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A COMPARISON: AREA (1968) AND CRC GUIDE (1966)

by

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ABSTRACT

The objective of this report is to look at the stability portions of the most recent draft of the AREA Specification(1) and make a comparison with the provisions in the second edition of the CRC Guide.(2)

It is hoped that this report will be helpful to pinpoint those areas where perhaps further study is needed, since it will show potential differences in philosophy--some of which are necessary; and a number of which undoubtedly will be simply the result of differences in practice.

It is also hoped that the material presented may be of help in the future deliberations of both organizations.

(1) American Railway Engineering Association
SPECIFICATIONS FOR STEEL RAILWAY BRIDGES, AREA Bulletin 611, January 1968, as revised for approval in September 1968

(2) Column Research Council
GUIDE TO DESIGN CRITERIA FOR METAL COMPRESSION MEMBERS, 2nd Edition, John Wiley & Sons, Inc., New York, 1966

NOMENCLATURE

A.R.E.A.		C.R.C.	
Symbol	Defining Statement	Defining Statement	Symbol
A	Area of entire flexural member section	A coefficient. Area of cross section	A
A_f	Area of the compression flange	Area of flange	A_f
	--	Area of stiffener cross section	A_s
	--	Area of web	A_w
a	Length of perforation	Length of side of stiffened plate. Length of perforation in a perforated plate. Torsion bending constant for an I-section	a
	--	A coefficient	B
	--	Width of rectangular cross section. Width of pony truss bridge, center to center of trusses	b
	--	Effective plate width	b_e
	--	Half-width of flange	b_f
	--	A coefficient Transverse pony truss bridge frame spring constant	C
	--	Coefficients for lateral-torsional buckling	C_1, C_2 C_4
	--	Required transverse pony truss bridge frame spring constant	C_{req}'

NOMENCLATURE (Cont'd)

A.R.E.A.		C.R.C.	
Symbol	Defining Statement	Defining Statement	Symbol
c	Spacing of perforations	Distance to extreme fiber of beam or column section in bending. Distance center-to-center of perforations in a perforated plate. One-half distance between batten fasteners, measured longitudinally.	c
	--	Flexural rigidity of a plate per unit width	D
d	Overall depth of the member. Clear distance between stiffeners	Depth of a section. Transverse distance between lines of longitudinal fasteners in a perforated plate.	d
	--	Stress-strain modulus of elasticity	E
	--	Strain-hardening modulus (initial)	E_{st}
	--	Eccentricity of end load in a beam-column	e
	--	Assumed equivalent eccentricity (representing defects, etc.)	e_o
F_a	Axial stress that would be permitted if axial force alone existed	Allowable average compressive stress in axially loaded members	F_a

NOMENCLATURE (Cont'd)

A.R.E.A.		C.R.C.	
Symbol	Defining Statement	Defining Statement	Symbol
F_{b1}, F_{b2}	Compressive bending stress about axes 1-1 and 2-2, respectively, that would be permitted if bending alone existed.	--	
f	Extreme fiber stress in the compression flange	--	
f_a	Computed axial stress	--	
f_{b1}, f_{b2}	Computed compressive bending stress about axes 1-1 and 2-2, respectively, at the point under consideration	--	
	--	Elastic shear modulus	G
h	Clear distance between the flanges. Width of plate.	Clear depth of plate girder web between flange components. Depth of pony truss at truss vertical, measured from center of floorbeam to center of top chord.	h
	--	Moment-of-inertia of floorbeam in a pony truss	I_b
	--	Moment-of-inertia of truss vertical in a pony truss	I_c
	--	Moment-of-inertia of the compression flange of plate girders	I_f

NOMENCLATURE (Cont'd)

A.R.E.A.		C.R.C.	
Symbol	Defining Statement	Defining Statement	Symbol
	--	Optimum moment-of-inertia of web stiffener in a plate girder	I_o
	--	Moment-of-inertia of cross section about y axis	I_y
	--	Torsion constant	J
	--	Effective or equivalent length factor	K
k	Effective length factor	Coefficient of proportionality, w/p Coefficient applied in plate buckling	k
k_1, k_2	Effective length factor of the compression member about axes 1-1 and 2-2, respectively	--	
ℓ	Length of the compression member. Distance between points of lateral support for the compression flange	Length of member, particularly a laterally unbraced length	L
ℓ_1, ℓ_2	Length of the compression member about axes 1-1 and 2-2, respectively	--	
ℓ_o	Length of compression lacing-bar connections	Sublength of laced column; distance between lacing-bar connection or distance between centers of batten plates	L_o

NOMENCLATURE (Cont'd).

