H-t)t}$$

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PREFACE

Prior to the publication of this Manual, there was no comprehensive set of design rules for the use of steel tubing and pipe for structural purposes. This "Tentative Criteria for Structural Applications of Steel Tubing and Pipe" is based on world-wide research and practical experience available to the American Iron and Steel Institute at the time of writing. This research is continuing, and the "Tentative Criteria" will be amended as future research and experience dictates.

The "Tentative Criteria" contains references to both Canadian and United States specifications. It is proper that the Canadian Specifications be used by Canadian building code and standards organizations, and that American Society for Testing and Materials and American Institute of Steel Construction specifications are used by United States code groups.

NOTATION

Symbol	Definition
A	Cross-sectional area, in^2
C_c	Column slenderness ratio dividing elastic and inelastic buckling
C_m	Coefficient for bending term in interaction formula
C_v	Ratio of "critical" web stress, according to linear buckling theory, to shear stress of a material
D	Mean diameter of a round tubular member, in.; width of main rectangular member of a connection, in.
E	Modulus of elasticity of steel (29000 ksi)
F_a	Allowable compression stress under concentric loading, ksi
F_{as}	Allowable compression stress under concentric loading on bracing and secondary members, ksi
F_b	Allowable bending stress when bending stress only exists, ksi
F'_e	Euler stress divided by factor of safety, ksi
F_{ey}	Effective yield stress to account for internal pressure, ksi
F_v	Allowable shear stress, ksi
F_y	Yield point, ksi
H	Total outside depth of a rectangular tubular section, in.
I	Moment of inertia of the total section about the axis of bending, in^4
K	Effective length factor
L	Unbraced length of the member, in.; length of the member subjected to torsion, in.
L_b	Unbraced length in the plane of bending, in.
M	Bending moment
Q_a	Area factor for thin walled tubular sections
S	Elastic section modulus, in^3
T	Thickness of the main member of a connection, in.
W	Total outside width of a rectangular tubular section, in.
a	Area of the branch member of a connection, in^2
b	Effective design width of a compression element, in.
d	Diameter or width of a branch in a connection, in.
f	Computed stress in compression element, ksi
f_a	Computed axial stress on the gross section, ksi
f_b	Computed bending stress, ksi
g	Area of the flange deducted to obtain the effective area, in^2
h	Clear depth of the web between flanges, in.
p	Radial pressure, psi
p_{cr}	Critical external pressure for elastic buckling, psi
r	Radius of gyration, in.
r_b	Radius of gyration in the plane of bending, in.
t	Base thickness of steel, in.: thickness of a branch in a connection, in.
w	Flat width of an element exclusive of fillets, in. In the absence of knowledge of the actual flat width, it may be taken as W-3t.
θ	Angle between branch and main member of a connection, degrees

SECTION 1.0
SCOPE, FIELD OF APPLICATION

These criteria are intended for use in the design of round and rectangular rolled or manufactured steel tubular sections used as tension, compression, bending or torsion members of building structures. Although the effects of unavoidable pressure differences may occasionally have to be considered, the criteria do not apply to pressure piping systems, pressure vessels or to buried structures with non-uniform external pressures. The criteria are intended to apply only to the manufactured products of a pipe or tubing mill purchased to a structural product specification. Only unstiffened tubular sections are considered except for the possibility of stiffening at connections and locations of concentrated forces.

These criteria are generally based on the "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings", latest edition issued by the American Institute of Steel Construction (AISC Specification) and the "Specification for the Design of Cold-Formed Steel Structural Members" (AISI Specification) except where recent research indicates that modifications are desirable.

The following definitions of terminology are used throughout the "Tentative Criteria for Structural Applications of Steel Tubing and Pipe" (Criteria):

Tubular Section. A hollow structural member with constant wall thickness and constant cross section along the length. Tubular sections include welded or seamless, square and rectangular tubing, pipe or hollow structural shapes.

Class A Tubular Section. A tubular section made by seamless or continuous welding process and hot formed to final shape, or a round tubular section made by cold forming, or a rectangular tubular section made by cold forming, and subsequently stress relieved.

The producer, if requested, shall furnish representative data showing that the average stress to average strain relationship obtained from a short compression test of the full cross section of a cold-formed stress relieved tubular section shall have a proportional limit of at least 70% of yield.

Class B Tubular Section. A rectangular tubular section that is cold formed from a section produced by either a seamless or by an automatic welding process producing a continuous weld.

Rectangular. Tubular sections with square or rectangular cross section but with corners rounded within the tolerances of appropriate product specifications, sometimes referred to as shaped tubing.

Round. Tubular sections with a circular cross section.

Manufactured. The product of a pipe or tubing mill meeting the tolerances of a standard specification and made by the seamless, continuous weld, electric resistance welding with longitudinal or spiral seam, or double submerged arc welding processes. Tubular sections made by structural fabrication are not included.

SECTION 2.0
PLANS AND DRAWINGS

In addition to conforming with acceptable standards for design drawings, shop drawings and nomenclature as set forth in the AISC Specification or Canadian Standard S16, all design and shop drawings shall clearly indicate the material specification, grade and whether the tubular sections are to be Class A or Class B.

SECTION 3.0
LOADS AND FORCES

3.1 General loads in the design of tubular structures shall include consideration of dead loads, live loads and impact. In the absence of any applicable building code requirements, the minimum conditions of the AISC Specification shall apply. Other loads which must be considered when they are present, shall meet the requirements of the provisions in 3.1.1, 3.1.2, 3.1.3 and 3.1.4.

3.1.1 Horizontal forces on crane runways shall be those of the AISC Specification unless otherwise specified.

3.1.2 Wind forces on enclosed tubular structures shall be determined in the same manner as for any other framing system. Wind forces on exposed frameworks of round tubular sections shall be ⅔ of the forces on frameworks with similar configurations but using sections or shapes with flat elements.

A factor of $0.6\frac{W}{H} + 0.4 \geqq \frac{2}{3}$ may be applied to reduce wind forces on the short side of rectangular sections with the outer corner radius not less than 0.05 times the width, W.

W = outside width of section
H = outside depth of section

Further reductions may be applied for larger corner radii if justified by data.

In the absence of any applicable building code, a wind load analysis based on the report "Wind Forces on Structures," ASCE Transactions, Vol. 126, Part II, paper 3269, 1962 or other recognized procedure shall be used to determine the wind forces.

3.1.3 External or internal pressures which are created by wave action or liquid filling for fire protection or by temperature induced changes in the pressure of air in a sealed tubular section shall be considered. The tubular sections shall not be required to contain these pressures for operational purposes and only their effect on reducing the primary load capacity of the members shall be considered.

Precaution or drainage shall be provided to insure that pressure due to freezing of water during construction or service will not occur.

3.1.4 Structures in localities subject to earthquakes, hurricanes and other extraordinary conditions shall be designed with due regard for such conditions.

3.2 Minimum dead, live, impact and extraordinary loads, in the absence of any applicable building code, shall not be less than those recommended in the American National Standards Institute Standard A58-1-72.

SECTION 4.0 MATERIAL

4.1 Structural Steel—Material approved for use in this Criteria shall conform to the limitations on chemical composition, mechanical properties and dimensional tolerances of the following listing (latest date of issue). Materials meeting any of these Specifications are prequalified for design according to the criteria for the class under which they are listed. Materials manufactured according to the Specifications listed under Class B may be qualified as Class A if stress relieved.

CLASS A

ASTM A53, "Welded and Seamless Steel Pipe"
- Type F
- Grade B, hot formed seamless
- Electric resistance welded
- Cold drawn seamless

ASTM A500, "Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes"
- Rounds

ASTM A501, "Hot-Formed Welded and Seamless Carbon Steel Structural Tubing"

ASTM A618, "Hot-Formed Welded and Seamless High-Strength Low-Alloy Structural Tubing"

Canadian Standard G40.21, "Structural Quality Steel"
- Class H
- Class C rounds

CLASS B

ASTM A500, "Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes"
- Shapes

Canadian Standard G40.21, "Structural Quality Steel"
- Class C shapes

Certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A370 or Canadian Standard G40.20 and the governing specification shall constitute sufficient evidence of conformity with one of the above specifications. If specified on the order, the fabricator shall provide an affidavit stating that the structural steel furnished meets the requirements of the grade specified.

4.2 Bolts—High strength steel bolts shall conform to one of the following specifications, latest edition:

"High Strength Bolts for Structural Steel Joints, Including Suitable Nuts and Plain Hardened Washers", ASTM A325
"Quenched and Tempered Steel Bolts and Studs", ASTM A449
"Quenched and Tempered Alloy Steel Bolts for Structural Steel Joints", ASTM A490

Other bolts shall conform with the "Specification for Carbon Steel Externally and Internally Threaded Standard Fasteners", ASTM A307, latest edition, hereinafter designated as A307 bolts.

Manufacturers certification shall constitute sufficient evidence of conformity with the specification.

4.3 Filler Metal for Welding—Filler metal for various types of welding shall conform to one of the following specifications, latest edition:

4.3.1 Manual shielded metal-arc welding
"Specification for Mild Steel Covered Arc-Welding Electrodes", AWS A5.1
"Mild Steel Covered Arc-Welding Electrodes", CSA W48.1
"Specification for Low-Alloy Steel Covered Arc-Welding Electrodes", AWS A5.5
"Low-Alloy Steel Arc-Welding Electrodes", CSA W48.3

4.3.2 Submerged arc welding
"Specification for Bare Mild Steel Electrodes and Fluxes for Submerged Arc Welding", AWS A5.17, F60 or F70 AWS-flux classifications
"Specification for Bare Low-Alloy Steel Electrodes and Fluxes for Submerged Arc Welding", AWS 5.23
"Bare Mild Steel Electrodes and Fluxes for Submerged Arc Welding", CSA W48.6

4.3.3 Gas metal arc welding
"Specification for Mild Steel Electrodes for Gas Metal-Arc Welding", AWS, A5.18, E60S or E70S electrodes
"Solid Mild Steel Electrodes for Gas Metal-Arc Welding", CSA W48.4

4.3.4 Flux cored arc welding
"Specification for Mild Steel Electrodes for Flux-Cored-Arc Welding", AWS A5.20, E60T or E70T electrodes
"Mild Steel Electrodes for Flux Cored Arc Welding", CSA W48.5

Manufacturer's certification shall constitute sufficient evidence of conformity with the specifications.

SECTION 5.0
EFFECTIVE SLENDERNESS RATIOS

In determining the slenderness ratio (KL/r) of an axially loaded compression member, the length shall be taken as its effective length KL, and r as the corresponding radius of gyration of the cross section. Provisions for the appropriate value of the effective length factor K follow.

5.1 For compression members in trusses made with tubular webs and continuous tubular chords connected with joints of 100% efficiency as defined in Section 10.0, the effective length factors used to modify the actual length between bracing points in the plane of buckling shall be

a) K = 0.7 for web members

b) K = 0.9 for chord members

5.2 In trusses where tubular members are connected to members with other shapes, the effective length factor K shall be taken as 0.9 for inplane buckling for continuous chords, as unity for other members or determined by rational analysis.

5.3 For compression members in frames, the effective length factor K shall be selected in accordance with the provisions of the AISC Specification as follows:

5.3.1 In frames with sidesway prevented by adequate attachment to diagonal bracing, shear walls, an adjacent structure having adequate lateral stability, or to floor slabs or roof decks secured horizontally by walls or bracing systems parallel to the place of the frame, the effective length factor, K, shall be taken as unity, unless analysis shows that a smaller factor can be used.

5.3.2 In frames where lateral stability is dependent upon the bending stiffness of rigidly connected beams and columns, the effective length KL of compression members shall be determined by a rational method but shall not be less than the unbraced length.

5.4 For tubular sections used as bracing or other secondary members, the effective length factor, K, shall be taken as unity.

5.5 The slenderness ratio for tubular sections in compression shall not exceed 200.

The slenderness ratio for tubular sections used as tension members, preferably should not exceed 240 for main members or 300 for bracing and other secondary members.

SECTION 6.0
AXIAL COMPRESSION

6.1 For round tubular sections with a diameter/thickness ratio not exceeding $3300/F_y$ or for rectangular sections with the flat width/thickness ratio of the longest side not exceeding $238/\sqrt{F_y}$, the stress on the gross section shall not exceed the limitations of Sections 6.1.1 or 6.1.2.

6.1.1 For compression members when KL/r is less than C_c

a) Class A tubular sections

$$F_a = \frac{\left[1 - \frac{(KL/r)^2}{2C_c^2}\right] F_y}{\frac{5}{3} + \frac{3(KL/r)}{8C_c} - \frac{(KL/r)^3}{8C_c^3}}$$

$$\text{where } C_c = \sqrt{\frac{2\pi^2 E}{F_y}} = \frac{756}{\sqrt{F_y}}$$

b) Class B tubular sections

$$F_a = \frac{\left[1 - \frac{(KL/r)}{1.5C_c}\right] F_y}{\frac{5}{3} + \frac{KL/r}{4C_c}}$$

$$\text{where } C_c = \sqrt{\frac{3\pi^2 E}{F_y}} = \frac{925}{\sqrt{F_y}}$$

6.1.2 When KL/r for compression members exceeds C_c but does not exceed 200

$$F_a = \frac{12\pi^2 E}{23(KL/r)^2} = \frac{149{,}000}{(KL/r)^2}$$

6.2 For round tubular sections with the diameter/thickness ratio greater than $3300/F_y$ but less than $13000/F_y$, the axial stress shall not exceed the smallest value of

a) The allowable stress for primary buckling contained in the provisions of Section 6.1, or

b) $$F_a = \frac{662}{D/t} + .40\ F_y$$

6.3 For rectangular tubular sections with the flat width/thickness ratio of any side exceeding $238/\sqrt{F_y}$, the stress on the gross section shall not exceed the value calculated by the following procedure:

a) Compute the effective width of all sides with the flat width/thickness ratio exceeding $238/\sqrt{F_y}$ by

$$b = \frac{253t}{\sqrt{f}}\left(1 - \frac{50.3}{(w/t)\sqrt{f}}\right) \leq w$$

w = the actual flat width
t = the thickness
f = computed stress on the gross area

b) Compute the form factor

$$Q_a = 1 - \frac{\Sigma(w - b)t}{\text{Actual Area}}$$

c) Compute the allowable stress according to the provisions of Sections 6.1.1 or 6.1.2 but with a modified yield stress, $Q_a\ F_y$, replacing the actual yield stress, F_y, in the equations for allowable stress and C_c.

SECTION 7.0
BENDING

7.1 Normal bending stresses in round tubular sections shall be limited to the allowable stresses in 7.1.1 or 7.1.2. Class A sections with D/t less than $1300/F_y$ are suitable for moment redistribution in 7.2.4.

7.1.1 For round tubular sections with the diameter/thickness ratio no greater than $3300/F_y$, the stress on the extreme fiber shall not exceed

$$F_b = 0.72\ F_y$$

7.1.2 For round tubular sections with the diameter/thickness ratio greater than $3300/F_y$ but no greater than $13000/F_y$ the stress on the extreme fiber shall not exceed

$$F_b = \frac{662}{D/t} + .40\ F_y$$

7.2 Normal bending stresses in rectangular tubular sections shall be limited to the allowable stresses in the appropriate Section 7.2.1, 7.2.2 or 7.2.3. Class B compact sections with the flat width/thickness ratio less than $150/\sqrt{F_y}$ and Class A compact sections are suitable for moment redistribution in 7.2.4.

7.2.1. For rectangular tubes which qualify as compact sections, the stress on the extreme fiber **(F_b)** shall not exceed:

$0.72F_y$ for rectangular sections bending about the strong axis
$0.69F_y$ for square sections
$0.66F_y$ for rectangular sections bending about the weak axis

In order to qualify as a compact section, the member shall meet the following requirements:

i) the flat width/thickness ratio of the compression flange shall not exceed $210/\sqrt{F_y}$

ii) the depth/thickness ratio of the webs shall not exceed

$$\frac{H}{t} = \frac{412}{\sqrt{F_y}}\left(1 - 2.33\,\frac{f_a}{F_y}\right) \text{ when } \frac{f_a}{F_y} \leq 0.16$$

$$\text{or } \frac{H}{t} = \frac{257}{\sqrt{F_y}} \text{ when } \frac{f_a}{F_y} \geq 0.16$$

iii) rectangular sections with $H/W \leq 6$ bending about the strong axis shall have the compression flange laterally supported at intervals not to exceed

$$\frac{L}{W} F_y = 1950 + 1200\,\frac{M_1}{M_2}$$

except that it need not be less than 1200. M_1 is the smaller and M_2 is the larger bending moment at the end of the unbraced length and the M_1/M_2 ratio of end moments is positive when M_1 and M_2 have the same sign (reverse curvature bending) and negative when they are of opposite signs (single curvature bending).

7.2.2 For non-compact rectangular sections meeting all the following requirements,

i) the flat width/thickness ratio of the compression flange not exceeding $245/\sqrt{F_y}$

ii) the depth/thickness ratio not exceeding $760/\sqrt{0.6\,F_y} = \dfrac{980}{\sqrt{F_y}}$

The stress on the extreme fiber (F_b) shall not exceed $\mathbf{0.60F_y}$.

7.2.3 For rectangular tubes with the flat width/thickness ratio of the compression flange exceeding $245/\sqrt{F_y}$ but with the depth/thickness ratio not exceeding $760/\sqrt{F_b}$, the allowable stress computed on the basis of the properties of the total section shall not exceed

$$F_b = 0.6\left(1 - \frac{g}{A} - \frac{1}{4}\,\frac{gH^2}{I}\right) F_y$$

$$\text{where } g = wt\left[1 - \frac{327}{\frac{w}{t}\sqrt{F_y}}\left(1 - \frac{64.9}{\frac{w}{t}\sqrt{F_y}}\right)\right]$$

A = area of the total section
H = depth of the total section
I = moment of inertia of the total section
w = flat width of flange

7.2.4 Sections suitable for moment redistribution as defined in 7.1 and 7.2 that are continuous over supports or are rigidly framed to columns by means of rivets, high strength bolts or welds, may be proportioned for 9/10 of the negative moment produced by gravity loading which are maximum at points of support, provided that, for such members, the maximum positive moment shall be increased by 1/10 of the average negative moments. This reduction shall not apply to moments produced by loading on cantilevers. If the negative moment is resisted by a column rigidly framed to the beam or girder, the 1/10 reduction may be used in proportioning the column for the combined axial and bending loading, providing that the stress, f_a, due to any concurrent axial load on the member, does not exceed 0.15 F_a.

7.3 Shear stresses due to bending shall be limited to the provisions of 7.3.1 or 7.3.2.

7.3.1 For unstiffened round tubes with the diameter/thickness ratio no greater than $12100/F_y$, the transverse shear stress due to beam action shall not exceed

$$F_v = 0.40\ F_y$$

The shear stress may be calculated as the total shear force divided by half the cross-sectional area of the tube.

7.3.2 For rectangular tubes with the ratio of clear depth of web to thickness no greater than $425/\sqrt{F_y}$, the average web shear shall not exceed

$$F_v = 0.36\ F_y$$

For thinner sections the average shear stress shall not exceed

$$F_v = \frac{F_y}{2.89}(C_v)$$

where $C_v = \dfrac{240{,}000}{F_y(h/t)^2}$, where C_v is less than 0.8

or $C_v = \dfrac{440}{h/t\ F_y}$, when C_v is more than 0.8

The shear stress may be calculated as the total shear divided by the web area which is the section depth times twice the thickness.

SECTION 8.0
TORSION

8.1 The shear stress in round tubular sections due to torsion shall be limited to the allowable stresses in 8.1.1 and 8.1.2.

8.1.1a For Class A round tubular sections with the ratio of diameter to thickness no greater than the larger of

$$D/t = 1.77\ (E/F_y)^{4/5}/(L/D)^{2/5} = \frac{6575}{F_y^{4/5}(L/D)^{2/5}}$$

$$\text{or } D/t = (E/F_y)^{2/3} = \frac{944}{F_y^{2/3}}$$

the shear stress shall not exceed

$$F_v = 0.4\ F_y$$

8.1.1b For Class B round tubular sections with the ratio of diameter to thickness no greater than

$$D/t = 0.529\ (E/F_y)^{2/3} = \frac{500}{F_y^{2/3}}$$

the shear stress shall not exceed

$$F_v = 0.4\ F_y$$

8.1.2 For thin round tubular sections which exceed the diameter to thickness ratios in 8.1.1a or b, the shear stress shall be limited to

a) For Class A tubular sections, the larger of

$$F_v = \frac{0.82\,E}{\sqrt{L/D(D/t)^{5/4}}} = \frac{23800}{\sqrt{L/D(D/t)^{5/4}}}$$

$$\text{or } F_v = \frac{0.4\,E}{(D/t)^{3/2}} = \frac{11600}{(D/t)^{3/2}}$$

b) For Class B tubular sections

$$F_v = 0.4\,F_y\left[\frac{1-0.0363(L/D)^{0.4}(1+1.89(F_y/E)^{2/3}D/t)}{1-0.0727(L/D)^{0.4}}\right]$$

$$\text{for } D/t \le \frac{3.09}{(L/D)^{0.4}}\left(\frac{E}{F_y}\right)^{4/5} = \frac{11500}{(L/D)^{0.4}F_y^{0.8}}$$

$$\text{or } F_v = \frac{0.82\,E}{\sqrt{L/D(D/t)^{5/4}}} \text{ for } D/t > \frac{3.09}{(L/D)^{0.4}}\left(\frac{E}{F_y}\right)^{4/5}$$

However, the allowable stress need not be less than

$$F_v = 0.5\,F_y\left[1-0.378\left(\frac{F_y}{E}\right)^{2/3}(D/t)\right]$$

$$\text{for } D/t < 1.59\left(\frac{E}{F_y}\right)^{2/3}$$

$$\text{or } F_v = \frac{0.4\,E}{(D/t)^{3/2}} \text{ for } D/t \ge 1.59\left(\frac{E}{F_y}\right)^{2/3}$$

8.2 The shear stress in the longest side of rectangular tubular sections caused by torsion shall be limited to the allowable stresses in 8.2.1 or 8.2.2.

