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Course Description

Secrets of the Manual
April 27, 2017

This presentation will highlight efficient ways to take advantage of the various design aids in the AISC *Steel Construction Manual* (14th Edition). The speaker will walk through the different sections of the Manual and highlight useful resources. The speaker will then focus on shortcuts for connection design, including: stiffeners, web yielding/crippling, welds, and more. Shortcuts for member design including composite members will also be reviewed. Learning these shortcuts are a must for those designing steel structures and checking designs.

Learning Objectives

- Locate important design shortcuts in the Manual
- Identify appropriate uses for eccentrically loaded bolt group tables
- Apply table shortcuts for beam bearing and column stiffener checks
- Design beams using composite beam tables in the Manual

There's always a solution in steel.

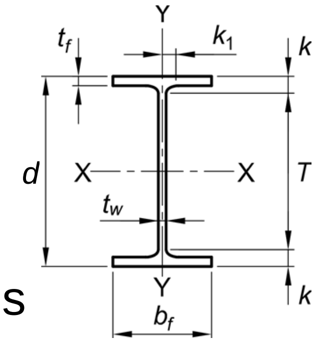
Secrets of the Manual

Based on the 14th Edition Manual and AISC 360-10

Presented by
Carol Drucker, PE, SE
Drucker Zajdel Structural Engineers
Chicago, IL

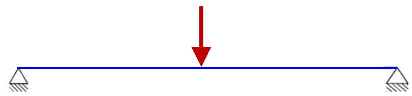
Manual Organization- General Parts

- Part 1: Dimension and Properties
- Part 2: General Design Considerations



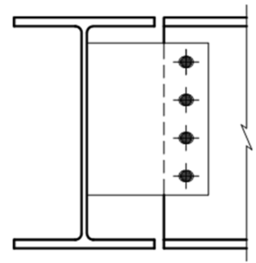
Manual Organization Main Member Design

- Part 3: Flexural Members
- Part 4: Compression Members
- Part 5: Tension Members
- Part 6: Members Subject to Combined Forces



Manual Organization-Connections

- Part 7: Design of Bolts
- Part 8: Design of Welds
- Part 9: Design of Elements
- Part 10: Simple Shear Connections
- Parts 11,12: Moment Connections
- Part 13: Bracing and Truss Connections
- Part 14: Base Plates, Anchor Rods, Column Splices
- Part 15: Hanger Connections, Brackets



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Manual Organization-Main Member/Connections

- Part 16: Specification, Commentary, and Codes
  - AISC 360-10
    - » Chapter A- N: Main Member, Stability, Connections, Fabrication, etc
    - » Appendix 1- Appendix 8: Ponding, Fatigue, Fire, Existing, Stability, Analysis, etc
    - » Commentary for Chapter A-N
    - » Commentary for Appendix 1-8
  - RCSC: Specification for Structural Joints Using High-Strength Bolts
  - AISC: 303-10: Code of Standard Practice

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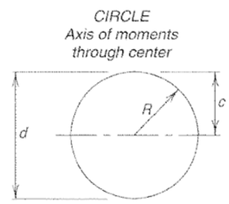
Manual Organization-Main Member/Connections

- Part 17: Miscellaneous Data and Mathematical Information
  - Metric Shapes
  - Gage metal thickness
  - Coefficient of thermal expansion
  - Material Weights
  - Properties of Shapes
- General Nomenclature
- Index



$$A = \frac{\pi d^2}{4} = \pi R^2 = .785398\ d^2 = 3.141593\ R^2$$

$$c = \frac{d}{2} = R$$

$$I = \frac{\pi d^4}{64} = \frac{\pi R^4}{4} = .049087\ d^4 = .785398\ R^4$$

$$S = \frac{\pi d^3}{32} = \frac{\pi R^3}{4} = .098175\ d^3 = .785398\ R^3$$

$$r = \frac{d}{4} = \frac{R}{2}$$

$$Z = \frac{d^3}{6}$$

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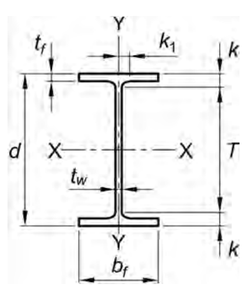
Secrets of the Manual

- **Part 1: Structural Properties**
  - *A, d,* t_w, b_f, t_f, etc

    =INDEX('Database v14.0.1'!$1:$1048576,MATCH("W12x96",'Database v14.0.1'!$C:$C,0),MATCH("bf",'Database v14.0.1'!$2:$2,0))



Link to shapes database: www.aisc.org/publications/steel-construction-manual-resources/

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Secrets of the Manual

- **Part 1: Structural Properties**
  - Angle Gage in Table 1-7A
  - Welded Flat Widths HSS

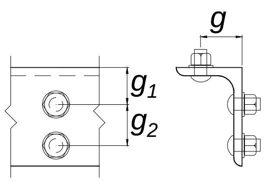
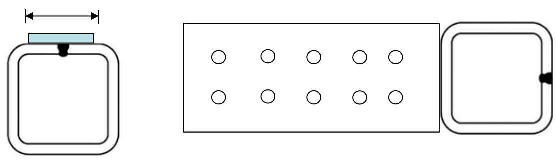
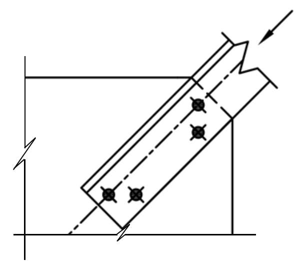


Table 1-7A
Workable Gages in Angle Legs, in.

Leg	8	7	6	5	4	3½	3	2½	2	1¾	1½	1⅜	1¼	1
g	4½	4	3½	3	2½	2	1¾	1⅜	1⅛	1	7/8	7/8	3/4	5/8
g_1	3	2½	2¼	2										
g_2	3	3	2½	1¾										

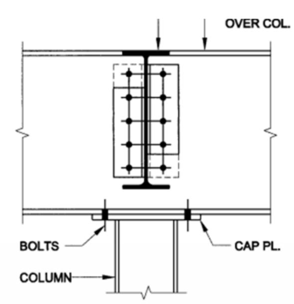
Note: Other gages are permitted to suit specific requirements subject to clearances and edge distance limitations.

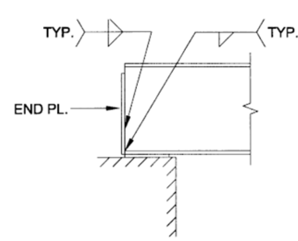




Secrets of the Manual

- **Part 2: General Design Considerations**
  - Bearing Connections
  - Table 2-4: Preferred Material Specification- Shapes
  - Table 2-6: ASTM Fasteners (Bolts, Rods)
  - ASD to LRFD $\Omega = \frac{1.5}{\phi}$





Secrets of the Manual

ASTM Designation	F_y Min. Yield Stress (ksi)	F_u Tensile Stress[a] (ksi)	Applicable S				
			W	M	S	HP	C
A36	36	58-80[b]	▒	■	■	▒	■
A53 Gr. B	35	60					

Table 2-4

Applicable ASTM Specifications for Various Structural Shapes

Steel Type	ASTM Designation		F_y Min. Yield Stress (ksi)	F_u Tensile Stress[a] (ksi)	Applicable Shape Series									
					W	M	S	HP	C	MC	L	HSS		Pipe
												Rect.	Round	
	A36		36	58-80[b]	▒	■	■	▒	■	■	■			
	A53 Gr. B		35	60										■
Carbon	A500	Gr. B	42	58									■	
			46	58								■		
		Gr. C	46	62									▒	
			50	62								▒		
	A501	Gr. A	36	58								▒	▒	
		Gr. B	50	70								▒	▒	
	A529[c]	Gr. 50	50	65-100	▒	▒	▒	▒	▒	▒	▒			
		Gr. 55	55	70-100	▒	▒	▒	▒	▒	▒	▒			
High-Strength Low-Alloy	A572	Gr. 42	42	60	▒	▒	▒	▒	▒	▒	▒			
		Gr. 50	50	65[d]	▒	▒	▒	■	▒	▒	▒			
		Gr. 55	55	70	▒	▒	▒	▒	▒	▒	▒			
		Gr. 60[e]	60	75	▒	▒	▒	▒	▒	▒	▒			
		Gr. 65[e]	65	80	▒	▒	▒	▒	▒	▒	▒			
	A618[f]	Gr. I & II	50[g]	70[g]								▒	▒	
		Gr. III	50	65								▒	▒	
	A913	50	50[h]	60[h]	▒	▒	▒	▒	▒	▒	▒			
		60	60	75	▒	▒	▒	▒	▒	▒	▒			
		65	65	80	▒	▒	▒	▒	▒	▒	▒			
		70	70	90	▒	▒	▒	▒	▒	▒	▒			
	A992		50	65[i]	■	▒	▒	▒	▒	▒	▒			
Corrosion Resistant High-Strength Low-Alloy	A242		42[j]	63[j]	▒									
			46[k]	67[k]	▒			▒						
			50[l]	70[l]	▒	▒	▒	▒	▒	▒	▒			
	A588		50	70	▒	▒	▒	▒	▒	▒	▒			
	A847		50	70								▒	▒	

■ = Preferred material specification
▒ = Other applicable material specification, the availability of which should be confirmed prior to specification
□ = Material specification does not apply

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ASTM Designation		F_y Min. Yield Stress (ksi)	F_u Tensile Stress[a] (ksi)	Diameter Range (in.)	High-Strength Bolts		Common Bolts	Nuts	Washers	Direct-Tension-Indicator Washers	Threaded Rods	Steel Headed Stud Anchors	Anchor Rods		
					Conventional	Twist-Off-Type Tension-Control							Hooked	Headed	Threaded & Nutted
F1554	Gr. 36	36	58-80	0.25 to 4									■	■	■
	Gr. 55	55	75-95	0.25 to 4									■	■	■
	Gr. 105	105	125-150	0.25 to 3									▒	▒	▒

Table 2-6

Applicable ASTM Specifications for Various Types of Structural Fasteners

ASTM Designation		F_y Min. Yield Stress (ksi)	F_u Tensile Stress[a] (ksi)	Diameter Range (in.)	High-Strength Bolts		Common Bolts	Nuts	Washers	Direct-Tension-Indicator Washers	Threaded Rods	Steel Headed Stud Anchors	Anchor Rods		
					Conventional	Twist-Off-Type Tension-Control							Hooked	Headed	Threaded & Nutted
A108		—	65	0.375 to 0.75, incl.								■			
A325[d]		—	105	over 1 to 1.5, incl.	■										
		—	120	0.5 to 1, incl.	■										
A490[d]		—	150	0.5 to 1.5	■										
F1852[d]		—	105	1.125		■									
		—	120	0.5 to 1, incl.		■									
F2280[d]		—	150	0.5 to 1.125, incl.		■									
A194 Gr. 2H		—	—	0.25 to 4				▒							
A563		—	—	0.25 to 4				■							
F436[b]		—	—	0.25 to 4					■						
F959		—	—	0.5 to 1.5						■					
A36		36	58-80	to 10							■		▒		▒
A193 Gr. B7[e]		—	100	over 4 to 7							▒		▒		▒
		—	115	over 2.5 to 4							▒		▒		▒
		—	125	2.5 and under							▒		▒		▒
A307 Gr. A		—	60	0.25 to 4			■								
A354 Gr. BD		—	140	2.5 to 4, incl.							▒		▒	▒	▒
		—	150	0.25 to 2.5, incl.							▒		▒	▒	▒
A449		—	90	1.75 to 3, incl.	c						▒		▒	▒	▒
		—	105	1.125 to 1.5, incl.	c						▒		▒	▒	▒
		—	120	0.25 to 1, incl.	c						▒		▒	▒	▒
A572	Gr. 42	42	60	to 6							▒		▒		▒
	Gr. 50	50	65	to 4							▒		▒		▒
	Gr. 55	55	70	to 2							▒		▒		▒
	Gr. 60	60	75	to 1.25							▒		▒		▒
	Gr. 65	65	80	to 1.25							▒		▒		▒
A588		42	63	Over 5 to 8, incl.							▒		▒		▒
		46	67	Over 4 to 5, incl.							▒		▒		▒
		50	70	4 and under							▒		▒		▒
A687		105	150 max.	0.625 to 3							▒		▒		▒
F1554	Gr. 36	36	58-80	0.25 to 4									■	■	■
	Gr. 55	55	75-95	0.25 to 4									■	■	■
	Gr. 105	105	125-150	0.25 to 3									▒	▒	▒

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Secrets of the Manual

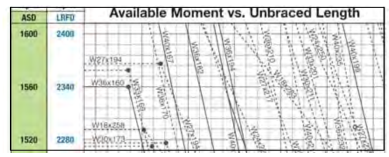
- **Part 3: Main Member**
  - Table 3-2: Selection by Z_x
  - Table 3-6: Maximum Total Uniform Load
  - Table 3-10: Available Moment vs Unbraced Length
  - Table 3-23: Beam Shear, Moment, and Deflection

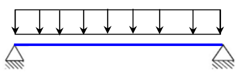
Table 3-2 (continued)
W-Shapes
Selection by Z_x

$F_y = 50$ ksi

Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.³	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.⁴	ASD	LRFD
W30×116	**378**	**943**	**1420**	**575**	**864**	**24.8**	**37.4**	**7.74**	**22.6**	**4930**	**339**	**509**
W21×147	373	931	1400	575	864	13.7	20.7	10.4	36.3	3630	318	477
W24×131	370	923	1390	575	864	16.3	24.6	10.5	31.9	4020	296	445
W18×158	356	888	1340	541	814	10.5	15.9	9.68	42.8	3060	319	479
W14×193	355	886	1330	541	814	5.30	7.93	14.3	79.4	2400	276	414
W12×210	348	868	1310	510	767	4.25	6.45	11.6	95.8	2140	347	520





Secrets of the Manual

- **Part 4, Part 5, Part 6: Main Member**
  - **Table 4-1: Compression of W-Sections**
  - Table 4-8 to 4-10: Axial Compression Double Angles
  - Table 4-12: Eccentrically Loaded Single Angles
  - Table 4-22: kl/r Table

P_{wo}, kips	192	287
P_{wi}, kips/in.	22.7	34.0
P_{wb}, kips	476	716
P_{fb}, kips	222	334

Table 4-1 (continued)
Available Strength in Axial Compression, kips
W-Shapes

$F_y = 50$ ksi

W14

Shape		W14×											
lb/ft		145		132		120		109		99		90	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, KL (ft), with respect to least radius of gyration, r_y	0	1280	1920	1160	1750	1060	1590	958	1440	871	1310	793	1190
	6	1250	1880	1130	1700	1030	1550	932	1400	848	1270	772	1160
	7	1240	1860	1120	1680	1020	1530	923	1390	839	1260	764	1150
	8	1230	1840	1110	1660	1010	1510	913	1370	830	1250	755	1140
	9	1210	1820	1090	1640	994	1490	901	1350	819	1230	745	1120
	10	1200	1800	1080	1620	980	1470	888	1340	807	1210	735	1100
	11	1180	1770	1060	1600	965	1450	874	1310	794	1190	723	1090
	12	1160	1750	1040	1570	948	1430	859	1290	780	1170	710	1070
	13	1140	1720	1020	1540	931	1400	843	1270	766	1150	697	1050
	14	1120	1690	1000	1510	912	1370	826	1240	750	1130	682	1030
	15	1100	1650	982	1480	892	1340	808	1210	733	1100	667	1000
	16	1080	1620	960	1440	872	1310	789	1190	716	1080	652	979
	17	1060	1590	937	1410	850	1280	770	1160	698	1050	635	955
	18	1030	1550	913	1370	828	1240	750	1130	680	1020	618	929
	19	1010	1510	888	1330	805	1210	729	1100	661	994	601	903
	20	980	1470	862	1300	782	1180	708	1060	642	964	583	877
	22	927	1390	810	1220	734	1100	664	998	602	904	547	822
	24	872	1310	756	1140	685	1030	620	931	561	843	509	766
	26	816	1230	702	1060	635	955	574	863	519	781	472	709
	28	759	1140	648	974	586	880	529	796	478	719	434	653
	30	703	1060	594	893	537	807	485	729	438	658	397	597
	32	647	973	542	814	489	735	441	663	398	598	361	543
	34	593	891	491	738	443	665	399	600	360	541	326	490
	36	540	812	442	664	398	598	359	539	323	485	292	439
	38	489	735	397	596	357	536	322	484	290	435	262	394
	40	441	663	358	538	322	484	290	437	261	393	237	356
Properties													
P_{wo}, kips		192	287	175	263	151	227	128	192	112	167	96.1	144
P_{wi}, kips/in.		22.7	34.0	21.5	32.3	19.7	29.5	17.5	26.3	16.2	24.3	14.7	22.0
P_{wb}, kips		476	716	407	611	312	469	220	330	173	260	129	194
P_{fb}, kips		222	334	199	298	165	249	138	208	114	171	94.3	142
L_p, ft		14.1		13.3		13.2		13.2		13.5		15.1	
L_r, ft		61.7		55.8		51.9		48.5		45.3		42.5	

Secrets of the Manual

- **Part 4, Part 5, Part 6: Main Member**
  - **Table 4-8 to 4-10: Axial Compression Double Angles**

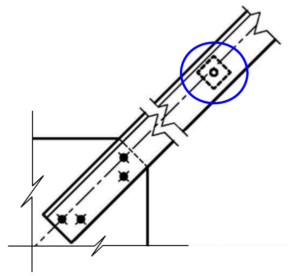
$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} \qquad \text{(E6-1)}$$

(i) When $\frac{a}{r_i} \le 40$

$$\left(\frac{KL}{r}\right)_m = \left(\frac{KL}{r}\right)_o \qquad \text{(E6-2a)}$$

(ii) When $\frac{a}{r_i} > 40$

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{K_i a}{r_i}\right)^2} \qquad \text{(E6-2b)}$$



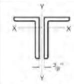
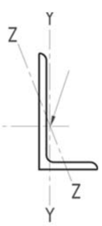


Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB
$F_y = 36$ ksi
2L6 LLBB

		Shape	2L6×4×								No. of connectors[a]
			7/8		3/4		5/8		9/16		
		lb/ft	54.4		47.2		40.0		36.2		
		Design	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, KL (ft), with respect to indicated axis	X-X Axis	0	345	518	300	450	252	379	229	343	
		4	333	501	290	435	244	366	221	332	
		6	319	479	277	417	234	351	212	318	
		8	300	451	261	393	220	331	200	300	
		10	277	416	242	363	204	307	185	278	
		12	252	378	220	331	186	279	169	254	
		14	224	337	197	296	166	250	151	228	
		16	197	296	173	260	146	220	133	201	b
		18	170	255	150	225	127	191	116	174	
		20	144	216	127	191	108	162	98.6	148	
		22	119	179	106	159	90.1	135	82.5	124	
		24	100	151	89.0	134	75.7	114	69.3	104	
		26	85.5	128	75.9	114	64.5	97.0	59.1	88.8	
		28	73.7	111	65.4	98.3	55.6	83.6	50.9	76.6	
		30	64.2	96.5	57.0	85.6	48.5	72.9	44.4	66.7	
	Y-Y Axis	0	345	518	300	450	252	379	229	343	
		4	319	480	273	410	224	336	199	298	
		6	304	456	259	390	213	320	189	284	
		8	283	425	241	363	198	298	176	265	
		10	251	378	214	322	176	264	156	235	
		12	222	334	189	284	155	233	138	208	
		14	192	289	163	244	133	200	119	179	2
		16	162	244	137	205	112	168	99.8	150	

Manual Organization

- **Part 4-6: Main Member**
  - **Table 4-12: Eccentrically Loaded Single Angles**



- The effects of eccentricity can be neglected using an effective slenderness ratio if (*Spec* E5):
  - Member are loaded at the ends in compression through same on leg
  - Member attached by welding or by a minimum of two bolts
  - No transfer loading

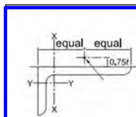


Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles
$F_y = 36$ ksi
L8-L7

	Shape	L8×4×						L7×4×					
		9/16[c]		1/2[c,f]		7/16[c,f]		3/4		5/8		1/2[c]	
	lb/ft	21.9		19.6		17.2		26.2		22.1		17.9	
	Design	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
ius of gyration, r_z	0	60.0	90.2	57.5	86.4	54.7	82.2	65.2	98.0	62.1	93.4	59.2	89.0
	1	59.3	89.2	56.8	85.4	54.1	81.3	64.4	96.9	61.4	92.4	58.5	87.9
	2	57.4	86.4	54.9	82.6	52.1	78.5	62.2	93.6	59.4	89.5	56.3	84.8
	3	54.4	82.0	51.9	78.3	49.2	74.3	58.8	88.7	56.2	84.8	52.8	79.8
	4	50.5	76.4	48.1	72.8	45.5	68.9	54.9	83.0	52.1	78.8	48.6	73.6
	5	46.2	70.1	43.9	66.6	41.4	62.8	50.4	76.5	47.5	72.1	43.8	66.7
	6	41.7	63.6	39.5	60.2	37.2	56.6	45.8	69.6	42.7	65.1	39.1	59.7
	7	37.4	57.0	35.3	53.9	33.1	50.6	41.1	62.7	38.1	58.2	34.6	52.9
	8	33.2	50.8	31.3	47.9	29.3	44.9	36.7	56.1	33.8	51.7	30.4	46.6
	9	29.4	45.0	27.6	42.4	25.8	39.6	32.6	49.8	29.7	45.6	26.6	40.9

