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Structural stability design provisions--a comparison of the provisions of the crc guide and the specificationsof aasho, AISC and AREA WRC Bulletin No.146, Nov. 1969 (69-15)

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Column Research Council

STRUCTURAL STABILITY DESIGN PROVISIONS

A Comparison of the Provisions of the CRC

Guide and the Specifications of

AASHO, AISC, and AREA

by

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ABSTRACT

A comparison is made in tabular form between the stability provisions of the Column Research Council "Guide to Design Criteria for Metal Compression Members" and the specifications of the American Association of State Highway Officials, the American Institute of Steel Construction, and the American Railway Engineering Association.

FOREWORD

The purpose of this report is to examine the stability provisions of the specifications of the American Association of State Highway Officials (AASHO), the American Institute of Steel Construction (AISC), and the American Railway Engineering Association (AREA) and to compare them with pertinent recommendations of the Column Research Council (CRC) "Guide to Design Criteria for Metal Compression Members."

The major specifications selected are those dealing with buildings and bridges, structures which have been of particular interest to the Council in the past. The findings of the Column Research Council are summarized in its "Guide to Design Criteria for Metal Compression Members." It is used as a reference and as the basis of many design provisions.

This comparison has been prepared under the authorization of the CRC Executive Committee as an aid in its own deliberations, as a useful reference document for its research workers, and as a means of pinpointing topics for which additional documentation might be needed. Also the report should be a help in future deliberations of the various specification-writing bodies.

As new editions of the three specifications and of the Guide become available, it is planned to issue revisions of this comparison.

The arrangement of the material in this Bulletin is according to the sequence of the "Guide". Topics such as arches, hybrid girders and box girders, which are not covered in the present edition of the Guide but are treated in the specifications, are listed at the end of the comparison. A table of contents, arranged in tabular form, preceeds the detailed tabulation of provisions.

A comparative nomenclature (symbols) is included in the Appendix. The sequence follows that used in the CRC Guide, but the symbols used in the individual specifications are retained in the appropriate listings.

The encouragement of Dr. L. S. Beedle and the members of the Executive Committee of the Column Research Council is sincerely acknowledged.

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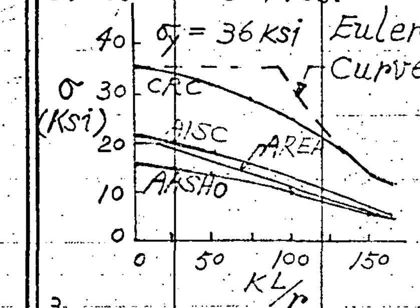
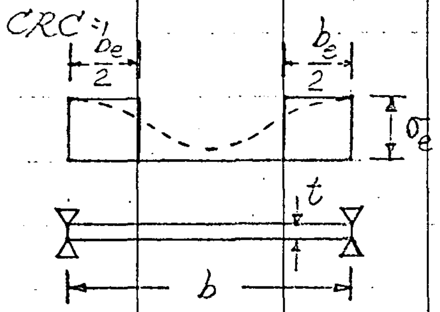
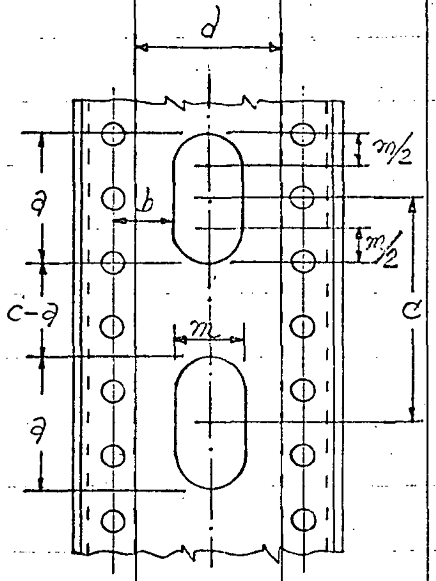
COMPARISON

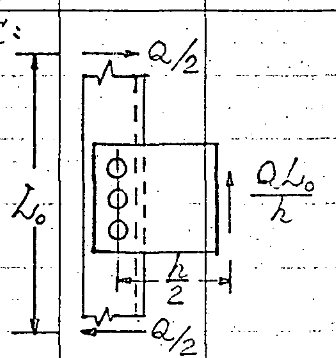
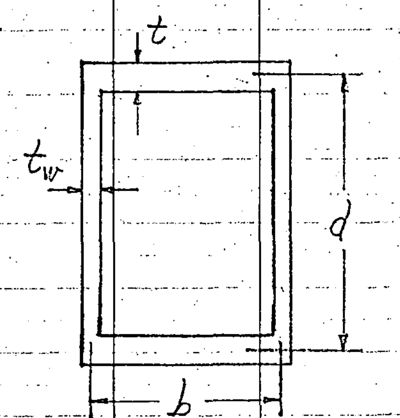
Topic	CRC Guide (2nd Ed. 1966) Sec.	CRC Guide (2nd Ed. 1966)	AASHO Specification (9^{th} Ed. 1965; and Interim Spec. 1966-67) Art.	AASHO Specification	AISC Specification (Final Draft, 1969) Sect.	AISC Specification	AREA Specification (1969 Edition) Art.	AREA Specification	Notes
		When $0<\frac{\sigma_c}{\sigma_y}<\frac{1}{2}$				When $\frac{Kl}{r}>C_c$			1. Elastic range.
	2.4	$\sigma_c=\sigma_e=\frac{\pi^2 E}{(KL/r)^2}$ (2.2)	App. C	$f_s=\frac{\pi^2 E}{\eta(L/r)^2}$	1.5.1.3.2	$F_a=\frac{12\pi^2 E}{23(Kl/r)^2}$ (1.5-2)	1.4.1	For ASTM A36 steel, when $kl/r \geq 143$ $F_a=\frac{147,000,000}{(kl/r)^2}$	2. Column curves. 
					1.8.4	Max. permitted $\frac{l}{r}=200$	2.4.1	For high-strength steel, when $kl/r \geq 27111/\sqrt{F_y}$ $F_a=\frac{147,000,000}{(kl/r)^2}$	3. AASHO: The limiting values of $\frac{l}{r}$ are given for various steels.
Columns, Centrally Loaded		When $\frac{1}{2}<\frac{\sigma_c}{\sigma_y}<1$				When $\frac{Kl}{r}<C_c$			Inelastic range.
	2.4	$\sigma_c=\sigma_y-\frac{\sigma_y^2}{4\pi^2 E}\left(\frac{KL}{r}\right)^2$ (2.10)	1.7.1	Riveted ends: $F_a=\frac{0.55F_y}{1.25}\left[1-\frac{(0.75\frac{l}{r})^2 F_y}{4\pi^2 E}\right]$ Pinned ends: $F_a=\frac{0.55F_y}{1.25}\left[1-\frac{(0.875\frac{l}{r})^2 F_y}{4\pi^2 E}\right]$	1.5.1.3.1	$F_a=\frac{\left[1-\frac{(Kl/r)^2}{2C_c^2}\right]F_y}{\frac{5}{3}+\frac{3(Kl/r)}{8C_c}-\frac{(Kl/r)^3}{8C_c^3}}$ (1.5-1) where $C_c=\sqrt{\frac{2\pi^2 E}{F_y}}$	1.4.1	For ASTM A36 steel, when $kl/r \leq 15$ $F_a=20,000$ psi; when $15<kl/r<143$ $F_a=21,500-100\frac{kl}{r}$	
							2.4.1	For high-strength steel, when $kl/r \leq 3388/\sqrt{F_y}$ $F_a=0.55F_y$; when $3388/\sqrt{F_y}<kl/r<27111/\sqrt{F_y}$ $F_a=0.60F_y-\left(\frac{F_y}{1662}\right)^{3/2}\frac{kl}{r}$	
					2.4	When $l/r \leq C_c$ $P_{cr}=1.7AF_a$ (2.4-1)			Plastic design
			1.7.12	For secondary members with $l/r \leq 140$, same formulas as for main members.	1.5.1.3.3	For bracing and secondary members, when $l/r>120$ $F_{as}=\frac{F_a \text{ (by Formula 1.5-1 or 1.5-2)}}{1.6-\frac{l}{200r}}$ (1.5-3)	1.4.1	For ASTM A36 steel, $F_a=20,000$ psi	AISC: Formula (1.5-3) specifies the same allowable stresses as those given by the Rankine-Gordon formula $F_a=\frac{F_y}{1+C(l/r)^2}$
							2.4.1	For high-strength steel, $F_a=0.55F_y$	
							1.5.1	For wind and sway bracing in compression, max. $l/r=120$	