8.2.1 For tubular sections with the clear length of the longest side to thickness ratio no greater than $380/\sqrt{F_y}$, the shear stress shall not exceed

$$F_v = 0.40\,F_y$$

8.2.2 For thinner sections with the ratio of the clear length of the longest side to thickness greater than $380/\sqrt{F_y}$, the stress shall not exceed

$$F_v = \frac{F_y}{2.89}(C_v)$$

where $C_v = \frac{240{,}000}{F_y(h/t)^2}$, where C_v is less than 0.8

or $C_v = \frac{440}{\frac{h}{t}\sqrt{F_y}}$ when C_v is more than 0.8

SECTION 9.0
COMBINED LOADING

9.1 Internal Pressure—For tubular sections which are unavoidably subjected to significant internal pressure, design for axial load, bending or torsion shall be based on the appropriate effective yield stress listed below:

a) For loadings which cause normal stresses

$$F_{ey} = \left(1-\frac{pD}{2t}\Big/F_y\right)F_y$$

b) For loadings which cause shear stresses

$$F_{ey} = \sqrt{1-\left(\frac{pD}{2t}\Big/F_y\right)^2}\,F_y$$

The value of D is the diameter of a round tubular section or the length of the longer side of a rectangular tube.

9.2 External Pressure—For round tubular sections which are unavoidably subjected to external pressure, the allowable stress in axial compression, bending or torsion shall be reduced by a factor of

$$\left(1 - \frac{\mathbf{p}}{\mathbf{p}_{cr}}\right) \text{ where } \mathbf{p}_{cr} = \frac{\mathbf{1.32\ E}}{\mathbf{(D/t)^3}} = \frac{\mathbf{38.3 \times 10^6}}{\mathbf{(D/t)^3}} \ \mathbf{(psi)}$$

9.3 Axial Compression and Bending—Members subjected to both axial compression and-bending stresses shall be proportioned to satisfy the following requirements:

a)

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{my} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}} \leq 1.0$$

b)

$$\frac{f_a}{0.60\ F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0$$

c) When $f_a/F_a < 0.15$, the following formula may be used in lieu of the formulas in a) and b)

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0$$

In formulas a), b) and c) the subscripts x and y, combined with subscripts, b, m and e, indicate the axis of bending about which a particular stress or design property applies, and

F_a = axial stress that would be permitted if axial force alone existed

F_b = compressive bending stress that would be permitted if bending moment alone existed

$F'_e = \dfrac{12\pi^2 E}{23(KL_b/r_b)^2}$ (In the expression for F'_e, L_b is the actual unbraced length in the plane of bending and r_b is the corresponding radius of gyration. K is the effective length factor in the plane of bending.)

f_a = computed axial stress

f_b = computed compressive bending stress at the point under consideration

C_m = a coefficient whose value shall be taken as follows:

1. For compression members in frames subject to joint translation (sidesway), $C_m = 0.85$.
2. For restrained compression members in frames braced against joint translation and not subject to transverse loading between their supports in the plane of bending

$C_m = 0.6 - 0.4 \dfrac{M_1}{M_2}$, but not less than 0.4, where M_1/M_2 is the ratio of the smaller to larger moments at the ends of that portion of the member unbraced in the plane of bending under consideration. M_1/M_2 is positive when the member is bent in reverse curvature and negative when it is bent in single curvature.

3. For compression members in frames braced against joint translation in the plane of loading and subjected to transverse loading between their supports, the value of C_m may be determined by rational analysis. However, in lieu of such

analysis, the following values may be used: (a) for members whose ends are restrained, $C_m = 0.85$; (b) for members whose ends are unrestrained, $C_m = 1.0$.

For round or square tubular sections with no lateral bracing along the length and with end conditions such that the effective length factor K is the same for any direction of bending, f_{bx} may be taken as the maximum bending stress, $\sqrt{M_x^2 + M_y^2}/S$ and f_{by} as zero.

9.4 Axial Tension and Bending — Members subjected to both axial tension and bending stresses shall be proportioned at all points along their length to satisfy the requirements

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0$$

where f_b is the computed bending tensile stress. However, the computed bending compressive stress, taken alone, shall not exceed the applicable value according to Section 7.0.

9.5 Axial Compression and Torsion — Members subjected to both axial compression and torsion shall be proportioned to satisfy the following requirement

$$\frac{f_a}{F_a} + \left(\frac{f_v}{F_v}\right)^2 \leq 1.0$$

where

f_v is the maximum shear stress

F_v is the allowable shear stress from Section 8.0

f_a is the axial stress on the gross section

F_a is the allowable axial stress from Section 6.0

9.6 Bending and Torsion — Members subjected to both bending and torsion stresses shall be proportioned to satisfy the following requirement at every location

$$\frac{f_b}{F_b} + \left(\frac{f_v}{F_v}\right)^2 \leq 1.0$$

where

f_v is the sum of the shear due to torsion and the shear due to bending at the location in question

F_v is the allowable shear stress in Section 8.0

f_b is the bending stress at the location in question

F_b is the allowable bending stress from Section 7.0

9.7 Axial Load, Bending and Torsion — Members simultaneously subjected to axial load, bending and torsion stresses shall be proportioned to meet the following requirements

$$\left(\frac{f_a}{F_a} + \frac{C_m f_b}{\left(1 - \frac{f_a}{F_e'}\right) F_b}\right) + \left(\frac{f_v}{F_v}\right)^2 \leq 1.0$$

or $$\left(\frac{f_a}{0.6\,F_y} + \frac{f_b}{F_b}\right) + \left(\frac{f_v}{F_v}\right)^2 \leq 1.0$$

when $f_a/F_a \leq 0.15$ the following formula may be used in lieu of both the above

$$\left(\frac{f_a}{F_a} + \frac{f_b}{F_b}\right) + \left(\frac{f_v}{F_v}\right)^2 \leq 1.0$$

SECTION 10.0
TUBULAR CONNECTIONS FOR STATIC LOADS

10.1 The following definitions of general terminology are used in this section and are illustrated in Figure 10.1:

Connection—the portion of a tubular structure in the general proximity of the intersection of two or more members.

Joint — the interface formed by the intersection of two members.

Main member—the continuous member passing through a connection.

Branch member—a member framing into the main member of a connection.

Direct interwelded connection — a connection formed by continuously welding the joint around the periphery of all branches framing into the connection. The branches may be either completely joined to the main member or partially to other branches.

Eccentricity—the offset of the intersection of the centerlines of branch members from the centerline of the main member. The eccentricity is negative when the intersection is toward the side of the main member into which the branches are joined.

T or Y connection—a single branch framing into a main member.

K connection—two branch members framing into the main member at the connection or two symmetric diagonal branches and a branch perpendicular to the main member.

Gap—in a K connection, the spacing between the toes of two branch members.

Lap—in a K connection, the length of overlaps in the sides of branches projected to the edge of the main member.

Width—width of a member is measured perpendicular to the plane of the main member and branch.

100% efficiency—developing the full strength of the members framing into the joint.

D—diameter or width of the main member.

d—the diameter or width of a branch.

T—the thickness of the main member.

t—the thickness of a branch.

a—the area of a branch.

θ—the angle between a branch and the main member.

Other terminology used to describe welds and joint preparation follow the definitions contained in AWS D1.1-75 "Structural Welding Code," Section 10.

10.2 The combined requirements of Sections 10.3 and 10.4 or 10.5 of this Criteria are for direct interwelded connections with all members in a single plane and developing 100% efficiency. Connections which do not conform with these requirements may be used if the design is verified by one of the following procedures:

10.2.1 An analysis of the connection according to the provisions and criteria of Section 10 of AWS D1.1-75.

10.2.2 A rational analysis shall be conducted to determine the maximum stresses in the main member, all branches and joining elements. The maximum normal stress shall not exceed 0.6 F_y or the local buckling strength of compression elements and the maximum shear stress shall not exceed 0.4 F_y.

10.2.3 The details of the connection shall be verified by tests to show that the efficiency is 100% or that the connection can develop 1.7 times the design load of each member.

10.3 The welds in joints of direct inter-welded connections shall conform to the requirements of Sections 10.3.1 through 10.3.3.

10.3.1 Fillet welds ,and complete or partial penetration groove welds which are accepted without welding procedure qualification in AWS D1. 1-75, Section 10 may be used in this Criteria without welding procedure qualification.

Connection and joint details, welding processes or welding procedures other than those included in the foregoing may be employed provided they shall have been qualified in accordance with the procedures specified in AWS D1.1-75, Section 10.

The electrodes and fluxes used in making welds shall be matched to the base metal in accordance with Table 10.1.

10.3.2 The allowable stresses in welds shall be those indicated in Table 10.2 provided the weld metal and base metal have been matched as per Table 10.1.

10.3.3 The effective areas of welds shall be the product of the effective length and effective throat thickness as defined below:

a) The effective length shall be the overall length of the full size weld measured along the centerline of the effective throat. The effective length may be conservatively taken as the circumference of the branch.

b) The effective throat of complete penetration groove welds shall be the thickness of the thinner part joined, with no increase for weld reinforcement.

c) The effective throat thickness for partial penetration groove welds shall be those specified in the prequalified joint details of AWS D1.1-75, Section 10.

d) The effective throat thickness for fillet welds shall be the shortest distance from the root to the face of the diagrammatic weld.

10.4 Connections with direct interconnection of round branches to a round main member shall be proportioned to the following requirements:

10.4.1 For T or Y connections with the branch subjected to an axial force

$$\mathbf{d/D \geq 0.21\ (F_y/35)}$$

$$\mathbf{D/T \leq 15\ (35/F_y)}$$

$$\mathbf{t/T \leq 0.5}$$

10.4.2 For K connections with the branches subjected to axial forces, limits for the main member and the tension branch are:

a) with zero eccentricity and up to 50% overlap in the branches

$$\mathbf{d/D \geq 0.30\ (F_y/35)}$$

$$\mathbf{D/T \leq 20\ (35/F_y)}$$

$$\mathbf{t/T \leq 0.6}$$

b) with negative eccentricity and up to 50% overlap in the branches

$$\mathbf{d/D \geq 0.30\ (F_y/35)}$$

$$\mathbf{D/T \leq 50\ (35/F_y)}$$

$$\mathbf{t/T \leq 1.0}$$

c) positive eccentricity is not permitted.

In addition to the limits listed in a) and b), the stress in the compression branch should not exceed the yield stress before the tensile branch reaches its full tensile capacity. If the ultimate buckling stress in the compression branch is less than F_y, the wall thickness of the member should be increased by the ratio of its yield stress to its buckling stress.

If there is over 50% overlap in the branches, there is no restriction on the wall thickness or diameter of the main member.

10.4.3 For connections in which the branches carry only bending moments and shear

$$\mathbf{d/D \geq 0.21\ (F_y/35)}$$

$$\mathbf{D/T \leq 15\ (35/F_y)}$$

$$\mathbf{t/T \leq 0.5}$$

10.5 Connections with direct interconnection of rectangular main members and round or rectangular branches shall be proportioned according to the following requirements:

10.5.1 For joints with axial forces in the branches and less than 50% overlap of the branches

a) if the average width of the branches does not exceed half the width of the main member

$$\mathbf{T \geq \frac{\sin \theta}{1.92}\, a}$$

b) if the average width of the branches exceeds half the width of the main member

$$\mathbf{T \geq \frac{\sin \theta}{1.92}\, a \left[\frac{1}{1 + 3\left(\frac{2d_{ave}}{D} - 1\right)} \right]}$$

subject to the limitation that the required T must not exceed

$$\mathbf{\frac{\sin \theta}{6.4}\, a}$$

If there is over 50% overlap in the branches of a K connection, there is no restriction on the thickness of the main member.

10.5.2 For connections with only bending moments and shear in the branch

a) if the width of the main member is approximately the same as the branch

i) the depth of the branch must not exceed the depth of the main member

ii) the ratio of moment, M (ft.-kips) to shear, V (kips) in the branch at the joint shall be within the range of 1 to 4

iii) the axial load of the main member shall not exceed 0.36 times its yield load and the main member shall not be subject to general buckling

iv) $\mathbf{t/T \leq 1.0}$

b) if the width of the main member is greater than the branch, items i), ii), and iii) of a) still apply and in addition

iv) $\mathbf{t/T \leq 0.5}$

v) $\mathbf{D/T \leq 16}$

TABLE 10.1
MATCHING FILLER METAL REQUIREMENTS

Steel Specifications	Electrode Specification
ASTM A53 Grade B and Type F ASTM A500 ASTM A501	SMAW AWS A5.1, A5.5 or CSA W48.1, W48.3; E60xx or E70xx
	SAW AWS A5.17, A5.23 or CSA W48.6; F6x-Exxx or F7x-Exxx
	GMAW AWS A5.18 or CSA W48.4; E70S-x or E70U-1
	FCAW AWS A5.20 or CSA W48.5; E60T-x or E70T-x (Except ExxT-2 & ExxT-3)
ASTM A618 CSA G40.21 Grades 50W & 55W	SMAW AWS A5.1, A5.5 or CSA 48.1, W48.3; E70xx
	SAW AWS 5.17, A5.23 or CSA W48.6 F7x-Exxx
	GMAW AWS A5.18 or CSA W48.4; E70S-x or E70U-1
	FCAW AWS A5.20 or CSA W48.5; E70T-x (Except E70T-2 & E70T-3)

TABLE 10.2
ALLOWABLE STRESSES IN WELDS

Type of weld	Tubular application	Kind of stress	Permissible unit stress	Required weld metal strength level[1]
Complete joint penetration groove weld	Butt splices of tubular members	Compression normal to the effective area[2]	Same as for base metal	Weld metal with a strength level equal to or less than matching weld metal may be used
		Tension or shear on effective area	Same as for base metal	Matching weld metal must be used. See Table 10.1
	Structural T-, Y-, or K-connections in structures designed for critical loading such as fatigue, which would normally call for complete joint penetration welds	Tension, compression, or shear on base metal adjoining weld. (Weld made from outside only)	Same as base metal	Matching weld metal must be used. See Table 10.1
		Tension, compression, or shear on effective area of groove welds, made conventionally from both sides or with backing		Matching weld metal must be used. See Table 10.1
Partial joint penetration groove weld	Butt splices of tubular members	Tension or compression parallel to axis of the weld[2]	Same as for base metal[3]	Weld-metal with a strength level equal to or less than matching weld metal may be used
		Compression normal to the effective throat[2] — Joint not designed to bear	0.50 x specified minimum tensile strength of weld, except that stress on adjoining base metal shall not exceed 0.60Fy	Weld metal with a strength level equal to or less than matching weld metal may be used
		Compression normal to the effective throat[2] — Joint designed to bear	Same as for base metal	
		Tension or shear on effective throat	0.30 x specified minimum tensile strength of weld metal, except that stress on adjoining base metal shall not exceed 0.50Fy, for tension, nor 0.40Fy for shear.	Weld metal with a strength level equal to or less than matching weld metal may be used
	Structural T-, Y-, or K-connection in ordinary structures	Load transfer across the weld as stress on the effective throat	0.30 x specified minimum tensile strength of weld metal, except that stress on adjoining base metal shall not exceed 0.60Fy for tension and compression, nor 0.40Fy, for shear.	Matching weld metal must be used. See Table 10.1
Fillet weld	Structural T-, Y-, or K-connection in ordinary structures; lap splice of tubular members.	Shear stress on effective throat regardless of direction of loading	0.30 x specified minimum tensile strength of weld metal, except that shear stress on adjoining base metal shall not exceed 0.40Fy.	Weld metal with a strength level equal to or less than matching weld metal may be used

[1] For matching weld metal see Table 10.1.

[2] Beam or torsional shear up to 0.30 minimum specified tensile strength of weld metal is permitted, except that shear on adjoining base metal shall not exceed 0.40Fy.

[3] Groove and fillet welds parallel to the longitudinal axis of tension and/or compression members, except in connection areas, are not considered as transferring stress and hence may take the same stress as that in the base metal, regardless of electrode (filler metal) classification.

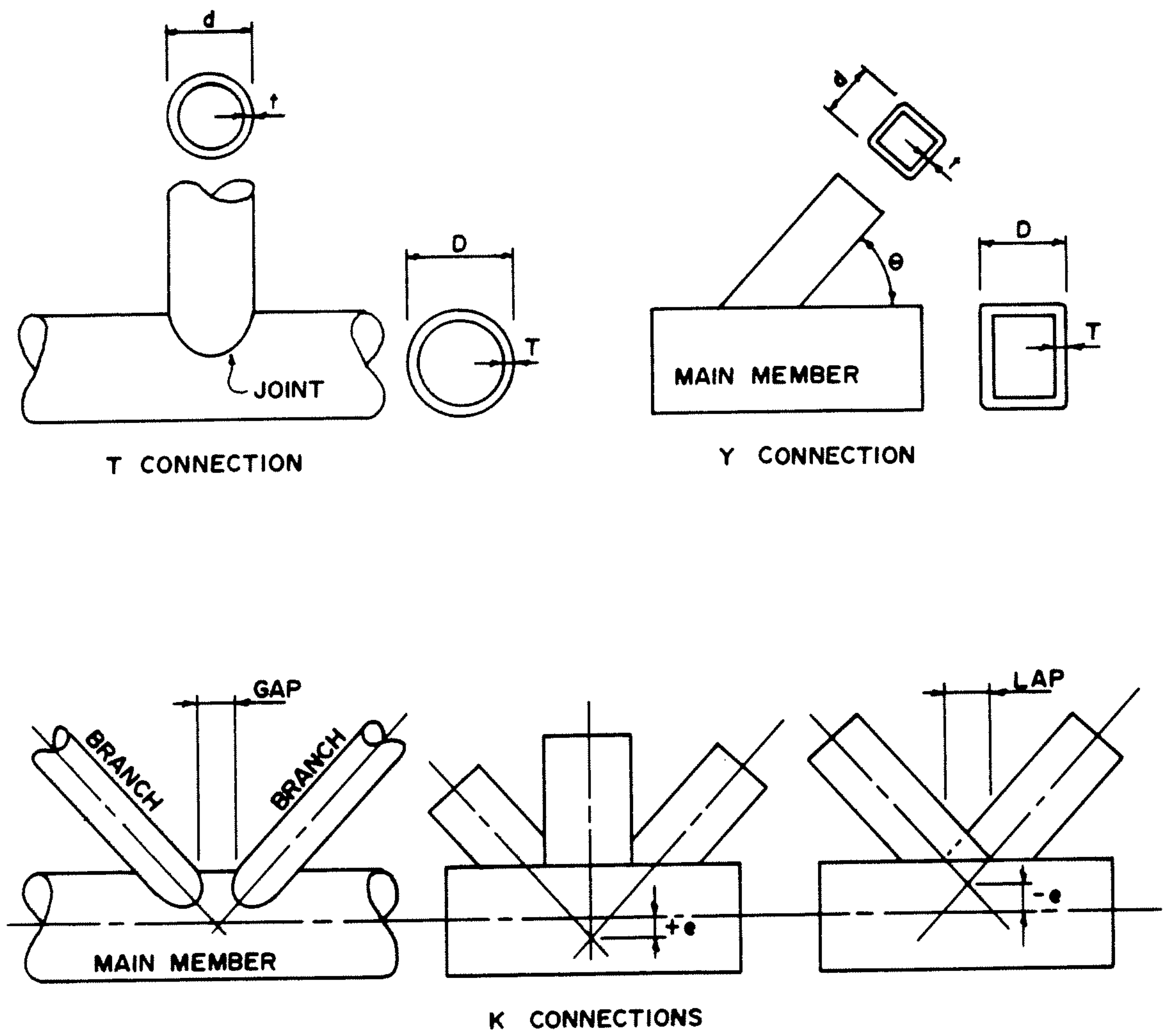

FIGURE 10.1 - DESCRIPTION OF TERMINOLOGY

SECTION 11.0
FATIGUE

11.1 The terminology and notation used in Section 11.0 is identical to that defined in Section 10.1.

11.2 The limitations of Section 11.0 apply to the design of tubular structures with direct interwelded connections.

11.3 Complete penetration groove welds where possible are recommended in connections subject to repeated loadings.

11.4 Only joints which meet the geometric requirements of Sections 11.4.1, 11.4.2 or 11.4.3 may be used for connections subject to repeated loadings.