Secrets of the Manual

- **Part 4, Part 5, Part 6: Main Member**
  - **Table 4-22: *KL/r* Table**

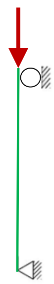
The *critical stress*, F_{cr}, is determined as follows:

(a) When $\frac{KL}{r} \leq 4.71\sqrt{\frac{E}{F_y}}$ (or $\frac{F_y}{F_e} \leq 2.25$)

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y \qquad \text{(E3-2)}$$

(b) When $\frac{KL}{r} > 4.71\sqrt{\frac{E}{F_y}}$ (or $\frac{F_y}{F_e} > 2.25$)

$$F_{cr} = 0.877 F_e \qquad \text{(E3-3)}$$



$\frac{KL}{r}$	F_y = 50 ksi F_{cr}/Ω_c ksi ASD	F_y = 50 ksi $\phi_c F_{cr}$ ksi LRFD
121	10.3	15.4
122	10.1	15.2
123	9.94	14.9

Table 4-22 (continued)
Available Critical Stress for Compression Members

$\frac{KL}{r}$	F_y = 35 ksi F_{cr}/Ω_c ksi ASD	F_y = 35 ksi $\phi_c F_{cr}$ ksi LRFD	$\frac{KL}{r}$	F_y = 36 ksi F_{cr}/Ω_c ksi ASD	F_y = 36 ksi $\phi_c F_{cr}$ ksi LRFD	$\frac{KL}{r}$	F_y = 42 ksi F_{cr}/Ω_c ksi ASD	F_y = 42 ksi $\phi_c F_{cr}$ ksi LRFD	$\frac{KL}{r}$	F_y = 46 ksi F_{cr}/Ω_c ksi ASD	F_y = 46 ksi $\phi_c F_{cr}$ ksi LRFD	$\frac{KL}{r}$	F_y = 50 ksi F_{cr}/Ω_c ksi ASD	F_y = 50 ksi $\phi_c F_{cr}$ ksi LRFD
121	9.91	14.9	121	10.0	15.0	121	10.2	15.4	121	10.3	15.4	121	10.3	15.4
122	9.79	14.7	122	9.85	14.8	122	10.1	15.2	122	10.1	15.2	122	10.1	15.2
123	9.67	14.5	123	9.72	14.6	123	9.93	14.9	123	9.94	14.9	123	9.94	14.9
124	9.55	14.3	124	9.59	14.4	124	9.78	14.7	124	9.78	14.7	124	9.78	14.7
125	9.43	14.2	125	9.47	14.2	125	9.62	14.5	125	9.62	14.5	125	9.62	14.5
126	9.31	14.0	126	9.35	14.0	126	9.47	14.2	126	9.47	14.2	126	9.47	14.2
127	9.19	13.8	127	9.22	13.9	127	9.32	14.0	127	9.32	14.0	127	9.32	14.0
128	9.07	13.6	128	9.10	13.7	128	9.17	13.8	128	9.17	13.8	128	9.17	13.8
129	8.95	13.4	129	8.98	13.5	129	9.03	13.6	129	9.03	13.6	129	9.03	13.6
130	8.83	13.3	130	8.86	13.3	130	8.89	13.4	130	8.89	13.4	130	8.89	13.4
131	8.71	13.1	131	8.73	13.1	131	8.76	13.2	131	8.76	13.2	131	8.76	13.2
132	8.60	12.9	132	8.61	12.9	132	8.63	13.0	132	8.63	13.0	132	8.63	13.0
133	8.48	12.7	133	8.49	12.8	133	8.50	12.8	133	8.50	12.8	133	8.50	12.8
134	8.37	12.6	134	8.37	12.6	134	8.37	12.6	134	8.37	12.6	134	8.37	12.6
135	8.25	12.4	135	8.25	12.4	135	8.25	12.4	135	8.25	12.4	135	8.25	12.4
136	8.13	12.2	136	8.13	12.2	136	8.13	12.2	136	8.13	12.2	136	8.13	12.2
137	8.01	12.0	137	8.01	12.0	137	8.01	12.0	137	8.01	12.0	137	8.01	12.0
138	7.89	11.9	138	7.89	11.9	138	7.89	11.9	138	7.89	11.9	138	7.89	11.9
139	7.78	11.7	139	7.78	11.7	139	7.78	11.7	139	7.78	11.7	139	7.78	11.7
140	7.67	11.5	140	7.67	11.5	140	7.67	11.5	140	7.67	11.5	140	7.67	11.5
141	7.56	11.4	141	7.56	11.4	141	7.56	11.4	141	7.56	11.4	141	7.56	11.4
142	7.45	11.2	142	7.45	11.2	142	7.45	11.2	142	7.45	11.2	142	7.45	11.2
143	7.35	11.0	143	7.35	11.0	143	7.35	11.0	143	7.35	11.0	143	7.35	11.0
144	7.25	10.9	144	7.25	10.9	144	7.25	10.9	144	7.25	10.9	144	7.25	10.9
145	7.15	10.7	145	7.15	10.7	145	7.15	10.7	145	7.15	10.7	145	7.15	10.7
146	7.05	10.6	146	7.05	10.6	146	7.05	10.6	146	7.05	10.6	146	7.05	10.6
147	6.96	10.5	147	6.96	10.5	147	6.96	10.5	147	6.96	10.5	147	6.96	10.5
148	6.86	10.3	148	6.86	10.3	148	6.86	10.3	148	6.86	10.3	148	6.86	10.3
149	6.77	10.2	149	6.77	10.2	149	6.77	10.2	149	6.77	10.2	149	6.77	10.2
150	6.68	10.0	150	6.68	10.0	150	6.68	10.0	150	6.68	10.0	150	6.68	10.0
151	6.59	9.91	151	6.59	9.91	151	6.59	9.91	151	6.59	9.91	151	6.59	9.91

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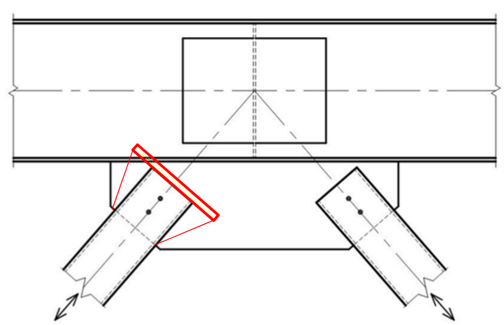
- **Table 4-22 – Use for Whitmore Check for Gussets**

$$\frac{KL}{r} = \frac{KL\sqrt{12}}{t_{gusset}}$$

Table 3. Summary of Proposed Effective Length Factors[a]

Gusset Configuration	Effective Length Factor	Buckling Length	$\frac{P_{exp}}{P_{calc}}$
Compact corner	—[b]	—[b]	1.36
Noncompact corner	1.0	l_{avg}	3.08
Extended corner	0.6	l_1	1.45
Single brace	0.7	l_1	1.45
Chevron	0.65	l_1	1.17

[a] Table 7 from Dowswell (2006) with revisions.
[b] Yielding is the applicable limit state for compact corner gusset plates; therefore, the effective length factor and the buckling length are not applicable.



Bo Dowswell (2012) "Technical Note: Effective Length Factors for Gusset Plates in Chevron Braced Frames" in AISC Engineering Journal 3rd Quarter. Free download for AISC members!

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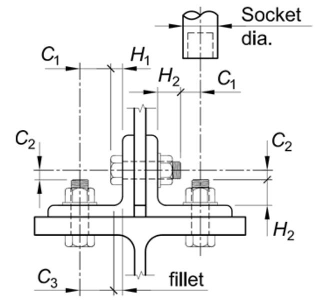
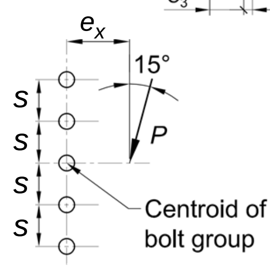
- **Part 7: Bolts**
  - Table 7-1, 7-2, and 7-3: Bolt Shear and Tension Values
  - Table 7-4: Bolt Dimensions
  - Table 7-6: Coefficients C for Eccentrically loaded bolt groups
  - Table 7-15 and 7-16: Bolt Installing Tolerance





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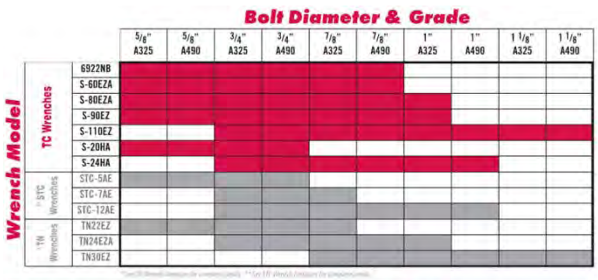
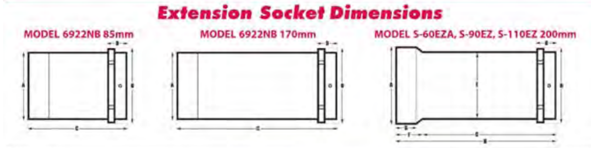
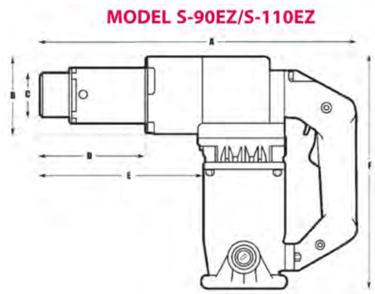
Bolt Installation-TC Bolts



Extension Socket Dimensions



DIMENSIONS	BOLT DIA.	A	B	C	D	E	F	G	H
6922NB 85mm	3/4"	1 25/32"	1 13/16"	2 1/2"	13/16"	n/a	n/a	n/a	n/a
6922NB 85mm	7/8"	1 25/32"	1 15/16"	2 9/16"	13/16"	n/a	n/a	n/a	n/a
6922NB 170mm	3/4"	1 11/16"	1 13/16"	5 13/16"	13/16"	n/a	n/a	n/a	n/a
6922NB 170mm	7/8"	1 13/16"	1 15/16"	5 27/32"	13/16"	n/a	n/a	n/a	n/a
S-60EZA 200mm	3/4"	1 7/8"	2 1/8"	6 25/32"	17/32"	1 3/4"	1 5/32"	1 1/32"	7 13/16"
S-90EZ 200mm	7/8"	2 1/8"	2 7/16"	6 15/32"	13/16"	1 25/32"	1 15/32"	1 5/32"	7 7/8"
S-110EZ 200mm	1"	2 1/4"	2 9/16"	6 7/16"	1 5/32"	1 31/32"	1 7/16"	1 3/16"	7 27/32"
S-110EZ 200mm	1 1/8"	2 1/2"	2 3/4"	6 13/16"	1 5/32"	2 3/32"	1 21/32"	1 11/32"	7 27/32"



Example: gwyinc.com

DIMENSIONS				
Model	S-60EZA		S-90EZ	
A	11"	280mm	13 3/4"	350mm
B	2 7/8"	73mm	3 3/8"	86mm
C	1 7/8"	48mm	1 15/16"	49mm
D	3 5/8"	92mm	5"	127mm
E	5 5/16"	135mm	7 1/16"	180mm
F	9 5/8"	245mm	10 1/4"	260mm

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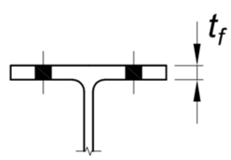
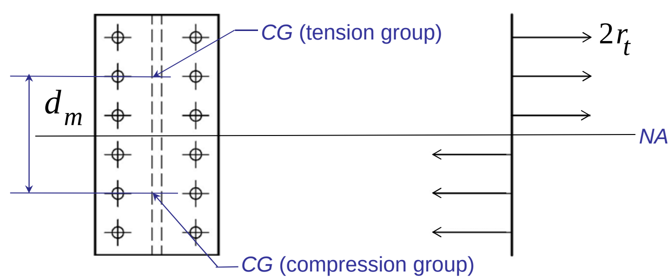


BOLT TENSION

$$r_t = \frac{M_c}{n'_b \, d_m}$$



- See Manual Fig. 7-7

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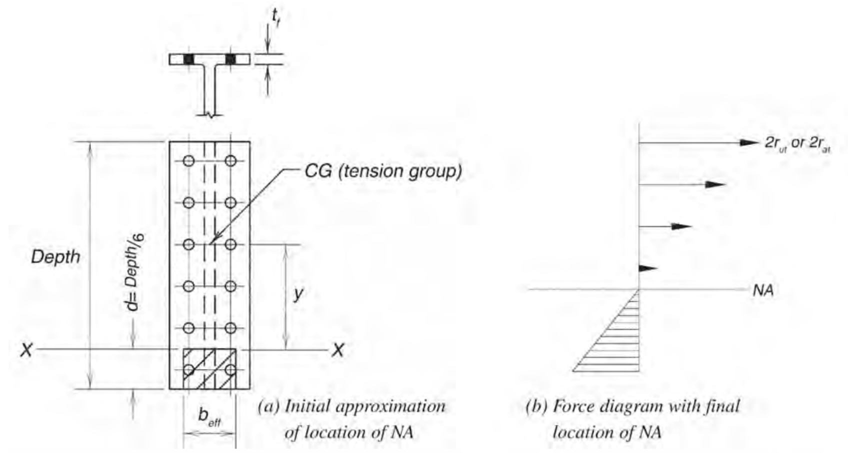

- See Manual Fig. 7-6 Case I

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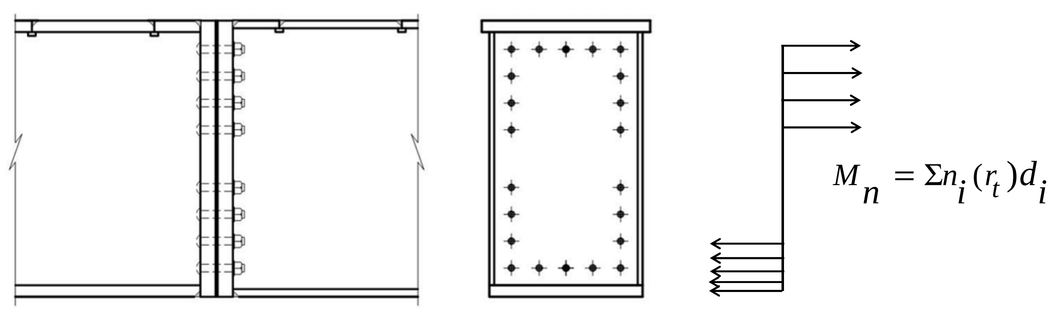

- See Manual Fig. 7-7 Expanded (similar to DG4* and DG16*)

*free download for AISC members!

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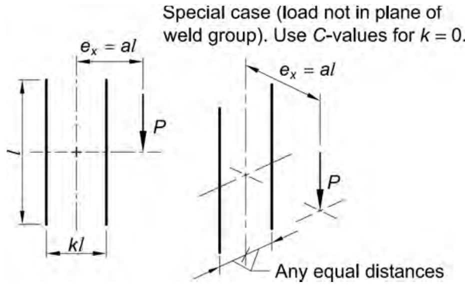
- **Part 8: Welds**
  - Figure 8-16: Susceptible and Improved Details
  - Table 8-2: Prequalified Welds (AWS)
  - Table 8-3: Electrode Strength Coefficient, C_1
  - Table 8-4 to Table 8-11: Coefficients C for Eccentrically Loaded Weld Groups



$$D_{min} = \frac{P_u}{\phi C C_1 l}$$

$$C_1 = \frac{(0.9)(80\text{ ksi})}{70\text{ ksi}}$$

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- **Part 8: Welds: Table 8-12**
  - **Cost of Welds**
  - **Fillet and Groove Welds**

5/16	1
5/8	6

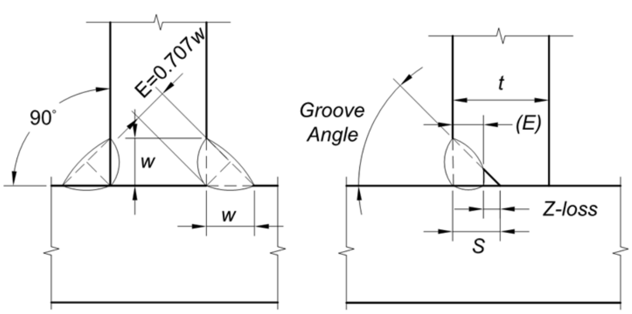


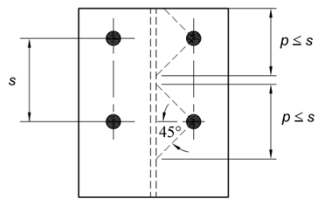
Table 8-12
Approximate Number of Passes for Welds

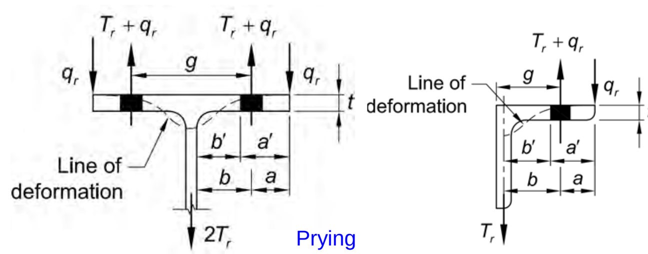
Weld Size* in.	Fillet Welds	Single-Bevel Groove Welds (Back-Up Weld Not Included) 30 Bevel	45 Bevel	Single-V Groove Welds (Back-Up Weld Not Included) 30 Groove Angle	60 Groove Angle	90 Groove Angle
3/16	1	—	—	—	—	—
1/4	1	1	1	2	3	3
5/16	1	1	1	2	3	3
3/8	3	2	2	3	4	6
7/16	4	2	2	3	4	6
1/2	4	2	2	4	5	7
5/8	6	3	3	4	6	8
3/4	8	4	5	4	7	9
7/8	—	5	8	5	10	10
1	—	5	11	5	13	22
1 1/8	—	7	11	9	15	27
1 1/4	—	8	11	12	16	32
1 3/8	—	9	15	13	21	36
1 1/2	—	9	18	13	25	40
1 3/4	—	11	21	13	25	40

*Plate thickness for groove welds.

Secrets of the Manual

- **Part 9: Connections**
  - Cope Beams: Single Coped and Double Coped
  - Prying Action
  - Rotational Ductility
  - Filler and Shims
  - Table 9-4: Beam Bearing





Prying

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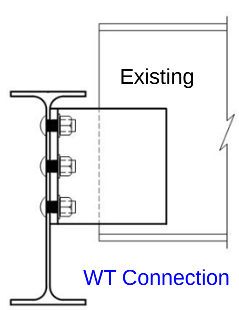
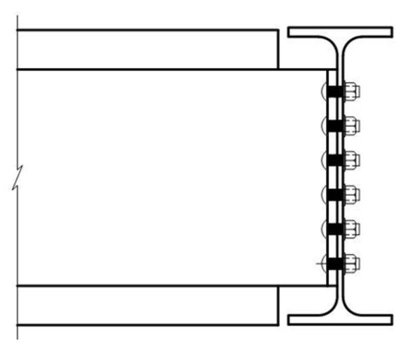
- **Part 9: Connections**
  - Cope Beams: Single Coped and Double Coped
  - Prying Action
  - **Rotational Ductility**
  - Filler and Shims
  - Table 9-4: Beam Bearing



WT Connection



End Plate Connection

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- **Part 9: Shear Rupture at Welds**
  - t_{min} to develop weld:
    - Shear Rupture = Fillet Weld Strength

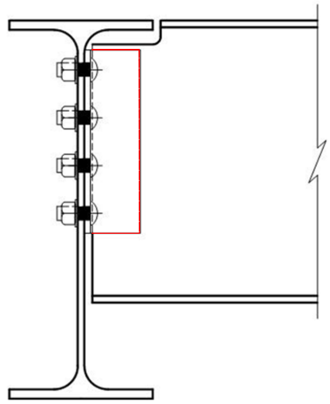
$$t_{min} = \frac{\phi_w(0.6)(F_{exx})(2)(0.707)(w)}{\phi(0.6)(F_u)} \qquad Manual\ Eq.(9-2)$$

$$= \frac{0.75(0.60)(70\text{ ksi})(2)(0.707)(\frac{D}{16})}{0.75(0.6)(F_u)}$$

$$= \frac{2(1.392)(D)}{0.75(0.6)(F_u)}$$

$$= \frac{6.19D}{F_u} \qquad Manual\ Eq.(9-3)$$

(2-Sided Connections)



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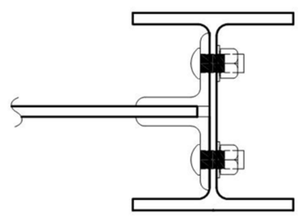
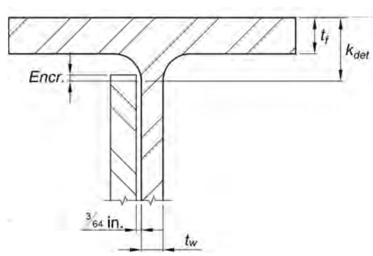
Secrets of the Manual

- **Part 10: Connections**
  - Figure 10-3: Encroachment
  - Table 10-1 to Table 10-9: Connection Tables
  - Figures: Skewed Plate Welds





$k_{det} - t_f$, in.	Encr., in.
5⁄16	1⁄8
3⁄8 to 1⁄2	3⁄16
9⁄16 to 13⁄16	1⁄4
7⁄8 to 1¼	5⁄16
15⁄16 to 1⅜	3⁄8

Fig. 10-3. Fillet encroachment (riding the fillet).