A.R.E.A.		C.R.C.	
Symbol	Defining Statement	Defining Statement	Symbol
	--	Panel length in a pony truss bridge	ℓ
	--	Bending moment	M
	--	Moment resisted by each group of fasteners	M_b
	--	Applied end moment	M_o
	--	Moment in a beam-column without regard to moment caused by deflection	$M_{o(x-x)}$, $M_{o(y-y)}$
	--	Ultimate bending moment in the absence of axial load in a beam-column	M_u, $M_{u(x-x)}$, $M_{u(y-y)}$
	--	A factor-of-safety. Number of parallel planes of battens in a battened column	n
P	Allowable compressive axial load on member	Column axial load	P
	--	Chord stress in a truss at maximum load	P_c
	--	Euler buckling load	P_e
	--	Ultimate load of axially loaded column	P_u
	--	Column axial load at full-yield condition	P_y

NOMENCLATURE (Cont'd)

A.R.E.A.		C.R.C.	
Symbol	Defining Statement	Defining Statement	Symbol
p	Allowable extreme fiber stress	--	
	--	Transverse shear in centrally loaded column	Q
r	Radius-of-gyration of the compression member	Radius-of-gyration of member	r
r_1, r_2	Radius-of-gyration of the compression member about axes 1-1 and 2-2, respectively	--	
	--	Radius-of-gyration of column flange	r_f
	--	Radius-of-gyration of one chord in a battened column	r_o
	--	Radius-of-gyration about the centroidal axis x-x (strong axes)	r_x
r_y	Radius-of-gyration of the entire section about the axis in the plane of the web	Radius-of-gyration about the centroidal axis y-y (weak axis)	r_y
S	Unit shearing stress, gross section, in web at point considered	--	
	--	Section modulus about x-x axis	S_x

NOMENCLATURE (Cont'd)

A.R.E.A.		C.R.C.	
Symbol	Defining Statement	Defining Statement	Symbol
t	Thickness of plate Thickness of web	A thickness	t
	--	Thickness of compression flange	t_f
	--	Thickness of web plates of box-section beam. Thickness of web	t_w
U	Maximum transverse shearing force in the plane of the perforated plate	--	
V	Shearing force normal to the member in the plane of lacing or plates	Transverse shear force in plate girder	V
v	Allowable unit shear specified for plate girder webs	--	
	--	Uniformly distributed total lateral load in a beam-column	w
	--	Aspect ratio a/h for stiffened plates Load ratio P/P_e	α
	--	Buckling parameter for a stiffened plate EI/Dh	γ
	--	Optimum relative stiffness of stiffener to web in a plate girder EI_o/Da, EI_o/Dh	γ_o
	--	Buckling parameter for a stiffened plate A_s/ht_w	δ

NOMENCLATURE (Cont'd)

A.R.E.A.		C.R.C.	
Symbol	Defining Statement	Defining Statement	Symbol
	--	Elastic strain at yield stress	ϵ_y
	--	Moment coefficient for lateral torsional buckling	K
	--	Slenderness function $\sqrt{\sigma_y/\sigma_c}$	λ
	--	Normal stress	σ
	--	Critical stress	σ_c
	--	Average stress at Euler buckling load	σ_e
	--	Yield stress level	σ_y
	--	Shear stress	τ
	--	Shear stress at buckling load for plate girder	τ_c
	--	Ultimate shear stress for plate girder	τ_u
	--	Shear stress at tension yield in plate girder	τ_y

217.36 Sept., '68
(as per AREA sequence)

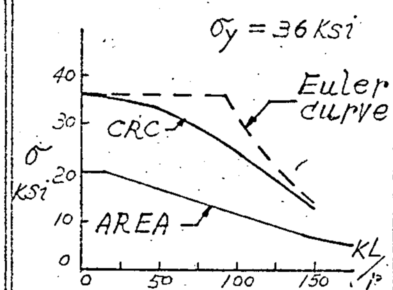
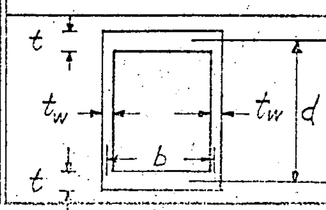
AREA BRIDGE SPECIFICATIONS AND THE CRC GUIDE