Topic	CRC Sec.	CRC	AASHO Art.	AASHO	AISC Sect.	AISC	AREA Art.	AREA	Notes
Columns, Effect of Initial Curvature	2.5	$\sigma_a = \frac{1}{2}\left[\left(\sigma_y + \sigma_e\left(1+\frac{\delta_o c}{p^2}\right)\right) - \sqrt{\left(\sigma_y+\sigma_e\left(1+\frac{\delta_o c}{p^2}\right)\right)^2 - 4\sigma_y\sigma_e}\right]$ (2.16b)	App. C	When $\frac{l}{p} \leq (\cos^{-1}\alpha)\left[\frac{E\left(1+0.25+\frac{e_o c}{p^2}\right)}{f_y}\right]^{1/2}$ $\frac{f}{s} = \frac{\frac{f_y}{\eta}}{1+0.25+\frac{e_o c}{p^2}}$ (C) When $\frac{l}{p}$ larger than specified $\frac{f}{s} = \frac{\frac{f_y}{\eta}}{1+\left(0.25+\frac{e_o c}{p^2}\right)B \operatorname{Cosec}\Phi} = \frac{P}{A}$ (A)					AASHO: Formulas (A) and (C) include effects of eccentricity of loads.
Columns, Effective Length	2.8	K = 1.0 for pin-end conditions. K = 0.65 for fixed-end conditions. Effective-length factors for other end conditions and for columns in continuous frames are given. (Figs. 2.14 and 2.21)			1.8.2 1.8.3	When sidesway prevented, K = 1.0 When sidesway not prevented, Kl shall be determined by a rational method. (CRC alignment chart)	1.4.1	Kl is the effective length. $K = \frac{7}{8}$ for members with pin-end connections. $K = \frac{3}{4}$ for members with riveted, bolted, or welded end connections.	AASHO: Effective-length factors are not specified. It is implied in Art. 1.7.1 that K is 0.875 for pin-end conditions and is 0.75 for riveted-end conditions.
Columns, Lateral Bracing	2.10	Transverse braces are designed for 2 % of the maximum compressive axial force in the compression element that is being braced.							

Topic	CRC Sec.	CRC	AASHO Art.	AASHO	AISC Sect.	AISC	AREA Art.	AREA	Notes
Compression Members, Local Buckling			1.7.16	Outstanding legs: Main member $b'/t \le 12$ Secondary member $b'/t \le 16$	1.9.1.2	Outstanding legs: The width-to-thickness ratio is not greater than: Single-angle struts; double-angle struts with seperators $76.0/\sqrt{F_y}$ Struts comprising double angles in contact; angles or plates projecting from girders, columns or other compression members; compression flanges of beams; stiffeners on plate girders $95.0/\sqrt{F_y}$ Stems of tees $127/\sqrt{F_y}$	1.6.2 2.6.2	For ASTM A36 steel, the width of outstanding elements of members in compression shall not exceed the following: 1. Legs of angles or flanges of beams or tees: $10t$, $12t$, $14t$ 2. Plates: $12t$ 3. Stems of tees: $16t$ For high-strength steel, the width of outstanding elements of members in compression shall not exceed the following: 1. Legs of angles or flanges of beams or tees: $1900t/\sqrt{F_y}$, $2300t/\sqrt{F_y}$, $2700t/\sqrt{F_y}$ 2. Plates: $2300t/\sqrt{F_y}$ 3. Stems of tees: $3000t/\sqrt{F_y}$	1. AASHO, ANSC, AREA: Limiting values of b/t ratio to avoid local buckling. 2. AISC: When the actual width-to-thickness ratio exceeds specified values, design provisions are specified in Appendix C.
	3.3	For elastic buckling: $\left(\frac{KL}{r}\right)_{equiv} = \frac{3.3}{\sqrt{k}}\left(\frac{b}{t}\right)$ (3.3) For buckling suppressed until strain hardening: $\left(\frac{b}{t}\right)_{max} = 13\sqrt{\frac{k\sqrt{E_{st}}}{\sigma_y}}$ (3.4)	1.7.89	Plates supported on one side, outstanding legs of angles and perforated plates: $b/t \le \frac{1625}{\sqrt{f_a}}$ Plates supported on two edges or webs of main component segments: $b/t \le \frac{4000}{\sqrt{f_a}}$ Solid cover plates supported on two edges or webs connecting main members or segments: $b/t \le \frac{5000}{\sqrt{f_a}}$ Perforated cover plates supported on two edges: $b/t \le \frac{6000}{\sqrt{f_a}}$	1.9.2.2	For stiffened elements, width-to-thickness ratio is not greater than: Flange of square and rectangular sections of uniform thickness $238/\sqrt{F_y}$ All other uniformly compressed stiffened elements $253/\sqrt{F_y}$			CRC: An equivalent KL/r ratio of the column is used if local buckling controls.

Topic	CRC Sec.	CRC	AASHO Art.	AASHO	AISC Sect.	AISC	AREA Art.	AREA	Notes
Compression Members, Effective Width of Plate	3.4	$\frac{b_e}{t} = 1.9\sqrt{\frac{E}{\sigma_e}}\left[1-0.475\sqrt{\frac{E}{\sigma_e}}\left(\frac{t}{b}\right)\right]$ (3.7) or, $\frac{b_e}{b} = \sqrt{\frac{\sigma_c}{\sigma_e}}\left[1-0.25\sqrt{\frac{\sigma_c}{\sigma_e}}\right]$ (3.8)		—	App. C3	For the flanges of square and rectangular sections of uniform thickness: $b_e = \frac{253t}{\sqrt{f}}\left(1-\frac{50.3}{(b/t)\sqrt{f}}\right) \leq b$ (C3-1) For other uniformly compressed elements: $b_e = \frac{253t}{\sqrt{f}}\left(1-\frac{44.3}{(b/t)\sqrt{f}}\right) \leq b$ (C3-2)		—	CRC: 
								For laced segments:	
							1.6.1	For ASTM A36 steel: $t \geq \frac{b}{32\sqrt{\frac{F_c}{f}}} \geq \frac{b}{64}$	
							2.6.1	For high-strength steel: $t \geq \frac{b\sqrt{F_y}}{6000\sqrt{\frac{F_c}{f}}} \geq \frac{b\sqrt{F_y}}{12\,000}$	
Laced Columns	3.12	K is modified to K': For $\frac{KL}{r} > 40$: $K' = K\sqrt{1+\frac{300}{(KL/r)^2}}$ For $\frac{KL}{r} \leq 40$: $K' = 1.1K$ (3.17)	1.7.84	l/r of member between lacing ≤ 40 and, $\leq \frac{2}{3}(\frac{l}{r}$ of member)	1.18.2.6	l/r of laced segments $\leq l/r$ of member	1.6.4.2	l/r of member between lacing ≤ 40 and, $\leq \frac{2}{3}(\frac{l}{r}$ of member)	
			1.7.84	Shearing force: $V = \frac{P}{100}\left[\frac{100}{\frac{l}{r}+10} + \frac{l/r}{3{,}300{,}000/F_y}\right]$	1.18.2.6	Lacing shall be proportioned to resist a shearing stress normal to the axis of the member equal to 2 percent of the total compressive stress in the member	1.6.4.1	Shearing force: For ASTM A36 steel: $V = \frac{P}{100}\left(\frac{100}{l/r+10} + \frac{l/r}{100}\right)$	
							2.6.3.1	For high-strength steel: $V = \frac{P}{100}\left(\frac{100}{l/r+10} + \frac{l/r}{3{,}600{,}000/F_y}\right)$	
			1.7.90	Stay plates near ends of column and at points where lacing is interrupted.	1.18.2.5	Tie plates at each end of column and at points where lacing is interrupted	1.6.3	Stay plates near ends of column and at points where lacing is interrupted.	