11.4.1 For T or Y connections with a round branch and round main member

$$d/D \geq 0.21\ (F_y/35)$$

$$D/t \leq 15\ (35/F_y)$$

$$t/T \leq 0.5$$

11.4.2 For K connections with round branches and round main member

$$d/D \geq 0.30\ (F_y/35)$$

$$d/t \geq 10.0$$

$$\text{overlap} > 50\%$$

11.4.3 For K connections with rectangular main members

$$\text{overlap} > 50\%$$

11.5 In the design of tubular structures subject to repeated variation of live load stress, consideration shall be given to the number of stress cycles, the expected range of stress and type of detail. After selecting the load condition in Section 11.5.1 and the stress category in Section 11.5.2, the allowable stress shall be determined in Section 11.5.3.

11.5.1 Loading condition shall be classified as follows:

Loading Condition	Number of Loading Cycles From	To
1	20,000	100,000
2	100,000	500,000
3	500,000	2,000,000
4	over 2,000,000	

11.5.2 The type and location of material shall be categorized as follows:

Condition	Kind of Stress	Stress Category
Base Material in Tubular Section	normal tensile stress or range of normal tensile and compressive stress	A
T or Y connection	shear stress on shear area defined as thickness of main member times periphery of the joint	V
K connection with 50% ≤ lap < 100%	normal tensile stress or range of normal tensile and compressive stress in branch	U
K connection with lap = 100%	nomal tensile stress or range of normal tensile and compressive stress in branch	F

11.5.3 The maximum stress shall not exceed the basic allowable stresses provided in Sections 6.0, 7.0 and 8.0 and the maximum range of stress shall not exceed the following:

Allowable Stress Range (ksi)

Stress Category	Loading Condition 1	2	3	4
A	40	32	24	24
F	17	14	11	9
U	14	10	7	6
V	3.2	2.7	2.2	1.8

SECTION 12.0
CORROSION PROTECTION

12.1 Exterior Protection—Exterior painting and protection shall be in accordance with the standards and provisions of the AISC Specification.

12.2 Interior Protection—Provision for interior protection shall be in accordance with Section 12.2.1, 12.2.2 or 12.2.3.

12.2.1 Unless otherwise specified, tubular members which will be concealed by interior building finish or encased in concrete need not have any interior painting or protection.

12.2.2 If the section is open and through ventilation and the entrance of water can occur, the interior surface shall be given a protective coating and drainage shall be provided.

12.2.3 Tubular sections which are hermetically sealed or tubular sections which have a pressure equalizing hole and are not subject to water entering by gravity, require no interior surface protection.

Tubular sections may be hermetically sealed by completely welding closures at the ends and complete welding of any penetrations along the length.

When perfect sealing is not assured, a pressure equalizing hole should be placed in the tubular section at a location where water cannot enter by gravity flow.

Concrete filled tubular sections shall not be sealed.

12.3 Severe Conditions—Consideration of galvanic protection or special coatings and wrappings shall be given in situations of severely corrosive conditions which may occur locally at material interfaces or generally in a severe environment.

COMMENTARY

This Commentary on the foregoing "Tentative Criteria" is presented to enable the reader to have greater knowledge of its content by giving more information and a discussion of the basis of its provisions.

SECTION 1.0
INTRODUCTION

Tubular sections are efficient structural members for carrying compression and torsional forces. Consequently, they are becoming increasingly popular in structural design, especially in the modern forms of trusses and complex space frames. Their low ratio of exposed surface area to volume compared to other shapes can reduce maintenance expenses and their low resistance to external fluid flow provides a distinct advantage in frameworks exposed to wind or water currents. The use of tubular sections has been limited in the past by difficulty in joining, but modern fabricating technology is rapidly overcoming this disadvantage.

Pressure differences between the inside and outside of a tubular section can occasionally occur in a few applications, even if only momentarily. Since these pressures can interact with the primary forces being carried, they must be considered in the design. However, the requirements for guaranteed pressure containing systems are more stringent than for structural members, and consequently the Criteria does not apply to members where pressure containment is essential. Buried structures, where soil interaction is an important factor in the ultimate strength, also do not fall under the scope and intent of the Criteria.

For purposes of this Criteria, tubular sections are defined as hollow members with constant wall thickness and a round, square or rectangular cross section which is constant along the length. Although the term pipe is commonly associated with a product used for fluid transmission, pipe products used for structural purposes only are included in the definition for tubular sections. The Criteria apply only to tubular sections which are manufactured in a pipe or tubing mill. The normal range of diameters for round tubular sections is approximately ¼ inch to 44 inches with wall thicknesses varying from less than 0.1 inches in smaller sizes to over 1.625 inches in larger diameters. It should be noted that larger sizes would generally have to be obtained by special order from mills with the capability of producing these sizes. Manufactured tubular sections are made by hot formed seamless processes or by forming skelp to the desired shape and joining the edges with a continuous welded seam. Published information is available describing the details of the various methods used to manufacture tubular sections (1.1, 1.2).

The division between Class A and Class B sections is based on the residual stress levels which can be expected from a manufacturing process. A cold formed rectangular section can be placed in the Class A group if it is given some form of partial stress relief. The new Canadian Standard G40.21 (1.5) is the only standard for a stress relieved cold formed product. Therefore, a manufacturer must establish a procedure which can qualify, preferably through the stress-strain relation obtained in a stub column test, that properties are similar to a Class A tube. Technical Memorandum No. 3 of the Column Research Council (1.6) presents a procedure for conducting a stub column test. In addition to defining yield, the Memorandum defines the proportional limit on the basis of an offset of 10 microin./in. For the purpose of this Criteria, a section may be considered as stress relieved if the results of three stub column tests indicate that the proportional limit is 70% or more of yield. Of course, the final dimensions of the product must be within the tolerances of standard tubular products.

Fabricated tubes are defined as sections formed by shaping plates and joining them with one or more longitudinal seam welds but not in a tube mill or in accordance with a product specification. There is good justification for excluding fabricated tubes and stiffened shells from the Criteria. The buckling strength of tubular sections is greatly influenced by geometric imperfections. Although it is certainly possible to fabricate large tubes and shells to the same degree of perfection as obtained in manufactured tubular sections, this quality is not universally assured by standard product specifications.

The Criteria brings together design guidelines from several sources. The basis for the recommendations is the design philosophy and factors of safety contained in the AISC Specification (1.3). Since much of the AISC Specification reflects the behavior of wide flange shapes, modifications have been made where tubular members have been shown to behave differently. The reasons for such modifications are explained in the Commentary.

In areas where the AISC and AISI Specifications give little direct guidance for the design of tubular structures (such as joints and thin walled members), basic research and criteria (primarily Ref. 1.4) from other sources have been used. Much of the basic research has come from the CIDECT (Comite International pour le Developpement et l'Etude de la Construction Tubulaire) programs which have sponsored many projects in Europe and Canada concerning tubular construction. These have been incorporated into this Criteria with modifications to provide safety factors comparable with those in the AISC Specification. Consequently, this Criteria is intended for the design of structures normally encountered in Civil Engineering practice and experience. When the general uncertainty in loading or quality of control is substantially different, the Criteria may not apply.

REFERENCES:

1.1 Graham, R. R., "Manufacture and Use of Structural Tubing," Journal of Metals, September, 1965.

1.2 "The Manufacture of Steel Tubular Products," publication of the United States Steel Corp., Pittsburgh, Pa., 1964.

1.3 "Specification for the Design, Fabrication & Erection of Structural Steel for Buildings," American Institute of Steel Construction.

1.4 "Specification for the Design of Cold-Formed Steel Structural Members," American Iron and Steel Institute, 1968 edition.

1.5 Canadian Standard G40.21, "Structural Quality Steels."

1.6 Johnston, B. G. (editor), "Guide to Design Criteria for Metal Compression Members," Column Research Council, (Technical Memorandum No. 3: Stub Column), John Wiley & Sons, 1966.

SECTION 2.0
PLANS AND DRAWINGS

Differences in the residual stress levels between Class A and Class B tubular sections affect the relative buckling strength of tubular members of the same size. In order to obtain the maximum advantage for each type of tubular section, separate design formulas for Class A and Class B tubular sections are given in later sections of the Criteria. Consequently, precautions must be taken in the drawings to assure that the tubular section used in construction is the same as that considered in the design of the structure. It is especially important to indicate if rectangular tubes have been designed as Class A or Class B. These tubes can be manufactured by either hot or cold forming, and cold formed rectangular tubes which are stress relieved are satisfactory for the Class A category.

There are no special differences in the quality of drawings or type of nomenclature required for tubular and other forms of structural steel construction which would preclude using the AISC or Canadian Standards (2.1, 2.2). However, it should be noted that since tubular sections have their particular advantage in space frame structures, special care may often be demanded to insure that the drawing for tubular structures adequately describes the configuration of the members in complex frameworks. This is especially true for the details of the connections.

REFERENCES:

2.1 AISC Specification, Section 1.1

2.2 Canadian Standard S16, "Steel Structures for Buildings," Section 4.

SECTION 3.0
LOADS AND FORCES

In many instances the type of framing system has no influence on the type or magnitude of the loads which 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, the Criteria makes reference to any applicable building code or to the appropriate provisions of Section 1.3 of the AISC Specification. The AISC precautionary provision for extraordinary conditions such as earthquakes and hurricanes is repeated.

There are, however, two situations in which the use of tubular sections may modify the forces which must be considered. These involve wind forces on exposed frameworks and pressure forces created by the enclosed nature of the section.

Section 3.1.2 considers wind forces on exposed frameworks which can occur either in the final structural configuration or during construction. The shape of a round tubular section offers lower resistance to fluid flow than shapes with flat elements and, therefore, reduces the wind forces. Since the AISC Specification gives very little guidance in determining the magnitudes of wind forces, the Criteria refers to the ASCE report on wind forces (3.1) when building codes do not apply.

In the ASCE report, wind forces are determined by an equation of the general form

$$\mathbf{F = CqA} \qquad (3.1)$$

where C is a coefficient

q is the dynamic pressure of the free stream $= \frac{1}{2} \rho v^2$ (lb/ft^2)

A is the exposed area (ft^2)

ρ is the mass density of air (lb-sec^2/ft^4)

v is the wind velocity (ft/sec)

For normal air the density is 0.002378 lb-sec^2/ft^4 and the force in pounds per foot of length of tube is

$$\mathbf{w = 0.000213\ CV^2D} \qquad (3.2)$$

where V is the wind velocity in miles per hour, and

D is the diameter of the round tubular section or the length of the side of a rectangular tube perpendicular to the wind in inches.

The value of C for a square shape with sharp corners is given as 2.03. Recent CIDECT studies (3.2) indicate that the rounded corners of square tubular shapes will reduce the drag coefficient. As shown in Figure 3.1, further reductions occur when rectangular sections are oriented with the short side perpendicular to the wind. However, the drag coefficient varies considerably with the orientation of the section relative to the wind and with the Reynold's number. The maximum values of the coefficient as indicated by the flat portions of the curves in Figure 3.1 are conservative values to use for design purposes. Table 3.1 lists these values along with their proportion of the typical value used for square tubular shapes.

TABLE 3.1—DRAG COEFFICIENTS

Section	Corner Radius	Wind Perpendicular to	C	C/2.03
Square tube	.05 W	Side	2.03	1.0
Round pipe		Diameter	1.25	0.62
8″ x 4″ rectangular	.05 W	Short Side	1.4	0.69
12″ x 4″ rectangular	.12 W	Short Side	1.0	0.49
10″ x 6″ rectangular	.20 W	Short Side	0.8	0.39

The preceding discussion concerns a single member exposed to the wind. The problem is more complex when trussworks of many members are involved. Effects of the solidarity ratio, shielding and wind angle must be considered, and apparently minor changes may have major effects. The recommendation of the report (3.1) is "in cases in which wind loads are of primary importance, wind-tunnel tests are advisable." In some cases it may be possible to use forces obtained from tests of other structures with nearly the same configurations. The opinion is expressed, however, that for trussed towers made of round members, drag coefficients are two-thirds those for structures with members composed of flat elements. This recommendation does not take account of the drop in drag coefficient at high Reynold's numbers in Figure 3.1. Based on the data in Table 3.1, the Criteria extends this reduction concept to include wind acting on the short side of tubes. Using the two data points for sections with the sharpest corners (square and 8" x 4" rectangular), the reduction on factor on wind force is approximated by 0.6(W/H) + 0.4 which uses the aspect ratio of the tube as the variable. The linear reduction is cut off in the Criteria at the ⅔ reduction for round sections, even though the data indicates that with a large corner radius the drag may be less than on a round section. There is no reduction of wind force on the long side of a rectangular tube.

In paragraph 3.1.3 which concerns pressure loads, it must be emphasized that there is no intention that the provisions of the Criteria should be applied to pressurized systems. The degree of inspection and proof testing of the product and construction as specified in the Criteria are insufficient to guarantee pressure containment. However, due to the enclosed nature of the tubular section, there may be rare situations in which unavoidable pressures could develop. These pressures would typically be caused by dynamic fluid conditions (wave action), fluid filling or temperature changes in the constant volume of air in a sealed member. The reduction in primary strength due to pressure is considered in Section 10 of the Criteria. Estimates of the amount of pressure are 0.433 psi per foot of water level difference between the inside and outside or 2.8 psi per 100°F change in temperature in a sealed section.

Experience has indicated that pressure due to ice inside a tubular section can be sufficient to burst the member. Therefore, it is important to avoid any possibility of freezing by providing drainage, complete sealing or other precautions during both construction or in service conditions.

REFERENCES:

3.1 "Wind Forces on Structures," ASCE Transactions, Vol. 126, Part II, paper 3269.

3.2 Hayus, F., "Drag Measurements on One Square Section and Two Rectangular Sections With Different Corner Radii," English Translation of CIDECT Report 1-NK-I-68-41, September, 1968.

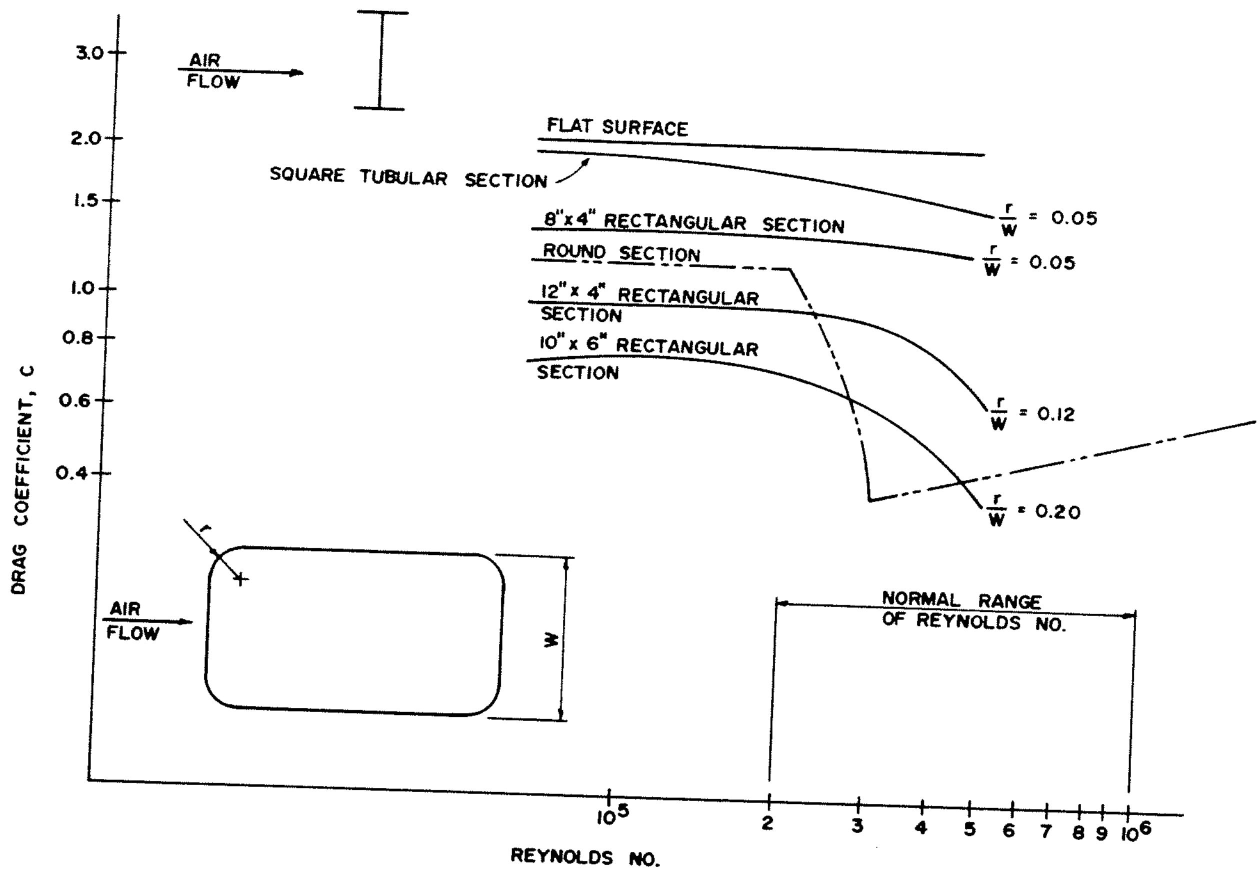

FIGURE 3.1 – VARIATION IN DRAG COEFFICIENT

SECTION 4.0
MATERIALS

Since welded connections are so important in tubular construction, the chemistry of the base metal of tubes must be controlled to insure that it is readily weldable. Except for A53 Type F, the ASTM specifications for manufactured tubes listed in the Criteria have all been approved by AWS as suitable for welded tubular structures. The Canadian standards have similar chemistry requirements and cover steels which are also readily weldable. Many specifications for pipe could also be included on the list but an attempt has been made to limit the listing to those which will be most commonly encountered in tubular construction. Material purchased under other specifications could have restricted availability.

The materials have been grouped into Class A and Class B design categories according to the level of residual stresses expected from the method of manufacture. Evidence indicates that tubes in the two classes could have different strength or behavior characteristics under axial compression, bending and torsion. Therefore, different design criteria are used in some instances for members in each class. Material meeting the Specifications listed are prequalified for design according to the criteria for the particular class with no evidence regarding the residual stress level being required. However, materials included in the list for Class B may be designed as Class A if they are properly stress relieved. In such cases the manufacturer may be required to supply evidence that the method used to stress relieve the tubes does produce a low residual stress level. The recommended method for qualifying a stress relieving procedure is through the stub column test discussed in the first section of the Commentary to this Criteria.

The specifications listed provide material having a minimum specified yield strength from 25 to 55 ksi. Structural steel normally has a minimum yield greater than 36 ksi but A53 Type F with a minimum yield strength of 25 ksi, is included because it is the most readily available tubular product and would be perfectly acceptable in many secondary structural applications. Its chemistry is such that it is also readily weldable. Another common designation for this pipe is Grade A25 of API Standard 5L.

In the strength range normally considered for most structural applications, A53 Grade B is the most readily available. Although its minimum specified yield strength is 35 ksi, its properties are similar to the familiar A36 structural steel. Equivalent material is also available—API 5L Grade B and ASTM A139 Grade B. A53 has a limiting size of 26 inches, but may be furnished in larger sizes; A139 is available in diameters up to 92 inches and API 5L include sizes up to and including 48 inch O.D.

Carbon steel rectangular or round structural tubing is manufactured in the U. S. under A500 and A501 specifications. The difference is whether the finished tube is shaped by cold or hot forming processes.The minimum strength of A500 tubing is enhanced by the cold working incurred in shaping but large residual stresses may be present in the finished product. A500 has three grades including a 50 ksi grade in shaped tubes (46 ksi in rounds). The maximum size for rectangular shapes is generally a periphery of 32 inches although some manufacturers produce shaped tubing with a 64-inch periphery.

The three grades of the A618 specifications provide tubular sections comparable to 50 ksi minimum yield strength weathering steels, ASTM A441 and A572 Grade 50 steels. Round and rectangular shapes are available under this specification. Sizes are similar to those for A500 and A501 products. High strength round pipe in larger sizes can be obtained under API 5LX specifications on special order from manufacturers.

The Canadian Standard G40.21 contains Grade 50, which is similar to A618 products, and a Grade 55 with 55 ksi yield strength. The chemistry requirements are identical in Class H or Class C categories in the standard and the difference is solely whether the tube is hot formed, cold formed, or cold formed, stress relieved.

Continuous weld and electric weld pipe and tubing, as applicable under the specifications of Section 4.1, are characterized by a longitudinal seam weld. Seamless pipe and tubing contain no welded seams and are available in sizes up to 26 inch O.D. These characterizations would be applicable to any round or rectangular tubular section in the listed specification with the exception of round pipe in larger sizes which may also be supplied with a spiral seam weld.

The portions of the Criteria dealing

with bolts and filler metal for welding are taken directly from the AISC Specification or CSA Standard S16.

SECTION 5.0
EFFECTIVE SLENDERNESS RATIOS

The high torsional stiffness and strength of a tubular section provides an increased restraining effect on members framing into it when compared to other shapes. For example, the joint between a tubular web member and a continuous tubular chord is usually formed by fully welding around the periphery of the web member. The chord, therefore, provides considerable end restraint both in and out of the plane of the truss. The tubular web 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 reduced effective lengths in equations for buckling. In Section 5.1, the use of K equal to 0.7 for web members and 0.9 for the chord between bracing points is based on the recommendations from CIDECT research (5.1). When members other than tubes are present, the provisions of 5.2 using K as unity or the use of $K = 0.9$ for buckling of continuous chords in the plane of the truss are consistent with current practice (5.2).