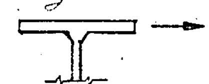
A COMPARISON

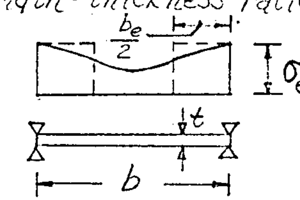
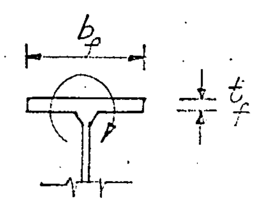
Topic	AREA Specification (1)		CRC Guide (2)		Notes
	Art.	Formula and/or Philosophy	Sec	Formula and/or Philosophy	
Beam – Columns in biaxial bending	1.3.14.1	Combined Stresses: Axial compression and bending: When $f_a/F_a \leq 0.15$ $\frac{f_a}{F_a} + \frac{f_{b1}}{F_{b1}} + \frac{f_{b2}}{F_{b2}} \leq 1.0$ When $f_a/F_a > 0.15$ $\frac{f_a}{F_a} + \frac{f_{b1}}{F_{b1}\left[1-\frac{f_a}{200\times10^6}\left(\frac{k_1 l_1}{r_1}\right)^2\right]} + \frac{f_{b2}}{F_{b2}\left[1-\frac{f_a}{200\times10^6}\left(\frac{k_2 l_2}{r_2}\right)^2\right]} \leq 1.0$ At points braced in the planes of bending: $\frac{f_a}{20{,}000} + \frac{f_{b1}}{F_{b1}} + \frac{f_{b2}}{F_{b2}} \leq 1.0$	6.7	$\frac{P}{P_u} + \frac{M_{o(x-x)}}{M_{u(x-x)}\left[1-\left(P/P_{e(x-x)}\right)\right]} + \frac{M_{o(y-y)}}{M_{u(y-y)}\left[1-\left(P/P_{e(y-y)}\right)\right]} \leq 1$ (6.19)	AREA is a format modified from 1969 AISC, which is based on CRC Sec. 6.7, which is generalized form of CRC Sec. 6.4.

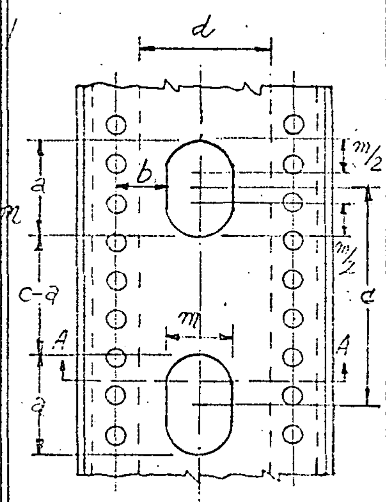
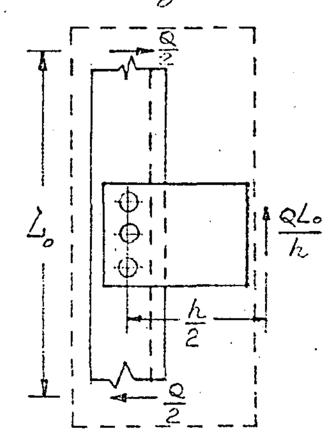
1. AREA Specifications for Steel Railway Bridges, AREA Bulletin 611, January 1968, as revised for approval in Sept. 1968.
2. The Column Research Council Guide to Design Criteria for Metal Compression Members, 2nd Edition, 1966