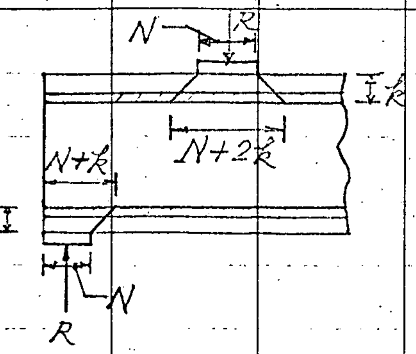
Topic	CRC Sec.	CRC	AASHO Art.	AASHO	AISC Sect.	AISC	AREA Art.	AREA	Notes
Columns with Perforated Plates	3.13	1. The perforations may be circular with straight edges, elliptical or circular. 2. $(c-a) \geq d$ 3. A and I based upon the net section. 4. If $a/r_f \leq 20$, and $a/r_f < L/3r_x$, the permissible load can be determined by the appropriate specification column stress applied to the column net section. 5. The net area of web at the perforation should be sufficient to resist $1/n$ times the transverse shear force, where n is the number of perforated plates. 6. The b/t ratio should conform to specification requirements for plates in high compression members.	1.7.84 1.7.84	(1) $a/m \leq 2$ (2) $(c-a) \geq d$ (3) The clear distance between the end perforation and the end of the cover plate shall not be less than 1.25 times the distance between points of support. (4) The point of support shall be the inner line of fasteners or fillet welds connecting the perforated plate to the flanges. (5) The periphery of the perforation at all points shall have a minimum radius of $1\frac{1}{2}$ inches. (6) For thickness of metal see Compression Members, Local Buckling (1.7.89). Shearing force formula for laced columns applies.	1.9.2.2 1.18.2.7	Width-to-thickness ratio at access holes is not greater than $317/\sqrt{F_y}$ $a/m \leq 2$ $(c-a) \geq d$ The periphery of the holes at all points shall have a minimum radius of $1\frac{1}{2}$ inches.	1.6.4.3 2.6.1 1.6.4.3 1.6.4.1 and 2.6.3.1	(a) Perforations shall be ovaloid or elliptical. (b) $a \leq 2m$, $a/r_f \leq 20$, $a/r_f < L/3r_x$ (c) $(c-a) \geq d$ (d) Thickness of perforated cover plate: $t \geq b/50$ also $t \geq w/12$ also $t \geq \frac{3cV}{2vh(c-a)}$ also, for high-strength steel: $t \geq \frac{b\sqrt{F_y}}{7500\sqrt{\frac{p_c}{f}}} \geq \frac{b\sqrt{F_y}}{15000}$ and, $t \geq \frac{w\sqrt{F_y}}{2300}$ (e) Splices are permitted with limitations. (f) The gross section of the plate through the perforation shall be considered as a part of the area of the member. Shearing force formula for laced columns applies.	CRC:  b (CRC) = w (AREA) d (CRC) = b (AREA)
Columns with Cover Plates							 1.6.1 2.6.1 1.6.1 2.6.1	For segments connected by solid cover plates: For ASTM A36 steel: $t \geq \frac{b}{32\sqrt{\frac{p_c}{f}}} \geq \frac{b}{64}$ For high-strength steel: $t \geq \frac{b\sqrt{F_y}}{6000\sqrt{p_c/f}} \geq \frac{b\sqrt{F_y}}{12000}$ Thickness of cover plate: For ASTM A36 steel: $t \geq b/40\sqrt{p_c/f} \geq b/80$ For high-strength steel: $t \geq b\sqrt{F_y}/7500\sqrt{p_c/f} \geq b\sqrt{F_y}/15000$	

Topic	Sec.	CRC	Art.	AASHO	Sect.	AISC	Art.	AREA	Notes
Columns with Batten Plates	 3.14	$\frac{KL}{r}=\sqrt{\left(\frac{L}{r}\right)^2+\frac{\pi^2}{12}\left(\frac{L_o}{r_o}\right)^2}$ (3.18) Moment resisted by each group of fasteners: $M_b=\frac{Q L_o}{2n}$ (3.19)		—		—		—	CRC: 
Beams with Box Sections	4.2	$\left(\frac{KL}{r}\right)_{equiv}=\sqrt{\frac{5.1 L S_x}{\sqrt{J I_y}}}$ (4.7) where $J=\frac{2b^2d^2}{\frac{b}{t}+\frac{d}{t_w}}$ (4.5)		—	1.5.1.4.1 1.5.1.4.4	Compact section: $F_b=0.66F_y$ Non compact section when $b/t \le 238/\sqrt{F_y}$ and $l \le (2500/F_y)d$ $F_b=0.60F_y$	1.4.1 2.4.1	For ASTM A36 steel: $P_c=20{,}000-0.4(L/R)_e^2$ For high-strength steel: $P_c=0.55F_y-\frac{0.55F_y^2}{1.8\times10^9}(L/R)_e^2$ where $\left(\frac{L}{R}\right)_e^2=\frac{3.95 l S_x\sqrt{ES/t}}{A\sqrt{I_y}}$	CRC: 
Beams in Bending	4.6	For doubly symmetric I sections: In elastic range: when $L/r_y > 4.64(d/t)$ $\sigma_c=\frac{3.06E}{Ld/r_y t}$ (4.17a) when $L/r_y < 4.64(d/t)$ $\sigma_c=\frac{14.2E}{(L/r_y)^2}$ (4.17b) In inelastic range, use Eq. (2.1b)	1.7.1	When compression flange is fully supported: $F_b=0.55F_y$ When compression flange is partially supported or is unsupported: $F_b=0.55F_y\left[1-\frac{(l/r')^2F_y}{4\pi^2E}\right]$ with $r'^2=b^2/12$ $F_b=0.55F_y\left[1-\frac{3(l/b)^2F_y}{\pi^2E}\right]$	1.5.1.4.1 1.5.1.4.2 1.5.1.4.6	Compact section with adequate lateral support: $F_b=0.66F_y$ Compact section except when $52.2/\sqrt{F_y} < b_f/2t_f < 95.0/\sqrt{F_y}$ $F_b=F_y\left[0.733-0.0014\left(\frac{b_f}{2t_f}\right)\sqrt{F_y}\right]$ (1.5-5) Non compact section when $\sqrt{\frac{102\times10^3C_b}{F_y}}<\frac{l}{r}<\sqrt{\frac{510\times10^3C_b}{F_y}}$ $F_b=\left[\frac{2}{3}-\frac{F_y(l/r)^2}{1530\times10^3C_b}\right]F_y$ (1.5-6a) when $\frac{l}{r}>\sqrt{\frac{510\times10^3C_b}{F_y}}$ $F_b=\frac{170\times10^3C_b}{(l/r)^2}$ (1.5-6b) or, for rectangular compression flange $F_b=\frac{12\times10^3C_b}{ld/A_f}$ (1.5-7) when $l\le 76.0b_f/\sqrt{F_y}$ $F_b=0.60F_y$	 1.4.1 2.4.1 1.7.1 and 2.7.1	For welded or rolled members: For ASTM A36 steel: $P_c=20{,}000-0.4(l/r_y)^2$ or $=\frac{10{,}500{,}000}{ld/A_f}$ The larger of the values computed by the formulas, but not to exceed 20,000 psi. For high-strength steel: $P_c=0.55F_y-\frac{0.55F_y^2}{1.8\times10^9}\left(\frac{l}{r_y}\right)^2$ or $=\frac{10{,}500{,}000}{ld/A_f}$ The larger of the values computed by the formulas, but not to exceed $0.55F_y$. For riveted or bolted members, only the first equations are permitted. The ratio of the distance between points of lateral supports and the radius of gyration of the portion of cross section above the neutral axis about an axis in the plane of the web shall not exceed 157 for A36 steel and $2990/\sqrt{F_y}$ for high-strength steel.	AASHO: The limiting values of l/b are given for various steels, which is obtained from $l/b \le \sqrt{\frac{\eta\pi^2E}{12F_y}}$