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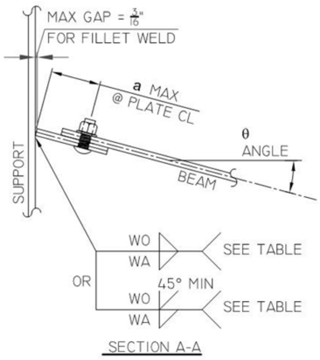
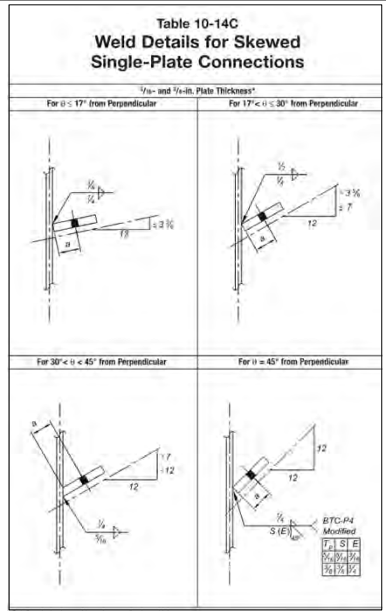
Skewed Plate Welds



SHEAR PLATE WELD DESIGN			
tp PLATE IN	θ ANGLE DEG	WO OBTUSE IN	WA ACUTE IN
3/8	0°	1/4	1/4
	0° < θ ≤ 17°	3/8	1/4
	17° < θ ≤ 30°	1/2	1/4
	30° < θ ≤ 45°	S = 3/8 (E* = 1/4)	1/4
	θ > 45°	S = 1/4 (E**= 1/4)	1/4
1/2	0°	5/16	5/16
	0° < θ ≤ 17°	1/2	5/16
	17° < θ ≤ 22°	9/16	5/16
	22° < θ ≤ 45°	S = 1/2 (E* = 3/8)	5/16
	θ > 45°	S = 5/16 (E**=5/16)	5/16

* PJP: BTC-P4 (MODIFIED), OR OPTIONALLY CJP
**PJP: BTC-P4-GF, F OR H POSITIONS. 45° BEVEL MIN.

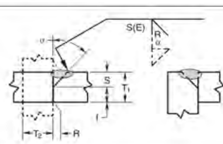


Skewed Plate Welds

PJP

Table 8-2 (continued)
Prequalified Welded Joints
Partial-Joint-Penetration Groove Welds

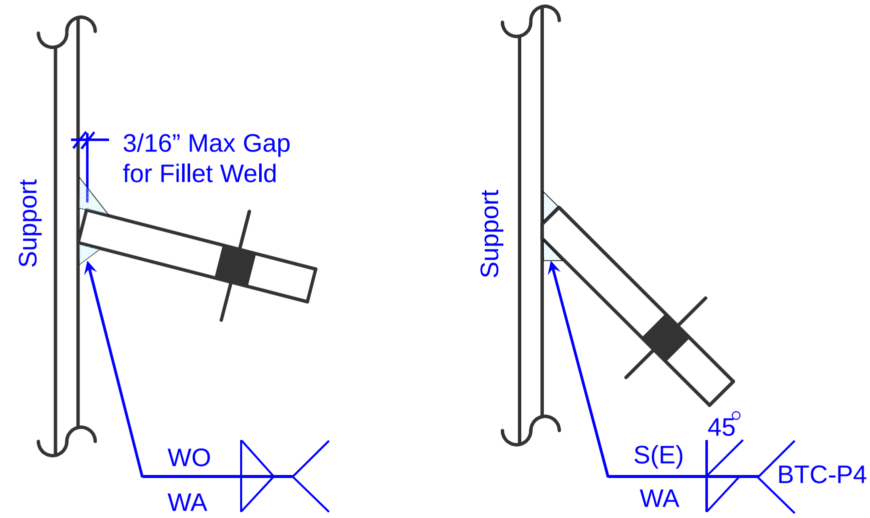
Single-bevel-groove weld (4)
Butt joint (B)
T-joint (T)
Corner joint (C)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Total Weld Size (E)	Notes
		T_1	T_2	Root Opening Root Face Groove Angle	Tolerances: As Detailed (see 3.12.3)	Tolerances: As Fit-Up (see 3.12.3)			
SMAW	BTC-P4	U	U	R = 0 f = 1/8 min. α = 45°	+1/16, −0 +U, −0 +10°, −0°	+1/8, −1/16 ±1/16 +10°, −5°	All	S−1/8	b, e, f, g, j, k
GMAW FCAW	BTC-P4-GF	1/4 min.	U	R = 0 f = 1/8 min. α = 45°	+1/16, −0 +U, −0 +10°, −0°	+1/8, −1/16 ±1/16 +10°, −5°	F, H V, OH	S S−1/8	a, b, f, g, j, k
SAW	TC-P4-S	7/16 min.	U	R = 0 f = 1/4 min. α = 60°	±0 +U, −0 +10°, −0°	+1/16, −0 ±1/16 +10°, −5°	F	S	b, f, g, j, k

Skewed Plate Welds

- **PLs Over ½"**



Skewed Plate Welds

- **1-Sided Welds**

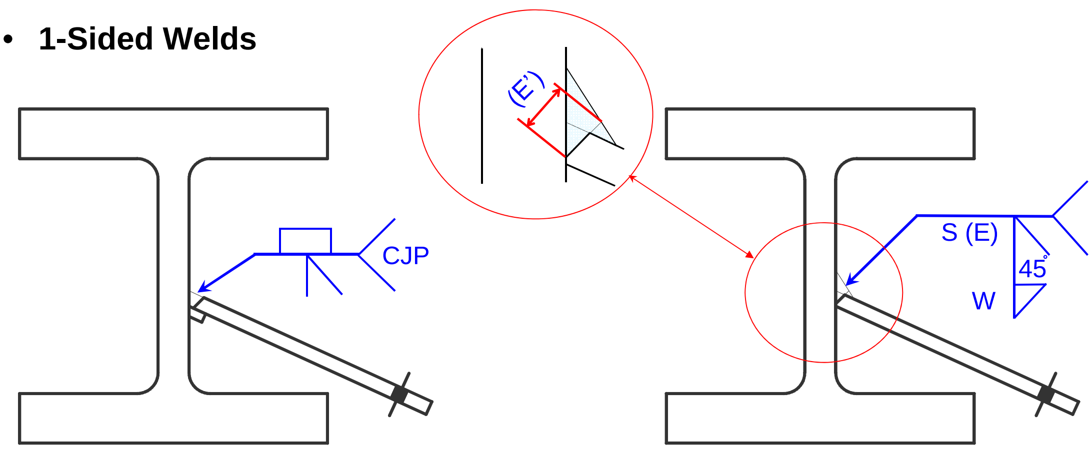


Plate Welds

- **Alternate Bent Plate Detail Large Skews**
  - Typically 1/2” Max Plate Thickness
  - Design from Bend Line
  - “Design of Skewed Connections” by Larry Kloiber and William A. Thornton, EJ 3rd Quarter 2001

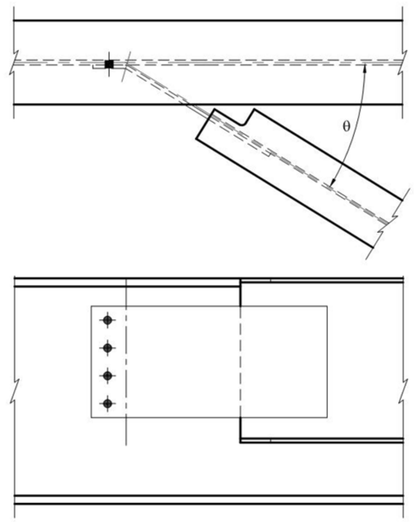
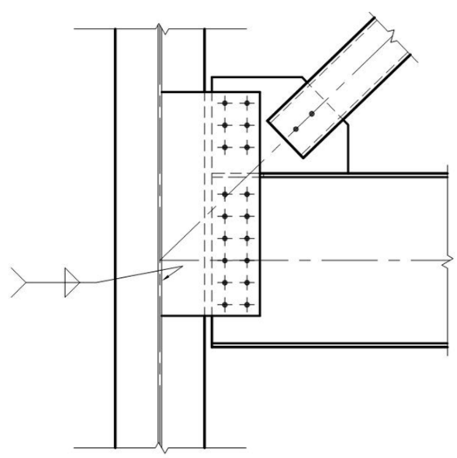


Plate Welds

- **Examples when rotational ductility and a fillet weld size of 5/8*t* is not always needed**
  - Bracing Connections
  - Moment Connections



Welded Angle Legs

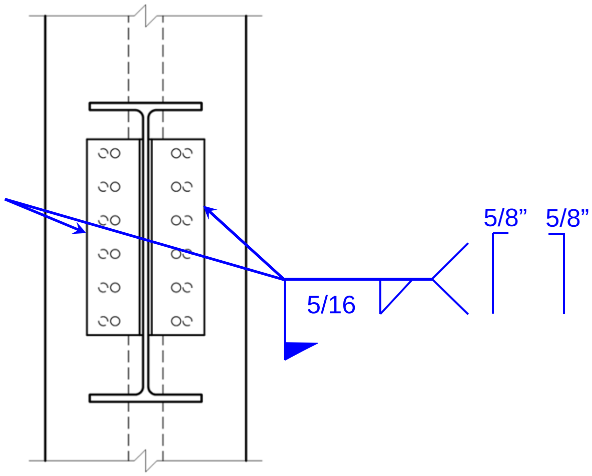
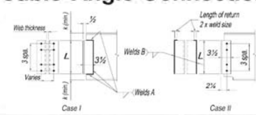


Table 10-2
Available Weld Strength of Bolted/Welded Double-Angle Connections



		Welds A (70 ksi)				Welds B (70 ksi)			
n	L, in.	Weld Size, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Minimum Web Thickness, in.	Weld Size, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Minimum Support Thickness, in.
12	35 1/2	5/16	393	589	0.476	3/8	366	550	0.286
		1/4	314	471	0.381	5/16	305	458	0.238
		3/16	236	353	0.286	1/4	244	366	0.190
11	32 1/2	5/16	365	548	0.476	3/8	331	496	0.286
		1/4	292	438	0.381	5/16	276	414	0.238
		3/16	219	329	0.286	1/4	221	331	0.190
10	29 1/2	5/16	337	505	0.476	3/8	295	443	0.286
		1/4	269	404	0.381	5/16	246	369	0.238
		3/16	202	303	0.286	1/4	197	295	0.190
9	26 1/2	5/16	309	463	0.476	3/8	259	389	0.286
		1/4	247	371	0.381	5/16	216	324	0.238
		3/16	185	278	0.286	1/4	173	259	0.190
8	23 1/2	5/16	281	422	0.476	3/8	223	335	0.286
		1/4	225	337	0.381	5/16	186	279	0.238
		3/16	169	253	0.286	1/4	149	223	0.190
7	20 1/2	5/16	253	379	0.476	3/8	187	280	0.286
		1/4	202	303	0.381	5/16	156	234	0.238
		3/16	152	227	0.286	1/4	125	187	0.190
6	17 1/2	5/16	222	334	0.476	3/8	150	226	0.286
		1/4	178	267	0.381	5/16	125	188	0.238
		3/16	133	200	0.286	1/4	100	150	0.190
5	14 1/2	5/16	191	287	0.476	3/8	115	172	0.286
		1/4	153	229	0.381	5/16	95.5	143	0.238

Secrets of the Manual

- **Part 11-Part 12: Partially Retrained and Fully Restrained Moment Connections**
  - Extended End-Plate Moment Connections
    - Design assumptions 1-11
      - End-plate effective width = b_f + 1 in.
      - CJP welds are treated as PJP welds between beam flange-to-web fillets. No weld access holes
    - Also see DG4* and DG16*

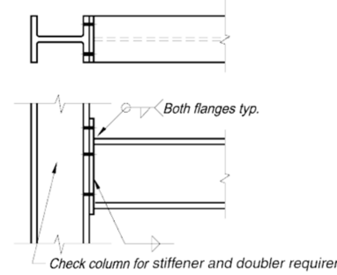
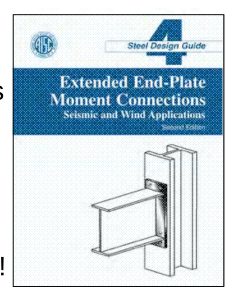


Fig. 12-5. Extended end-plate FR moment connection.



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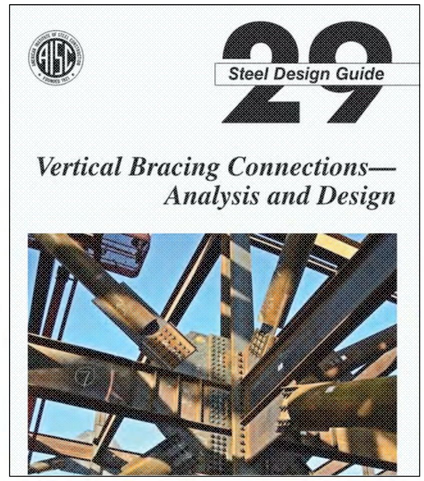
Secrets of the Manual 44

Secrets of the Manual

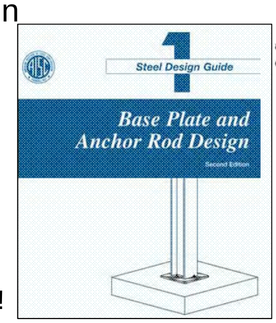
- **Part 13: Bracing**
  - See DG29 and Design Examples

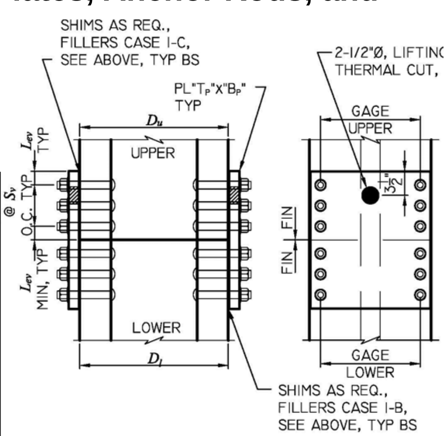


Link to design examples: www.aisc.org/publications/steel-construction-manual-resources/

Secrets of the Manual

- **Part 14: Beam Bearing Plates, Column Base Plates, Anchor Rods, and Column Spices**
  - Table 14-2: Recommended Maximum Sizes for Anchor-Rod Holes in Base Plates (also see DG1*)
  - Tables 14-3: Cases I-XII





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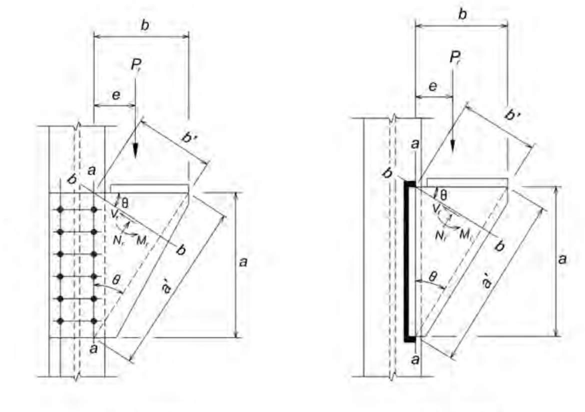
Secrets of the Manual

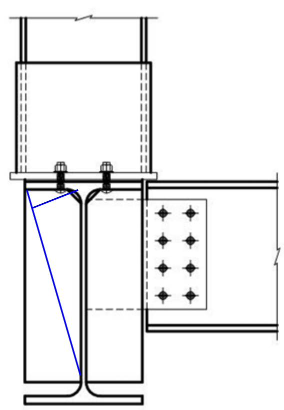
- **Part 15: Hanger Connections, Bracket Plates, and Crane Rail Connections**
  - Figure 15-2: Bracket Plates
  - Table 15-3: Z_{net} *plates*
  - Table 15-4 to 15-6:
    Clevis, Pins, and Turnbuckles

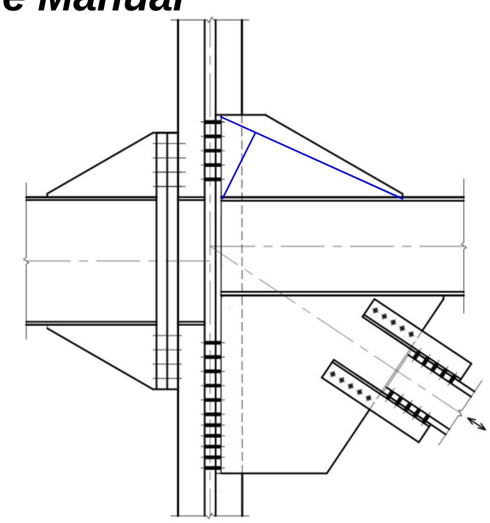


(a) bolted (b) welded

Secrets of the Manual

- **Part 15: Brackets Cont.**





Secrets of the Manual

- **Part 15: Clevises and Pins**
  - For design: *Specification* Section D5 and J7
  - Example: clevelandcityforge.com

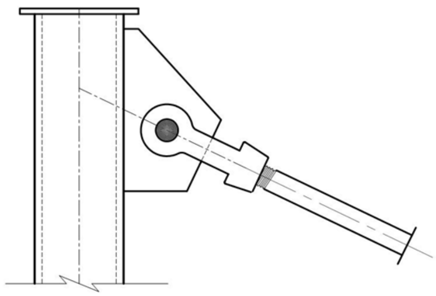
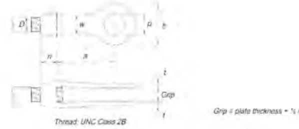


Table 15-4
Dimensions and Weights of Clevises



Clevis Number	Dimensions, in.							Weight, lb	Available Strength, kips*	
	Max. D	Max. p	b	n	a	w	t		ASD	LRFD
2	5/8	3/4	1 7/16	5/8	3 9/16	1 1/16	5/16 (+1/32, -0)	1	5.83	8.75
2 1/2	7/8	1 1/2	2 1/2	1	4	1 1/4	5/16 (+1/32, -0)	2.5	12.5	18.8
3	1 3/8	1 3/4	3	1 1/4	5 1/16	1 1/2	1/2 (+1/16, -1/32)	4	25.0	37.5
3 1/2	1 1/2	2	3 1/2	1 1/2	6	1 3/4	1/2 (+1/16, -1/16)	6	30.0	45.0
4	1 3/4	2 1/4	4	1 3/4	5 13/16	2	1/2 (+1/16, -1/16)	9	35.0	52.5
5	2 1/8	2 1/2	5	2 1/4	7	2 1/2	5/8 (+3/32, -0)	16	62.5	93.8
6	2 1/2	3	6	2 3/4	8	3	3/4 (+3/32, -0)	26	90.0	135
7	3	3 3/4	7	3	9	3 1/2	7/8 (+1/8, -1/16)	36	114	171
8	4	4 1/4	8	4	10 1/8	4	1 1/2 (+1/8, -1/16)	90	225	338

Notes:
Weights and dimensions of clevises are typical; products of all suppliers are essentially similar. User shall verify with the manufacturer that product meets available strength specifications above.
* Tabulated available strengths are based on $\phi = 0.50$, $\Omega = 3.00$. Strength at service load corresponds to a 3:1 safety factor using maximum pin diameter.