It is important to note that even though end restraint is present, it is still reasonable to determine the member forces with the assumptions of pinned joints in the truss and secondary stresses due to end fixity may be neglected. In a truss with the joints having 100% efficiency, the failure load depends on the axial strength of members framing into it and tests have shown that the secondary stresses have little effect (5.1). Of course, this is not true for trusses subjected to fatigue loadings.

The provisions of 5.3, 5.4, and 5.5 are taken directly from the AISC Specification. There is no evidence to show that tubes used as members of frames or secondary members would behave differently than other types of sections. Consequently, the provision of Section 1.8 of the AISC Specification are adopted along with the recommendations in the AISC Commentary for values of K and the use of the alignment chart. Recent publications (5.3) have indicated that effective length factors greater than unity may not be appropriate in many frames subject to sidesway, and modifications to the AISC provisions would be equally applicable to tubular sections as to other shapes.

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SECTION 6.0
AXIAL COMPRESSION

The strength of tubular sections in axial compression is influenced by the method of producing the tube and its shape as well as its dimensions. Consequently, an investigation of available literature reveals many different proposed equations to predict the ultimate strength. The total picture is further complicated, especially for round sections, by the large differences between theoretical predictions and experimental results for local buckling. The Criteria presents two different column curves in an effort to take full advantage of the maximum strength of the various types of tubes whenever warranted.

Rather than repeat the excellent discussions which can be found elsewhere concerning the behavior of various types of tubes (6.7, 6.9, 6.11), this commentary will summarize the results and explain the basis for the formulas in the Criteria. Some of the major considerations which must be included in a comprehensive tube criteria are the following:

1. As in any thin-walled member with constant cross section, both overall and local buckling in either the elastic or inelastic range can be the cause of compressive failure. Overall, or primary, buckling strength under axial load is governed by the familiar slenderness ratio (KL/r), the yield strength, residual stresses and initial out-of-straightness. Local buckling of rectangular tubular sections is based on the principles of plate buckling theory and is governed by the ratio of width to thickness. In very short round sections, local buckling is similar to an infinitely wide plate and the ratio of length to thickness is of prime importance. For longer round sections, the buckled configuration consists of approximately square waves along the length and around the circumference. The strength is a function of the ratio of the diameter to thickness.
2. Cold-formed rectangular sections, formed from round tubes, which are not stress relieved have a rounded stress-strain curve and residual stresses varying from about 40 to 80% of the yield stress which reduce the primary mode buckling strength of intermediate length members below that of hot-formed sections with similar yield strength.
3. Cold-formed round and all hot-formed tubular sections have lower residual stresses and are, therefore, stronger in the intermediate lengths in the primary buckling mode.
4. The local buckling strength of round tubular sections is extremely sensitive to initial distortion from the perfect cylindrical surface. Manufactured sections generally have less initial distortion than a fabricated cylinder and, consequently, local buckling occurs at higher stresses.

Section 6.1 deals with the strength in the primary buckling mode. For members meeting one of the requirements on thickness, local buckling will not occur below the yield strength and primary buckling is the only mode which need be considered in design. Both the diameter/thickness limit for round sections and the flat width/thickness limit for rectangular sections are based on the AISI Specification (6.14). For rectangular tubes, the AISI Specification prescribes a limit of $184/\sqrt{f}$ where f is defined as the actual compressive stress. Setting f equal to 0.6 F_y as the basic allowable compressive design stress, leads to a limiting width/thickness ratio of $238/\sqrt{F_y}$ which appears in the AISC Specification (6.13) for box sections. The flat width is only the flat portion of the flange, exclusive of the corner radius. In the absence of a knowledge of the actual flat width, it may be taken as the full width less three times the wall thickness.

From the summary of conclusions listed above, it is apparent that residual stresses are an important factor in the design of tubular compression members. The CRC Guide (6.6) contains a detailed description of how residual stresses affect the member capacity in any shape. In recent years, several test programs on tubular compression members have been undertaken. Although some work is still in progress, enough data has been collected to verify that the manufacturing process affects the compressive strength of tubes. Test data for tubular sections in axial compression are shown in Figures 6.1 for round sections and Figure 6.2 for rectangular sections. Table 6.1 summarizes the studies which led to the data in the Figures. It should be noted that solid points and crosses represent data from tests on cold formed tubes while open circles and squares are for hot formed tube studies. Both plots are non-dimensional to the same scale for ease of comparison. The yield load used to non-dimensionalize the ordinates is the section area times the tensile test yield strength. The 0.2% offset tensile yield strength, according to ASTM designation A370, was used because it is a standard for yield in structural steels which do not have a distinct yield point. For consistency, the tensile yield is also used for the hot formed sections, except for the European data on rounds, where the values from CIDECT curves showing the mean and two standard deviations were reproduced. The solid curves on the plots are the equations in 6.1.1 of the Criteria without the factor of safety while the dashed curves are the design equations.

The data in Figure 6.2 for rectangular tubes indicates a rather clear distinction between hot and cold formed tubes. Analytical studies which consider the effects of residual stresses in rectangular tubes (6.11) and both residual stresses and out-of-straightness (6.1,6.2) also indicate that differences of up to 20% can exist between hot and cold formed products. On the basis of this information, two column curves are used in Section 6.1.1. The data on round tubes in Figure 6.1 does not indicate as clear a distinction between hot and cold formed sections. It should be noted that much of the cold formed data (that from reference 6.12) is for thin walled mechanical tubing, (.040 in. to .097 in. thick) which today is not normally used for structural purposes. In most cases, for the products tested there is little difference between the tensile and compressive yield. However, the series of 2 inch galvanized pipe had 20% difference, resulting in the low data points for hot formed tubes in Figure 6.1. These points still follow the general shape of the Class A curve. Based on the data presented in Figure 6.1, all round tubes are placed in the Class A category. However, for small diameter mechanical tubing sections with thin walls, a designer could conservatively design cold formed round sections according to Class B criteria.

The use of different column curves for various products has been proposed and used elsewhere. Several column curves for ultimate strength which have been proposed in literature are plotted in Figure 6.3 along with the reference source. Curve A is the 1960 CRC column curve (6.6) and is the basis of the AISC equation (6.13). It is based on Bleich's (6.3) parabolic equation to include residual stresses

$$F_{cr} = F_y - \frac{F_r}{\pi^2 E}(F_y - F_r)\left(\frac{KL}{r}\right)^2$$

with the residual stress level (F_r) taken at 50% of yield. This curve fits the hot formed data reasonably well. The linear curve B is based on the experimental data for cold formed members (6.9). These two curves have been selected for use in the Criteria for the following reasons:

1. They are readily adaptable to the form of the present AISC Specification in that they involve a single equation which is applicable until the curve meets the Euler curve. Curves C through E are generated numerically and require up to five equations to describe them.

2. The variable factor of safety concept used in the AISC Specification can be readily applied to curves A and B. This factor of safety varies from 1.67 to 1.92 and since the higher value is used for the Euler curve, it reflects the lower strengths at intermediate lengths indicated by the other curves and some of the test data. The linear variation in factor of safety used in 6.1.1 for curve B is recommended in recent drafts of the CRC Guide.

Section 6.2 contains provisions for round tubular members in which local buckling is a possible mode of failure. If the member is sufficiently long, general buckling may still occur and this condition must be checked. For local buckling, the provision of the AISI Specification (6.14) is used. As noted earlier, local buckling strength of round sections is extremely sensitive to initial imperfections (6.4). The equation in (6.2.b) is based on test data for manufactured sections. Beyond the upper limit of $13000/F_y$ for D/t, the strength decreases very rapidly. Also, an equation for elastic buckling which is based on data for fabricated tubes is (6.6).

$$F_{cr} = \frac{8000}{D/t}$$

With a factor of safety of 1.6, this equation gives an allowable stress for local buckling which is less than the equation in (6.2.b) in some instances. This point emphasizes the necessity for strictly adhering to manufacturing tolerances for thin sections and for insuring that the member will not be dented during shipping or construction. It is also inherently assumed that unreinforced perforations do not exist. Figure 6.4 shows the relation of primary and local buckling for a material with 50 ksi yield strength. The solid curve is the variation in allowable stress to prevent local buckling while the scales on the right indicate the allowable stress variation with effective slenderness based on the primary buckling equations of Section 6.1.1. Primary buckling will generally govern except for short columns with thin walls.

The procedure in Section 6.3 for rectangular tubes follows the concepts of Appendix C of the AISC Specification. These concepts recognize that the local buckling of the walls of a rectangular member does not terminate the load carrying ability of a member but does interact to reduce the critical load for general buckling. The form factor is effectively a reduction in the yield strength in Appendix C of AISC. The Criteria modifies Appendix C in that the effective yield strength, Q_aF_y, is applied to the equations for tubular sections rather than the AISC column equations.

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TABLE 6.1
SUMMARY OF AXIAL TEST PROGRAMS ON TUBULAR COLUMNS

ROUND

Ref.	Type	Size	Thickness	No. of Tests	Symbol in Fig. 6.1
6.12	Galv. Pipe	1″ to 3″	.092″-.134″	30	¤
	CF, low carbon	3/8″ to 3″	.042″-.086″	136	▲
	CF, medium carbon	1″ to 3″	.040″-.097″	58	+
CIDECT	HF Seamless	3½″ and 4¾″	.197″-.217″	89	○
	HF welded	4½″	.250″	10	○-
	CF welded	3½″ x 10¾″	.126″-.280″	65	●

RECTANGULAR

Ref.	Type	Size	Thickness	No. of Tests	Symbol in Fig. 6.2
6.5	HF	3½″ x 3½″ to 10″ x 10″	3/16″-1/2″	10	△
6.10	CF stress relieved	2″ x 4″	.1875″	2	×
CIDECT	HF seamless	5″ x 5″	.1875″	65	▭
	HF welded	3″ x 3″ and 3½″ x 3½″	.193″ .250″	88	▭-
	CF welded	3″ x 3″ to 8″ x 8″	.110″-.375″	132	■

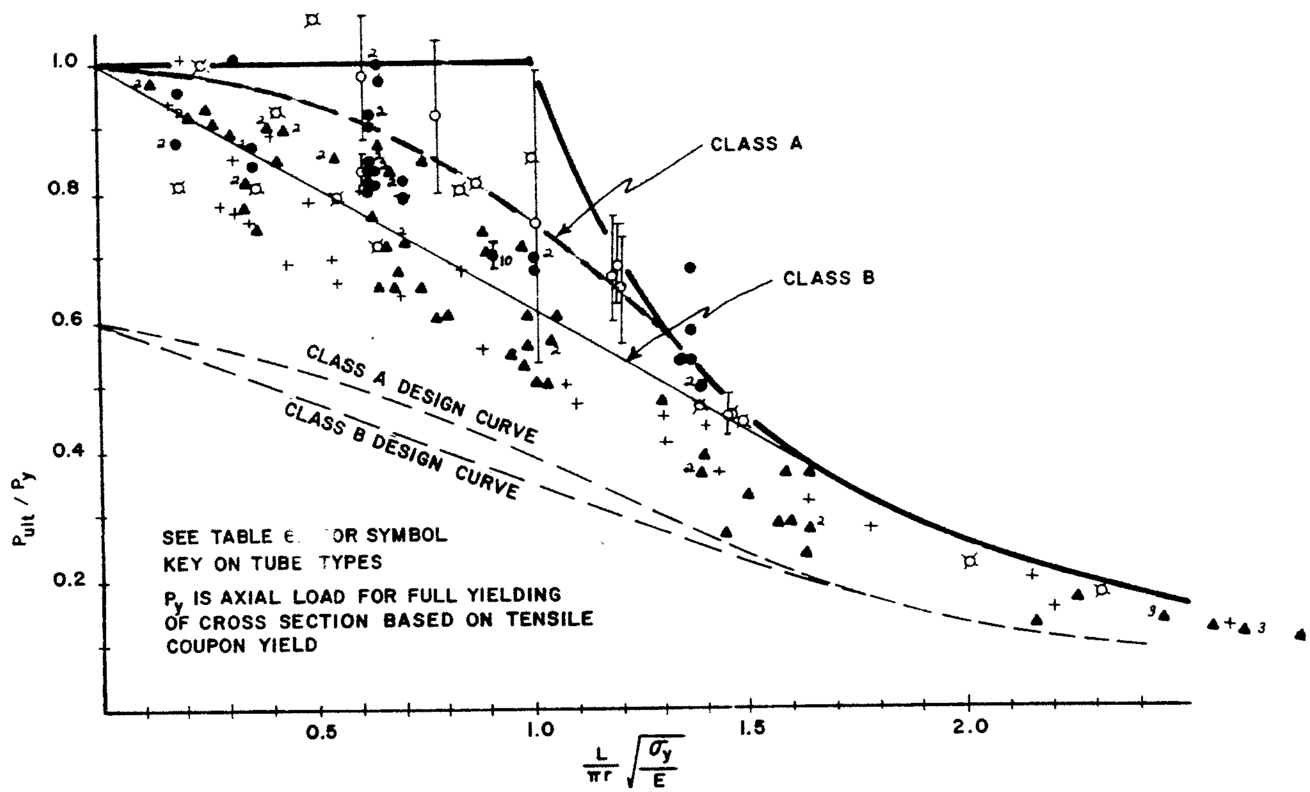

FIGURE 6.1 - TEST DATA FOR AXIALLY LOADED ROUND TUBES

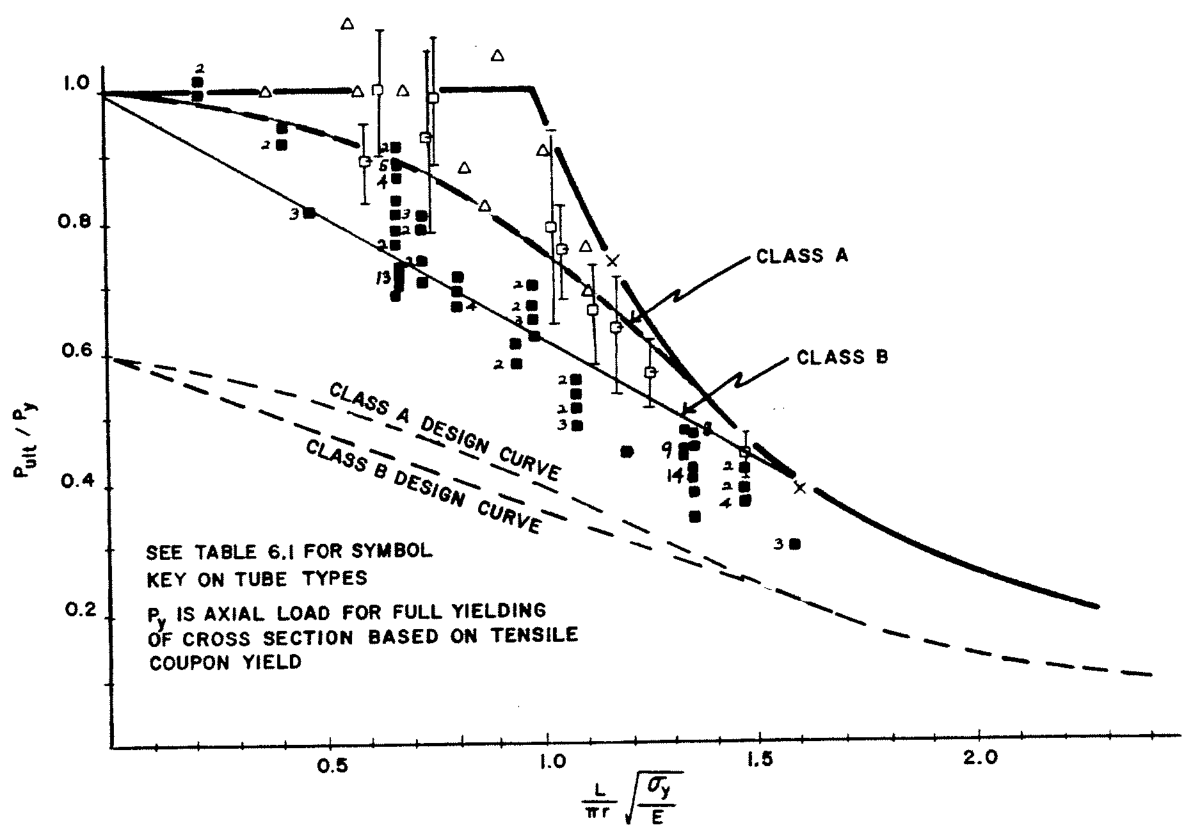

FIGURE 6.2 - TEST DATA FOR AXIALLY LOADED RECTANGULAR TUBES

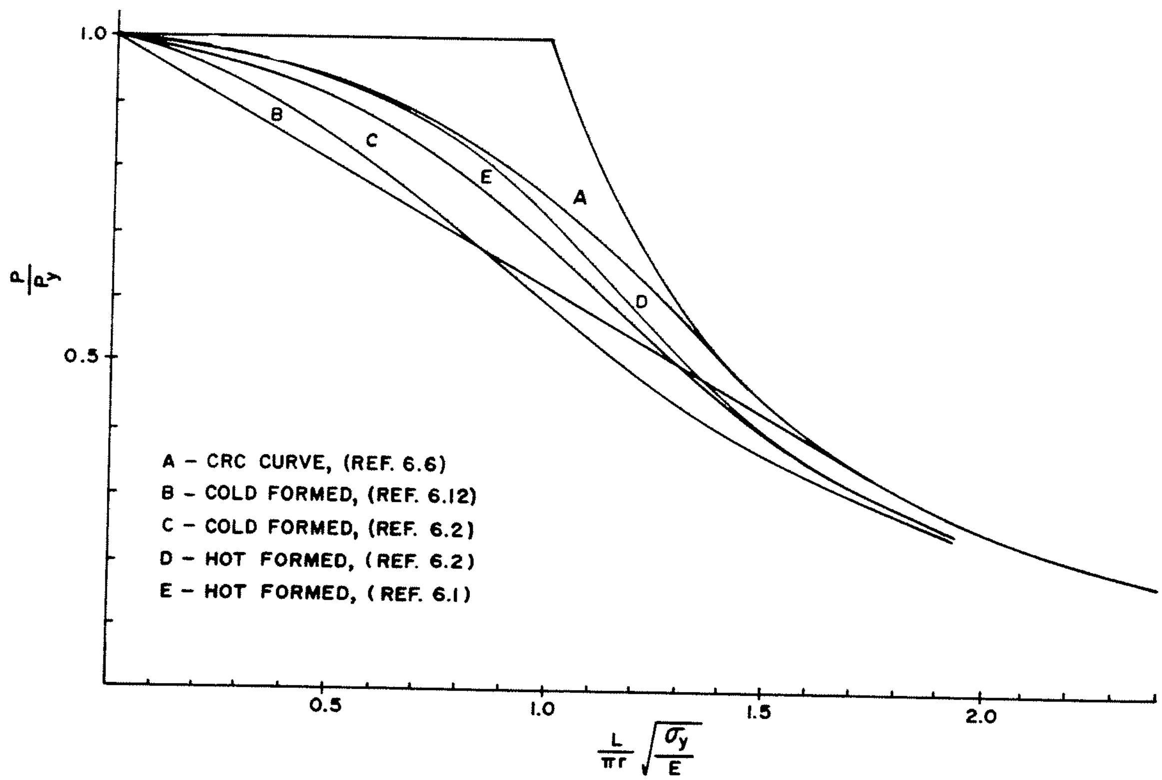

FIGURE 6.3 - COLUMN CURVES PROPOSED FOR TUBULAR SECTIONS

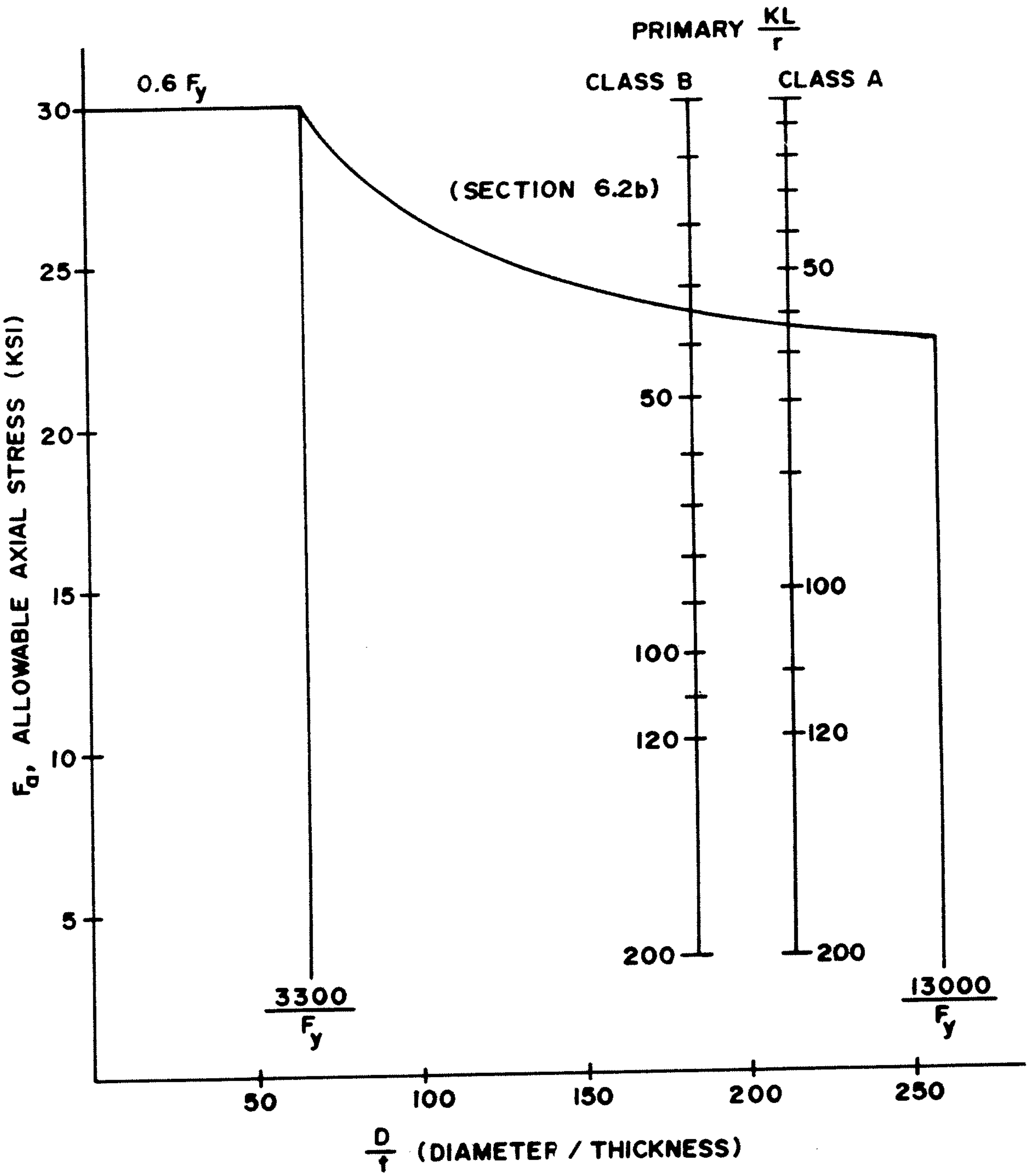

FIGURE 6.4 – ALLOWABLE STRESS IN THIN WALLED ROUND SECTION (F_y = 50KSI)

SECTION 7.0
BENDING

The provisions for the allowable normal bending stresses are based on the several possible modes of failure which can occur in tubular members. These include plastic mechanisms, inelastic local buckling and elastic local buckling. Round members are covered in Section 7.1 while rectangular members are covered in Section 7.2. Both sections are complete in that they include provisions for thick and thin walled members. Section 7.3 treats shearing stresses due to bending in both round and rectangular tubular sections.