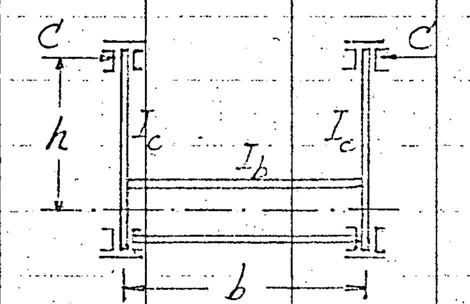
Topic	CRC Sec.	CRC	AASHO Art.	AASHO	AISC Sect.	AISC	AREA Art.	AREA	Notes
Beams and Plate Girders, Lateral Support	4.11	Designed for 2% of the total compressive force that exists concurrently in the compression flange of the laterally braced beam or girder.	1.7.22	Cross frames or diaphragms at each end and at intervals not to exceed 25 feet.	1.10.5	Designed for 1% of the total compression flange stress.	1.3.11 1.11.4	Designed for $2\frac{1}{2}$ % of the total axial stress in both members in that panel. Cross frames or diaphragms at each end and at intervals specified according to deck construction.	
					2.9	When $+1.0 > M/M_p > -0.5$ $l_{cr}/r_y = \frac{1375}{F_y} + 25$ When $-0.5 > M/M_p > -1.0$ $l_{cr}/r_y = \frac{1375}{F_y}$ (2.9-1)			Plastic design.
Plate Girders, Web Depth to Thickness Ratio	5.4	Vertical buckling of compression flange: $\frac{h}{t_w} < \frac{0.48E}{\sqrt{\sigma_y(\sigma_y+\sigma_{rc})}}$ (5.7)	1.7.71	Buckling of web plate by flexural bending: (A) Girders not stiffened longitudinally $t \geq \frac{D\sqrt{f_b}}{23,000} \geq \frac{D}{170}$ (B) Girders stiffened longitudinally $t \geq \frac{D\sqrt{f_b}}{46000} \geq \frac{D}{340}$	1.10.2	Vertical buckling of compression flange: $\frac{h}{t} \leq \frac{14,000}{\sqrt{F_y(F_y+16.5)}}$ For $a/h \leq 1.5$ $\frac{h}{t} \leq \frac{2000}{\sqrt{F_y}}$	1.7.3 2.7.2	Buckling of web plate by flexural bending: For ASTM A36 steel: $\frac{D}{t} < 170\sqrt{\frac{P_c}{f}}$ For high-strength steel: $\frac{D}{t} < \frac{32500}{\sqrt{F_y}}\sqrt{\frac{P_c}{f}}$	
Beams and Plate Girders, Local Buckling of Compression Flange	5.4	$\frac{b_f}{t_f} \leq 12 + \frac{L}{b_f}$ (5.12)	1.7.70	(A) Welded girders $\frac{b}{t} \leq \frac{3250}{\sqrt{f_b}} \leq 24$ (B) Riveted girders $\frac{b'}{t} \leq \frac{1625}{\sqrt{f_b}} \leq 12$	1.9.1.2	$\frac{b_f}{t_f} \leq \frac{95.0}{\sqrt{F_y}}$		See Compression Members, Local Buckling (1.6.2; 2.6.2)	CRC: Local buckling of compression flange will not occur prior to lateral torsional buckling of the member.
					2.7	For rolled I or WF shapes: F_y — $b_f/2t_f$ 36 — 8.5 42 — 8.0 45 — 7.4 50 — 7.0 55 — 6.6 60 — 6.3 65 — 6.0			Plastic design.