Secrets of the Manual

- **Part 16: Specification for Structural Steel Buildings (360-10),**
  - User Notes
  - Commentary

G2. MEMBERS WITH UNSTIFFENED OR STIFFENED WEBS

1. Shear Strength

This section applies to webs of singly or doubly symmetric members and channels subject to shear in the plane of the web.

The *nominal shear strength*, V_n, of unstiffened or stiffened webs according to the *limit states* of *shear yielding* and *shear buckling*, is

$$V_n = 0.6F_yA_wC_v \quad \text{(G2-1)}$$

(a) For webs of rolled I-shaped members with $h/t_w \leq 2.24\sqrt{E/F_y}$:

$$\phi_v = 1.00 \text{ (LRFD)} \qquad \Omega_v = 1.50 \text{ (ASD)}$$

and

$$C_v = 1.0 \quad \text{(G2-2)}$$

User Note: All current ASTM A6 W, S and HP shapes except W44×230, W40×149, W36×135, W33×118, W30×90, W24×55, W16×26 and W12×14 meet the criteria stated in Section G2.1(a) for F_y = 50 ksi (345 MPa).

Secrets of the Manual

- **Part 16: Specification for Structural Steel Buildings (360-10),**

(b) For webs of all other doubly symmetric shapes and singly symmetric shapes and channels, except round *HSS*, the web shear coefficient, C_v, is determined as follows:

(i) When $h/t_w \le 1.10\sqrt{k_v E/F_y}$

$$C_v = 1.0 \quad \text{(G2-3)}$$

(ii) When $1.10\sqrt{k_v E/F_y} < h/t_w \le 1.37\sqrt{k_v E/F_y}$

$$C_v = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \quad \text{(G2-4)}$$

(iii) When $h/t_w > 1.37\sqrt{k_v E/F_y}$

$$C_v = \frac{1.51 k_v E}{(h/t_w)^2 F_y} \quad \text{(G2-5)}$$

Secrets of the Manual

- RCSC
  - Masking Requirements: Fig C-3.1
  - Section 4:PT and SC Joint Applications
  - Section 6: Washer Requirements
  - Section 8: Bolt Installation Methods

Table 6.1. Washer Requirements for Pretensioned and Slip-Critical Bolted *Joints* with Oversized and Slotted Holes in the Outer Ply

ASTM Designation	Nominal Bolt Diameter, d_b, in.	Hole Type in Outer Ply		
		Oversized	Short-Slotted	Long-Slotted
A325 or F1852	½ - 1½	ASTM F436 [a]		5⁄16 in. thick plate washer or continuous bar [b,c]
A490 or F2280	≤ 1	ASTM F436 [a]		5⁄16 in. thick plate washer or continuous bar [b,c]
	> 1	ASTM F436 with 5⁄16 in. thickness [a,b,d]		ASTM F436 washer with either a 3⁄8 in. thick plate washer or continuous bar [b,c]

[a] This requirement shall not apply to heads of round head tension-control bolt assemblies that meet the requirements in Section 2.7 and provide a bearing circle diameter that meets the requirements of ASTM F1852 or F2280.

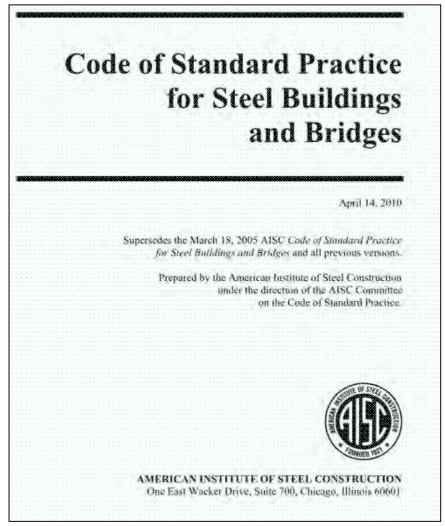
[b] Multiple washers with a combined thickness of 5⁄16 in. or larger do not satisfy this requirement.

[c] The plate washer or bar shall be of structural-grade steel material, but need not be hardened.

[d] Alternatively, a 3⁄8 in. thick plate washer and an ordinary thickness F436 washer may be used. The plate washer need not be hardened.

Secrets of the Manual

- **Part 16: Code of Standard Practice**
  - Section 3.1.2: Delegate Design
    - Fabricator shall submit representative samples of connections
    - Shop and Erection drawing review
  - Section 6.4 Fabrication Tolerance
  - Section 10: AESS Steel


Code of Standard Practice for Steel Buildings and Bridges

April 14, 2010

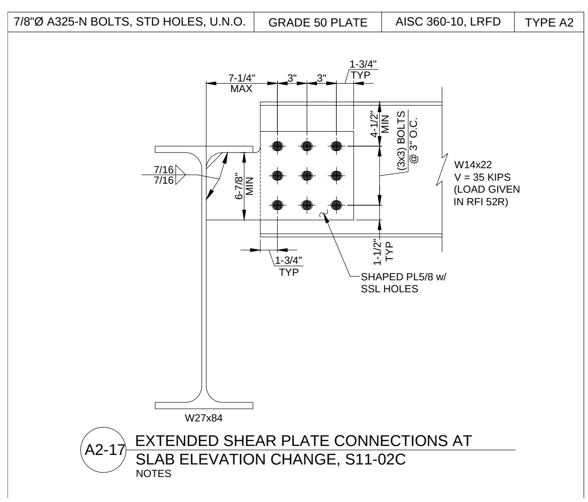
Supersedes the March 18, 2005 AISC *Code of Standard Practice for Steel Buildings and Bridges* and all previous versions.

Prepared by the American Institute of Steel Construction under the direction of the AISC Committee on the Code of Standard Practice.

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
One East Wacker Drive, Suite 700, Chicago, Illinois 60601

Connection Details for Delegated Design

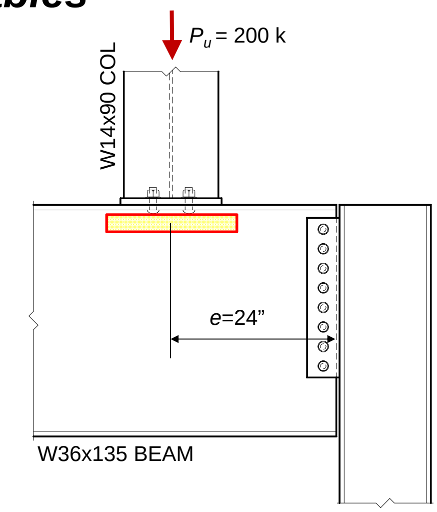
- Show all information to allow for checking
- Member forces and sizes
- Slopes
- ASD or LRFD with *Specification* Edition
- Plate thickness, edge distances, etc



Efficient Use of Tables

Bearing Example

- Given:
  - AISC Specification 360-10
  - AISC 14th Edition, LRFD
  - W Shapes ASTM A992, F_y = 50 ksi
  - 1" cap plate, Grade 50, F_y = 50 ksi
  - 7/8" dia. A325-N, STD holes, ϕr_v=24.3 kip
  - Load P_u = 200 kips
  - $d \geq e \geq d/2$

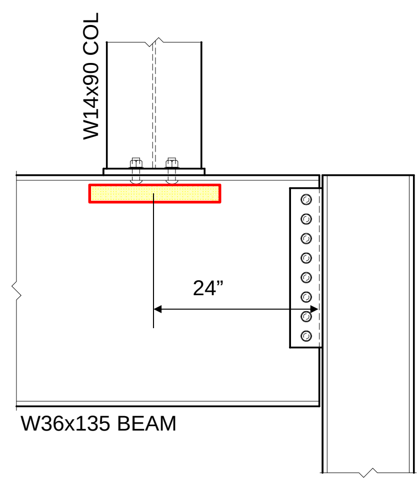


Efficient Use of Tables

- Web Local Yielding $(\phi = 1.00)$
  - AISC *Specification* Eq. J10-3

$$\phi R_n = \phi F_{yw} t_w (2.5k + l_b)$$

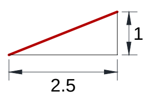
Determine l_b

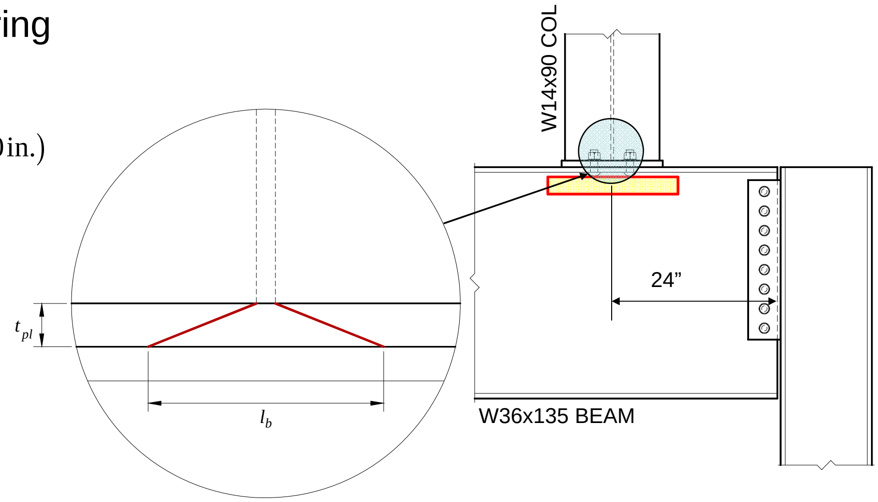


Efficient Use of Tables

- Length of bearing

$$l_b = t_{wc} + 5t_{pl}$$
$$= 0.440\,\text{in.} + 5(1.00\,\text{in.})$$
$$= 5.44\,\text{in.}$$





Efficient Use of Tables

- Web Local Yielding $(\phi = 1.00)$

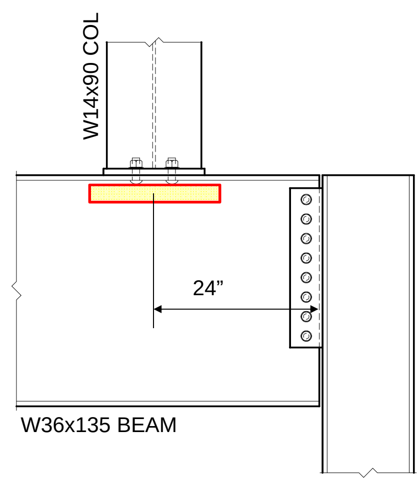
$l_b = 5.44\,\text{in.}$

– AISC *Specification* Eq. J10-3

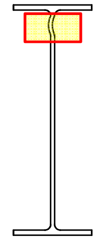
$$\phi R_n = \phi F_{yw} t_w (2.5k + l_b)$$
$$= 1.00(50\,\text{ksi})(0.600\,\text{in.})(2.5(1.54\,\text{in.}) + (5.44\,\text{in.}))$$
$$= 279\,\text{kips} \geq P_u = 200\,\text{kips} \quad \textbf{o.k.}$$

(b) When the concentrated force to be resisted is applied at a distance from the member end that is less than or equal to the depth of the member, *d*,

$$R_n = F_{yw} t_w (2.5k + l_b) \qquad \text{(J10-3)}$$



Efficient Use of Tables



- Web Local Crippling $(\phi = 0.75)$
  - AISC *Specification* Eq. J10-4

(a) When the concentrated compressive *force* to be resisted is applied at a distance from the member end that is greater than or equal to $d/2$:

$$R_n = 0.80 t_w^2 \left[1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5} \right] \sqrt{\frac{E F_{yw} t_f}{t_w}} \qquad \text{(J10-4)}$$

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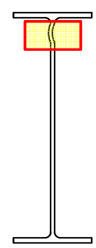
Efficient Use of Tables



- Web Local Crippling $(\phi = 0.75)$

$l_b = 5.44\,\text{in.}$

  - AISC *Specification* Eq. J10-4

$$\phi R_n = \phi 0.80 t_w^2 \left(1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5} \right) \sqrt{\frac{E F_{yw} t_f}{t_w}}$$

$$= 0.75(0.80)(0.600\,\text{in.})^2 \left(1 + 3\left(\frac{5.44\,\text{in.}}{35.6\,\text{in.}}\right)\left(\frac{0.600\,\text{in.}}{0.790\,\text{in.}}\right)^{1.5} \right) \sqrt{\frac{(29000\,\text{ksi})(50\,\text{ksi})(0.790\,\text{in.})}{(0.600\,\text{in.})}}$$

$$= 389\,\text{kips} \geq P_u = 200\,\text{kips} \;\; \textbf{o.k.}$$

Secrets of the Manual 60

Efficient Use of Tables

- Web Local Yielding $(\phi = 1.00)$

$l_b = 5.44\,\text{in.}$

– AISC 14th Edition Eq. 9-45a

LRFD
$\phi R_n = \phi R_1 + l_b(\phi R_2)$ (9-45a)

$$\phi R_n = \boxed{\phi R_1} + l_b\left(\boxed{\phi R_2}\right)$$

$$= 116\,\text{kips} + (5.44\,\text{in.})\left(30.0\,\frac{\text{kips}}{\text{in.}}\right)$$

$$= 279\,\text{kips} \geq P_u = 200\,\text{kips} \quad \textbf{o.k.}$$

Shape	R_1/Ω	ϕR_1	R_2/Ω	ϕR_2	R_3/Ω	ϕR_3	R_4/Ω	ϕR_4
	kips	kips	kips/in.	kips/in.	kips	kips	kips/in.	kips/in.
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
×135	77.0	116	20.0	30.0	99.5	149	5.55	8.32

Secrets of the Manual 61

Efficient Use of Tables

- Web Local Crippling $(\phi = 0.75)$

$l_b = 5.44\,\text{in.}$

– AISC 14th Edition Eq. 9-49a

LRFD
$\phi R_n = 2[(\phi R_3) + l_b(\phi R_4)]$ (9-49a)

$$\phi R_n = 2\left[\boxed{\phi R_3} + l_b\left(\boxed{\phi R_4}\right)\right]$$

$$= 2\left[149\,\text{kips} + (5.44\,\text{in.})\left(8.32\,\frac{\text{kips}}{\text{in.}}\right)\right]$$

$$= 389\,\text{kips} \geq P_u = 200\,\text{kips} \quad \textbf{o.k.}$$

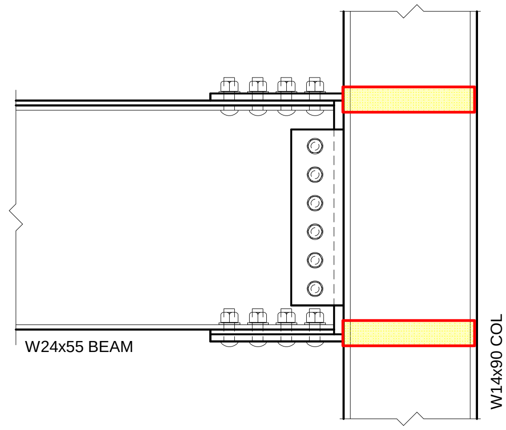
Shape	R_1/Ω	ϕR_1	R_2/Ω	ϕR_2	R_3/Ω	ϕR_3	R_4/Ω	ϕR_4
	kips	kips	kips/in.	kips/in.	kips	kips	kips/in.	kips/in.
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
×135	77.0	116	20.0	30.0	99.5	149	5.55	8.32

Secrets of the Manual 62

Efficient Use of Tables

Column Stiffener Checks

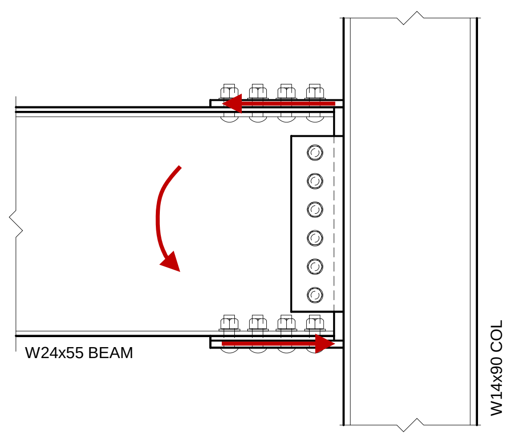
- Given:
  - AISC Specification 360-10
  - AISC 14th Edition, LRFD
  - W Shapes ASTM A992, F_y = 50 ksi
  - 3/4"x9" flange plates, grade 50, F_y = 50 ksi
  - 7/8" dia. A325-N, STD holes, ϕr_v=24.3 kip
  - Load M_u = 300 kips-ft



Efficient Use of Tables

- Flange Force

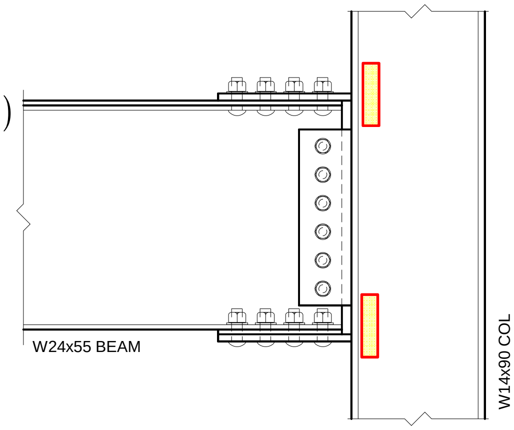
$$P_{fl} = \frac{M_u}{d}$$
$$= \frac{300\,\text{kips-ft}}{23.6\,\text{in.}}$$
$$= 153\,\text{kips}$$



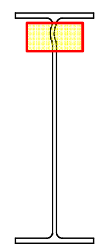
Efficient Use of Tables

- Web Local Yielding $(\phi = 1.00)$
  - AISC *Specification* Eq. J10-2

$$\phi R_n = \phi F_{yw} t_w \left(5k_{des} + t_{pl}\right)$$
$$= 1.00(50\,\text{ksi})(0.440\,\text{in.})\left(5(1.31\,\text{in.}) + (0.75\,\text{in.})\right)$$
$$= 161\ \text{kips} \geq P_{fl} = 153\,\text{kips}\ \ \textbf{o.k.}$$



Efficient Use of Tables



- Web Local Crippling $(\phi = 0.75)$
  - AISC *Specification* Eq. J10-4

$$\phi R_n = \phi 0.80 t_w^2 \left(1 + 3\left(\frac{t_{pl}}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right)\sqrt{\frac{EF_{yw}t_f}{t_w}}$$
$$= 0.75(0.80)(0.440\,\text{in.})^2\left(1 + 3\left(\frac{0.75\,\text{in.}}{14\,\text{in.}}\right)\left(\frac{0.440\,\text{in.}}{0.710\,\text{in.}}\right)^{1.5}\right)\sqrt{\frac{(29000\,\text{ksi})(50\,\text{ksi})(0.710\,\text{in.})}{(0.440\,\text{in.})}}$$
$$= 192\,\text{kips} \geq P_u = 153\,\text{kips}\ \ \textbf{o.k.}$$