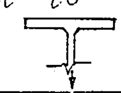
Topic	AREA Specification		CRC Guide	Notes
Beam - Columns eccentrically loaded	No special provision see 1.3.14.1 above	6.2	Allowable average stress for design use: "Exact" design formula: $F_a = \dfrac{\sigma_y/n}{\left[1+\left(\frac{ec}{r^2}+\frac{e_o c}{r^2}\right)\sec\frac{L}{2r}\sqrt{nF_a/E}\right]}$ (6.2) Approximate formula: (equal end eccentricities) $F_a = \dfrac{\sigma_y/n}{1+\left(\frac{1+0.23n\alpha}{1-n\alpha}\right)\frac{ec}{r^2}}$ (6.6) Formula for beam-column subjected to uniform total lateral load W and axial load P: $F_a = \dfrac{\sigma_y/n}{1+\left[\frac{1+0.026(nF_a/\sigma_e)}{1-(nF_a/\sigma_e)}\right]\frac{k}{8}\frac{Lc}{r^2}}$ (6.7) where $W = kP$	Beam-Column design based on load at initial yield. (Based on Secant formula)
Beam - Columns laterally supported	No special provision see 1.3.14.1 above	6.3	Interaction formulas: $\dfrac{P}{P_u} + \dfrac{M}{M_u} \leqslant 1$ (6.8) In the elastic range: $\dfrac{P}{P_u} + \dfrac{M_o}{M_u[1-(P/P_e)]} \leqslant 1$ (6.10) Eccentrically loaded columns having equal end eccentricity e at both ends: $\dfrac{P}{P_u} + \dfrac{Pe}{M_u[1-(P/P_e)]} \leqslant 1$ (6.11)	Beam-Column strength in bending without lateral buckling.

Topic	AREA Specification		CRC Guide		Notes
Beam – Columns Wide-flange shape		No special provision See 1.3.14.1 above	6.3	The Galambos – Ketter interaction formula: $A\frac{M}{M_u} + B\frac{P}{P_y} + C\left(\frac{P}{P_y}\right)^2 \leq 1$ (6.12)	
Beam – Columns I-shaped laterally unsupported		No special provision See 1.3.14.1 above	6.4	$\frac{P}{P_u} + \frac{M_o}{M_u[1-(P/P_e)]} \leq 1$ (6.14) Hill and Clark	
centrally loaded columns	1.4.1	Allowable unit stress (psi): $kl/r \leq 15$ 20,000 $15 \leq kl/r \leq 143$ $21{,}500 - 100\frac{kl}{r}$ $kl/r \geq 143$ $\frac{147{,}000{,}000}{(kl/r)^2}$	2.4	$\sigma_c = \sigma_y - \frac{\sigma_y^2}{4\pi^2 E}\left(\frac{KL}{r}\right)^2$ (2.10) Tangent-modulus theory	
Effective-length of compression members	1.4.1	kl is the effective length k = 7/8 for members with pin-end connections. k = 3/4 for members with riveted, bolted, or welded end connections	2.8	Euler buckling load: $\sigma_e = \frac{\pi^2 E}{(KL/r)^2}$ (2.2) Where K is the effective-length factor. Recommended K value when ideal conditions are approximated: K = 1.0 for members with pin-end connections. K = 0.65 for members with fixed-end connections.	CRC: Effective-length factors K for centrally loaded columns with various idealized end conditions are given in Fig. 2.14.
Laterally unsupported beams – rectangular or box sections	1.4.1	Allowable unit stress = 20,000 psi	4.2	$\left(\frac{KL}{r}\right)_{equiv} = \sqrt{\frac{5.1 L S_x}{\sqrt{J I_y}}}$ (4.7) $J = \frac{2b^2 d^2}{\frac{b}{t} + \frac{d}{t_w}}$ (4.5) $\sigma_{cr} \approx$ equivalent column	