Topic	CRC Sec.	CRC	AASHO Art.	AASHO	AISC Sect.	AISC	AREA Art.	AREA	Notes
Plate Girders, Shear Strength	5.5	$\tau_u = \frac{\sigma_y}{\sqrt{3}}\left[\frac{\tau_c}{\tau_y} + \frac{1-\tau_c/\tau_y}{1.15\sqrt{1+\alpha^2}}\right]$ (5.22)	1.7.1	$F_v = 0.33 F_y$	1.10.5.2	$F_v = \frac{F_y}{2.89}(C_v) \le 0.4F_y$ (1.10-1) where $C_v = \frac{45000\,k}{F_y(h/t)^2} \le 0.8$ $C_v = \frac{190}{h/t}\sqrt{\frac{k}{F_y}} \ge 0.8$ or, when $C_v \le 1.0$ $F_v = \frac{F_y}{2.89}\left[C_v + \frac{1-C_v}{1.15\sqrt{1+(a/h)^2}}\right]$ $\le 0.4F_y$ (1.10-2)	1.4.1 2.4.1	Allowable shear stresses: For ASTM A36 steel: 12,500 psi For high-strength steel: $0.35\,F_y$	1. CRC and AISC: Based on ultimate strength in shear. 2. AASHO and AREA: Shear strength formula applicable with stiffener spacing requirement.
					2.5	$V_u \le 0.55\,F_y t d$ (2.5-1)			Plastic design.
Plate Girders, Combined Bending and Shear	5.6	$\sigma = \sigma_y \frac{1+\frac{1}{2}(A_w/A_f)\left[1-(\tau/\tau_u)^2\right]}{1+\frac{1}{6}(A_w/A_f)}$ (5.25)			1.10.7	$f_b \le \left(0.825 - 0.375\frac{f_v}{F_v}\right)F_y \le 0.6F_y$ (1.10-6)			AISC: For A514 girders, F_v by formula (1.10-1) if $f_b > 0.75F_b$.
Plate Girders, Bending	5.6	For inelastic range, when $0 < \lambda < \sqrt{2C_1}$, $\sigma_c/\sigma_y = 1 - \frac{\lambda^2}{4C_1}$ For elastic range, when $\lambda > \sqrt{2C_1}$ $\sigma_c/\sigma_y = C_1/\lambda^2$ (5.26) where $\lambda = \frac{1}{\pi}\frac{L}{r}\sqrt{\epsilon_y} = \frac{L}{\pi}\sqrt{\frac{\epsilon_y(A_f + A_w/6)}{I_f}}$ and, $C_1 = 1.75 - 1.05K + 0.3K^2$ $-0.46 < K < +1$		See Beams in Bending (1.7.1)	1.10.6	$F_b' \le F_b\left[1 - 0.0005\frac{A_w}{A_f}\left(\frac{h}{t} - \frac{760}{\sqrt{F_b}}\right)\right]$ (1.10-4)		See Beams in Bending (1.4.1; 2.4.1)	AISC: Considering stress reduction due to lateral web deflection.
Plate Girders, Transverse Stiffener Spacing	5.5 5.13	According to shear strength Eq. (5.22) The spacing between stiffeners at end panels shall be such that $f_v < \frac{\tau_c}{N}$	1.7.72	$d \le \frac{11000t}{\sqrt{f_v}} \le D$ Stiffeners may be omitted if $t \ge \frac{D\sqrt{f_v}}{7500} \ge \frac{D}{150}$ For the first two stiffeners at the ends of simply-supported girders shall be one-half the value specified.	1.10.5.3	Intermediate stiffeners are not required when $h/t < 260$ and $f_v < F_v$ by Formula (1.10-1). When stiffeners are required, spacing is according to shear strength, and $a/h \le \left(\frac{260}{h/t}\right)^2 < 3.0$ The spacing between stiffeners at end panels shall be such that the smaller panel dimension, a or h, shall not exceed $348t/\sqrt{f_v}$	1.7.8 2.7.3	For ASTM A36 steel: Intermediate stiffeners are needed if $D/t > 60$. $d \le 72$ inches or, $d \le \frac{10500t}{\sqrt{S}}$ For high-strength steel: Intermediate stiffeners are needed if $D/t > 11400/\sqrt{F_y}$ $d \le 72$ inches or, $d \le \frac{10500t}{\sqrt{S}}$	

Topic	CRC Sec.	CRC	AASHO Art.	AASHO	AISC Sect.	AISC	AREA Art.	AREA	Notes
Plate Girders, Transverse Stiffeners Flexural Rigidity	5.7	$\gamma_o^c = \frac{14}{(a/h)^3}$ (5.27) or, $\gamma_o^c = 4\left[7\left(\frac{h}{a}\right)^2 - 5\right]$ (5.28) or, for double stiffeners $\gamma_o^c = 27.75\left(\frac{h}{a}\right)^2 - 7.5$ (5.29) and, for single stiffeners $\gamma_o^c = 21.5\left(\frac{h}{a}\right)^2 - 7.5$ (5.30)	1.7.72	$I = \frac{d_o t^3 J}{10.92}$ where $J = 25\frac{D^2}{d^2} - 20 \geq 5.0$	1.10.5.4	The moment of inertia shall not be less than $(h/50)^4$	1.7.8	The width of the outstanding leg of each angle, or the width of the welded stiffener plate, shall be not more than 16 times its thickness and not less than 2 inches plus 1/30 of the depth of the girder.	
Plate Girders, Transverse Stiffener Area	5.8	$A_s = \frac{1}{2}\left(1 - \frac{\tau_c}{\tau_y}\right)\left[\alpha - \frac{\alpha^2}{\sqrt{1+\alpha^2}}\right] h t_w$ (5.32)			1.10.5.4	$A_{st} = \frac{1-C_v}{2}\left[\frac{a}{h} - \frac{(a/h)^2}{\sqrt{1+(a/h)^2}}\right] YDht$ (1.10-3)			
Plate Girders, Longitudinal Stiffeners	5.9	Stiffeners at $h/5$ $\gamma_o^c = 3.87 + 5.1\alpha + (8.82 + 77.6\delta)\alpha^2$ where $\delta = A_s/ht_w$, $\gamma_o^c = EI_o/Dh$ $\alpha = a/h$	1.7.73	Stiffeners at $D/5$ $I = Dt^3\left(2.4\frac{d_o^2}{D^2} - 0.13\right)$ The thickness of the stiffener shall not be less than $\frac{b'\sqrt{f_b}}{2250}$			5.2.8	Stiffeners at $D/5$ $I_s = Dt^3\left(2.4\frac{d^2}{D^2} - 0.13\right)$ The thickness of the stiffener shall not be less than $\frac{b'\sqrt{f_b}}{2250}$	CRC = Rigidities for stiffeners at different locations are also given.
Beams and Plate Girders, Bearing Stiffeners			1.7.67 1.7.74	Bearing stiffeners required when $f_v \geq 0.75 F_v$ and shall be designed as columns considering effective width of web. The thickness of the stiffener shall not be less than $\frac{b'}{12}\sqrt{\frac{F_y}{33000}}$	1.10.5.1	Bearing stiffeners shall be placed in pairs at unframed ends and where required at points of concentrated load (see Web Crippling), and shall be designed as columns considering effective width of the web. For width-to-thickness ratio, see Compression Members, Local Buckling (1.9.1.2; 1.9.2.2)	1.7.7	Stiffeners shall be placed in pairs at end bearings and at points of bearing of concentrated loads. For the width of outstanding stiffener plates, see Compression Members, Local Buckling. (1.6.2 ; 2.6.2)	

Topic	CRC		AASHO		AISC		AREA		Notes
	Sec.		Art.		Sect.		Art.		
Beams and Plate Girders, Web Crippling		—		—	1.10.10.1	For interior loads: $\frac{R}{t(N+2k)} \leq 0.75F_y$ (1.10-7) For end-reactions: $\frac{R}{t(N+k)} \leq 0.75F_y$ (1.10-8)		—	
Plate Girders, Edge Loading	5.13	For loaded edge clamped, others simply supported: $\sigma_n = \left(5.5 + \frac{4h^2}{a^2}\right)\left[\frac{\pi^2 E}{12(1-\nu^2)(h/t_w)^2}\right]$ (5.35a) For all edges simply supported: $\sigma_n = \left(2 + \frac{4h^2}{a^2}\right)\left[\frac{\pi^2 E}{12(1-\nu^2)(h/t_w)^2}\right]$ (5.35b)		—	1.10.10.2	The total compression stresses shall not exceed the following: when flange restrained against rotation: $\left[5.5 + \frac{4}{(a/h)^2}\right]\frac{10,000}{(h/t)^2}$ (1.10-9) When flange is not restrained against rotation: $\left[2 + \frac{4}{(a/h)^2}\right]\frac{10,000}{(h/t)^2}$ (1.10-10)		—	
Beam-Columns, Initial Yield	6.2	For elastic range, when $\sigma_{max} \leq \sigma_y$ $\sigma_{max} = \frac{P}{A}\left[1 + \left(\frac{ec}{r^2} + \frac{e_0 c}{r^2}\right)\sec\left(\frac{L}{2r}\sqrt{\frac{P}{AE}}\right)\right]$ (6.1) or, $F_a = \frac{\sigma_y/n}{\left[1 + \left(\frac{ec}{r^2} + \frac{e_0 c}{r^2}\right)\sec\left(\frac{L}{2r}\sqrt{\frac{nF_a}{E}}\right)\right]}$ (6.2) Approximate formula, for equal end eccentricities: $F_a = \frac{\sigma_y/n}{1 + \left(\frac{1+0.23n\alpha}{1-n\alpha}\right)\frac{ec}{r^2}}$ (6.6) when subjected to uniform total lateral load $W = kP$: $F_a = \frac{\sigma_y/n}{1 + \left[\frac{1+0.028\left(\frac{nF_a}{\sigma_E}\right)}{1-\left(\frac{nF_a}{\sigma_E}\right)}\right]\frac{k}{8}\frac{Lc}{r^2}}$ (6.7)	App. C	For effects of initial curvature and eccentricity, see Columns, Effect of Initial Curvature. For effects of transverse loading: $\frac{P}{S} = \frac{\frac{f_y}{n} - \frac{Mc}{I}}{1 + \left[0.25 + (e_g + d)\frac{c}{r^2}\right]\sec\frac{\phi}{2}}$ (E)		See Beam-Columns, Interaction Without Lateral Buckling. (1.6.1; 2.4)		See Beam-Columns, Biaxial Bending. (1.3.14.1; 2.3.2.1)	CRC and AASHO: Initial out-of-straightness considered.