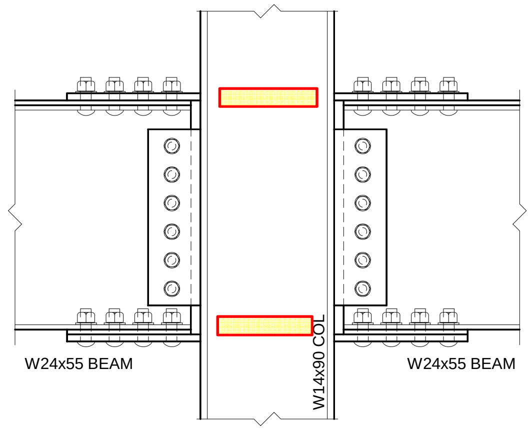
Efficient Use of Tables

- Web Compression Buckling $(\phi = 0.9)$
  - AISC *Specification* Eq. J10-8

$$\phi R_n = \phi \frac{24 t_w^3 \sqrt{E F_{yw}}}{h}$$

$$= (0.9) \frac{24(0.440\,\text{in.})^3 \sqrt{(29000\,\text{ksi})(50\,\text{ksi})}}{14\,\text{in.} - 2(1.31\,\text{in.})}$$

$$= 194\,\text{kips} \geq P_{fl} = 153\,\text{kips} \;\; \textbf{o.k.}$$



Efficient Use of Tables

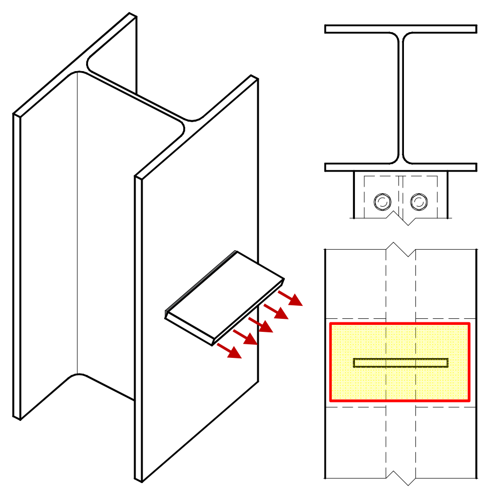
- Flange Local Bending $(\phi = 0.9)$
  - AISC *Specification* Eq. J10-1

$$\phi R_n = \phi 6.25 F_{yf} t_f^2$$

$$= 0.9(6.25)(50\,\text{ksi})(0.710\,\text{in.})^2$$

$$= 142\,\text{kips} \leq P_{fl} = 153\,\text{kips} \;\; \textbf{n.g.}$$

<u>Stiffeners are needed</u>



See Blodgett 5.7-8

Efficient Use of Tables

- Web Local Yielding $(\phi = 1.00)$
  - AISC *14th Edition Eq. 4-2a*

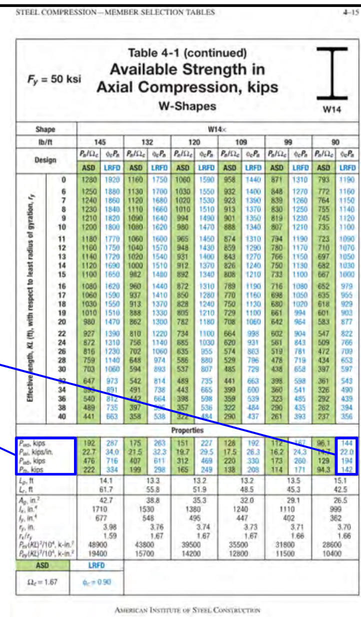
LRFD
$\phi R_n = P_{wo} + P_{wi} l_b \qquad (4\text{-}2a)$

$$\phi R_n = \boxed{P_{wo}} + \boxed{P_{wi}} l_b$$

$$= 144\,\text{kips} + \left(22.0\,\frac{\text{kips}}{\text{in.}}\right)(0.75\,\text{in.})$$

$$= 161\,\text{kips} \geq P_{fl} = 153\,\text{kips} \;\; \textbf{o.k.}$$

P_{wo}, kips	144
P_{wi}, kips/in.	22.0
P_{wb}, kips	194
P_{fb}, kips	142



Efficient Use of Tables

- Web Local Crippling $(\phi = 0.75)$
  - AISC *14th Edition Eq. 9-49a*

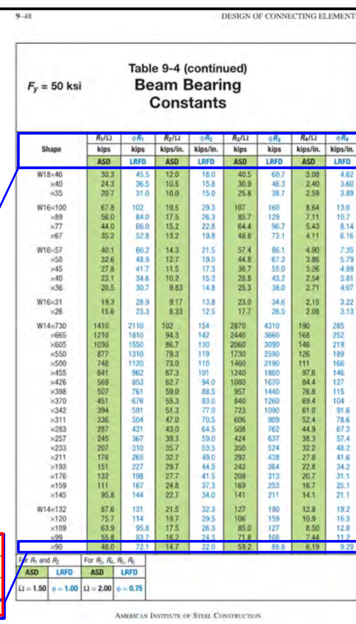
LRFD
$\phi R_n = 2[(\phi R_3) + l_b(\phi R_4)] \qquad (9\text{-}49a)$

$$\phi R_n = 2\left[\boxed{\phi R_3} + l_b \boxed{(\phi R_4)}\right]$$

$$= 2\left[88.8\,\text{kips} + (0.75\,\text{in.})\left(9.29\,\frac{\text{kips}}{\text{in.}}\right)\right]$$

$$= 192\,\text{kips} \geq P_u = 153\,\text{kips} \;\; \textbf{o.k.}$$

Shape	R_1/Ω kips ASD	ϕR_1 kips LRFD	R_2/Ω kips/in. ASD	ϕR_2 kips/in. LRFD	R_3/Ω kips ASD	ϕR_3 kips LRFD	R_4/Ω kips/in. ASD	ϕR_4 kips/in. LRFD
×90	48.0	72.1	14.7	22.0	59.2	88.8	6.19	9.29



Efficient Use of Tables

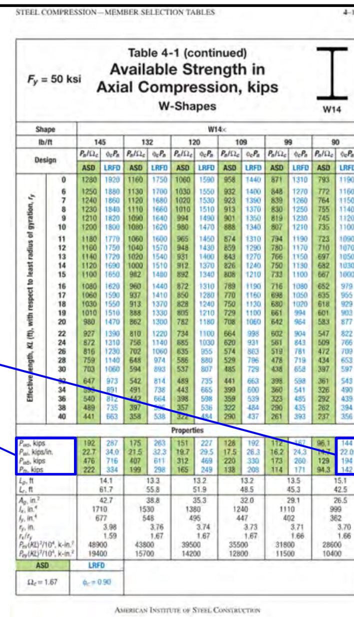
- Web Compression Buckling $(\phi = 0.9)$
  - AISC *14th Edition Eq. 4-3a*

LRFD	
$\phi R_n = P_{wb}$	(4-3a)

$\phi R_n = \boxed{P_{wb}}$

$= 194\,\text{kips} \geq P_{fl} = 153\,\text{kips}$ **o.k.**

P_{wo}, kips	144
P_{wi}, kips/in.	22.0
P_{wb}, kips	194
P_{fb}, kips	142



Efficient Use of Tables

- Flange Local Bending $(\phi = 0.9)$
  - AISC *14th Edition Eq. 4-4a*

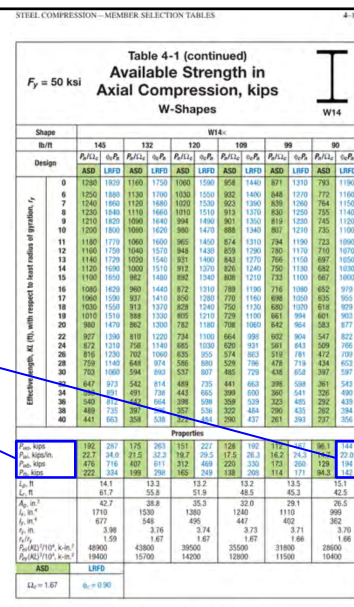
LRFD	
$\phi R_n = P_{fb}$	(4-4a)

$\phi R_n = \boxed{P_{fb}}$

$= 142\,\text{kips} \leq P_{fl} = 153\,\text{kips}$ **n.g.**

Stiffeners are needed

P_{wo}, kips	144
P_{wi}, kips/in.	22.0
P_{wb}, kips	194
P_{fb}, kips	142

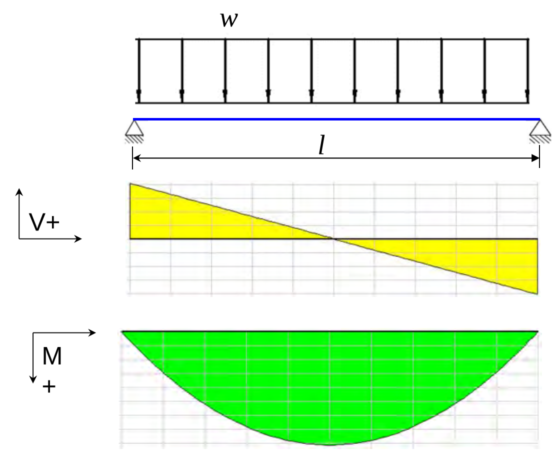


Polling Question

Understanding Tables

Determine End Reaction from UDL

- Given:
  - AISC Specification 360-10
  - AISC 14th Edition, LRFD
  - W14x22 ASTM A992, F_y = 50 ksi
  - L= 20 ft
  - End Reaction = 50% UDL

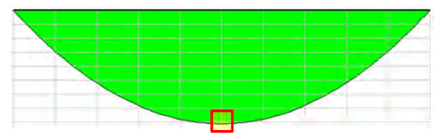


Understanding Tables

- Shear Based on Flexural Yielding Strength $(\phi = 0.9)$
  - AISC *Specification* Eq. F2-1

$$\phi M_n = \phi F_y Z_x$$
$$= 0.9(50\,\text{ksi})(33.2\,\text{in.}^3)$$
$$= 1494\,\text{kip-in (1 ft/12 in.)}$$
$$= 125\,\text{kip-ft}$$

$$wl_{flexure} = \frac{\phi M_n (8)}{l}$$
$$= \frac{125\,\text{kip-ft}(8)}{(20\,\text{ft})}$$
$$= 49.8\text{ kips}$$



$$M_{\max} = \frac{w(l)^2}{8}$$

Understanding Tables

- Shear Based on Shear Strength $(\phi = 1)$
  - AISC *Specification* Eq. G2-1

$$\phi V_n = \phi(0.6)(F_y)(A_w)(C_v)$$
$$= 1(0.6)(50\,\text{ksi})(13.7\text{ in.})(0.23\text{ in.})(1)$$
$$= 94.5\text{ kips}$$

$$V_{\max} = \frac{w(l)}{2}$$

$$wl_{shear} = \phi V_n (2)$$
$$= 94.5\text{ kips}(2)$$
$$= 189\text{ kips}$$

Understanding Tables

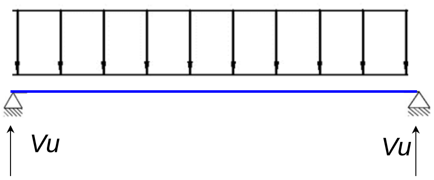
- Determine End Reaction Based on Percent UDL

$$V_u = \min\left((wl_{flexure})\frac{(\%\text{UDL})}{100\%}, \frac{wl_{shear}}{2}\right)$$

$$= \min\left(49.8\text{ kip}\frac{(50\%)}{100\%}, \frac{189\text{ kips}}{2}\right)$$

$$= \min(24.9\text{ kip}, 94.5\text{ kip})$$

$$= 24.9\text{ kip}$$



wl= 49.8

663	996
82.8	125
50.6	76.1
4.78	7.27
63.0	94.5

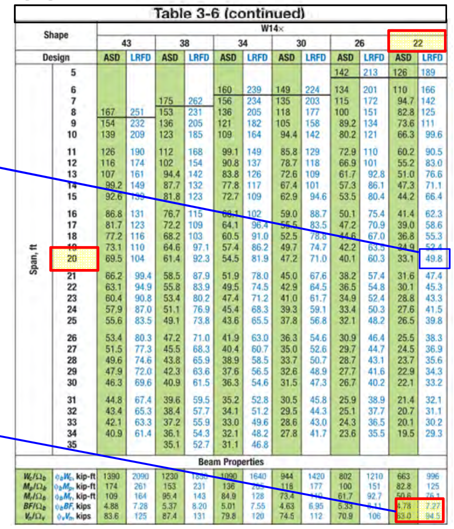


Table 3-6 (continued)

Shape		W14×											
		43		38		34		30		26		22	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	5									142	213	126	189
	6					160	239	149	224	134	201	110	166
	7			175	262	156	234	135	203	115	172	94.7	142
	8	167	251	153	231	136	205	118	177	100	151	82.8	125
	9	154	232	136	205	121	182	105	158	89.2	134	73.6	111
	10	139	209	123	185	109	164	94.4	142	80.2	121	66.3	99.6
	11	126	190	112	168	99.1	149	85.8	129	72.9	110	60.2	90.5
	12	116	174	102	154	90.8	137	78.7	118	66.9	101	55.2	83.0
	13	107	161	94.4	142	83.8	126	72.6	109	61.7	92.8	51.0	76.6
	14	99.2	149	87.7	132	77.8	117	67.4	101	57.3	86.1	47.3	71.1
	15	92.6	139	81.8	123	72.7	109	62.9	94.6	53.5	80.4	44.2	66.4
	16	86.8	131	76.7	115	68.1	102	59.0	88.7	50.1	75.4	41.4	62.3
	17	81.7	123	72.2	109	64.1	96.4	55.5	83.5	47.2	70.9	39.0	58.6
	18	77.2	116	68.2	103	60.5	91.0	52.5	78.8	44.6	67.0	36.8	55.3
	19	73.1	110	64.6	97.1	57.4	86.2	49.7	74.7	42.2	63.5	34.9	52.4
	20	69.5	104	61.4	92.3	54.5	81.9	47.2	71.0	40.1	60.3	33.1	49.8
	21	66.2	99.4	58.5	87.9	51.9	78.0	45.0	67.6	38.2	57.4	31.6	47.4
	22	63.1	94.9	55.8	83.9	49.5	74.5	42.9	64.5	36.5	54.8	30.1	45.3
	23	60.4	90.8	53.4	80.2	47.4	71.2	41.0	61.7	34.9	52.4	28.8	43.3
	24	57.9	87.0	51.1	76.9	45.4	68.3	39.3	59.1	33.4	50.3	27.6	41.5
	25	55.6	83.5	49.1	73.8	43.6	65.5	37.8	56.8	32.1	48.2	26.5	39.8
	26	53.4	80.3	47.2	71.0	41.9	63.0	36.3	54.6	30.9	46.4	25.5	38.3
	27	51.5	77.3	45.5	68.3	40.4	60.7	35.0	52.6	29.7	44.7	24.5	36.9
	28	49.6	74.6	43.8	65.9	38.9	58.5	33.7	50.7	28.7	43.1	23.7	35.6
	29	47.9	72.0	42.3	63.6	37.6	56.5	32.6	48.9	27.7	41.6	22.9	34.3
	30	46.3	69.6	40.9	61.5	36.3	54.6	31.5	47.3	26.7	40.2	22.1	33.2
	31	44.8	67.4	39.6	59.5	35.2	52.8	30.5	45.8	25.9	38.9	21.4	32.1
	32	43.4	65.3	38.4	57.7	34.1	51.2	29.5	44.3	25.1	37.7	20.7	31.1
	33	42.1	63.3	37.2	55.9	33.0	49.6	28.6	43.0	24.3	36.5	20.1	30.2
	34	40.9	61.4	36.1	54.3	32.1	48.2	27.8	41.7	23.6	35.5	19.5	29.3
	35			35.1	52.7	31.1	46.8						
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	1390	2090	1230	1850	1090	1640	944	1420	802	1210	663	996
M_p/Ω_b	$\phi_b M_p$, kip-ft	174	261	153	231	136	205	118	177	100	151	82.8	125
M_r/Ω_b	$\phi_b M_r$, kip-ft	109	164	95.4	143	84.9	128	73.4	110	61.7	92.7	50.6	76.1
BF/Ω_b	$\phi_b BF$, kips	4.88	7.28	5.37	8.20	5.01	7.55	4.63	6.95	5.33	8.11	4.78	7.27
V_n/Ω_v	$\phi_v V_n$, kips	83.6	125	87.4	131	79.8	120	74.5	112	70.9	106	63.0	94.5

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Understanding Tables

- UDL Cautions:

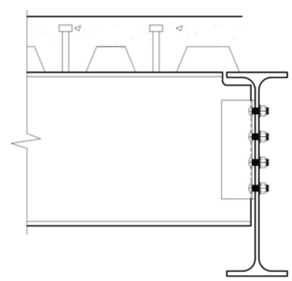
  – For 100% composite, $\phi M_n \approx 297$ kip-ft.

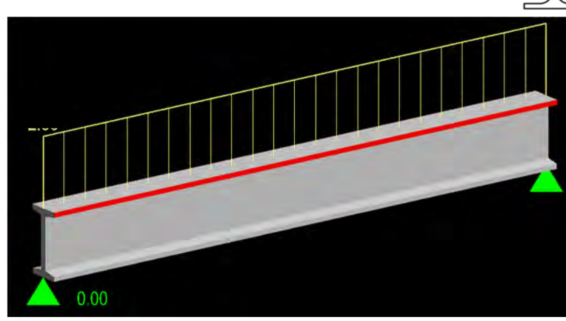
  This gives:

$$V_u = \frac{\phi M_n(8)}{l(2)}$$

$$= \frac{297\text{ kip-ft}(8)}{20\text{ ft }(2)}$$

$$= 59.4\text{ kips} \approx 119\%\text{ UDL}$$



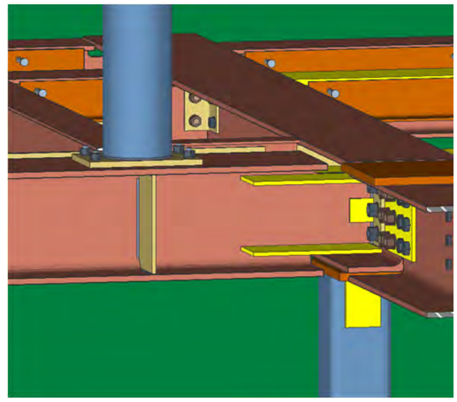


Secrets of the Manual 78

Understanding Tables

- UDL Cautions:
  - For Point Loads

$$\phi V_{u\_max} = \phi(0.6)(F_y)(A_w)(C_v)$$
$$= 1(0.6)(50\,\text{ksi})(13.7\text{ in.})(0.23\text{ in.})(1)$$
$$= 94.5\text{ kips} \approx 190\%\text{ UDL}$$



User Note: All current ASTM A6 W, S and HP shapes except W44×230, W40×149, W36×135, W33×118, W30×90, W24×55, W16×26 and W12×14 meet the criteria stated in Section G2.1(a) for $F_y = 50$ ksi (345 MPa).

Secrets of the Manual 79

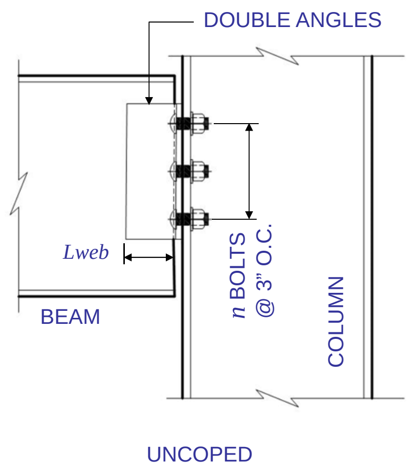
Understanding Tables

- Working backwards:
  - Given: Connection strength, Find: $\phi V_{n\_connection}$
    Minimum beam length, l_{min}

$$l_{min\_flexure} = \frac{\phi M_n(8)(\%\text{UDL})}{100\%(\phi V_{n\_connection})}$$

$$l_{min\_shear} = \frac{\phi M_n(8)}{\phi V_n(2)}$$

$$l_{min} = \max(l_{min\_flexure}, l_{min\_shear})$$



Secrets of the Manual 80

Understanding Tables

- Working backwards:
  - Given: Connection strength, $\phi V_{n\_connection} = 71\text{ kip}$
  - Find: Minimum beam length, $l_{\min}$

$$l_{\min\_flexure} = \frac{\phi M_n(8)(\%\text{UDL})}{100\%(\phi V_{n\_connection})} = \frac{(125\text{ kip-ft})(8)(50\%)}{100\%(71\text{ kip})} = 7.04\text{ ft}$$

$$l_{\min\_shear} = \frac{\phi M_n(8)}{\phi V_n(2)} = \frac{(125\text{ kip-ft})(8)}{(94.5\text{ kip})(2)} = 5.29\text{ ft}$$

$$l_{\min} = \max(l_{\min\_flexure}, l_{\min\_shear}) = 7.04\text{ ft}$$

V_u= 142k/2=71k

Shape		W14× 43		38		34		30		26		22	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	5									142	213	126	189
	6					160	239	149	224	134	201	110	166
	7			175	262	156	234	135	203	115	172	94.7	142
	8	167	251	153	231	136	205	118	177	100	151	82.8	125
	9	154	232	136	205	121	182	105	158	89.2	134	73.6	111
	10	139	209	123	185	109	164	94.4	142	80.2	121	66.3	99.6
	11	126	190	112	168	99.1	149	85.8	129	72.9	110	60.2	90.5
	12	116	174	102	154	90.8	137	78.7	118	66.9	101	55.2	83.0
	13	107	161	94.4	142	83.8	126	72.6	109	61.7	92.8	51.0	76.6
	14	99.2	149	87.7	132	77.8	117	67.4	101	57.3	86.1	47.3	71.1
	15	92.6	139	81.8	123	72.7	109	62.9	94.6	53.5	80.4	44.2	66.4
	16	86.8	131	76.7	115	68.1	102	59.0	88.7	50.1	75.4	41.4	62.3
	17	81.7	123	72.2	109	64.1	96.4	55.5	83.5	47.2	70.9	39.0	58.6
	18	77.2	116	68.2	103	60.5	91.0	52.5	78.8	44.6	67.0	36.8	55.3
	19	73.1	110	64.6	97.1	57.4	86.2	49.7	74.7	42.2	63.5	34.9	52.4
	20	69.5	104	61.4	92.3	54.5	81.9	47.2	71.0	40.1	60.3	33.1	49.8
	21	66.2	99.4	58.5	87.9	51.9	78.0	45.0	67.6	38.2	57.4	31.6	47.4
	22	63.1	94.9	55.8	83.9	49.5	74.5	42.9	64.5	36.5	54.8	30.1	45.3
	23	60.4	90.8	53.4	80.2	47.4	71.2	41.0	61.7	34.9	52.4	28.8	43.3
	24	57.9	87.0	51.1	76.9	45.4	68.3	39.3	59.1	33.4	50.3	27.6	41.5
	25	55.6	83.5	49.1	73.8	43.6	65.5	37.8	56.8	32.1	48.2	26.5	39.8
	26	53.4	80.3	47.2	71.0	41.9	63.0	36.3	54.6	30.9	46.4	25.5	38.3
	27	51.5	77.3	45.5	68.3	40.4	60.7	35.0	52.6	29.7	44.7	24.5	36.9
	28	49.6	74.6	43.8	65.9	38.9	58.5	33.7	50.7	28.7	43.1	23.7	35.6
	29	47.9	72.0	42.3	63.6	37.6	56.5	32.6	48.9	27.7	41.6	22.9	34.3
	30	46.3	69.6	40.9	61.5	36.3	54.6	31.5	47.3	26.7	40.2	22.1	33.2
	31	44.8	67.4	39.6	59.5	35.2	52.8	30.5	45.8	25.9	38.9	21.4	32.1
	32	43.4	65.3	38.4	57.7	34.1	51.2	29.5	44.3	25.1	37.7	20.7	31.1
	33	42.1	63.3	37.2	55.9	33.0	49.6	28.6	43.0	24.3	36.5	20.1	30.2
	34	40.9	61.4	36.1	54.3	32.1	48.2	27.8	41.7	23.6	35.5	19.5	29.3
	35			35.1	52.7	31.1	46.8						
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	1390	2090	1230	1850	1090	1640	944	1420	802	1210	663	996
M_p/Ω_b	$\phi_b M_p$, kip-ft	174	261	153	231	136	205	118	177	100	151	82.8	125
M_r/Ω_b	$\phi_b M_r$, kip-ft	109	164	95.4	143	84.9	128	73.4	110	61.7	92.7	50.6	76.1
BF/Ω_b	$\phi_b BF$, kips	4.88	7.28	5.37	8.20	5.01	7.55	4.63	6.95	5.33	8.11	4.78	7.27
V_n/Ω_v	$\phi_v V_n$, kips	83.6	125	87.4	131	79.8	120	74.5	112	70.9	106	63.0	94.5

Understanding Tables

Maximum Total Uniform Load Tables

Table 3-6. W-Shapes—Maximum Total Uniform Load

Maximum total uniform loads on braced ($L_b \le L_p$) simple-span beams bent about the strong axis are given for W-shapes with F_y = 50 ksi (ASTM A992). The uniform load constant, $\phi_b W_c$ or W_c/Ω_b (kip-ft), divided by the span length, L (ft), provides the maximum total uniform load (kips) for a braced simple-span beam bent about the strong axis. This is based on the available flexural strength as discussed for Table 3-2.