Round sections subjected to bending reach their ultimate capacity in one of three basic failure modes. For very thick sections the compressive capacity of the material is reached, which for steel means that large distortion occurs with no drop-off in the load. Thinner round sections fail by excessive ovalization of the cross section. This is a type of inelastic instability problem in which the decrease in moment capacity caused by the reduction in the section modulus due to flattening occurs more rapidly than the increase in moment. Finally, very thin cylinders fail in a diamond shaped local buckling pattern. The division between ovalization and local buckling is taken as $3300/F_y$ which is the same as that recommended in the AISI Specification (7.6).

The concept of plastic design based on developing the full plastic moment and rotational capacity to redistribute moments has recently been investigated for hot formed tubes (7.4). It is concluded that ovalization will not impair the development of the plastic hinge in tubes with D/t less than $1300/F_y$. This study has not been extended to cold formed tubes. However, since first yielding occurs only locally at the extreme fiber and the shape factor is greater than 1.30 as opposed to lower values for a wide flange shape, some increase in allowable stress beyond the AISC basic value of $0.6F_y$ used for wide flange shapes is warranted for all round tubes. The theoretical elastic stress at which failure by ovalization would occur is far above the yield strength of the material covered in the Criteria. Therefore, it is assumed that the sections will be able to develop a moment capacity equal to the moment at first yielding times the shape factor and the allowable stress is consequently increased to $0.72\ F_y$. Since this is a minimum strength concept, it applies to both hot and cold formed sections. However, in recognition that plastic behavior is not the basis for the increased allowable stress and that ovalization may limit the rotational capacity of the inelastic member, the provision for redistribution of the moment is limited to Class A tubes with small D/t.

Recent studies have indicated that there is no significant increase in the local buckling strength of cylinders subjected to bending over that for axially compressed cylinders as was suggested in earlier literature. Therefore, the allowable stress for thin round sections is obtained by using the equation of Section 6.2.b.

The provisions for rectangular members are based on the concepts of the AISC Specification (7.5). Section 7.2.1 defines compact sections which can develop the full plastic moment capacity. CIDECT research (7.2) indicates that cold formed sections meeting the compact section limitations of section 7.2.1 will develop a plastic moment and, therefore, a higher allowable stress is permitted. The allowable stresses in Section 7.2.1 reflect the variation in shape factor for the type of member. A higher proportion of the material is in the web of tubing as opposed to wide flange sections, resulting in higher shape factors. Figure 7.1 is a histogram showing the variation in shape factors for the standard rectangular shapes. For rectangular members bending about the major axis, a lower limit of 1.2 is used and the basic 0.6 F_y AISC allowable stress is increased 20% to 0.72 F_y. The square sections have lower shape factors and the increase is limited to 15%. Rectangular sections bending about the minor axis are close to the web-flange proportions of a wide flange member and the 10% increase for compact sections in the AISC Specification is used. The AISC provision for redistribution of the moments is applied to all Class A compact sections and to certain Class B sections which meet a more stringent width/thickness limitation of $150/\sqrt{F_y}$. This departure from AISC, which permits redistribution for only hot formed members, is also based on CIDECT research (7.2) that shows plastic failure mechanisms will occur in cold formed beams meeting the heavier wall thickness limitation.

Two changes from the 1969 AISC Specification are made in the requirements for a compact section; the w/t limit and the lateral bracing requirement. Raising the w/t limit from $190/\sqrt{F_y}$ to $210/\sqrt{F_y}$ is again based on the CIDECT research. The lateral bracing change is based on the fact that the tubular shape has superior torsional stiffness than the wide flange shape and consequently much less tendency for lateral-torsional buckling. This has been recognized in Supplement No. 3 to the AISC Specification (7.7). Rectangular members bending about the minor axis are theoretically not subject to lateral buckling and, therefore, the new AISC bracing requirements for compact box-shaped members is applied in the Criteria only to tubes bending about the major axis. The AISC limits compact sections to shapes whose depth is not more than six times the width, but no manufactured tube exceeds this limit. Since these lateral bracing requirements are new and not fully discussed in published literature, a theoretical discussion following a development similar to that used by Galambos (7.1) for wide flange shapes is presented for rectangular shapes.

In the rectangular section, the pure torsional stiffness is larger than the warping stiffness as opposed to the wide flange section for which the reverse is true. Therefore, the critical moment for lateral torsional buckling can be expressed as

$$M_{cr} = \frac{\pi}{L}\sqrt{EI_yGK} \qquad (7.1)$$

in which the pure torsional stiffness factor is approximately

$$K = \frac{2W^2H^2t}{W+H} \qquad (7.2)$$

and

L is the unbraced length
E is the tensile modulus of elasticity
G is the shear modulus of elasticity
I_y is the moment of inertia about the the minor axis.

To obtain the condition needed for a plastic hinge, it is assumed that the material is fully in the strain hardened region in the vicinity of the hinge (constant moment) and that equation 7.1 becomes

$$ZF_y = \frac{\pi}{L}\sqrt{E_s G_s I_y \frac{2W^2H^2t}{W+H}} \qquad (7.3)$$

in which Z is the plastic section modulus and the subscript "s" indicates properties in the strain hardened region.

Equation 7.3 can be rearranged as

$$\frac{LF_y}{W} = \frac{\pi H}{Z}\sqrt{E_s G_s}\sqrt{I_y \frac{2t}{W+H}} \qquad (7.4)$$

Using $E_s = 900$ ksi and $G_s = 3100$ ksi, the values of LF_y/W as a function of w/t for constant values of W/H are shown in Figure 7.2. Also super-imposed in this figure are the limits of maximum w/t to prevent local flange buckling of a compact section, the range of H/t limits for the compact section, and the bracing requirements in the AISC Specification. Based on this analysis, the AISC requirement appears conservative for rectangular tubes. The new AISC bracing requirement

$$\frac{L}{W}F_y = 1950 + 1200\frac{M_1}{M_2} \qquad (7.5)$$

takes advantage of a moment gradient but does not consider the increased resistance to lateral buckling as the W/H ratio approaches unity.

All members which meet the requirements listed in Section 7.2.2 are designed to the AISC allowable bending stress of 0.6 F_y. The proper lower limiting value of w/t is difficult to determine for the non-compact section. AISI (7.6) requires $184/\sqrt{.6\ F_y} = (237/\sqrt{F_y})$ specifically for rectangular tubes, AISC (7.5) requires $238/\sqrt{F_y}$ for box sections and the CIDECT research (7.2) recommends $245/\sqrt{F_y}$. The value $245/\sqrt{F_y}$ has been selected because

1. It reflects the most recent research on rectangular products.
2. It provides a reasonable range of w/t between compact sections and sections requiring stresses reduced due to effects of local buckling.
3. The transition to reduced allowable stresses for members with greater w/t involves a step reduction in allowable stress of only 5% or less as shown in Figure 7.4.

As in the new AISC provisions, no lateral bracing is required since tubes are not manufactured with H/W greater than six.

The limit on depth to thickness ratio is that used by AISC for reduction in flange stress (7.5 Sect. 1.10.6). It is not exceeded by any of the currently available standard rectangular tubes.

Appendix C of the AISC Specification forms the basis for the allowable stress for thin walled rectangular shapes. The theory in Appendix C can be simplified when it is applied only to box sections with constant thickness. Appendix C permits an allowable bending stress of 0.6 F_y if the stress is computed using the section modulus in Figure 7.3.a) or b). The effective width of the compression flange is

$$b = \frac{253t}{\sqrt{f}}\left(1 - \frac{50.3}{(w/t)\sqrt{f}}\right) \leq w \qquad (7.6)$$

where f is the compressive stress in the flange. This becomes

$$b = \frac{327w}{\frac{w}{t}\sqrt{F_y}}\left(1 - \frac{64.9}{\frac{w}{t}\sqrt{F_y}}\right) < w \qquad (7.7)$$

when the stress level is 0.6 F_y.

The quantity Q is defined as the ratio of the effective section modulus to the full elastic section modulus, or $Q = S_e/S$. The requirements of Appendix C can, therefore, be expressed as

$$0.6\ F_y = \frac{M}{QS} \qquad (7.8)$$

or alternately as used in the Criteria

$$0.6\ QF_y \geq \frac{M}{S} \qquad (7.9)$$

For the thin walled sections, an expression for Q is easily formulated. The area which is no longer effective is shaded in Figure 7.3.a) and its magnitude is

$$g = (w - b)t = wt\left(1 - \frac{327}{\frac{w}{t}\sqrt{F_y}}\left(1 - \frac{64.9}{\frac{w}{t}\sqrt{F_y}}\right)\right) \qquad (7.10)$$

The neutral axis of the member will shift downward by an amount equal to

$$y = \frac{gH/2}{A - g} \qquad (7.11)$$

(This assumes $H/2 \approx \frac{1}{2}H - \frac{1}{2}t$)

The effective section modulus is

$$S_e = \frac{I + Ay^2 - g(\frac{1}{2}H + y)^2}{\frac{1}{2}H + y} \qquad (7.12)$$

The factor Q is obtained by dividing the effective section modulus by $I/\frac{1}{2}H$, which upon substitution of equation 7.11 for **y** can be algebraically reduced to

$$Q = 1 - \frac{g}{A} - \frac{g(\frac{1}{2}H)^2}{I} \qquad (7.13)$$

If the alternate effective section shown in Figure 7.3.b) is used, there is no shift in the neutral axis and

$$Q = 1 - \frac{2g(\frac{1}{2}H)^2}{I} \qquad (7.14)$$

The approach in the Criteria is based on the idea that it is simpler in design to use the full section properties to calculate the bending stresses and reduce the effective yield stress rather than work with an effective section modulus. Two important factors should be noted however. The first is that the allowable stress is only an average and the maximum compressive stress is actually 0.6 F_y at the corners. Second, if the maximum tensile stress must be computed for fatigue or other considerations, it is less than 0.6 F_y by a factor of $(1 - 2g/A)$.

Figure 7.4 shows a typical variation in the allowable bending stress as a function of w/t.

The criteria for shear stresses in Section 7.3 due to bending in round tubular members makes no provision for the use of stiffeners to prevent shear buckling or for reduction in allowable stress to account for buckling. Therefore, it is assumed that the thickness limitations will allow the full yield strength in shear to be developed.

For round sections the allowable stress is the same as the AISC Specification and the stress is calculated by $f_v = VQ/It$. For a thin round section, I is $\pi R^3 t$ and Q at the centroid is $2R^2 t$. This gives $f_v = V/(\pi Rt/2)$, in which the denominator is recognized as half the area of the tube.

The criteria for the limiting D/t ratio is based on equating the critical elastic buckling shear stress to the yield stress in shear, taken as 0.6 F_y. Roark (7.3, Table XVI, case N) notes that very short cylinders under transverse shear may fail by buckling at the neutral axis when the shear stress there reaches a value 1.25 times the critical shear stress due to torsion. For short tubes with $L/D < 2.5$, the critical stress in torsion is given by the empirical expression.

$$(f_v)_{cr} = 0.2E\frac{t}{D} + 5E\left(\frac{t}{D}\right)^2 \qquad (7.15)$$

At the longest length for which the formula applies (2.5D), it is slightly conservative to neglect the second term. Neglecting the second term becomes very conservative for shorter lengths. Therefore, a safe limit of D/t to prevent elastic buckling due to shear at the neutral axis is obtained by

$$1.25\left(0.2E\frac{t}{D}\right) = 0.6\ F_y \qquad (7.16)$$

which is solved for

$$\frac{D}{t} = \frac{12100}{F_y} \qquad (7.17)$$

It should be noted that for very thin long tubes, the critical torsional stress could be considerably less.

$$(f_v)_{cr} = 0.825E\left(\frac{t}{D}\right)^{3/2} \qquad (7.18)$$

If this expression were used in place of equation 7.15, it would lead to D/t limits of approximately half those given by equation 7.17. However, it has been assumed that shear buckling limitations would be a governing factor only in very short members or where the shear gradient is steep. In thin tubes with the maximum shear stress constant along the length, bending stresses will become critical at lengths less than 2.5D. Therefore, the equation for short tubes is satisfactory.

In rectangular beams there is again no provision for stiffeners. However, a provision is included for a reduced allowable stress when the webs are thin. As is common for wide flange members, the shear stress is calculated by an approximate expression using just the web area. It can be shown that the approximate expression gives a stress lower than the true maximum by a factor equal to the shape factor of the section. Therefore, in order to maintain the same safety factor as the AISC Specification, the allowable stress has been reduced by the ratio of the average shape factors for wide flange shapes to those for tubes, about 10%.

The limiting clear depth to thickness ratio corresponds to the limit in the AISC Specification above which stiffeners would be provided (article 1.10.5.2). Modifying the AISC equation to account for the change in allowable stress, the following expression is obtained for unstiffened limit,

$$\frac{F_y}{2.89} C_v = 0.36\ F_y$$

where $$C_v = \frac{190}{h/t}\sqrt{\frac{5.34}{F_y}} \qquad (7.19)$$

When this is solved for $h/t = 425/\sqrt{F_y}$, it provides a limit to insure that the full allowable shear stress can be reached without buckling. For thin sections the expression for reduced allowable stresses is obtained from Section 1.10.5.2 of the AISC Specification assuming there are no stiffeners and the aspect ratio of the webs is very large.

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7.4 Sherman, D. R. and Glass, A. M., "Ultimate Bending Capacity of Circular Tubes," Proceeding Offshore Technology Conference, OTC 2119, May, 1974.

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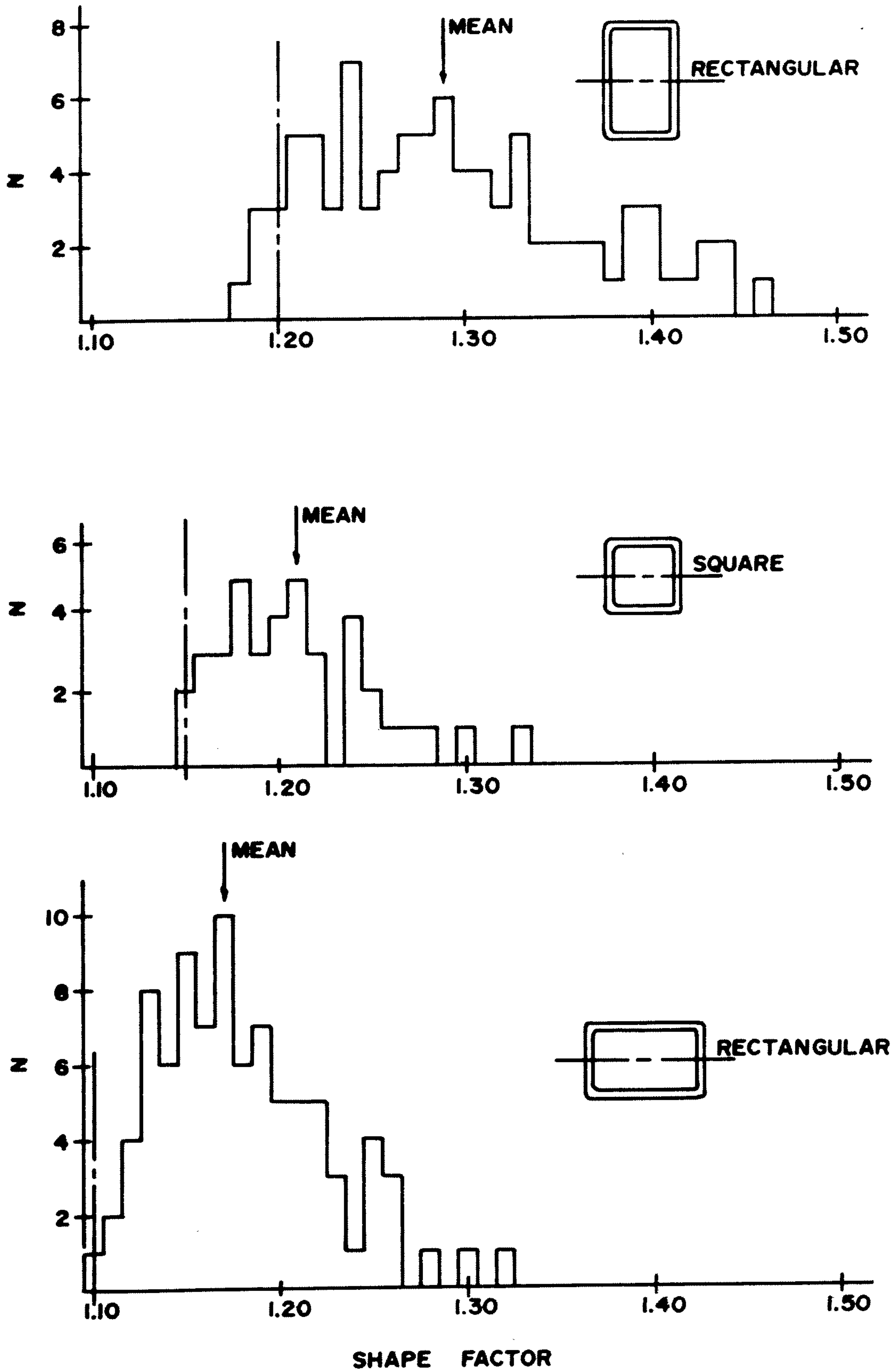