Topic	CRC		AASHO		AISC		AREA		Notes
	Sec.		Art.		Sect.		Art.		
Beam-Columns, Interaction without Lateral Buckling	6.3	Interaction formula: $\frac{P}{P_u}+\frac{M}{M_u}\le 1$ (6.8) For end moments and axial loads, in the elastic range: $\frac{P}{P_u}+\frac{M_o}{M_u[1-(P/P_e)]}\le 1$ (6.10) For eccentrically loaded columns having equal end eccentricity: $\frac{P}{P_u}+\frac{Pe}{M_u[1-(P/P_e)]}\le 1$ (6.11)			1.6.1	$\frac{f_a}{F_a}+\frac{C_{mx}f_{bx}}{(1-\frac{f_a}{F'_{ex}})F_{bx}}+\frac{C_{my}f_{by}}{(1-\frac{f_a}{F'_{ey}})F_{by}}\le 1.0$ (1.6-1a) $\frac{f_a}{0.60F_y}+\frac{f_{bx}}{F_{bx}}+\frac{f_{by}}{F_{by}}\le 1.0$ (1.6-1b) when $\frac{f_a}{F_a}\le 0.15$, use $\frac{f_a}{F_a}+\frac{f_{bx}}{F_{bx}}+\frac{f_{by}}{F_{by}}\le 1.0$ (1.6-2)		See Beam-Columns, Biaxial Bending. (1.3.14.1; 2.3.2.1)	AASHO: For combined effects of end moments and axial loads, see Beam-Columns, Initial Yield, Appendix C.
					2.4	$\frac{P}{P_{cr}}+\frac{C_m M}{(1-\frac{P}{P_e})M_m}\le 1.0$ (2.4-2) $\frac{P}{P_y}+\frac{M}{1.18M_p}\le 1.0$; $M\le M_p$ (2.4-3) where $P_e=\frac{23}{12}AF'_e$ $M_m=M_p$, if braced in the weak direction. $M_m=\left[1.07-\frac{(l/r_y)\sqrt{F_y}}{3160}\right]M_p\le M_p$ (2.4-4), if unbraced in the weak direction.			Plastic design
Beam-Columns, Interaction Laterally Unsupported	6.4	$\frac{P}{P_u}+\frac{M_o}{M_u[1-(P/P_e)]}\le 1$ (6.14)				See Beam-Columns, Interaction Without Lateral Buckling. (1.6.1; 2.4)		See Beam-Columns, Biaxial Bending. (1.3.14.1; 2.3.2.1)	
Beam-Columns, Unequal End Moments	6.6	$\frac{M_{eq}}{M_a}=0.6+0.4\frac{M_b}{M_a}\ge 0.4$ (6.17)		See Beam-Columns, Initial Yield, (Appendix C)	1.6.1	$C_m=0.6-0.4\frac{M_1}{M_2}\ge 0.4$			CRC: Moment diagram is a straight line.

Topic	CRC Sec.	CRC	AASHO Art.	AASHO	AISC Sect.	AISC	AREA Art.	AREA	Notes
Beam-Columns, Biaxial Bending	6.7	$\frac{P}{P_u}+\frac{M_{o(x-x)}}{M_{u(x-x)}[1-(P/P_{e(x-x)})]}+\frac{M_{o(y-y)}}{M_{u(y-y)}[1-(P/P_{e(y-y)})]}\leq 1$ (6.19)				See Beam-Columns, Interaction Without Lateral Buckling. (1.6.1; 2.4)	1.3.14.1 and 2.3.2.1 1.3.14.1 2.3.2.1	Axial compression and bending: when $f_a/F_a \leq 0.15$ $\frac{f_a}{F_a}+\frac{f_{b1}}{F_{b1}}+\frac{f_{b2}}{F_{b2}}\leq 1.0$ when $f_a/F_a > 0.15$ $\frac{f_a}{F_a}+\frac{f_{b1}}{F_{b1}[1-\frac{f_a}{200\times10^6}(\frac{k_1 l_1}{r_1})^2]}+\frac{f_{b2}}{F_{b2}[1-\frac{f_a}{200\times10^6}(\frac{k_2 l_2}{r_2})^2]}\leq 1.0$ and, at points braced in the planes of bending: For ASTM A36 steel: $\frac{f_a}{20,000}+\frac{f_{b1}}{F_{b1}}+\frac{f_{b2}}{F_{b2}}\leq 1.0$ For high-strength steel: $\frac{f_a}{0.55F_y}+\frac{f_{b1}}{F_{b1}}+\frac{f_{b2}}{F_{b2}}\leq 1.0$	
Pony Trusses, Buckling of the Compression Chord	7.2	Compression-chord buckling load: $P_c = A\,\sigma_c$ (7.4) Required transverse frame spring-constant: $C_{req} = 1.46\,\frac{P_c}{l}$ (7.5) Transverse frame spring constant furnished: $C = \frac{E}{h^2[(h/3I_c)+(b/2I_b)]}$ (7.6a)	1.7.86	The top chord shall be considered as a column with elastic lateral supports at the panel points. The critical buckling force of the column shall exceed the maximum force from dead load, live load and impact in any panel of the top chord by not less than 50 percent.					CRC: The design formulas are also applicable to the design of plate girders with elastically braced compression flanges, see Section 7.6. 2. C_{req} may be unconservative for short pony trusses. 3. More accurate solution by table or curves is given in the Guide. 