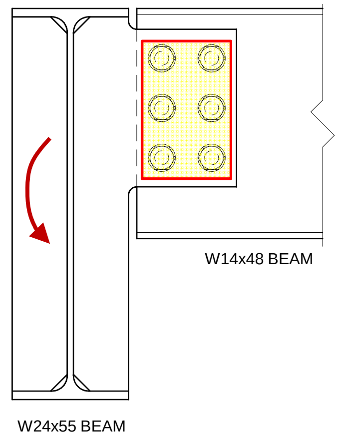
The strong-axis available shear strength, $\phi_v V_n$ or V_n/Ω_v, can be determined using the tabulated value. Above the heavy horizontal line in the tables, the maximum total uniform load is limited by the strong-axis available shear strength.

The tabulated values can also be used for braced simple-span beams with equal concentrated loads spaced as shown in Table 3-22a if the concentrated loads are first converted to an equivalent uniform load.

Efficient Use of Tables

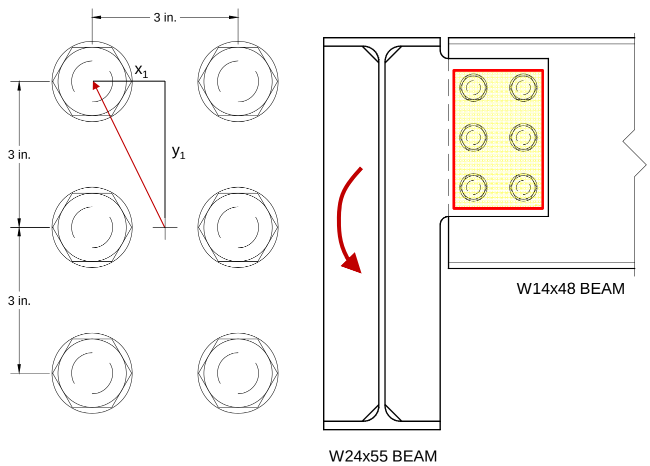
Instantaneous Center of Rotation Method
Spandrel example

- Given:
  - AISC Specification 360-10
  - AISC 14th Edition, LRFD
  - W Shapes ASTM A992, F_y = 50 ksi
  - 3/8" plates, grade 50, F_y = 50 ksi
  - 7/8" dia. A325-N, STD holes, ϕr_v=24.3 kip
  - Bolt spacing: 3" vertical, 3" horizontal
  - Load: Façade moment M_f = 100 kip-in



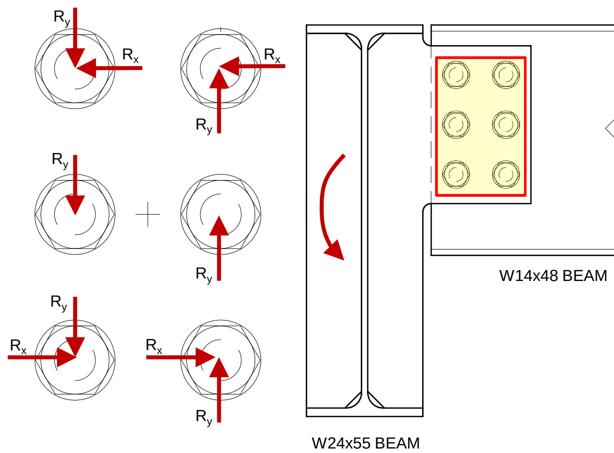
Efficient Use of Tables

$$I_x = \sum_{i=1}^{6} x_i^2 = 6(1.5\,\text{in.})^2$$
$$= 13.5\,\text{in}^2$$
$$I_y = \sum_{i=1}^{6} y_i^2 = 4(3\text{in.})^2$$
$$= 36\,\text{in}^2$$
$$I_p = I_x + I_y$$
$$= 49.5\,\text{in}^2$$



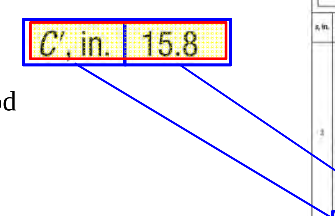
Efficient Use of Tables

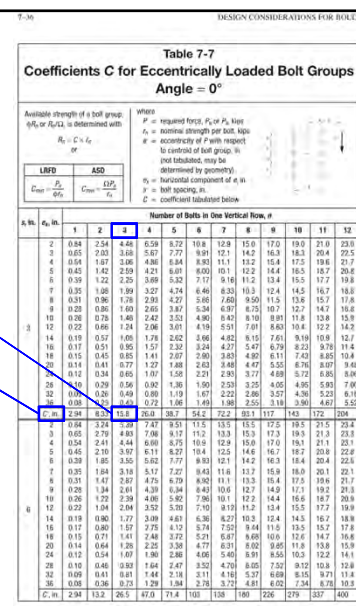
$$R_x = \frac{M_f \max\limits_{i=1..6}(y_i)}{I_p} = \frac{(100\,\text{kip-in.})(1.5\,\text{in.})}{49.5\,\text{in.}^2}$$
$$= 3.03\,\text{kip}$$
$$R_y = \frac{M_f \max\limits_{i=1..6}(x_i)}{I_p} = \frac{(100\,\text{kip-in.})(3.0\,\text{in.})}{49.5\,\text{in.}^2}$$
$$= 6.06\,\text{kip}$$
$$R = \sqrt{R_x^2 + R_y^2} = \sqrt{(3.03\,\text{kip})^2 + (6.06\,\text{kip})^2}$$
$$= 6.78\,\text{kip}$$



Efficient Use of Tables

$$M_{\max\_1} = M_f \frac{(\phi r_{nv})}{R} = 100 \text{ kip-in} \frac{(24.3\,\text{kips})}{6.78 \text{ kips}}$$
$$= 360 \text{ kips-in.}$$





Use the Instantaneous Center of Rotation Method with C'

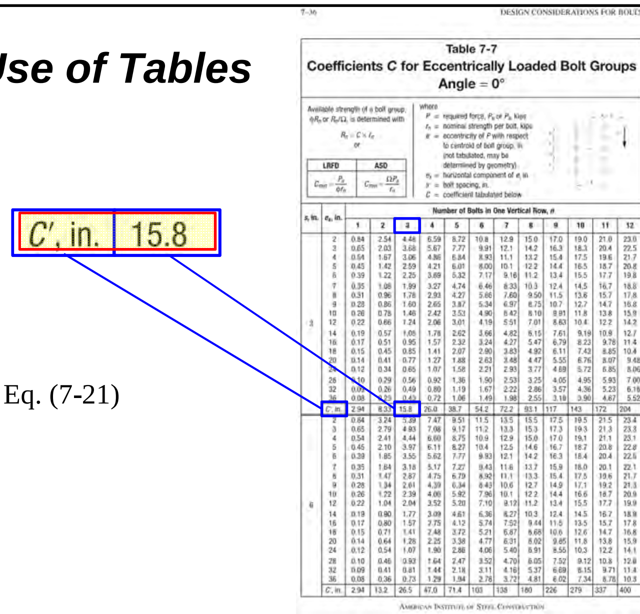
$C' = 15.8$ (From Table 7-7)

$$M_{\max\_2} = C'(\phi r_{nv}) = (15.8\text{in.})(24.3\,\text{kips})$$
$$= 384 \text{ kip-in.}$$

$$\frac{M_f}{R} = \frac{100\,\text{kip-in.}}{6.78\,\text{in.}} = 14.8\,\text{in.} \le C' = 15.8 \text{ in}$$

Efficient Use of Tables

$$C' = \left[\sum l_i \left(1 - e^{-\left(\frac{10 l_i \Delta_{max}}{l_{max}} \right)} \right)^{0.55} \right], \text{in.} \qquad \text{Manual Eq. (7-21)}$$



Secrets of the Manual

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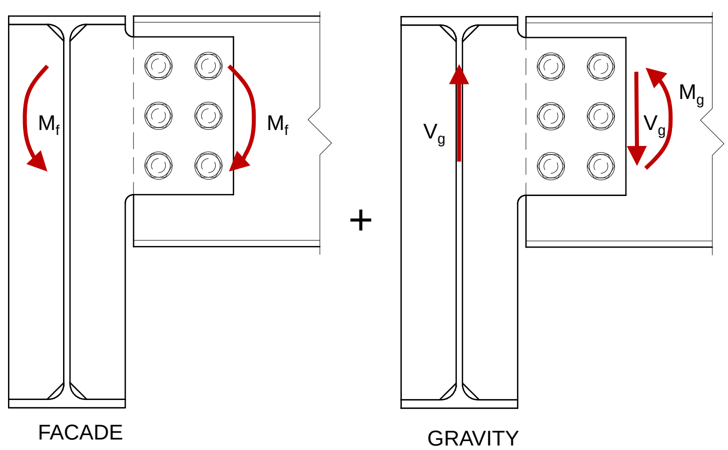
Efficient Use of Tables

- Façade and Gravity Moments Should Act Opposite



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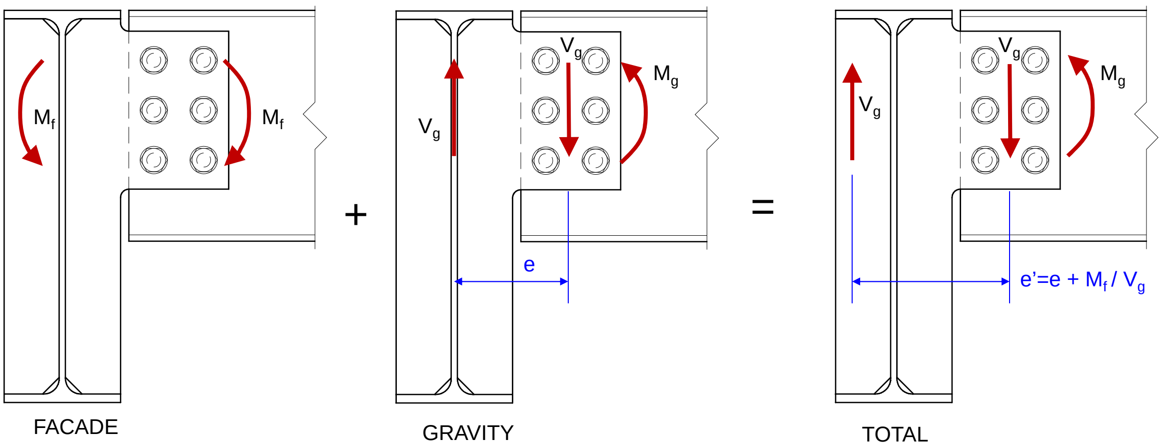
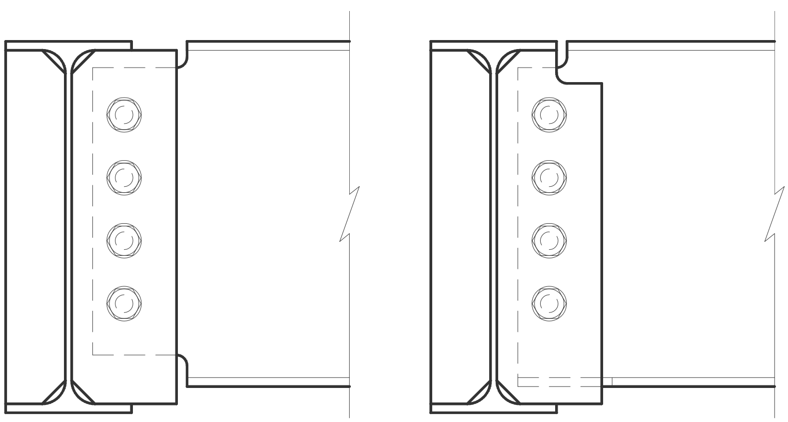
Efficient Use of Tables



Efficient Use of Tables



NON EXTENDED SPANDREL CONNECTIONS

Efficient Use of Tables

- Pick a clevis: 2" rod, F_y = 36 ksi, full strength

$$A_{rod} = \frac{\pi\left(d_{rod}^2\right)}{4} = \frac{\pi\left(2\,\text{in.}\right)^2}{4}$$
$$= 3.14\,\text{in.}^2$$
$$T_{\max} = \phi A_{rod} F_{y\_rod} = 0.9\left(3.14\,\text{in.}^2\right)\left(36\,\text{ksi}\right)$$
$$= 102\,\text{kips}$$

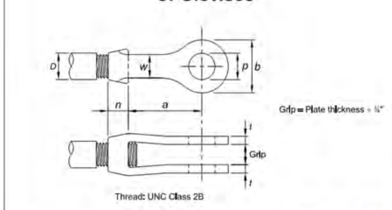
Select Clevis Number 6,

Design strength = 135 kips ≥ 102 kips

6	2 1/2	3	6	2 3/4	8	3	3/4 (+3/32, -0)	26	90.0	135

* Tabulated available strengths are based on ϕ = 0.50, Ω = 3.00. Strength at service load corresponds to a 3:1 safety factor using maximum pin diameter.

Table 15-4
Dimensions and Weights of Clevises

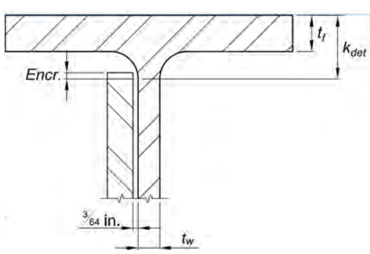


Clevis Number	Dimensions, in.							Weight, lb	Available Strength, kips*	
	Max. D	Max. p	b	n	a	w	t		ASD	LRFD
2	5/8	3/4	1 7/16	5/8	3 9/16	1 1/16	5/16 (+1/32, -0)	1	5.83	8.75
2 1/2	7/8	1 1/2	2 1/2	1	4	1 1/4	5/16 (+1/32, -0)	2.5	12.5	18.8
3	1 3/8	1 3/4	3	1 1/4	5 1/16	1 1/2	1/2 (+1/16, -1/32)	4	25.0	37.5
3 1/2	1 1/2	2	3 1/2	1 1/2	6	1 3/4	1/2 (+1/16, -1/16)	6	30.0	45.0
4	1 3/4	2 1/4	4	1 3/4	5 15/16	2	1/2 (+1/16, -1/16)	9	35.0	52.5
5	2 1/8	2 1/2	5	2 1/4	7	2 1/2	5/8 (+3/32, -0)	16	62.5	93.8
6	2 1/2	3	6	2 3/4	8	3	3/4 (+3/32, -0)	26	90.0	135
7	3	3 3/4	7	3	9	3 1/2	7/8 (+1/8, -1/16)	36	114	171
8	4	4 1/4	8	4	10 1/4	4	1 1/2 (+1/8, -1/16)	90	225	338

ASD	LRFD	Notes:
Ω = 3.00	ϕ = 0.50	Weights and dimensions of clevises are typical; products of all suppliers are essentially similar. User shall verify with the manufacturer that product meets available strength specifications above. * Strength at service load corresponds to a 3:1 safety factor using maximum pin diameter.

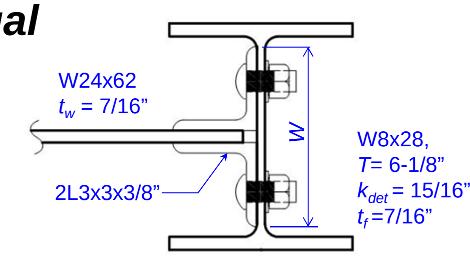
Secrets of Manual

- Check Double-Angle Tolerance using Fig 10-3



$k_{det} - t_f$, in.	Encr., in.
5⁄16	1⁄8
3⁄8 to 1⁄2	3⁄16
9⁄16 to 13⁄16	1⁄4
7⁄8 to 1 1⁄4	5⁄16
1 5⁄16 to 1 3⁄8	3⁄8

Fig. 10-3. Fillet encroachment (riding the fillet).