FIGURE 7.1 - DISTRIBUTION OF SHAPE FACTORS IN STANDARD RECTANGULAR TUBULAR SECTIONS

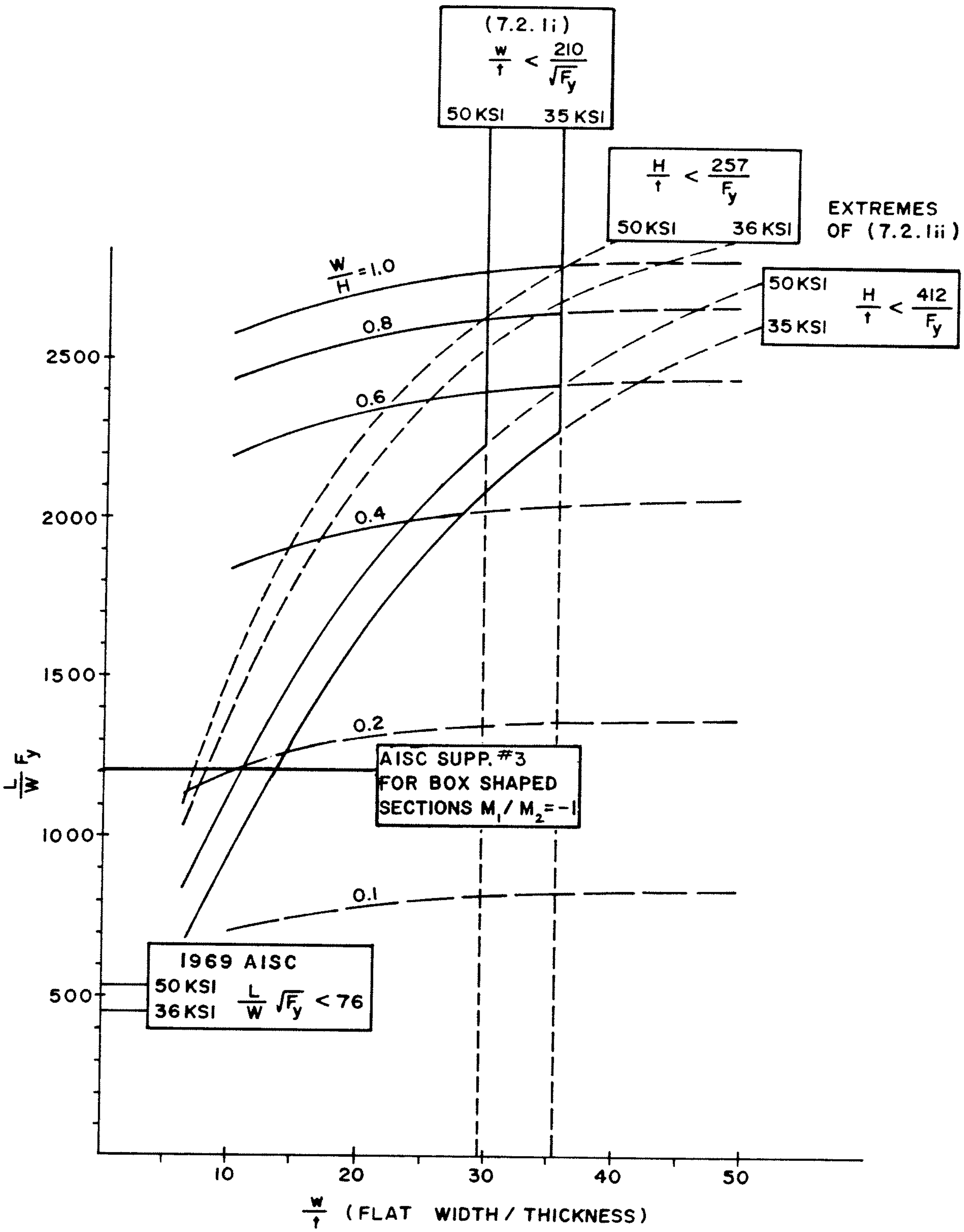
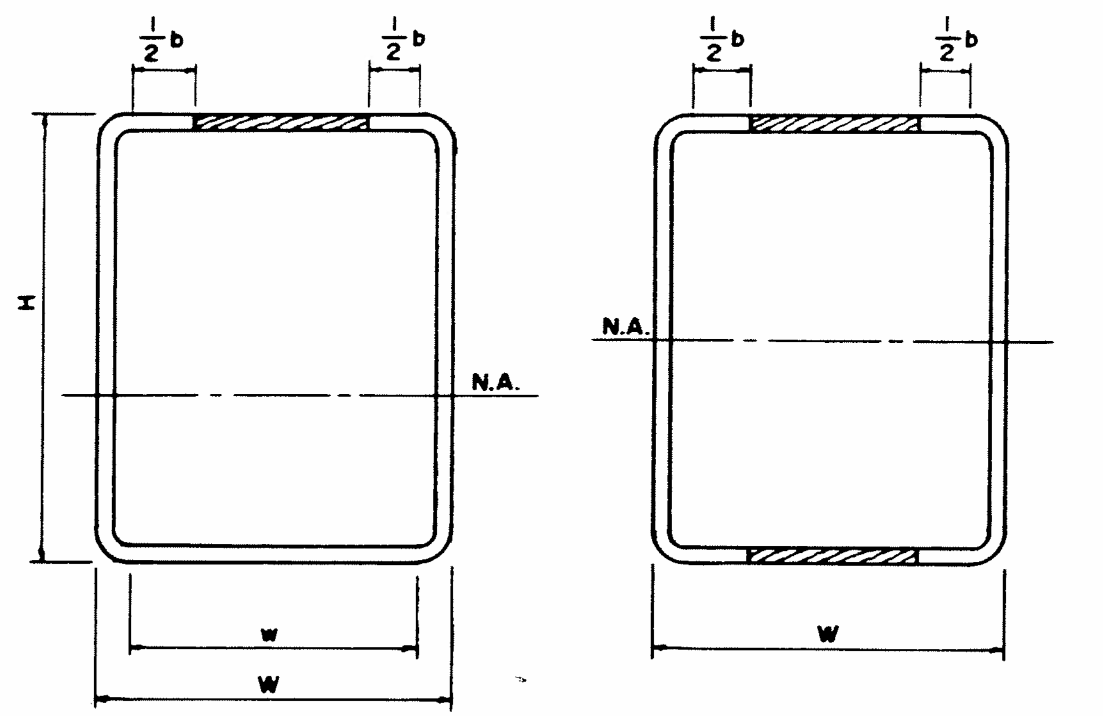


FIGURE 7.2 — THEORETICAL CRITICAL BRACING LENGTH FOR PLASTIC MOMENT IN RECTANGULAR TUBES WITH $M_1 / M_2 = -1$



a) EFFECTIVE SECTION b) PERMISSIBLE APPROXIMATION

FIGURE 7.3 - EFFECTIVE SECTION IN THIN WALLED RECTANGULAR TUBES

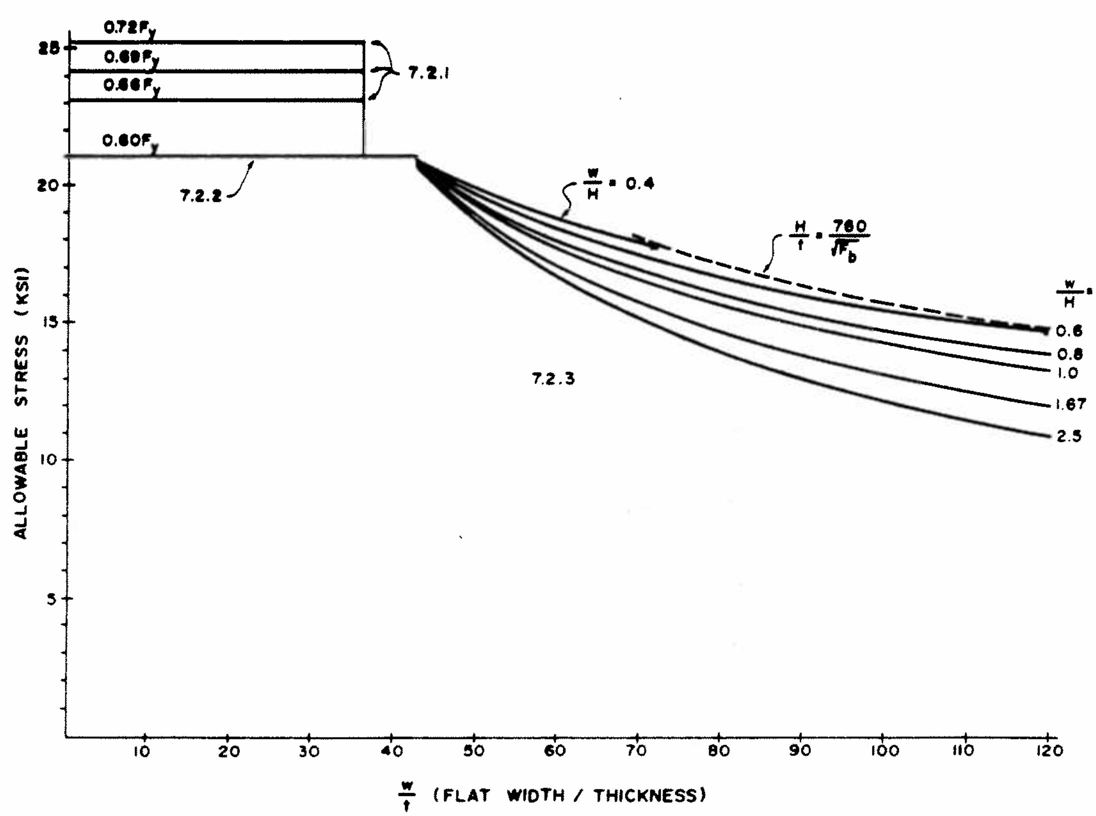


FIGURE 7.4 — ALLOWABLE BENDING STRESS IN THIN WALLED RECTANGULAR TUBES (F_y = 35 KSI)

SECTION 8.0
TORSION

A primary use of tubular members is in space frame construction where significant torsional moments must often be carried by the members. The closed cross section of a tube makes it far more efficient in resisting torsion than an open section such as a wide flange or channel shape. The patterns of shear flow due to torsion in a closed and open shape are compared in Figure 8.1. If the same torsional moment is resisted, the fact that the moment arm between opposite shears is far greater in the closed shape means that the shear stress will be much less. While normal stresses due to warping of the cross section are a significant factor in resisting the torsional moment in an open section, they are very small in a closed section and the total moment can be assumed to be resisted by shear stresses.

In thin walled tubular sections, the shear stress is assumed to be uniformly distributed. For round sections, the shear stress is given by

$$f_v = \frac{2T}{\pi(D-t)^2 t} \qquad (8.1)$$

while for rectangular sections, it is

$$f_v = \frac{T}{2(w-t)(H-t)t} \qquad (8.2)$$

Steel tubular sections have not been investigated in torsion to the extent that they have been in compression and bending. It appears, however, that the local buckling strength is not as sensitive to initial imperfections. Although the effects of normal imperfections are included in the empirical coefficients of the equation of Section 8, the difference between Class A and Class B sections is more important, and this distinction is made.

Section 8.1 is concerned with round sections while 8.2 deals with rectangular sections. Provisions for thin walled members where local buckling may occur are included in both sections.

In Section 8.1, when considering local buckling of round tubular sections subjected to torsion, most structural members will be either long or of moderate length. The elastic buckling of long tubes is uneffected by end conditions and the critical stress is (8.1)

$$\tau_{cr} = \frac{K_s E}{(D/t)^{3/2}} \qquad (8.3)$$

The theoretical value of K_s is 0.73 but a value of 0.6 is generally recommended to take account of initial imperfections (8.1). Schilling (8.2) recommends an equation for the critical elastic stress for tubular sections of moderate length, which are affected by end conditions.

$$\tau_{cr} = 1.23\,\frac{\sqrt{D/L}\;E}{(D/t)^{5/4}} \qquad (8.4)$$

This equation also includes a reduction for initial imperfections and represents a simply supported end condition, i.e.: the wall is free to rotate at the end.

Equations 8.3 and 8.4 are plotted in Figure 8.2. Although there is some inconsistency in the literature concerning the division between long and moderately long tubular sections, it appears from Figure 8.2 that the expressions for moderate length sections are valid for most practical lengths. It is also evident that it would be extremely uneconomical to neglect the increase in torsional capacity for moderate length members. Therefore, the provisions of the Criteria include the length effect for simple end conditions but neglect the approximately 10% increase in buckling strength if the ends were fixed.

For tubular sections which have a distinct yield point, Schilling (8.2) recommends that the elastic equation can be used up to the shear yield strength of the material. For thicker sections, design can be based on the yield strength. The Criteria follows this recommendation in Section 8.1(a) and uses an allowable shear stress of 0.4 F_y for tubes with D/t less than the intersection of the elastic curves and the assumed shear yield of 0.6 F_y. The allowable stress of 0.4 F_y is the same as the AISC provisions for shear in members which will not buckle elastically due to shear. For thin Class A round tubular sections, the allowable stress in Section 8.1.2a is based on equation 8.4 reduced by a factor of $\frac{0.4}{0.6} = \frac{2}{3}$ to keep the same factor of safety as thick sections. Equation 8.3 for long tubular sections is used as a lower limit for both the D/t cutoff and allowable stress in thin sections.

In Class B tubular sections with a rounded stress-strain curve the elastic buckling expressions are reduced by a plasticity factor. Schilling (8.2) uses the ratio of secant to elastic modulus, E_s/E, which requires a knowledge of the stress-strain curve and involves a successive trial design procedure. Felton and Dobbs (8.1), however, in working with long tubular sections with rounded stress-strain curves, found that inelastic buckling data could be approximated by a linear variation with D/t from the 0.75 F_y intercept to 0.30 F_y on the curve of equation 8.3. This is shown in Figure 8.3. The assumption has been made in the Criteria, that a linear relation could also be used to modify equation 8.4 for inelastic action. Since inelastic action would tend to eliminate the effect of length, the linear transition for moderate tubes begins at the intersection of the transition for long tubular sections and the shear yield strength of the material, 0.6 F_y. Thick Class B tubular sections are defined in Section 8.1.lb as the D/t corresponding to this intercept.

The other end of the linear transition is again at a stress of 0.3 F_y in equation 8.4. In Section 8.1.2b, the allowable stress for thin tubular sections is based on the linear transition and equation 8.4 with the ⅔ reduction for consistent factor of safety. The D/t division between the two expressions corresponds to a stress of 0.3 F_y in equation 8.4. The derivation of the expression for the linear transition is somewhat involved algebraically and uses an approximation that $(E/F_y)^{2/15}$ is 2.35 for the range of materials included in the Criteria.

Figure 8.4 shows how the allowable stress varies with D/t and L/D for Class A and Class B materials with 35 ksi yield strength. As in the case of bending or compression, this plot is intended to illustrate the effect of the equations and it is recognized that cold formed members would have a higher yield strength.

In Section 8.2 which deals with rectangular tubular sections, the allowable stresses for torsion are based on the AISC provisions for shear in beams with unstiffened webs (8.3, Section 1.10.5.2). The shear distribution due to torsion is uniform in the longest sides which is the same distribution assumed in girder webs. (Actually, the maximum stress in the girder web is slightly higher.) Therefore, it seems reasonable that the criteria for elastic buckling would be nearly the same in both cases.

A shear stress of 0.4 F_y is used as

the basic allowable stress if the full yield strength can be developed. The limiting length to thickness ratio is obtained by solving the AISC equation

$$F_v = 0.4\ F_y = \frac{F_y}{2.89}(C_v)$$

where $C_v = \dfrac{440}{H/t\ \sqrt{F_y}}$, which applies to a side with a large aspect ratio. For thin sections the expression for reduced allowable stress in the Criteria is obtained from the AISC Specification assuming there are no stiffeners and the aspect ratio is very large.

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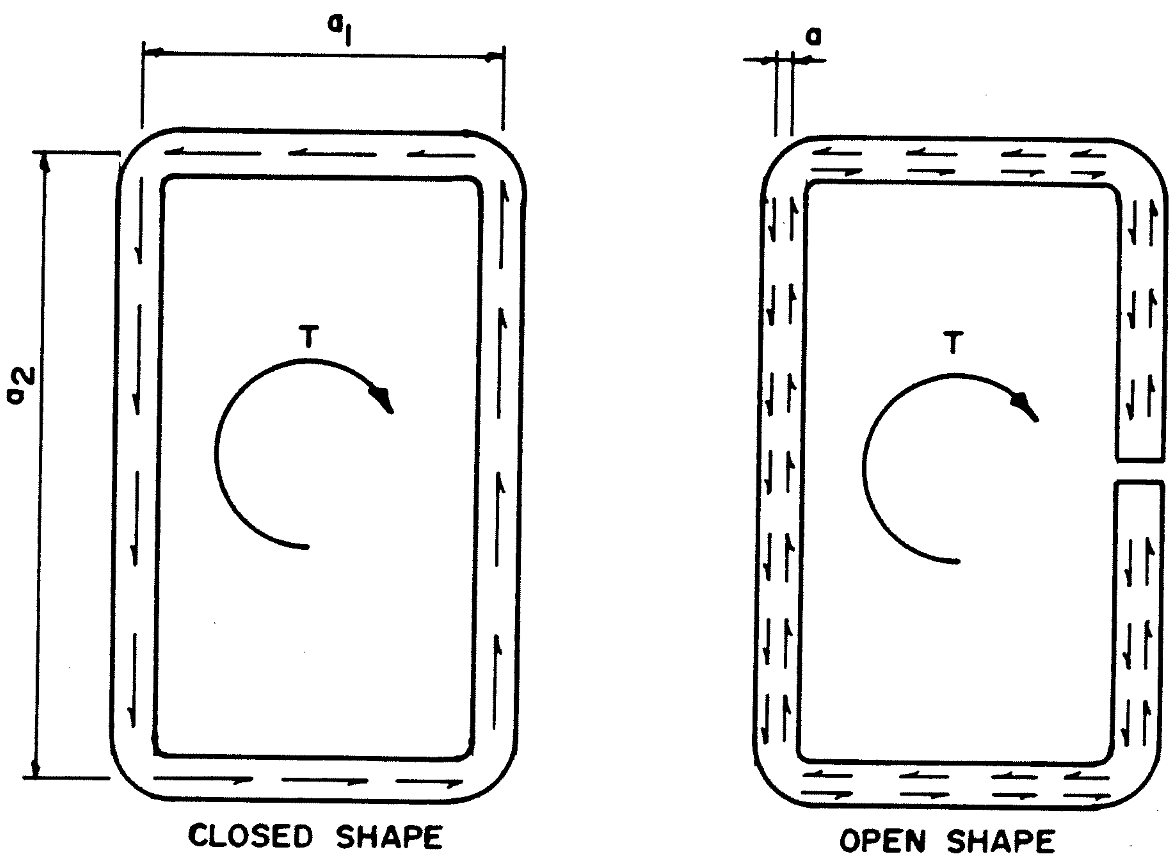