Topic	CRC	AASHO	AISC	AREA	Notes
	Sec.	Art.	Sect.	Art.	
Through Girders	See Pony Trusses, Buckling of the Compression Chord (7.2)				AREA: Design of top flange of through plate girder is not specified, presumably according to bending of plate girders. Details for bracing are specified in Art. 1.11.1.
Hybrid Girders		See Interim Specification, Art. INT. 11(67)	See Section 1.10		
Composite Beams and Girders		See Art. 1.7.97 to Art. 1.7.102 and Interim Specification Art. INT. 5(67) to INT. 9(67)	See Section 1.11	See Section 5.1	
Box Girders		See Interim Specification Art. INT. 10(67)			
Arch		See Art. 1.7.91 and Section 9			

APPENDIX — NOMENCLATURE (SYMBOLS)

CRC Symbol	CRC Defining Statement	AASHO Symbol	AASHO Defining Statement	AISC Symbol	AISC Defining Statement	AREA Symbol	AREA Defining Statement
A	Area of cross section.	A	Gross cross-sectional area of column, in sq. inches.	A	Gross area of member.	A	Total area enclosed within the center lines of the box-type member webs and flanges, in sq. inches.
A_f	Area of flange.			A_f	Area of compression flange.	A_f	Area of the smaller flange, in square inches, excluding any portion of the web.
A_s	Area of stiffener cross section.			A_{st}	Cross-sectional area of stiffener or pair of stiffeners.		
A_w	Area of web.			A_w	Area of girder web.		
a	Length of side of stiffener plate.	d	Required distance between stiffeners, in inches.	a	Clear distance between transverse stiffeners.	d	Clear distance between transverse stiffeners.
		d_o	Actual distance between stiffeners, in inches.				
	Length of perforation in a perforated plate					a	Length of perforation.
b	Width of plate	b	Width of plate.	b	Actual width of stiffened compression element.	h	Width of plate.
	Width of pony truss bridge center to center of trusses.						
	Length of short side of a box section, center to center of long sides.						
	Transverse distance from edge of a perforation to nearest line of longitudinal fasteners.					w	Distance from the line of connection of the cover plate to the edge of the perforation.
b_e	Effective plate width.			b_e	Effective width of stiffened compression element.		
b_f	Half-width of flange.	b	Flange plate width.	b_f	Flange width of rolled beam or plate girder.		
		b'	Width of outstanding legs of elements in compression.				

CRC		AASHO		AISC		AREA	
Symbol	Defining Statement	Symbol	Defining Statement	Symbol	Defining Statement	Symbol	Defining Statement
		b'	Width of stiffeners.			b'	Width of stiffeners.
		B	$\sqrt{\alpha^2 - 2\alpha \cos \Phi + 1}$				
C	Transverse pony truss bridge frame spring constant, particularly the least one.						
C_1	Coefficients for lateral-torsional buckling; equal to $1.75 - 1.05\kappa + 0.3\kappa^2$			C_b	Bending coefficient dependent upon moment gradient; equal to $1.75 + 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2$		
				C_c	Column slenderness ratio dividing elastic and inelastic buckling; equal to $\sqrt{\frac{2\pi^2 E}{F_y}}$, except in Appendix C.		
				C_m	Coefficient applied to bending term in interaction formula and dependent upon column curvature caused by applied moments.		
				C_v	Ratio of "critical" web stress, according to the linear buckling theory, to the shear yield stress of web material; equal to $\frac{\pi^2 E k\sqrt{3}}{12(1-\nu^2)(h/t)^2 F_y}$ or $\frac{190}{h/t}\sqrt{\frac{k}{F_y}}$ or $\frac{190}{h/t}\sqrt{\frac{k}{F_y}}$ (See Sect. 1.10.5.2)		

CRC		AASHO		AISC		AREA	
Symbol	Defining Statement	Symbol	Defining Statement	Symbol	Defining Statement	Symbol	Defining Statement
c	Distance to extreme fiber of beam or column section in bending.	c	Distance from neutral axis to the extreme fiber in compression.	c	Distance from neutral axis to extreme fiber of beam.		
	Distance center-to-center of perforations in a perforated plate.					c	Spacing of perforations.
D	Flexural rigidity of a plate per unit width.						
				D	Factor dependent upon type of transverse stiffeners.		
d	Depth of a section.			d	Depth of beam or girder.	d	Overall depth of the member, in inches.
	Transverse distance between lines of longitudinal fasteners in a perforated plate.					b	Unsupported distance between the nearest lines of fasteners or welds, or between the roots of rolled flanges.
		d	Deflection due to the transverse components of externally applied loads, in inches.				
E	Stress-strain modulus of elasticity.	E	Modulus of elasticity of steel (29,000,000 psi).	E	Modulus of elasticity of steel (29,000 ksi).		
E_{st}	Strain-hardening modulus (initial).						
e	Eccentricity of end load in a beam-column.	e_g	Eccentricity of applied load at the end of column having the greater computed moment, in inches.				
		e_s	Eccentricity at opposite end.				
e_o	Assumed equivalent eccentricity (representing defects, etc.)						

CRC Symbol	CRC Defining Statement	AASHO Symbol	AASHO Defining Statement	AISC Symbol	AISC Defining Statement	AREA Symbol	AREA Defining Statement
F_a	Allowable average compressive stress in axially loaded members.	F_a	Allowable compressive stress in concentrically loaded columns.	F_a	Axial stress permitted in the absence of bending moment.	F_a	Axial stress that would be permitted if axial force alone existed.
						P_c	Allowable unit stress determined by the kl/r of the member.
						F_{as}	Axial compressive stress, permitted in the absence of bending moment for bracing and other secondary members.		
F_b	Allowable compressive bending stress.	F_b	Allowable compressive stress in extreme fibers of sections subject to bending.	F_b	Bending stress permitted in the absence of axial force.	F_{b1}; F_{b2}	Compressive bending stress about axes 1-1 and 2-2, respectively, that would be permitted if bending alone existed.
				F_b'	Allowable bending stress in compression flange of plate girders as reduced for hybrid girders or because of large web depth-to-thickness ratio.		
				F_e'	Euler stress divided by factor of safety; equal to $\frac{12\pi^2 E}{23(K l_b / r_b)^2}$		
		F_v	Allowable shear stress.	F_v	Allowable shear stress	v	Allowable unit shear specified for plate girder webs.
				f	Axial compression load on member divided by effective area.	f	Actual average unit stress in compression.
							Extreme fiber stress in the compression flange of plate girder.
f_a	Average compressive stress due to axial load.	f_a	Calculated compressive stress.	f_a	Computed axial stress.	f_a	Computed axial stress.

CRC		AASHO		AISC		AREA	
Symbol	Defining Statement	Symbol	Defining Statement	Symbol	Defining Statement	Symbol	Defining Statement
f_b	Compressive stress due to bending moment.	f_b	Calculated compressive bending stress in the flange.	f_b	Computed bending stress.	f_b	Calculated compressive bending stress in the flange.
						f_{b1}, f_{b2}	Computed compressive bending stress about axes 1-1 and 2-2, respectively, at the point under consideration.
		f_s	Permissible average unit stress for steel columns.				
h	Clear depth of plate girder web between flange components	D	Unsupported distance between flange components, in inches.	h	Clear distance between flanges of a beam or girder.	D	Clear distance between flanges, in inches.
	Depth of pony truss at truss vertical, measured from center of floorbeam to center of top chord.						
	Transverse distance between lines of fasteners in a battened column.						
I	Moment-of-inertia of cross section	I	Moment of inertia of section about an axis perpendicular to the plane of bending, in (inches)^4.				
I_b	Moment-of-inertia of floorbeam in a pony truss.						
I_c	Moment-of-inertia of truss vertical in a pony truss						
I_f	Moment-of-inertia of compression flange of plate girder						

CRC Symbol	CRC Defining Statement	AASHO Symbol	AASHO Defining Statement	AISC Symbol	AISC Defining Statement	AREA Symbol	AREA Defining Statement
I_o	Optimum moment-of-inertia of web stiffener in a plate girder.	I	Minimum moment of inertia of stiffeners about the edge in contact with the web plate.			I_E	Minimum required moment of inertia of longitudinal stiffeners about edge in contact with web plate.
I_y	Moment-of-inertia of cross section about y axis.					I_y	Moment of inertia of the box-type member about its minor axis, in (inches)^4.
J	Torsion constant.						
K	Effective or equivalent length factor.			K	Effective length factor.	k	Effective length factor.
K'	Modified effective length factor for laced columns.						
k	Coefficient of proportionality. Coefficient applied in plate buckling.			k	Coefficient relating linear buckling strength of a plate to its dimensions and condition of edge support. Distance from outer face of flange to web toe of fillet.		
						k_1, k_2	Effective length factor of the compression member about axes 1-1 and 2-2, respectively.
L	Length of member, particularly a laterally unbraced length.	l	Length of member or length of unsupported flange, in inches.	l	Actual unbraced length, in inches.	l	Length of the compression member or the distance between points of lateral support for the compression flange, in inch.
		L	Effective length of the column				
L_o	Sublength of laced column; distance between lacing-bar connections or distance between centers of batten plates.						