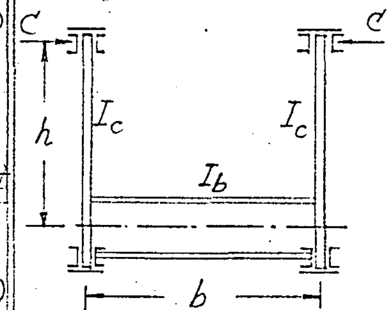
Topic	AREA Specification		CRC Guide		Notes
Laterally unsupported beams	1.4.1	Allowable unit stress: $p = 20{,}000 - 0.4\left(\frac{l}{r_y}\right)^2$ ---- (1) $p = \frac{10{,}500{,}000}{ld/A_f}$ ----(2) The larger of the values computed by the formulas (1) & (2), but not to exceed 20,000 psi	4.4	$\sigma_c = \frac{C_4\sqrt{I_y J}\,\sqrt{EG}}{S_x L}$ (4.11) $C_4 = \frac{C_1}{K}\pi\left[\sqrt{1+\frac{\pi^2 a^2}{(KL)^2}(C_2^2+1)} \pm \frac{C_2\pi a}{KL}\right]$ (4.12) Values of coefficients C_1 and C_2 are given in Fig. 4.5.	CRC: The basic procedure for doubly symmetrical I-shaped beams and plate girders AREA: Only use for rectangular shaped flanges and use average area in formula.
Lateral buckling of plate girder flanges	1.4.1	see 1.4.1 above	5.4	$\frac{\sigma_c}{\sigma_y} = 1 - \frac{\lambda^2}{4}$ for $0<\lambda<\sqrt{2}$ (inelastic) (5.9a) $\frac{\sigma_c}{\sigma_y} = \frac{1}{\lambda^2}$ for $\lambda>\sqrt{2}$ (elastic) (5.9b) where $\lambda = \frac{1}{\pi}\frac{L}{r}\sqrt{\epsilon_y} = \frac{L}{\pi}\sqrt{\frac{\epsilon_y(A_f + A_w/6)}{I_f}}$	(1) Failure of compression flange due to lateral buckling.  (2) AREA (2) only for doubly symmetrical shapes with rectangular flanges. (3) AISC has modified to eliminate this form $A_w/6$.
Slenderness ratio	1.5.1	The slenderness ratio (ratio of length to least radius of gyration) shall not exceed: 100 for main compression members 120 for wind and sway bracing in compression. 140 for single lacing 200 for double lacing 200 for tension members		——	
Minimum thickness	1.5.4	0.335 inch		——	

Topic	AREA Specification		CRC Guide		Notes
Local buckling (elastic)	1.6.2	The width of outstanding elements 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 : $16t$	3.3	$\left(\frac{KL}{r}\right)_{equiv} = \frac{3.3}{\sqrt{k}}\left(\frac{b}{t}\right)$ (3.3) k = plate buckling coefficient.	
Local buckling (plastic)		No provision, see 1.6.2 above	3.3	$\left(\frac{b}{t}\right)_{max} = 13\sqrt{\frac{k\sqrt{E_{st}}}{\sigma_y}}$ (3.4) Local buckling suppressed until strain-hardening	
Effective width		No provision, see 1.6.2 above	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) $\frac{b_e}{b} = \sqrt{\frac{\sigma_c}{\sigma_e}}\left[1-0.25\sqrt{\frac{\sigma_c}{\sigma_e}}\right]$ (3.8) post-buckling strength	Effective flat-plate width-thickness ratios 
Torsional (local) buckling of plate girder flanges		No provision, See 1.6.2 above	5.4	$\frac{b_f}{t_f} \leq 12 + \frac{L}{b_f}$ (5.12) Torsional (local) buckling of flange will not occur prior to lateral buckling of flange.	

Topic		AREA Specification		CRC Guide	Notes
Columns with perforated plates	1.6.4.1 1.6.4.3	Shearing force: $V = \frac{P}{100}\left(\frac{100}{l/r+10} + \frac{l/r}{100}\right)$ Thickness: for tension members: $t \geq \frac{1}{50}$ of the distance between the nearer lines of connections. for compression members: $t \geq \frac{1}{50}$ of the distance between the nearer lines of connections also $t \geq \frac{1}{12}$ of the distance from such a line of connections to the edge of the perforation at the center of perforation. for all members: $t \geq \frac{3cU}{2vh(c-a)}$	3.13	The design suggestions from the White-Thürlimann study and AASHO Specs. 1. The perforations may be circular with straight sides, elliptical or circular. 2. $(c-a) \geq d$ 3. The cross sectional area and moment-of-inertia are computed based upon the net section (A-A). 4. The permissible load can be determined by the appropriate specification applied to the column net section, if $a/r_f \leq 20$, and $a/r_f < L/r_x$. 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. Perforated plates need not be checked for shear if the rule (2) is followed. 6. The b/t ratio should conform to specification requirements for plates in main compression.	CRC: Fig. 3.13 
Columns with batten plates		————	3.14	Effective length (Bleich): $\frac{KL}{r} = \sqrt{\left(\frac{L}{r}\right)^2 + \frac{\pi^2}{12}\left(\frac{L_o}{h}\right)^2}$ (3.18) Moment resisted by each group of fasteners: $M_b = \frac{QL_o}{2n}$ (3.19) Chord bending moment (for computing combined stresses): $M = \frac{Qc}{2}$	CRC: Fig. 3.14. 