FIGURE 8.1 – SHEAR FLOW DUE TO TORSION

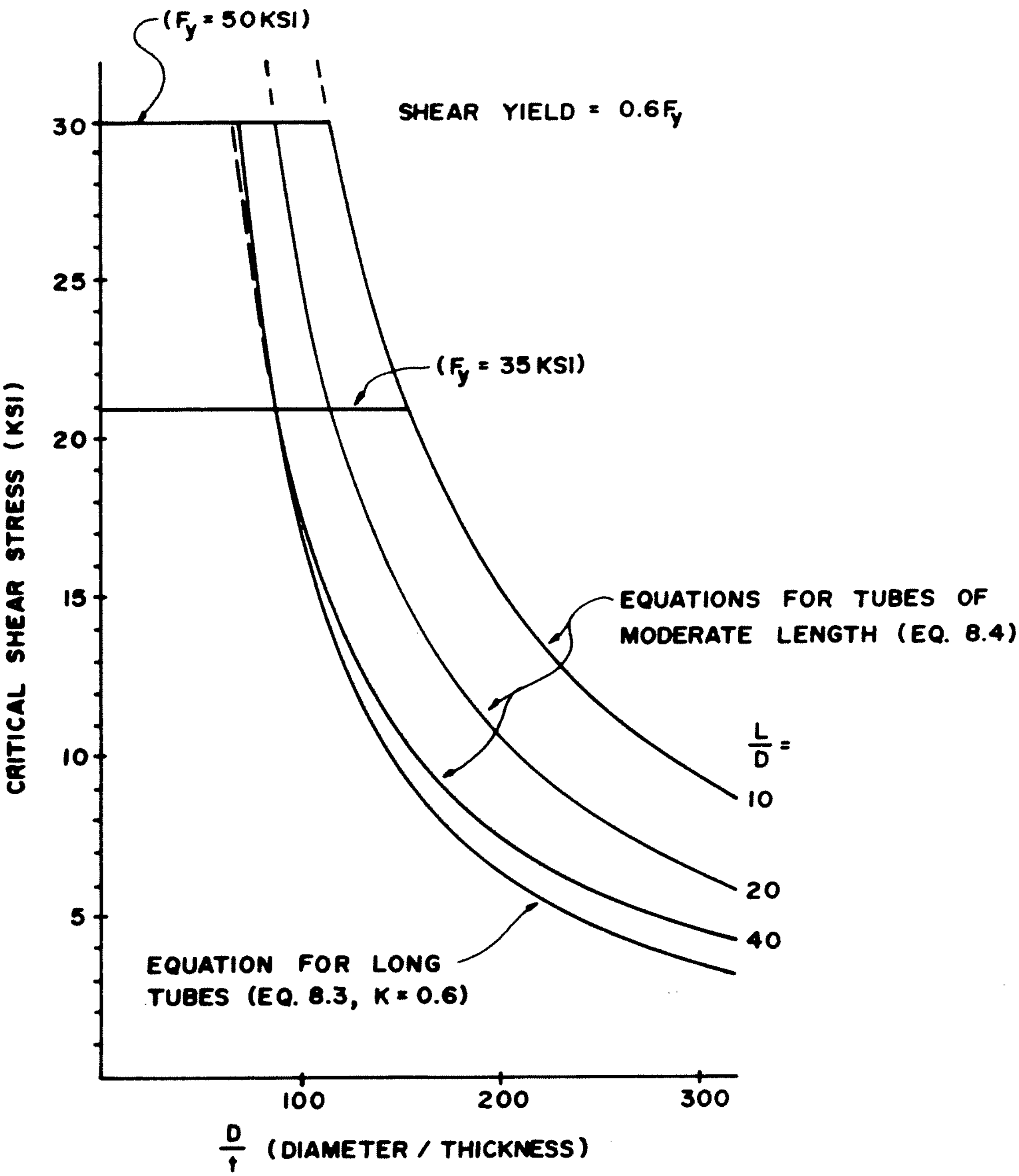

FIGURE 8.2 – ELASTIC BUCKLING EQUATIONS FOR THIN ROUND SECTIONS IN TORSION

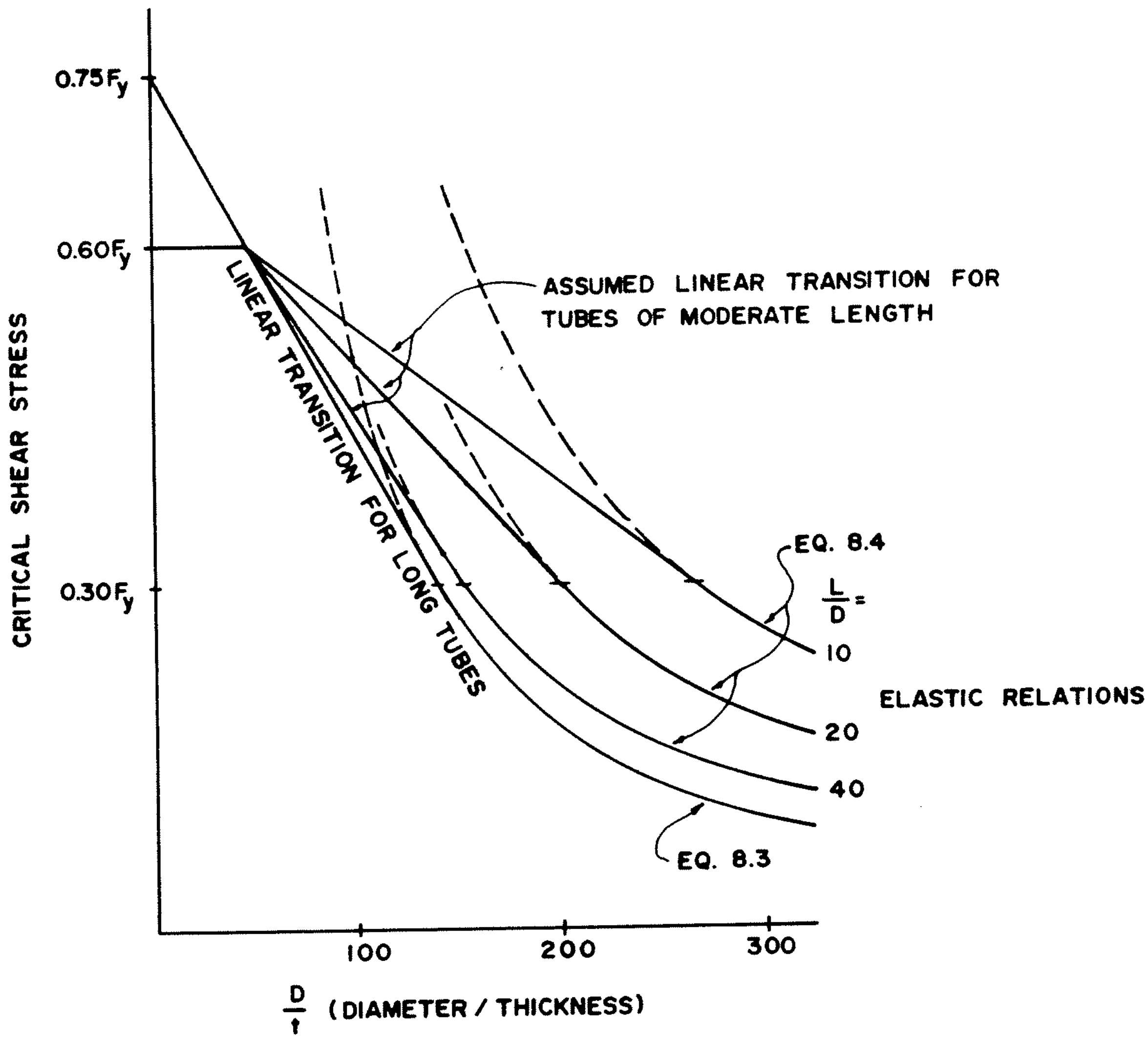

FIGURE 8.3 - INELASTIC TORSIONAL BUCKLING FOR THIN ROUND SECTIONS HAVING ROUNDED STRESS-STRAIN RELATION

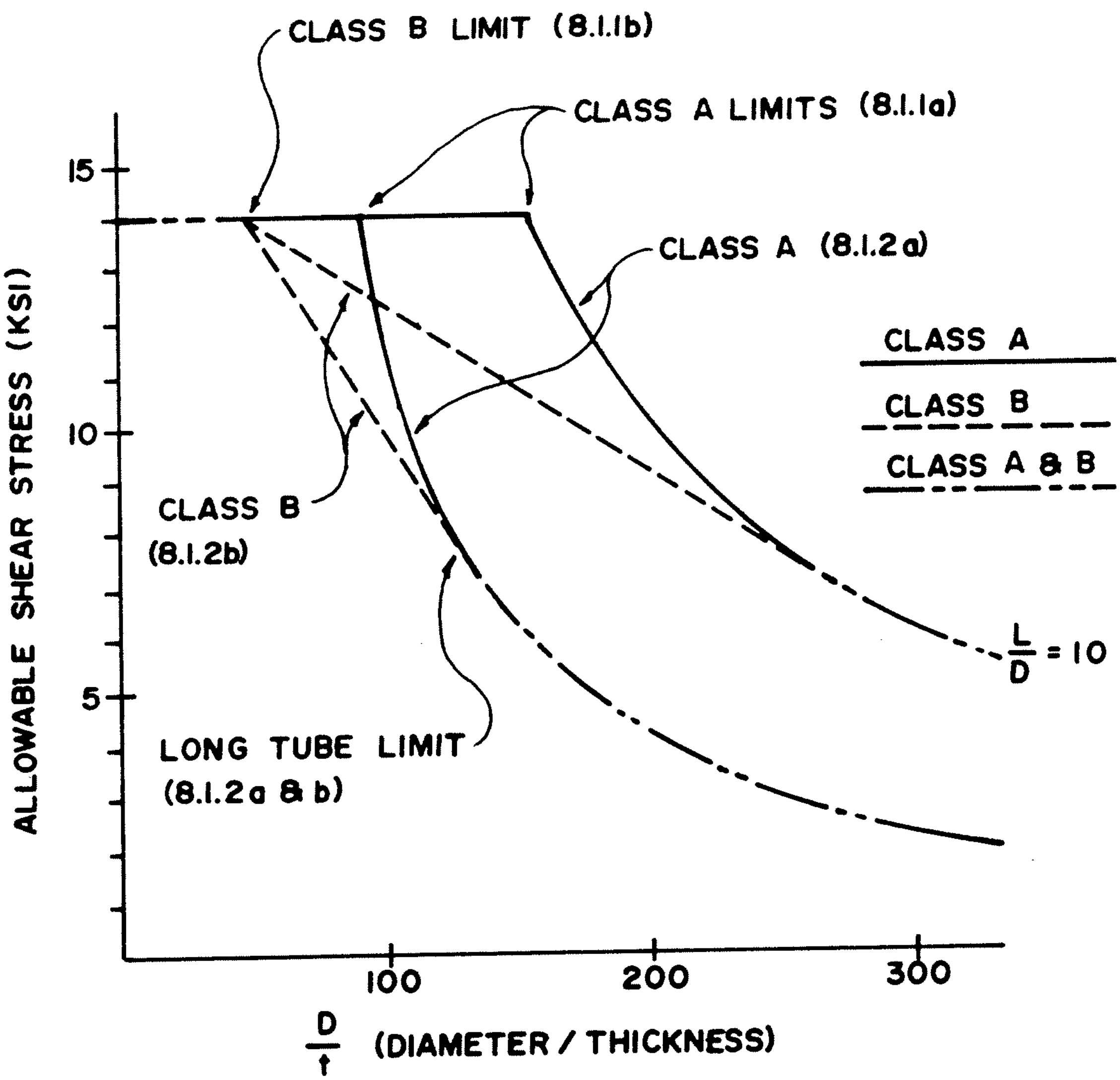

FIGURE 8.4 – ALLOWABLE STRESSES IN THIN ROUND TUBULAR SECTIONS LOADED IN TORSION (F_y = 35 KSI)

SECTION 9.0
COMBINED LOADINGS

The various portions of Section 9 provide criteria for combinations of loadings which could be encountered in tubular members. The AISC Specification provides only for combinations of axial loading and bending. Additional provisions to include the effects of pressure and torsion are included in the Criteria because they may often be significant factors in the design of tubular members.

Studies have indicated that internal pressure considered in Section 9.1 increases the elastic buckling strength of thin tubes (9.3, 9.7). However, since pressure is considered only as an accidental condition, the Criteria takes no advantage of the increase in buckling strength. On the other hand, internal pressure will cause yielding to occur at reduced axial loads and bending or torsional moments. Figure 9.1 shows the changes in a Mohr's Circle diagram resulting from superimposing an internal pressure on an initial primary loading. The first diagram is for primary loadings such as axial load and bending which cause normal compression while the last diagram is for primary shear stresses. Since hoop tension alone, without longitudinal pressure stresses, cause the most severe condition, only its effect is shown. Using the maximum shear stress as a yield criteria ($\tau_m = ½ F_y$), the change in radius of Mohr's Circle estimates the effect of internal pressure. The relations equating the radius to the maximum shear can be solved for the primary stress which will cause yielding at a particular pressure. This is used as the effective yield strength in Section 9.1. By using the effective yield stress in the design equations in the preceding sections for primary loadings, allowable stresses are reduced. When the full yield strength can be developed, no change occurs when elastic buckling governs the design and a transition is provided for intermediate thicknesses.

The amount of reduction in the design stress caused by internal pressure depends on the yield strength, geometric properties of the tube, and the governing design equation. However, for round tubular sections with D/t at the upper limit for neglecting local buckling ($3300/F_y$), about 7 psi are required for a 1% reduction in yield strength. Consequently, it can be seen that large differences in hydrostatic head are required to have a significant effect and temperature changes in sealed pipe are not very important. Only with thin tube, in the transition between primary and elastic local buckling, would the effect of internal pressure possibly be a significant factor.

Section 9.2 concerns the effect of external pressure. Two equations are available for the elastic buckling of cylinders under external pressure. One takes account of end effects and the other is for long cylinders.

$$p_{cr} = \frac{2.05\ E}{\left(\frac{L}{D}\right)\left(\frac{D}{t}\right)^{5/2}} \qquad (9.1)$$

$$p_{cr} = \frac{2.2\ E}{\left(\frac{D}{t}\right)^{3}} \qquad (9.2)$$

Both are highly dependent on D/t and are plotted in Figure 9.2. Since only very short and thin tubes are influenced by end effects, and it is conservative to neglect these effects, only the long cylinder equations are used in section 9.2. The expression is for elastic buckling but since the critical pressures are much less than the external pressure which would cause yielding, no inelastic buckling expression is necessary. Weingarten and Seide (9.9) recommend a linear interaction relation between external pressure and axial load. It is felt that a linear interaction would also be conservative if external pressure is present with bending or torsional loads. Consequently, the reduction factor in the design formulation is based on the linear interaction and is applied to all cases of primary loading. A safety factor of 1.67 has been applied to the pressure term.

External pressure has considerably more effect on the strength of members than internal pressure. For a tube with D/t at the upper limit for neglecting local buckling (92 for 36 ksi yield material), an external pressure of 0.5 psi will cause a 1% reduction in allowable axial stress as opposed to 7 psi internal pressure for the same percentage reduction.

Other combinations of primary loads are considered in Sections 9.3 through 9.7. Interaction relations which approximate the strength of a member under the action of more than one type of load are used as the criteria for the combined load cases. The concepts of interaction diagrams and the various forms that they may take are described in the CRC Guide and in an article by Shanley (9.10).

The Criteria uses the provisions of the AISC Specification for the cases of combined axial load and bending in Sections 9.3 and 9.4. These interaction relations are used in the AISC Specification because they closely approximate the data obtained from many tests of rolled wide flange members. Comparisons with data from numerical analysis and from a limited number of tests of round and rectangular members indicate that the relations are also reasonably accurate for tubular members where local buckling is not a factor (9.2, 9.6). The linear interaction also applies in the case of local buckling since the normal stresses from axial load and bending are additive. The moment amplification term is also reasonable to apply for local buckling near the center of the member.

In the case of biaxial eccentricity of loading round or square tubular sections, the AISC equations lead to inconsistent results if the eccentricity is not in the x or y directions (9.11). The interaction equation adds the x and y eccentricities instead of using the square root of the sum of the squares. The AISC equations could be as much as 40% conservative and the provision added at the end of Section 9.3 yields the same strength column for any direction of eccentricity.

Several forms of interaction relations have been proposed for other load combinations (9.1, 9.4, 9.5, 9.8). A common form combines normal and shear stresses elliptically by using the sum of the squares.

$$\left(\frac{\sigma}{\sigma_{\text{all}}}\right)^2 + \left(\frac{\tau}{\tau_{\text{all}}}\right)^2 \leq 1 \qquad (9.3)$$

A second form uses just the first power of the normal stresses.

$$\left(\frac{\sigma}{\sigma_{\text{all}}}\right) + \left(\frac{\tau}{\tau_{\text{all}}}\right)^2 \leq 1 \qquad (9.4)$$

These relations are plotted in Figure 9.3. The latter form is the more conservative, but not too conservative (9.5), and is the form used in the Criteria. In Section 9.7, the normal stresses for bending and axial load are added linearly as in Section 9.3 and include the moment amplification term.

(See page 54 for applicable references.)

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9.8 "Design and Construction Guide-Structural Steel Supports for Outdoor Advertising Signs," Prepared for U.S. Steel Corp. by Wilbur Smith & Assoc., Nov. 1968.

9.9 Weingarten, V. I. and Seide, D., "Elastic Stability of Thin-Walled Cylindrical and Conical Shells Under Combined External Pressure and Axial Compression," AIAA Journ., vol. 3, no. 5, May 1965.

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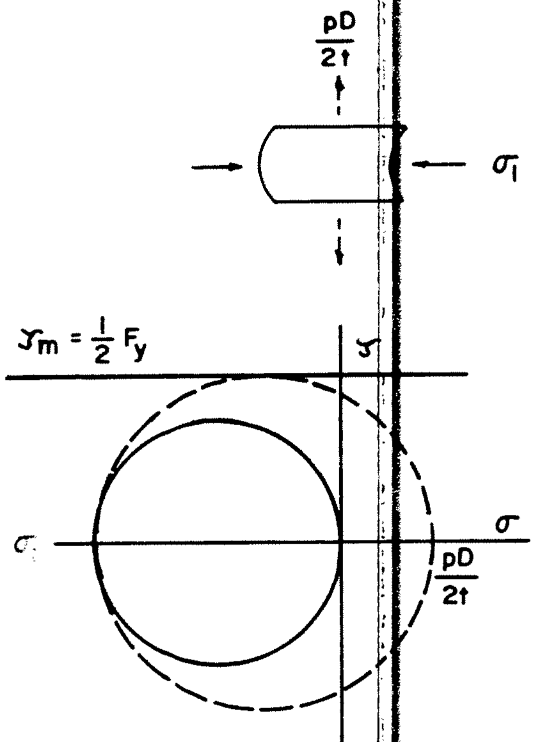
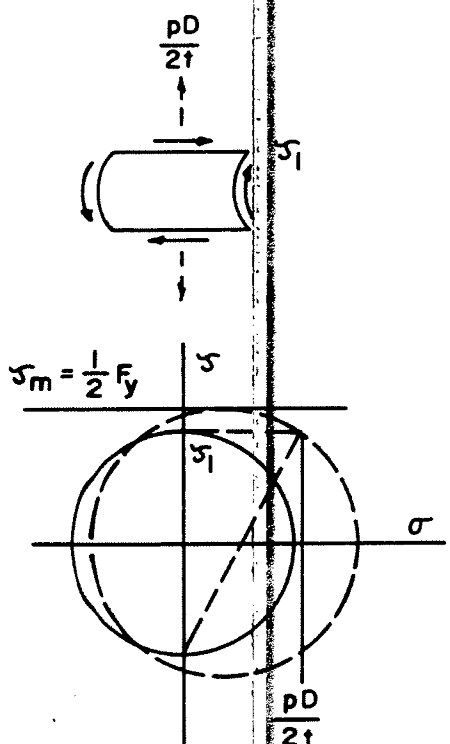


$$\tau_m = \frac{1}{2}\left(\sigma_1 + \frac{pD}{2t}\right) = \frac{1}{2}F_y$$

$$\frac{F_{ye}}{F_y} = \frac{\sigma_1}{F_y} = 1 - \frac{\frac{pD}{2t}}{F_y}$$

HOOP STRESS SUPERIMPOSED ON AXIAL COMPRESSION



$$\tau_m = \sqrt{\tau_1^2 + \left(\frac{pD}{2t}\right)^2}$$

$$\frac{F_{ye}}{F_y} = \frac{2\tau_1}{2\tau_m} = \sqrt{1 - \left[\frac{\frac{pD}{2t}}{F_y}\right]^2}$$

HOOP STRESS SUPERIMPOSED ON SHEAR

FIGURE 9.1 - EFFECT OF INTERNAL PRESSURE ON EFFECTIVE YIELD STRENGTH FOR PRIMARY LOAD

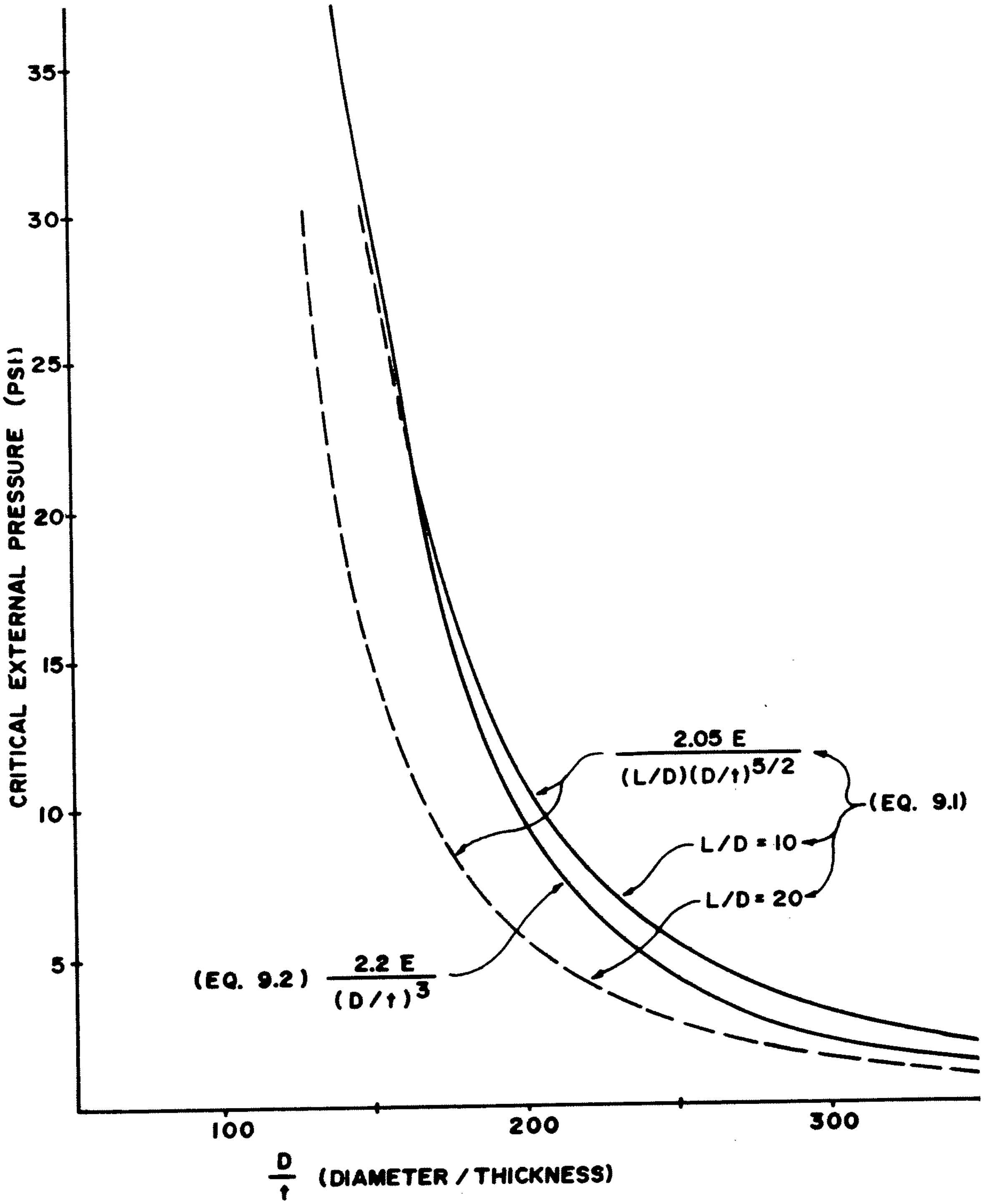


FIGURE 9.2 — ELASTIC BUCKLING OF CYLINDERS UNDER EXTERNAL PRESSURE ($E = 29 \times 10^6$ PSI)