CRC		[illegible]		AISC		AREA	
Symbol	Defining Statement	Symbol	Defining Statement	Symbol	Defining Statement	Symbol	Defining Statement
l	Panel length in a pony truss bridge.						
						l_1, l_2	Length of the compression member about axes 1-1 and 2-2, respectively.
				l_b	Actual unbraced length in plane of bending, in inches.		
				l_{cr}	Critical unbraced length adjacent to plastic hinge, in inches.		
M	Bending moment	M	Moment due to the transverse components of externally applied load, in inch pounds.	M	Moment (kip-ft)		
M_a	End moment acting on a beam-column at end a.			M_2	Larger moment at end of unbraced length of beam-column.		
M_b	End moment acting on a beam-column at end b.			M_1	Smaller moment at end of unbraced length of beam-column.		
	Moment resisted by each group of fasteners.						
M_{eq}	Equivalent uniform moment in a beam column.						
				M_m	Critical moment that can be resisted by a plastically designed member in absence of axial load.		
$M_o, M_{o(x-x)}, M_{o(y-y)}$	Moment in a beam-column without regard to moment caused by deflection.						

CRC Symbol	CRC Defining Statement	AASHO Symbol	AASHO Defining Statement	AISC Symbol	AISC Defining Statement	AREA Symbol	AREA Defining Statement
				M_p	Plastic moment.		
$M_u, M_{u(x-x)}, M_{u(y-y)}$	Ultimate bending moment in the absence of axial load in a beam-column.						
m	Width of a perforation in a perforated plate.						
N, n	A factor-of-safety.	η	Factor of safety based on yield point.				
				N	Length of bearing of applied load (inches).		
n	Number of perforated plates used in a column. Number of parallel planes of battens in a battened column.						
P	Column axial load.	P	Allowable compressive axial load on members or load parallel to the axis of the member, in lbs.	P	Applied load (kips)	P	Allowable compressive axial load on member.
P_c	Chord stress in a truss at maximum load.						
				P_{cr}	$1.70\ A F_a$		
$P_e, P_{e(x-x)}, P_{e(y-y)}$	Euler buckling load			P_e	$1.92\ A F_e'$		
P_u	Ultimate load of axially loaded column.						
P_y	Column axial load at full-yield condition.			P_y	Plastic axial load; equal to profile area times specified minimum yield stress.		

CRC Symbol	CRC Defining Statement	AASHO Symbol	AASHO Defining Statement	AISC Symbol	AISC Defining Statement	AREA Symbol	AREA Defining Statement
Q	Transverse shear in centrally loaded column.	V	Shearing force normal to the compression members in pounds.			V	Shearing force normal to the compression members.
				R	Reaction or concentrated transverse load applied to beam or girder (kips)		
r	Radius-of-gyration of member	r	Radius of gyration of member, in inches	r	Governing radius of gyration.	r	Radius of gyration of the compression member, in inches.
						r_1, r_2	Radius of gyration of the compression member about axes 1-1 and 2-2, respectively.
				r_b	Radius of gyration about axis of concurrent bending.		
r_f	Radius-of-gyration of column flange.	r'	Radius of gyration of the compression flange about the axis in the plane of the web.				
r_o	Radius-of-gyration of one chord in a battened column.						
r_x	Radius-of-gyration about the centroidal axis $x-x$ (strong axis).						
r_y	Radius-of-gyration about the centroidal axis $y-y$ (weak axis).			r_y	Lesser radius of gyration.	r_y	Radius of gyration of the compression flange and that portion of the web area on the compression side of the axis of bending, about the axis in the plane of the web, in inches.
S_x	Section modulus about $x-x$ axis.					S_x	Section modulus of the box-type member about its major axis, in (inches).
						s	Length of flange or web component of the box-type member.

CRC		[illegible]		AISC		AREA	
Symbol	Defining Statement	Symbol	Defining Statement	Symbol	Defining Statement	Symbol	Defining Statement
t	A thickness.	t	Plate or web thickness	t	Girder, beam or column web thickness.	t	Thickness of plate or web.
t_f	Thickness of compression flange.	t	Flange plate thickness	t_f	Flange thickness.	t	Flange plate thickness.
t_w	Thickness of web.	t	Thickness of web	t	Thickness of web.	t	Thickness of web.
						U	Maximum transverse shearing force in the plate of the plate.
W	Uniformly distributed total lateral load in a beam-column.						
Y	Ratio of yield stress of web steel to yield stress of stiffener steel.						
α	Aspect ratio a/h for stiffened plates. Load ratio P/P_e.						
		α	$\left(\frac{e_s c}{r^2}+0.25\right)\Big/\left(\frac{e_o c}{r^2}+0.25\right)$ when e_o and e_s lie on the same side of the column axis, α is positive; when on opposite sides, α is negative.				
γ_o	Optimum relative stiffness of stiffener to web in a plate girder.	J	Required ratio of rigidity of one transverse stiffener to that of the web plate				
δ	Buckling parameter for a stiffened plate A_s/ht_w.						
δ_o	Maximum initial out-of-straightness of a column.						

CRC Symbol	CRC Defining Statement	AASHO Symbol	AASHO Defining Statement	AISC Symbol	AISC Defining Statement	AREA Symbol	AREA Defining Statement
ϵ_y	Elastic strain at yield stress						
κ	Moment coefficient for lateral torsional buckling.						
λ	Slenderness function. $\sqrt{\sigma_y/\sigma_e}$, $\sqrt{\sigma_y/\sigma_c}$						
ν	Poisson's ratio			ν	Poisson's ratio, may be taken as 0.3 for steel.		
σ	Normal stress						
σ_a	Average normal stress.						
σ_c	Critical stress						
σ_e	Average stress at Euler buckling load.						
σ_{max}	Maximum combined stress due to column load and bending moment.						
σ_n	Transverse normal stress in a plate-girder web.						
σ_{rc}	Maximum residual compressive stress.						
σ_y	Yield stress level.	F_y	Specified minimum yield point.	F_y	Specified minimum yield stress of the type of steel being used (ksi).	F_y	Yield point of the steel as specified in Art. 2.2.1
τ	Shear stress	f_v	Average calculated unit shearing stress in the gross section of the web plate at the point considered.	f_v	Computed shear stress.	S	Unit shearing stress in the gross section of the web at the point under consideration, in psi.
τ_c	Shear stress at buckling load for plate girders						
τ_u	Ultimate shear stress for plate girders.			V_u	Statical shear produced by "ultimate" load in plastic design.		
τ_y	Shear stress at tension yield in plate girders.						
		Φ	$\frac{L}{r}\sqrt{\frac{\eta f_s}{E}}$ radians				

SUBJECT INDEX

(According to section number in CRC Guide)

SUBJECT INDEX

(Cont'd)

SUBJECT INDEX

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