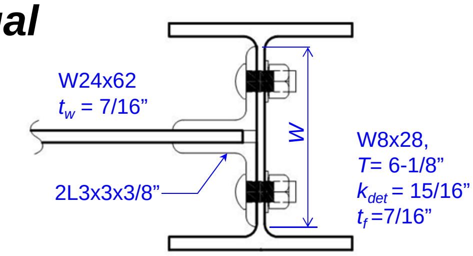


$$k_{det} - t_f = \frac{15}{16}\text{in.} - \frac{7}{16}\text{in.} = \frac{1}{2}\text{in.}$$

$$Encr. = \frac{3}{16}\text{in.}$$

Secrets of Manual

- Check Double-Angle Tolerance using Fig 10-3



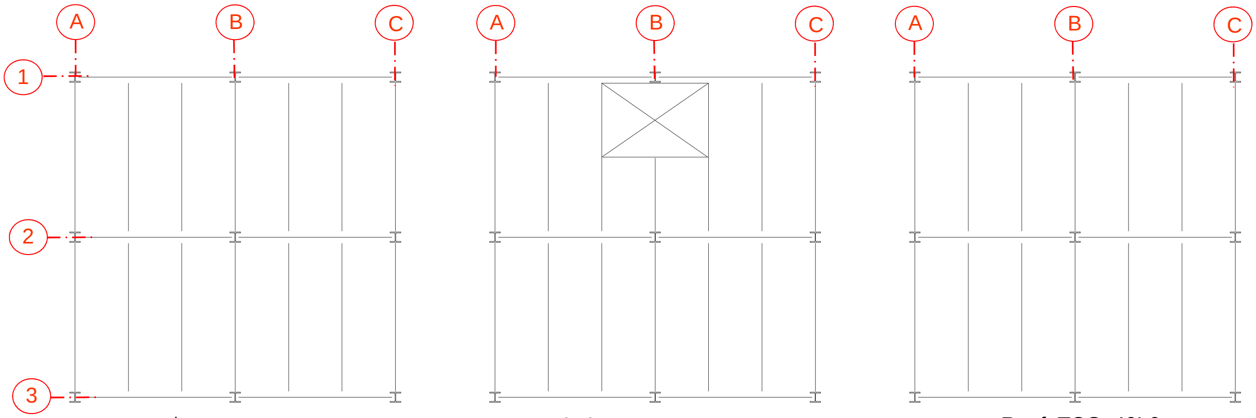
$$W_{max} = T + (2)\,Encr. = 6\frac{1}{8}\text{in.} + (2)\frac{3}{16}\text{in.} = 6\frac{1}{2}\text{in.}$$

$$W = t_w + (2)\,b_{angle} = \frac{7}{16}\text{in.} + (2)\,3\text{ in.} = 6\frac{7}{16}\text{in.}$$

$$W \le W_{max} \quad \textbf{o.k.}$$

Efficient Use of Tables

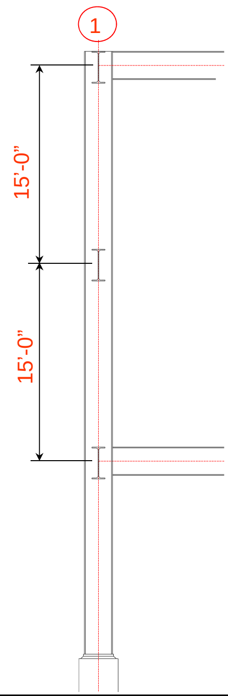
Determine Strength of Column at Grid B/1



Efficient Use of Tables

Determine Strength of Column at Grid B/1

- Given:
  - AISC *Specification* 360-10
  - AISC 14th Edition, LRFD
  - W14x90 ASTM A992, F_y = 50 ksi
    - r_x = 6.14 in.
    - r_y = 3.70 in.
  - L_x = 30 ft, L_y = 15 ft
  - K =1.0



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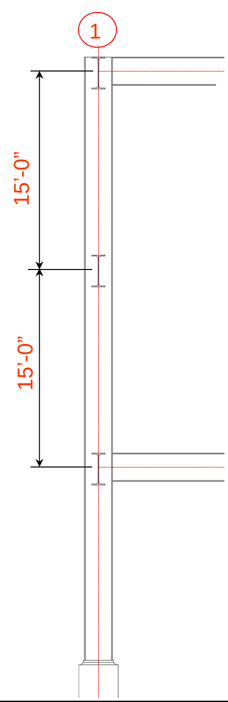
Efficient Use of Tables

- Determine Unbraced Length
  - Since unbraced length differs in the two axes, select member based on *y-y* axis then check equivalent *y-y* axis length for *x-x* axis.

$$\frac{KL_x}{r_x} = \frac{KL_y}{r_y}$$

$$\frac{KL_x}{KL_y} = \frac{r_x}{r_y}$$

$$KL_{y\_EQ} = \frac{KL_x}{\left(\dfrac{r_x}{r_y}\right)}$$



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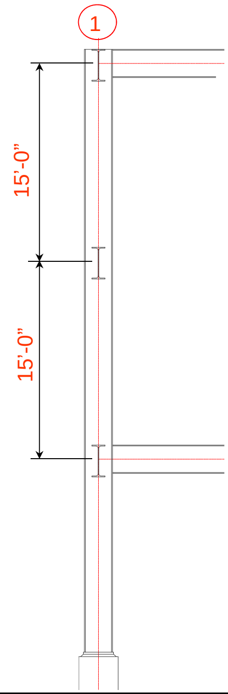
Efficient Use of Tables

- Determine Unbraced Length

$$\frac{r_x}{r_y} = \frac{6.14 \text{ in.}}{3.70 \text{ in.}} = 1.66 \qquad \text{(See Table T4-1a)}$$

$$KL_{y\_EQ} = \frac{KL_x}{\left(\frac{r_x}{r_y}\right)} = \frac{30.0 \text{ ft}}{1.66} = 18.1 \text{ ft} \qquad \textbf{(Controls)}$$

$$KL_y = 15.0 \text{ ft}$$



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Efficient Use of Tables

- Determine Strength
  - Using KL_{y_EQ} = 18.1 ft and interpolation:

$$\phi P_n = 926 \text{ kips}$$

979
955
929
903
877

F_y = 50 ksi

Table 4-1 (continued)
Available Strength in Axial Compression, kips
W-Shapes

W14

Shape		W14×											
lb/ft		145		132		120		109		99		90	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, KL (ft), with respect to least radius of gyration, r_y	0	1280	1920	1160	1750	1060	1590	958	1440	871	1310	793	1190
	6	1250	1880	1130	1700	1030	1550	932	1400	848	1270	772	1160
	7	1240	1860	1120	1680	1020	1530	923	1390	839	1260	764	1150
	8	1230	1840	1110	1660	1010	1510	913	1370	830	1250	755	1140
	9	1210	1820	1090	1640	994	1490	901	1350	819	1230	745	1120
	10	1200	1800	1080	1620	980	1470	888	1340	807	1210	735	1100
	11	1180	1770	1060	1600	965	1450	874	1310	794	1190	723	1090
	12	1160	1750	1040	1570	948	1430	859	1290	780	1170	710	1070
	13	1140	1720	1020	1540	931	1400	843	1270	766	1150	697	1050
	14	1120	1690	1000	1510	912	1370	826	1240	750	1130	682	1030
	15	1100	1650	982	1480	892	1340	808	1210	733	1100	667	1000
	16	1080	1620	960	1440	872	1310	789	1190	716	1080	652	979
	17	1060	1590	937	1410	850	1280	770	1160	698	1050	635	955
	18	1030	1550	913	1370	828	1240	750	1130	680	1020	618	929
	19	1010	1510	888	1330	805	1210	729	1100	661	994	601	903
	20	980	1470	862	1300	782	1180	708	1060	642	964	583	877
	22	927	1390	810	1220	734	1100	664	998	602	904	547	822
	24	872	1310	756	1140	685	1030	620	931	561	843	509	766
	26	816	1230	702	1060	635	955	574	863	519	781	472	709
	28	759	1140	648	974	586	880	529	796	478	719	434	653
	30	703	1060	594	893	537	807	485	729	438	658	397	597
	32	647	973	542	814	489	735	441	663	398	598	361	543
	34	593	891	491	738	443	665	399	600	360	541	326	490
	36	540	812	442	664	398	598	359	539	323	485	292	439
	38	489	735	397	596	357	536	322	484	290	435	262	394
	40	441	663	358	538	322	484	290	437	261	393	237	356
Properties													
P_{wo}, kips		192	287	175	263	151	227	128	192	112	167	96.1	144
P_{wi}, kips/in.		22.7	34.0	21.5	32.3	19.7	29.5	17.5	26.3	16.2	24.3	14.7	22.0
P_{wb}, kips		476	716	407	611	312	469	220	330	173	260	129	194
P_{fb}, kips		222	334	199	298	165	249	138	208	114	171	94.3	142
L_p, ft		14.1		13.3		13.2		13.2		13.5		15.1	
L_r, ft		61.7		55.8		51.9		48.5		45.3		42.5	
A_g, in.^{2}		42.7		38.8		35.3		32.0		29.1		26.5	
I_x, in.^{4}		1710		1530		1380		1240		1110		999	
I_y, in.^{4}		677		548		495		447		402		362	
r_y, in.		3.98		3.76		3.74		3.73		3.71		3.70	
r_x/r_y		1.59		1.67		1.67		1.67		1.66		1.66	
$P_{ex}(KL)^2/10^4$, k-in.^{2}		48900		43800		39500		35500		31800		28600	
$P_{ey}(KL)^2/10^4$, k-in.^{2}		19400		15700		14200		12800		11500		10400	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Secrets of the Manual

Efficient Use of Tables

- Check Using Specification

$$\frac{KL_x}{r_x} = \frac{1(30.0\text{ ft})(12\text{ in./ft})}{6.14\text{ in.}} = 58.6 \quad \text{(Controls)}$$

$$\frac{KL_y}{r_y} = \frac{1(15\text{ ft})(12\text{ in./ft})}{3.7\text{ in.}} = 48.6$$

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} = 113$$

The *nominal compressive strength*, P_n, shall be determined based on the *limit state* of *flexural buckling*.

$$P_n = F_{cr} A_g \quad \text{(E3-1)}$$

The *critical stress*, F_{cr}, is determined as follows:

(a) When $\frac{KL}{r} \le 4.71\sqrt{\frac{E}{F_y}}$ (or $\frac{F_y}{F_e} \le 2.25$)

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y \quad \text{(E3-2)}$$

(b) When $\frac{KL}{r} > 4.71\sqrt{\frac{E}{F_y}}$ (or $\frac{F_y}{F_e} > 2.25$)

$$F_{cr} = 0.877 F_e \quad \text{(E3-3)}$$

where

F_e = elastic *buckling* stress determined according to Equation E3-4, as specified in Appendix 7, Section 7.2.3(b), or through an elastic buckling analysis, as applicable, ksi (MPa)

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} \quad \text{(E3-4)}$$

Efficient Use of Tables

- Check Using Specification

$$F_e = \frac{\pi^2 E}{\left(\frac{kL}{r}\right)^2} = \frac{\pi^2(29{,}000\text{ ksi})}{(58.6)^2} = 83.3 \quad \text{(E3-4)}$$

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y = \left[0.658^{\left(\frac{50\text{ ksi}}{83.3\text{ ksi}}\right)}\right] 50\text{ ksi} = 38.9\text{ ksi} \quad \text{(E3-2)}$$

$$\phi F_{cr} = (0.9)38.9\text{ ksi} = 35.0\text{ ksi}$$

58	23.4	35.2
59	23.2	34.9

$$\phi P_n = 0.9 F_{cr} A_g = (35.0\text{ ksi})(26.5\text{ in.}^2) = 927\text{ kips} \approx 926\text{ kips} \quad \textbf{o.k.}$$

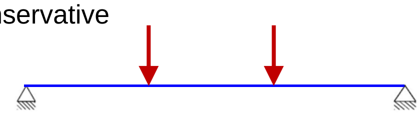
Table 4-22 (continued)
Available Critical Stress for Compression Members

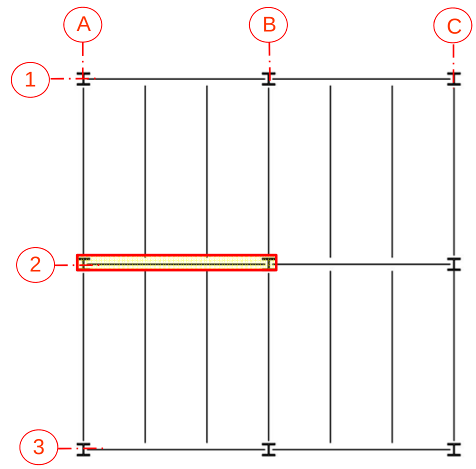
	F_y = 35 ksi			F_y = 36 ksi			F_y = 42 ksi			F_y = 46 ksi			F_y = 50 ksi	
$\frac{KL}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{KL}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{KL}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{KL}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{KL}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD
41	19.2	28.9	41	19.7	29.7	41	22.7	34.1	41	24.6	37.0	41	26.5	39.8
42	19.2	28.8	42	19.6	29.5	42	22.6	33.9	42	24.5	36.8	42	26.3	39.5
43	19.1	28.7	43	19.6	29.4	43	22.5	33.7	43	24.3	36.6	43	26.2	39.3
44	19.0	28.5	44	19.5	29.3	44	22.3	33.6	44	24.2	36.3	44	26.0	39.1
45	18.9	28.4	45	19.4	29.1	45	22.2	33.4	45	24.0	36.1	45	25.8	38.8
46	18.8	28.3	46	19.3	29.0	46	22.1	33.2	46	23.9	35.9	46	25.6	38.5
47	18.7	28.1	47	19.2	28.9	47	22.0	33.0	47	23.8	35.7	47	25.5	38.3
48	18.6	28.0	48	19.1	28.7	48	21.8	32.8	48	23.6	35.4	48	25.3	38.0
49	18.5	27.9	49	19.0	28.5	49	21.7	32.6	49	23.4	35.2	49	25.1	37.7
50	18.4	27.7	50	18.9	28.4	50	21.6	32.4	50	23.3	35.0	50	24.9	37.5
51	18.3	27.6	51	18.8	28.3	51	21.4	32.2	51	23.1	34.8	51	24.8	37.2
52	18.3	27.4	52	18.7	28.1	52	21.3	32.0	52	23.0	34.5	52	24.6	36.9
53	18.2	27.3	53	18.6	28.0	53	21.2	31.8	53	22.8	34.3	53	24.4	36.7
54	18.1	27.1	54	18.5	27.8	54	21.0	31.6	54	22.6	34.0	54	24.2	36.4
55	18.0	27.0	55	18.4	27.6	55	20.9	31.4	55	22.5	33.8	55	24.0	36.1
56	17.9	26.8	56	18.3	27.5	56	20.7	31.2	56	22.3	33.5	56	23.8	35.8
57	17.7	26.7	57	18.2	27.3	57	20.6	31.0	57	22.1	33.3	57	23.6	35.5
58	17.6	26.5	58	18.1	27.1	58	20.5	30.7	58	22.0	33.0	58	23.4	35.2
59	17.5	26.4	59	17.9	27.0	59	20.3	30.5	59	21.8	32.8	59	23.2	34.9
60	17.4	26.2	60	17.8	26.8	60	20.2	30.3	60	21.6	32.5	60	23.0	34.6

Efficient Use of Tables

Determine Strength of Beam Between Grids A/2 to B/2

- Given:
  - AISC *Specification* 360-10
  - AISC 14th Edition, LRFD
  - W18x35 ASTM A992, F_y = 50 ksi
  - L= 30 ft
  - Braced points only at beam-to-girder connections
  - Equally spaced bays
  - C_b = 1.0, conservative





Efficient Use of Tables

- From Table 3-2

  L_p = 4.31 ft

  L_r = 12.3 ft

$$\frac{y-y_0}{x-x_0}=\frac{y_1-y_0}{x_1-x_0}$$

$$\text{Solving for } y:\quad y=y_0+(y_1-y_0)\frac{x-x_0}{x_1-x_0}$$

$$\phi M_{nx}=\phi M_{px}+(\phi M_{rx}-\phi M_{px})\frac{L-L_p}{L_r-L_p}$$

$$=249\text{ kip-ft}+(151\text{ kip-ft}-249\text{ kip-ft})\left(\frac{10\text{ ft}-4.31\text{ ft}}{12.3\text{ ft}-4.31\text{ ft}}\right)=179\text{ kip-ft}$$

Z_x

Table 3-2 (continued)
W-Shapes
Selection by Z_x

F_y = 50 ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W18×35	**66.5**	**166**	**249**	**101**	**151**	**8.14**	**12.3**	**4.31**	**12.3**	**510**	**106**	**159**
W12×45	64.2	160	241	101	151	3.80	5.80	6.89	22.4	348	81.1	122
W16×36	64.0	160	240	98.7	148	6.24	9.36	5.37	15.2	448	93.8	141
W14×38	61.5	153	231	95.4	143	5.37	8.20	5.47	16.2	385	87.4	131
W10×49	60.4	151	227	95.4	143	2.46	3.71	8.97	31.6	272	68.0	102
W8×58	59.8	149	224	90.8	137	1.70	2.55	7.42	41.6	228	89.3	134
W12×40	57.0	142	214	89.9	135	3.66	5.54	6.85	21.1	307	70.2	105
W10×45	54.9	137	206	85.8	129	2.59	3.89	7.10	26.9	248	70.7	106
W14×34	**54.6**	**136**	**205**	**84.9**	**128**	**5.01**	**7.55**	**5.40**	**15.6**	**340**	**79.8**	**120**

Efficient Use of Tables

- Check if Compact for Flexure

Shape
W18×35
W12×45
W16×36
W14×38
W10×49
W8×58
W12×40
W10×45

Z_x

Table 3-2 (continued)
W-Shapes
Selection by Z_x

F_y = 50 ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.³	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.⁴	ASD	LRFD
W18×35	**66.5**	**166**	**249**	**101**	**151**	**8.14**	**12.3**	**4.31**	**12.3**	**510**	**106**	**159**
W12×45	64.2	160	241	101	151	3.80	5.80	6.89	22.4	348	81.1	122
W16×36	64.0	160	240	98.7	148	6.24	9.36	5.37	15.2	448	93.8	141
W14×38	61.5	153	231	95.4	143	5.37	8.20	5.47	16.2	385	87.4	131
W10×49	60.4	151	227	95.4	143	2.46	3.71	8.97	31.6	272	68.0	102
W8×58	59.8	149	224	90.8	137	1.70	2.55	7.42	41.6	228	89.3	134
W12×40	57.0	142	214	89.9	135	3.66	5.54	6.85	21.1	307	70.2	105
W10×45	54.9	137	206	85.8	129	2.59	3.89	7.10	26.9	248	70.7	106
W14×34	**54.6**	**136**	**205**	**84.9**	**128**	**5.01**	**7.55**	**5.40**	**15.6**	**340**	**79.8**	**120**
W16×31	**54.0**	**135**	**203**	**82.4**	**124**	**6.86**	**10.3**	**4.13**	**11.8**	**375**	**87.5**	**131**
W12×35	51.2	128	192	79.6	120	4.34	6.45	5.44	16.6	285	75.0	113
W8×48	49.0	122	184	75.4	113	1.67	2.55	7.35	35.2	184	68.0	102
W14×30	**47.3**	**118**	**177**	**73.4**	**110**	**4.63**	**6.95**	**5.26**	**14.9**	**291**	**74.5**	**112**
W10×39	46.8	117	176	73.5	111	2.53	3.78	6.99	24.2	209	62.5	93.7
W16×26[v]	**44.2**	**110**	**166**	**67.1**	**101**	**5.93**	**8.98**	**3.96**	**11.2**	**301**	**70.5**	**106**
W12×30	43.1	108	162	67.4	101	3.97	5.96	5.37	15.6	238	64.0	95.9
W14×26	**40.2**	**100**	**151**	**61.7**	**92.7**	**5.33**	**8.11**	**3.81**	**11.0**	**245**	**70.9**	**106**
W8×40	39.8	99.3	149	62.0	93.2	1.64	2.46	7.21	29.9	146	59.4	89.1
W10×33	38.8	96.8	146	61.1	91.9	2.39	3.62	6.85	21.8	171	56.4	84.7
W12×26	**37.2**	**92.8**	**140**	**58.3**	**87.7**	**3.61**	**5.46**	**5.33**	**14.9**	**204**	**56.1**	**84.2**
W10×30	36.6	91.3	137	56.6	85.1	3.08	4.61	4.84	16.1	170	63.0	94.5
W8×35	34.7	86.6	130	54.5	81.9	1.62	2.43	7.17	27.0	127	50.3	75.5
W14×22	**33.2**	**82.8**	**125**	**50.6**	**76.1**	**4.78**	**7.27**	**3.67**	**10.4**	**199**	**63.0**	**94.5**
W10×26	31.3	78.1	117	48.7	73.2	2.91	4.34	4.80	14.9	144	53.6	80.3
W8×31[f]	30.4	75.8	114	48.0	72.2	1.58	2.37	7.18	24.8	110	45.6	68.4
W12×22	**29.3**	**73.1**	**110**	**44.4**	**66.7**	**4.68**	**7.06**	**3.00**	**9.13**	**156**	**64.0**	**95.9**
W8×28	27.2	67.9	102	42.4	63.8	1.67	2.50	5.72	21.0	98.0	45.9	68.9
W10×22	**26.0**	**64.9**	**97.5**	**40.5**	**60.9**	**2.68**	**4.02**	**4.70**	**13.8**	**118**	**49.0**	**73.4**
W12×19	**24.7**	**61.6**	**92.6**	**37.2**	**55.9**	**4.27**	**6.43**	**2.90**	**8.61**	**130**	**57.3**	**86.0**
W8×24	23.1	57.6	86.6	36.5	54.9	1.60	2.40	5.69	18.9	82.7	38.9	58.3
W10×19	**21.6**	**53.9**	**81.0**	**32.8**	**49.4**	**3.18**	**4.76**	**3.09**	**9.73**	**96.3**	**51.0**	**76.5**
W8×21	20.4	50.9	76.5	31.8	47.8	1.85	2.77	4.45	14.8	75.3	41.4	62.1

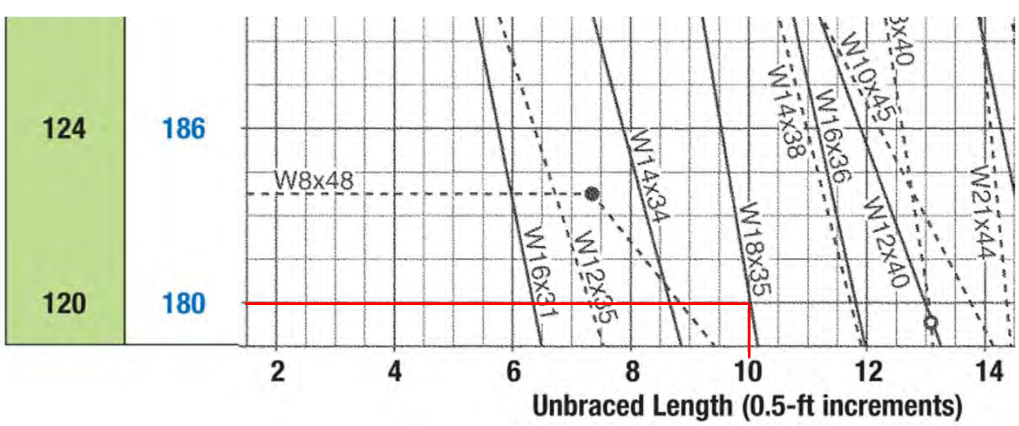
ASD	LRFD	
$\Omega_b = 1.67$	$\phi_b = 0.90$	[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.
		[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

ASD	LRFD	
$\Omega_b = 1.67$	$\phi_b = 0.90$	[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.
$\Omega_v = 1.50$	$\phi_v = 1.00$	[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

Secrets of the Manual

Efficient Use of Tables

- From Manual Table 3-2, C_b = 1 (See Table 3-1 for other C_b)



Secrets of the Manual

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Efficient Use of Tables

- From *Specification:*

F1. GENERAL PROVISIONS

The *design flexural strength*, $\phi_b M_n$, and the *allowable flexural strength*, M_n/Ω_b, shall be determined as follows:

(1) For all provisions in this chapter

$$\phi_b = 0.90 \text{ (LRFD)} \qquad \Omega_b = 1.67 \text{ (ASD)}$$

and the *nominal flexural strength*, M_n, shall be determined according to Sections F2 through F13.