Topic	AREA Specification		CRC Guide		Notes
Laced columns	1.6.42	flange $\frac{l_o}{r} < 40$ and $\frac{l_o}{r} < \frac{2}{3}\frac{l}{r}$ (member)		—	
		—	3.12	for $\frac{KL}{r} > 40 : K' = K\sqrt{1 + \frac{300}{(\frac{KL}{r})^2}}$ (3.17) for $\frac{KL}{r} \leq 40 : K' = 1.1K$	
Thickness of web plates of plate girders	1.7.3.	Buckling of web plate by flexural bending: $\frac{h}{t} < 170\sqrt{\frac{p}{f}}$ p = allowable extreme fiber stress. f = extreme fiber stress in the compression flange.		—	
Vertical buckling of plate girder flanges		—	5.4	Vertical buckling of top flange: $\frac{h}{t_w} < \frac{0.48E}{\sqrt{\sigma_y(\sigma_y + \sigma_{RC})}}$ (5.7)	CRC covers case for no external load applied except to stiffeners. 
Shear strength of plate girders		—	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)	CRC based on ultimate strength in shear.
Plate girders		—	5.6	Interaction formula: $\sigma = \sigma_y \frac{1 + \frac{1}{4}(A_w/A_f)[1 - (\tau/\tau_u)^2]}{1 + \frac{1}{6}(A_w/A_f)}$ (5.25)	

Topic		AREA Specification		CRC Guide	Notes
Plate girders		—	5.6	$\frac{\sigma_c}{\sigma_y} = 1 - \frac{\lambda^2}{4C_1} \quad 0 < \lambda < \sqrt{2C_1}$ $\frac{\sigma_c}{\sigma_y} = \frac{C_1}{\lambda^2} \quad \lambda > \sqrt{2C_1}$ (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 $C_1 = 1.75 - 1.05K + 0.3K^2$ $-0.46 < K < +1$	Ultimate strength with moment gradient considered.
Transverse stiffener spacing of plate girders	1.7.8	Intermediate (transverse) stiffeners are needed if depth $h > 60t$. stiffeners spacing: $d \leq 72$ inches $d \leq \frac{10500t}{\sqrt{S}}$ d = clear distance between stiffeners. t = thickness of web. S = shearing stress	5.5	According to Sec. 5.5 the ultimate strength in shear.	
Plate girder transverse stiffeners	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.	5.7	Relative Rigidity: $\gamma = EI/Da$ Bleich: $\gamma_o = 4[7(h/a)^2 - 5]$ (5.28) Rockey: $\gamma_o = 27.75(h/a)^2 - 7.5$ (5.29) for double stiffeners $\gamma_o = 21.5(h/a)^2 - 7.5$ (5.30) for single stiffeners where $\gamma_o = EI_o/Da$	
			5.8	Area: $A_s = \frac{1}{2}\left(1 - \frac{\tau_c}{\tau_y}\right)\left[\alpha - \frac{\alpha^2}{\sqrt{1+\alpha^2}}\right] h t_w$ (5.32)	

Topic		AREA Specification		CRC Guide	Notes
Plate girder longitudinal stiffeners		———	5.9	Bleich: $\gamma_o^* = (12.6 + 50\delta)\alpha^2 - 3.4\alpha^3$ (5.33) where $\delta = A_s/ht_w$, $\gamma^* = EI/Dh$ $\gamma_o^* = EI_o/Dh$, and $\alpha = a/h$	Also see CRC Fig. 5.10 and Table 5.1 for relative rigidity.
Pony trusses		———	7.2	Engesser theory: Compression-chord buckling load: $P_c = A\,\sigma_c$ (7.4) Required spring-constant = $C_{req} = \frac{\pi^2}{4K^2}\frac{P_c}{\ell}$ (7.3) $= 1.46\,\frac{P_c}{\ell}$ (7.5) Holt's solution: See Table 7.1, P. 175 Transverse-frame spring-Constant $C = \frac{E}{h^2\left[(h/3I_c)+(b/2I_b)\right]}$ (7.69)	The design formulas are also applicable to the design of plate girders with elastically braced compression flanges. (CRC. Sec. 7.6)  CRC: Fig. 7.3.
Through girders	1.11.1	"The top flanges of through plate girders shall be braced at the panel points by brackets with web plates. The brackets shall extend to the top flange of the main girder and be as wide as the clearance will allow. They shall be attached securely to a stiffener on the girder and to the top flange of the floorbeam. On solid floor bridges the brackets shall be not more than 12 ft apart."		———	

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