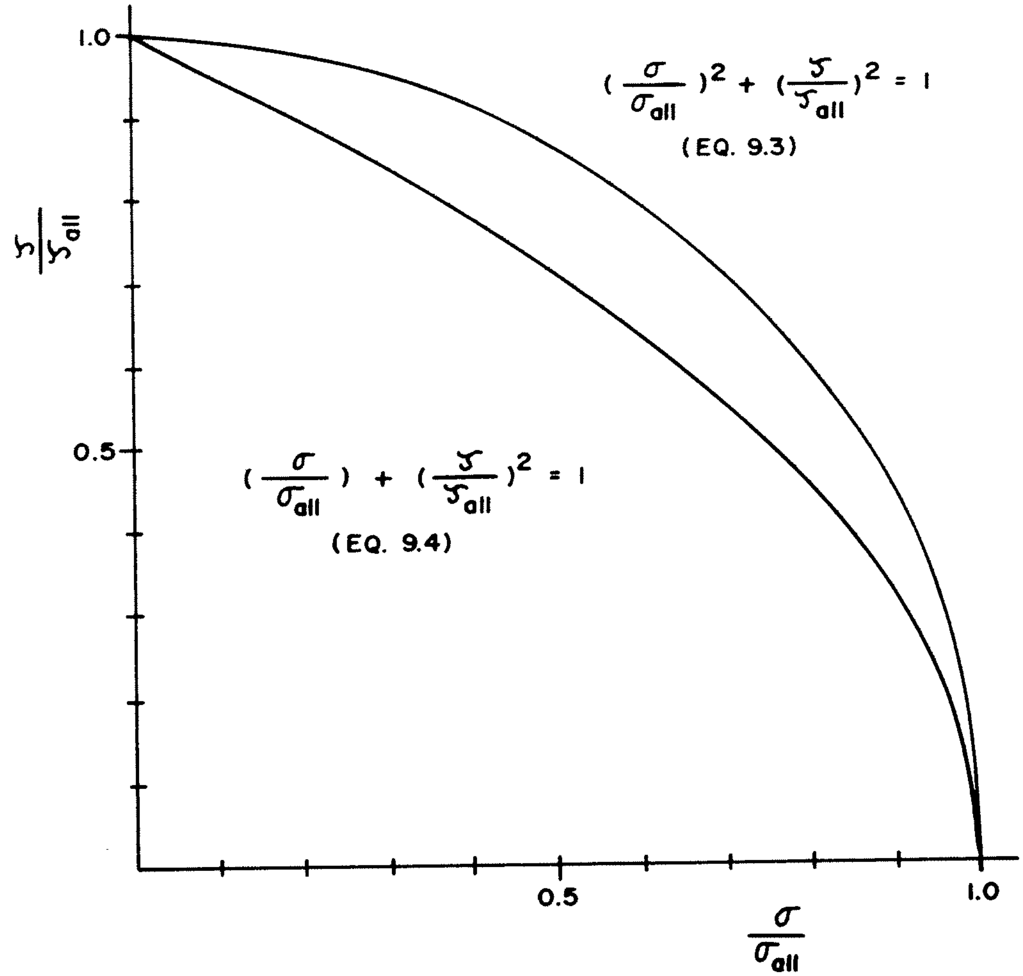


FIGURE 9.3 - SHAPES OF BASIC INTERACTION CURVES BETWEEN NORMAL STRESSES AND SHEAR STRESSES

SECTION 10.0
TUBULAR CONNECTIONS FOR STATIC LOADS

Since special terminology is used in discussing tubular connections, several important terms are defined in the beginning of Section 10. In reviewing the literature, other terms are often encountered (such as the interchangeable use of branch, brace or web), however, the definitions in Section 10.1 are limited to those which appear in the remainder of the Criteria. Figure 10.1 illustrates many of these terms.

Several investigations of tubular connections resulting in a large series of reports have been conducted in both the U.S. and Europe. These investigations have provided a general knowledge of the behavior of tubular connections and have established guidelines for many problem areas. However, due to the large number of variables in possible member sizes and joint geometries, only a few connection configurations have had sufficient study to set quantitative design specifications. The Criteria considers only connections formed by fully welding members directly to one another. Limits on the sizes of the members are given to insure that the connection will develop the full capacity of the members framing into it. Most of the information has been verified experimentally for hot formed tubes only. The CIDECT programs are currently investigating connections of cold formed members to confirm the applicability of these equations and limits to cold formed tubes.

To cover other situations where the requirements of the Criteria are considered too conservative because 100% efficiency is not required or because fabrication of the geometry is too difficult, Section 10.2 establishes procedures by which other connection configurations may be qualified. These procedures include application of the AWS Criteria, an analytical method to show that the stress levels are within the range accepted by the AISC Specifications (10.16), and experimental verification of the detail. The AWS procedure is a general method in which nominal punching shear stresses are calculated and compared to empirically derived limiting values. The procedure is applicable for axial forces and bending in either round or rectangular branches. The limiting values for 100% efficiency cited in the Criteria are consistent with the AWS procedure but provide a relatively quick and conservative check for the adequacy of a connection.

Several analytical approaches are available. For connections of a single round branch to a round main member, various procedures have been developed to calculate the stresses in the connection. References 10.11 and 10.15 discuss several. Some of the procedures have successfully been used for some time in the design of nozzles for pressure vessels. It should be noted, however, that these procedures are often involved and usually apply only within certain ranges of variables. The use of finite element analysis with large computer programs is a general analytical procedure, but is practical for situations where the same connection details will be used repeatedly. For tubes framing directly into rectangular main members, simple equations and design tables are available for permissible loads in the members forming the branches in connections with less than 100% efficiency (10.7).

There are also a number of special connections which do not involve direct connection of the members or are used to stiffen the main member:

1. gusset plates (10.1, 10.2, 10.7)
2. beam seats and clip angles (10.14)
3. doubler plates (10.7, 10.8)
4. wing stiffening plates (10.2, 10.6, 10.14)
5. spherical connectors (10.10, 10.12)
6. cropped or flattened tubes (10.2, 10.7)
7. concrete filled connections (10.5)
8. patented connectors

In general only limited tests have been conducted to determine the behavior and strength of these various configurations. However, the references cited include qualitative discussions which will assist the designer in selecting an adequate detail. For connections which are extensions of construction practice with wide flange shapes (gusset plates, beam seats and clip angles), standard practice and provisions of the AISC Specification should be followed whenever applicable. Care must be taken, however, to insure that the main member is stiffened or the load is sufficiently spread to avoid excessive bending in the wall.

Section 10.3 follows the AWS Structural Welding Code (10.17) which now contains an extensive section on welded tubular structures. This section includes the direct welding of tubular members in the T, Y or K configurations. The Code includes joint details, workmanship, fit-up tolerances and joint preparation, as well as qualification and inspection rejoint details are included for complete or partial penetration groove welds and fillet welds.

The matching of base metal and welding electrodes given in Table 10.1 is taken from the AWS Code, which differs from the AISC specification. Additions have been made in Table 10.1 to include the steels meeting Canadian standards accepted in the Criteria. The allowable stresses in Table 10.2 are also the recommendation of AWS.

Sections 10.4 and 10.5 of the Criteria establish limits on connection parameters to develop 100% efficient connections. The design philosophy for tubular connections with 100% efficiency comes from the studies at Berkeley (10.3, 10.4). The strongest and stiffest K connections in both round and rectangular tubes are those which include an overlap of the branch members. However, due to the extra and more complex cuts required for fit-up, joints with overlap are not as economical as gap joints. In using gap joints, sufficient gap width must be provided to insure adequate welding.

The provisions for direct joining of round tubes in Section 10.4 are based on the extensive studies at the University of California, Berkeley, and the University of Texas. The limits for K connections in 10.4.2 come indirectly from the Berkeley studies (10.3, 10.4). Since the limits were obtained on tests of material with 35 ksi yield stress, the ratio of $F_y/35$ has been included to account for higher strength steels. The studies at Texas for T and Y connections were primarily concerned with determining the maximum stress in the connection (10.11, 10.15). However, by noting which of these tests of T joints failed at loads greater than the yield load of the branch, the limits of 10.4.1 were deduced. It made no difference in the magnitude of the maximum stress if the load in the branch were tension or compression. The Texas studies also indicated that the maximum stress in the main member of a Y connection is not as great as in a T connection (10.15), so the same limits are applied to Y connections until more definitive tests are conducted. Finally, the Texas tests showed that the stress in the main member when the branch is subjected to bending about any axis is less than the stress due to an axial force in the branch (10.15). Consequently, the same limits are applied for bending even though they may be extremely conservative when the bending moment is in the plane of the connected members.

Section 10.5 concerns connections with a rectangular main member and round or rectangular branches. Since the corners of a rectangular member are rounded, a more satisfactory joint detail is obtained if the width of the branches is less than the width of the main member rather than having both members the same width. A difference of two inches has been recommended when the main member is large, i.e., the periphery greater than 16 inches, while a one inch difference is recommended for the case of smaller main members (10.7).

Provisions for the minimum wall thickness of a rectangular main member when the branches are subjected to axial forces are derived from equations developed at the University of Sheffield in England (10.9). These equations give allowable forces in the branches as a function of the tube widths, thickness of the main member and yield strength of the members. The allowable component of force perpendicular to the main member as given by the Sheffield equations is shown in Figure 10.2. By eliminating the load factor from the equations and equating the load to the yield force of the branch, the limiting thickness is obtained for a joint with 100% efficiency. Eccentricity does not effect the connection strength in connections with rectangular main members as it does when round main members are used (10.13). Of course, the moment caused by the connection eccentricity must still be applied to the members. If the branches overlap by more than 50%, very little force is transferred to the main member and its wall thickness is not a critical factor on the strength of the connection. Although the equations of Section 10.5.1 were developed for K connections with a gap between the branches, it is felt that they may be conservatively applied to T and Y connections.

The provisions for branches subjected to bending come from the investigations at Drexel Institute of Technology (10.8). Since the reference gives limits for the ultimate load conditions, item iii) in Section 10.5.2 has been adjusted to include a safety factor of 5/3 on the axial force in the main member. With the limitations listed, the full plastic moment capacity of the branch member can be developed. The interpretation of the condition that the members have the same width should allow for the difference noted earlier for good fit-up. In other words, when the main member has a periphery sixteen inches or greater, the branch may be classified as the same width if its actual width is within two inches of the width of the main member. For smaller main members, the difference limit is one inch.

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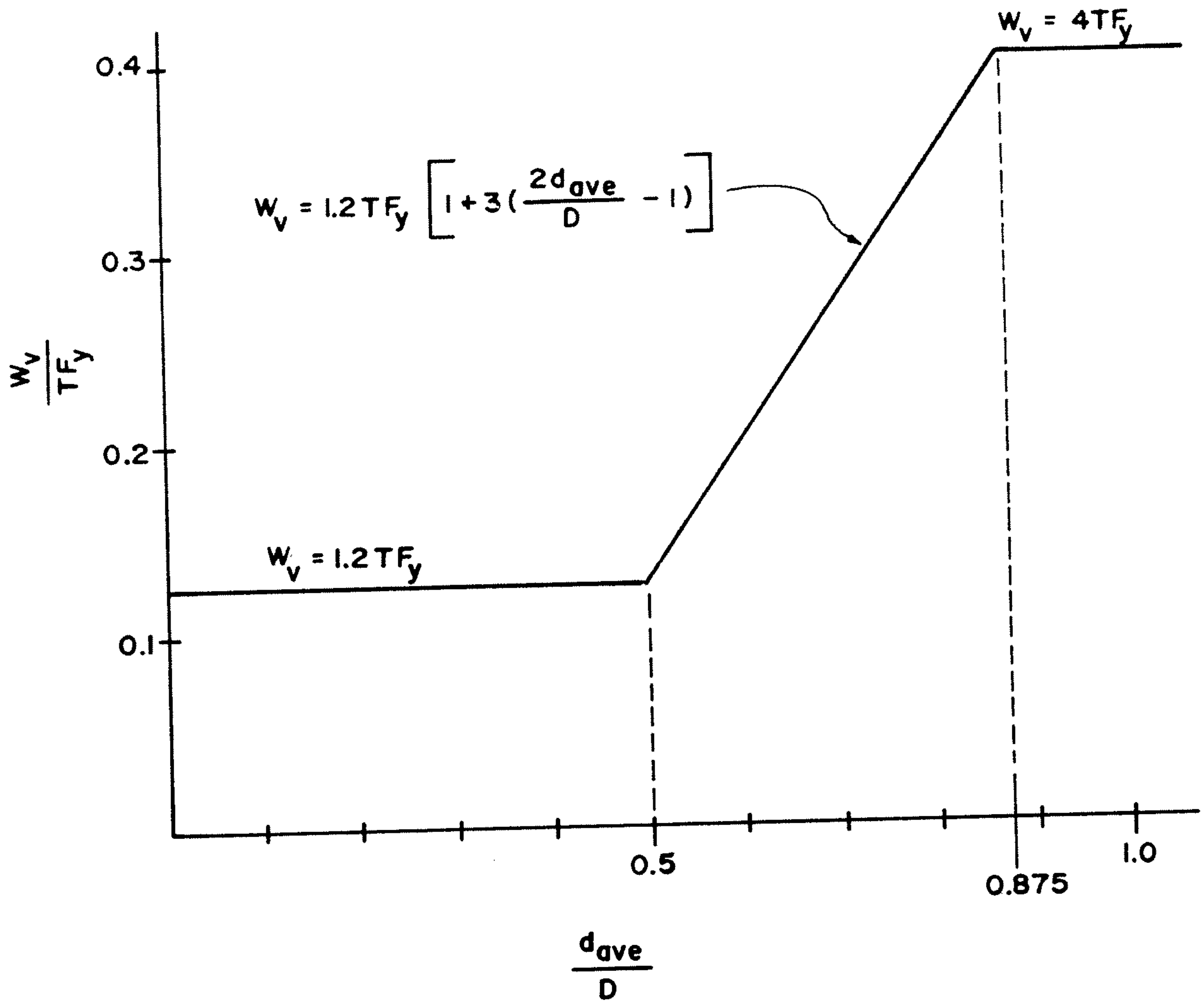

FIGURE 10.2 – SHEFFIELD EQUATIONS FOR ALLOWABLE VERTICAL COMPONENT OF FORCES IN BRANCHES OF K CONNECTION WITH RECTANGULAR MAIN MEMBER

SECTION 11.0
FATIGUE

Although considerable research on the fatigue behavior of tubular connections has been conducted, the body of data is not nearly as extensive as that which forms the basis for the AISC (11.4) requirements for other types of members. However, enough data is available on connections with full penetration welds to formulate a tentative criteria for certain types of connections.

Full penetration groove welds are recommended in tubular connections because they have better strength in fatigue. However, it is recognized that where geometry does not permit full penetration welds, a partial penetration weld may be required. In such cases, the latest issue of the AWS Structural Welding Code should be used to determine the weld size.

In following the philosophy of Section 10 of the Criteria, only connections which can develop 100% joint efficiency are used. Even these are somewhat restricted in fatigue design in that K connections with less than 50% overlap are not permitted. Research on gap joints with both round and rectangular main members (11.1, 11.2) indicate that a gap joint is a severe fatigue situation. The studies also show that reinforcing the joint with gusset plates and other forms of stiffeners may not increase the fatigue life. If stiffening is used, extreme care must be exercised to avoid stress concentrations.

The procedure for designing tubular structures for fatigue in Section 11.5 is parallel to that used in Appendix B of the AISC Specification (11.4). Since modified Goodman diagrams of fatigue data indicate that the number of cycles to failure is nearly a constant function of stress range, the AISC uses a simple design philosophy that only the stress range be specified for a particular cyclic life without regard to the stress ratio, R. Data from tests at the University of Texas (11.3) indicate that this relation is also valid for tubular T joints. Fortunately, this means that the criteria can be established with data from S-N curves for only one stress ratio.

The criteria for the base material in the tubular section is taken directly from the AISC provision without the support of direct data on tubular members. Studies at the University of Texas (11.3) provide the data for T and Y connections. Five series of tests on connections with differing proportions were conducted. The D/T ratios of the main members for these tests were either 48 or 28.2, both of which are thinner than the requirement for a 100% efficiency joint. Consequently, the stress range for category T is based only on the series with D/T equal to 28.2 which should be conservative for the 100% efficient connection. A factor of safety of 1.67 is used to obtain the allowable ranges for category V. Although the tests were conducted on only T connections, the results are also used for Y connections in the Criteria.

Data for the K connections come from the European CIDECT program. These results are summarized in S-N curves for $R = S_{min}/S_{max} = .20$ where S is the maximum stress in the branches (11.1). This data is converted to stress range by multiplying S by 0.8 and dividing by the factor of safety 1.67 to obtain the allowable range. The curves were extended by a linear extrapolation to cover the loading conditions used by AISC and the values for joints with 100% overlap closely match AISC stress category F. Data for joints with 47% did not match any of the existing AISC categories and, therefore, category U in the Criteria is new.

Although the data for the tubular connections is for axially loaded members, the criteria is also applied to connections with bending moments until more complete data is available.

In details which are not covered by the criteria or in cases where fatigue is an especially important design consideration, the fatigue provisions of AWS D1.1-75 (11.5) may be used.

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SECTION 12.0
CORROSION PROTECTION

Since the interior of a tubular section is difficult to inspect during the life of a structure, some concern has been expressed over the possibility of internal corrosion. Much divergence of opinion exists over the degree of protection required. However, there is a growing body of evidence that internal corrosion is not a large problem and good design practice can eliminate the need for expensive protection.

First, it has been scientifically shown that in a hermetically sealed tube, internal corrosion cannot progress beyond the point where the oxygen or moisture necessary for chemical oxidation is consumed (12.1). The oxidation depth is insignificant when the corrosion process must stop in a sealed tube even when a corrosive atmosphere is present at the time of sealing. Therefore, one engineering practice which is recommended by the Criteria is to design the details of the structure so that the tubes can be completely sealed by continuous welding at the ends and at points where penetrations must be made. Some fabricators go so far as to pressurize the tubes and look for leaks with soap film. This degree of inspection is shown primarily in exposed bridges and does not appear to be common practice or warranted in the protected steel members of buildings.

In some instances, special connectors are used which make complete sealing impractical. If fine openings exist, water can enter the tube through capillary action or by inspiration resulting from the partial vacuum created if the air in the tube is cooled rapidly (12.2). Air, of course, could also enter the tube and the corrosion process will continue until the opening rusts shut. By using a pressure equalizing hole, the entrance of water by inspiration can be prevented. However, the hole must be placed so that it is impossible for water to flow into the tube by gravity and the hole must be large enough to prevent capillary flow. Good external painting will minimize the capillary flow of water into the tube through fine cracks.

Some precaution is necessary concerning sealing a tubular section filled with concrete. In the event of a fire, water in the concrete will vaporize and may create internal pressures sufficient to burst a sealed tubular section. Measures should be taken to insure that the vaporized induced pressure will safely leak out.

In cases when moisture will not enter the tubular section, internal protection is not required. Therefore, in sealed members or in members where capillary flow, gravity flow and inspiration are prevented, the Criteria does not require an internal protective coating. However, in cases of open tubular sections where changes of the air volume by ventilation or direct water flow into the member is possible, conservative practice would recommend an internal protective coating be used. On the other hand, instances can be cited where tubular sections used as utility poles and tubes used in marine construction were open to water flow and accumulation but still showed no significant interior corrosion even after several decades (12.1). Even so, the Criteria recommends that for the load carrying members of structures, when sealing or prevention of internal moisture cannot be incorporated into the design, internal surface protection is necessary. The Criteria does not define the type of protection to be used when an internal coating is required. Conditions of exposure and individual preferences of the method of application are factors which have a bearing on the selection.

The exterior protection of tubular sections in buildings requires no special consideration beyond that specified for other shapes of structural steel (12.3). However, in situations of severe corrosive conditions, the tubular shape does lend itself more easily to special wrappings or protective measures than other shapes. Publications of the pipeline industries contain much information on methods of protecting pipes against corrosion in severe conditions.

Following the concepts of the AISC Specification, no minimum thickness is required in the Criteria. However, it should be noted that British standards for tubular structures (12.4) and bridge specifications (12.5) do require a minimum thickness to guard against excessive loss of strength due to corrosion. The bridge specifications typically use 5/16 inches for plate elements. The British standard applies the concept that if only one surface is subject to corrosive action, the minimum wall thickness may be half that required when two surfaces are exposed. Therefore, in severe conditions when guaranteed protection and inspection are difficult to obtain for main load carrying members, the designer may wish to consider a minimum thickness of 5/16 inches for open tubes and 5/32 inches for sealed tubes.

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CONVERSION FACTORS TO SI UNITS

For the convenience of those wishing to convert from the units expressed in the Notation in the "Criteria" to the International System of Units (SI) the following conversion factors are given:

TO CONVERT FROM	TO:	MULTIPLY BY:
Inches	Millimetres (mm)	25.40
square inches	square millimetres	645.16
cubic inches	cubic millimetres	16.387×10^3
inches^4	millimetres^4	$.4162 \times 10^6$
pounds per square in.	kilopascals (kPa)	6.895
kips per square in.	megapascals (MPa)	6.895

A more extensive list of conversion factors is to be found in the American Iron and Steel Institute, "Metric Practice Guide" 1975.