(2) The provisions in this chapter are based on the assumption that points of support for *beams* and girders are restrained against rotation about their longitudinal axis.

(3) For singly symmetric members in *single curvature* and all doubly symmetric members:

C_b, the *lateral-torsional buckling* modification factor for nonuniform moment diagrams when both ends of the segment are braced is determined as follows:

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \qquad \text{(F1-1)}$$

Table 3-1

Lateral Bracing Along Span	C_b
None Load at midpoint	1.32
At load point	1.67 1.67
None Loads at third points	1.14

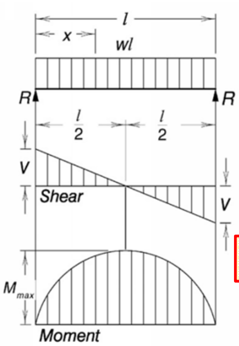
Efficient Use of Tables

- Deflection

Table 3-23

Shears, Moments and Deflections

1. SIMPLE BEAM — UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load .... $= wl$

$R = V$ .... $= \frac{wl}{2}$

V_x .... $= w\left(\frac{l}{2} - x\right)$

M_{max} (at center) .... $= \frac{wl^2}{8}$

M_x .... $= \frac{wx}{2}(l - x)$

Δ_{max} (at center) .... $= \frac{5wl^4}{384EI}$

Δ_x .... $= \frac{wx}{24EI}\left(l^3 - 2lx^2 + x^3\right)$

Efficient Use of Tables

- From *Specification:*

1. Yielding

$$M_n = M_p = F_y Z_x \quad \text{(F2-1)}$$

where

F_y = *specified minimum yield stress* of the type of steel being used, ksi (MPa)
Z_x = plastic section modulus about the x-axis, in.^{3} (mm^3)

2. Lateral-Torsional Buckling

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

$$M_n = C_b\left[M_p - \left(M_p - 0.7F_yS_x\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p \quad \text{(F2-2)}$$

(c) When $L_b > L_r$

$$M_n = F_{cr}S_x \le M_p \quad \text{(F2-3)}$$

where

L_b = length between points that are either braced against lateral displacement of the compression flange or braced against twist of the cross section, in. (mm)

$$F_{cr} = \frac{C_b\pi^2E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{Jc}{S_xh_o}\left(\frac{L_b}{r_{ts}}\right)^2} \quad \text{(F2-4)}$$

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Efficient Use of Tables

- From *Specification:*

The limiting lengths L_p and L_r are determined as follows:

$$L_p = 1.76r_y\sqrt{\frac{E}{F_y}} \quad \text{(F2-5)}$$

$$L_r = 1.95r_{ts}\frac{E}{0.7F_y}\sqrt{\frac{Jc}{S_xh_o} + \sqrt{\left(\frac{Jc}{S_xh_o}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}} \quad \text{(F2-6)}$$

2. Compression Flange Local Buckling

(a) For sections with noncompact flanges

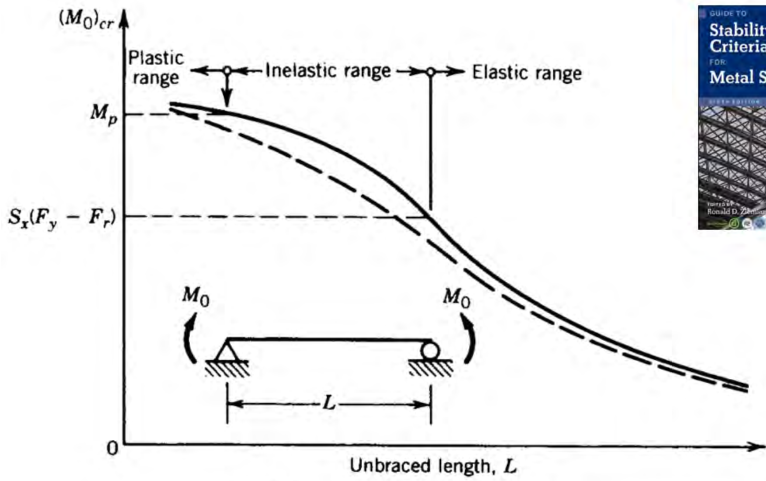
$$M_n = M_p - \left(M_p - 0.7F_yS_x\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \quad \text{(F3-1)}$$

(b) For sections with slender flanges

$$M_n = \frac{0.9Ek_cS_x}{\lambda^2} \quad \text{(F3-2)}$$

Efficient Use of Tables

- *Guide to Stability Design Criteria for Metal Structures,* Galambos



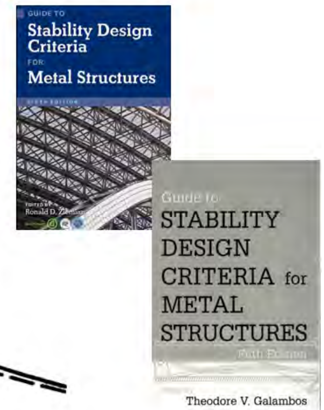


Fig. 5.1 Beam buckling curves.

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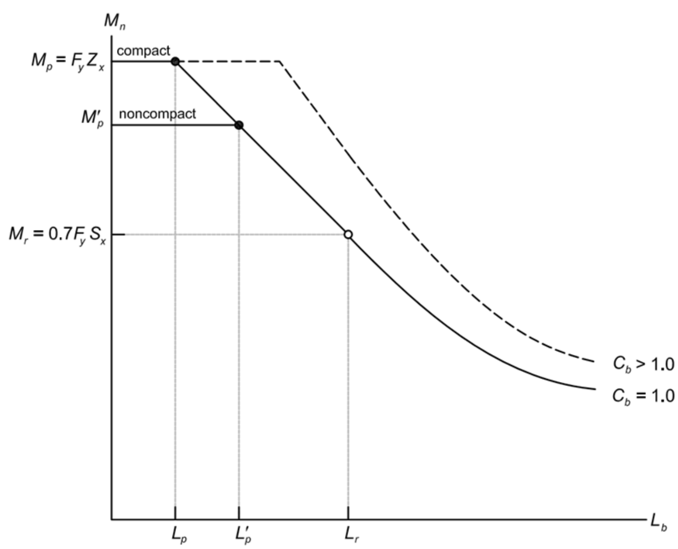
Efficient Use of Tables

- *Manual* Fig 3-1 (also see *Commentary* Figure C-F1.2)



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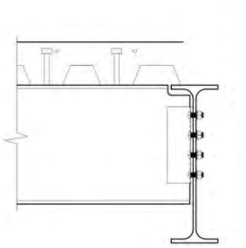
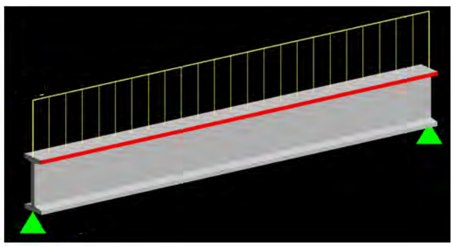
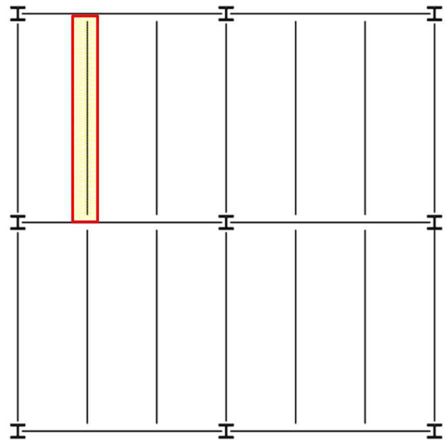
Efficient Use of Tables

Determine Strength of Composite Beam (shown)

- Given:
  - AISC *Specification* 360-10 & AISC 14th Edition, LRFD
  - W16x26 ASTM A992, F_y = 50 ksi, L = 30 ft
  - 3" NLWT concrete (f'_c = 4 ksi) on 3" metal deck
  - ¾" dia. stud







Secrets of the Manual 111

Efficient Use of Tables

- Determine Effective Width of Concrete Slab (*Specification* I3.1a)

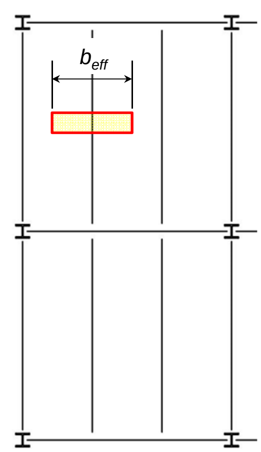
Each side of beam centerline, minimum of:

$$1)\ 1/8 \text{ of span} = \frac{L_b}{8} = \frac{(30\text{ ft})(12\text{ in/ft})}{8} = 45\text{ in., controls}$$

$$2)\ 1/2 \text{ c.c. beam spacing} = \frac{b_o}{2} = \left(\frac{1}{2}\right)\frac{(30\text{ ft})(12\text{ in/ft})}{(3\text{ spaces})} = 60\text{ in.}$$

3) distance to edge of slab = N/A

For this example, b_{eff} =(2 sides of beam centerline)(45 in) = 90 in.



Secrets of the Manual 112

Efficient Use of Tables

- Load Transfer Between Steel Beam and Concrete Slab (*Specification* I3.2d(1), Positive Flexural Strength where concrete slab is in compression)

(a) Concrete crushing (concrete fully in compression)

$$V'_c = 0.85 f'_c A_c = 0.85(4 \text{ ksi})(3 \text{ in})(90 \text{ in.}) = 918 \text{ kips}$$

(*Note*: only concrete above deck considered.)

(b) Tensile yielding of steel section (steel fully in tension)

$$V'_s = A_s F_y = (50 \text{ ksi})(7.68 \text{ in}^2) = 384 \text{ kips} < V'_c \quad \therefore \text{PNA in concrete}$$

(c) Shear strength of steel studs

$$V'_q = \sum Q_n = \min(V'_c, V'_s) \text{ for 100\% composite}$$

Secrets of the Manual 113

Efficient Use of Tables

- Stud Strength (*Specification* Eq. I8-1)

$$Q_n = 0.5 A_{sa} \sqrt{f'_c E_c} \le R_g R_p A_{sa} F_u$$

A_{sa} = area of stud, in^2

E_c = concrete modulus of elasticity = $w_c^{1.5} \sqrt{f'_c}$, ksi

F_u = specified minimum tensile strength of stud = 65 ksi

$$Q_n = 0.5\left(0.4418 \text{ in}^2\right)\sqrt{(4 \text{ ksi})\left[(145 \text{ pcf})^{1.5}\left(\sqrt{4 \text{ ksi}}\right)\right]}$$

$= 26.1$ kips/stud

$$R_g R_p A_{sa} F_u = 1.0(0.6)(0.4418 \text{ in}^2)(65 \text{ ksi})$$

$= 17.2$ kips/stud, controls

Condition	R_g	R_p
No decking	1.0	0.75
Decking oriented parallel to the steel shape		
$\frac{w_r}{h_r} \ge 1.5$	1.0	0.75
$\frac{w_r}{h_r} < 1.5$	0.85**	0.75
Decking oriented perpendicular to the steel shape Number of steel headed stud anchors occupying the same decking rib		
1	1.0	0.6+
2	0.85	0.6+
3 or more	0.7	0.6+

h_r = nominal rib height, in. (mm)
w_r = average width of concrete rib or haunch (as defined in Section I3.2c), in. (mm)
** for a single steel headed stud anchor
+ this value may be increased to 0.75 when $e_{mid\text{-}ht} \ge 2$ in. (51 mm)

Secrets of the Manual 114

Efficient Use of Tables

- Use Table 3-21 for Stud Strength

Table 3-21

Shear Stud Anchor Q_n

F_u = 65 ksi

Nominal Horizontal Shear Strength for One Steel Headed Stud Anchor, Q_n, kips

Deck condition		Stud anchor diameter, in.	Normal weight concrete w_c = 145 pcf, f'_c = 3 ksi	Normal weight concrete w_c = 145 pcf, f'_c = 4 ksi	Lightweight concrete w_c = 110 pcf, f'_c = 3 ksi	Lightweight concrete w_c = 110 pcf, f'_c = 4 ksi
R_p = 0.60	1	3/8	4.31	4.31	4.28	4.31
		1/2	7.66	7.66	7.60	7.66
		5/8	12.0	12.0	11.9	12.0
		$\boxed{3/4}$	17.2	$\boxed{17.2}$	17.1	17.2

Deck Perpendicular

Weak Studs per rib

(R_p = 0.60)

Efficient Use of Tables

- Determine Number of Studs

For 100% composite, studs required between points of maximum and zero bending moment (half span of beam):

$$\#\ \text{of studs} = \frac{\left[V'_q = \min(V'_c, V'_s)\right]}{Q_n} = \frac{384\ \text{kips}}{17.2\ \text{kips/stud}} = 22.3 \quad \therefore 46\ \text{studs for beam total length}$$

However, for 46 studs, there will be (2) studs per rib. For (2) studs per rib,

R_g =0.85 $\quad Q_n$ =14.6 kips/stud

bending moment (half span of beam):

$$\#\ \text{of studs} = \frac{\left[V'_q = \min(V'_c, V'_s)\right]}{Q_n} = \frac{384\ \text{kips}}{14.6\ \text{kips/stud}} = 26.3 \quad \therefore 54\ \text{studs for beam total length}$$

*See Design Example I.1 on page I-15 for more information

Efficient Use of Tables

- Determine 100% Composite Strength

$$a = \frac{\sum Q_n}{0.85 f'_c b_{eff}} = \frac{384 \text{ kips}}{0.85(4 \text{ ksi})(90 \text{ in.})} = 1.25 \text{ in.}$$

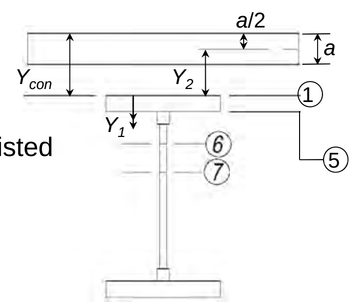
$$Y2 = (3 \text{ in. conc.} + 3 \text{ in. deck}) - \frac{1.25 \text{ in.}}{2} = 5.38 \text{ in.}$$

Y2 = Distance top of steel to beam to center line of the concrete flange force

- Sum moments about $a/2$:

$$\phi_b M_n = \phi A_s F_y \left(Y2 + \frac{d_{bm}}{2} \right) = 0.9 \left[\frac{384 \text{ kips}}{12 \text{ in./ft}} \left(5.38 \text{ in.} + \frac{15.7 \text{ in.}}{2} \right) \right] = 381 \text{ kip-ft}$$

Efficient Use of Tables



- For Use with Table 3-19 (see also Figure 3-3 in *Manual*)

  Y1 = distance from top of steel flange to any of the PNA locations listed

  PNA locations:

  TFL = Top of Flange (beam top flange), also 1

  BFL = Bottom of Flange (beam top flange), also 5

  2,3,4 = 4 equal spaces within flange

$$6 = \frac{\sum Q_n \text{ at BFL} + \sum Q_n \text{ at } 7}{2}$$

$$7 = 0.25 F_y A_s$$

$$Y2 = Y_{con} - \frac{a}{2} \qquad a = \frac{\sum Q_n}{0.85 f'_c b_{eff}}$$

and Y_{con} = top of steel to top of concrete

PNA[c]	$Y1$[a]	$\sum Q_n$	$Y2$[b], in.				
			2		2.5		
	in.	kip	ASD	LRFD	ASD	LRFD	ASD
TFL	0	384	189	284	198	298	208
2	0.0863	337	184	276	192	289	201
3	0.173	289	179	269	186	280	193
4	0.259	242	174	261	180	270	186
BFL	0.345	194	168	253	173	260	178
6	2.05	145	161	241	164	247	168
7	4.01	96.0	148	223	151	226	153

Efficient Use of Tables

- Table 3-19

From above,

$$V'_c = 918 \text{ kips}$$
$$V'_s = 384 \text{ kips}$$

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	$\sum Q_n$	$Y2$[b], in.													
	kip-ft					4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W16×26	110	166	TFL	0	384	227	341	237	356	246	370	256	384	265	399	275	413	285	428
			2	0.0863	337	218	327	226	340	234	352	243	365	251	377	259	390	268	403
			3	0.173	289	208	312	215	323	222	334	229	345	237	356	244	366	251	377
			4	0.259	242	198	297	204	306	210	315	216	324	222	333	228	343	234	352
			BFL	0.345	194	188	282	192	289	197	296	202	304	207	311	212	318	217	326
			6	2.05	145	175	263	179	268	182	274	186	279	189	285	193	290	197	296
			7	4.01	96.0	158	237	160	241	163	244	165	248	167	252	170	255	172	259

Enter Table with PNA at TFL and

$$\sum Q_n = 384 \text{ kips (100\% composite)}$$

$$a = \frac{\sum Q_n}{0.85 f'_c b_{eff}} = \frac{384 \text{ kips}}{0.85(4 \text{ ksi})(90 \text{ in.})} = 1.25 \text{ in.}$$

$$Y2 = (3 \text{ in. conc.} + 3 \text{ in. deck}) - \frac{1.25 \text{ in.}}{2} = 5.38 \text{ in.}$$

Efficient Use of Tables

- Moment Capacity based on Table 3-19 (100% composite)

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	$\sum Q_n$	$Y2$[b], in.													
	kip-ft					4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W16×26	110	166	TFL	0	384	227	341	237	356	246	370	256	384	265	399	275	413	285	428
			2	0.0863	337	218	327	226	340	234	352	243	365	251	377	259	390	268	403
			3	0.173	289	208	312	215	323	222	334	229	345	237	356	244	366	251	377
			4	0.259	242	198	297	204	306	210	315	216	324	222	333	228	343	234	352
			BFL	0.345	194	188	282	192	289	197	296	202	304	207	311	212	318	217	326
			6	2.05	145	175	263	179	268	182	274	186	279	189	285	193	290	197	296
			7	4.01	96.0	158	237	160	241	163	244	165	248	167	252	170	255	172	259

$$\phi_b M_n = 384 \text{ kft} - (384 \text{ kft} - 370 \text{ kft})\left(\frac{5.5 \text{ in.} - 5.38 \text{ in.}}{5.5 \text{ in.} - 5 \text{ in.}}\right) = 381 \text{ kip-ft}$$

- Compare to

$$\phi_b M_n = \phi A_s F_y \left(Y2 + \frac{d_{bm}}{2}\right) = 0.9\left[\frac{384 \text{ kips}}{12 \text{ in./ft}}\left(5.38 \text{ in.} + \frac{15.7 \text{ in.}}{2}\right)\right] = 381 \text{ kip-ft}$$

Efficient Use of Tables

- Using Table 3-19 for 63% composite

For 63% composite, enter Table with

$$\sum Q_{n\_63\%} = (0.63)(384 \text{ kips}) = 242 \text{ kips}$$

$$a = \frac{\sum Q_{n\_63\%}}{0.85 f'_c b_{eff}} = \frac{242 \text{ kips}}{0.85(4 \text{ ksi})(90 \text{ in.})} = 0.791 \text{ in.} \qquad Y2 = 6 \text{ in.} - \frac{0.791 \text{ in.}}{2} = 5.60 \text{ in.}$$

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	$\sum Q_n$	Y2[b], in.													
	kip-ft					4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W16×26	110	166	TFL	0	384	227	341	237	356	246	370	256	384	265	399	275	413	285	428
			2	0.0863	337	218	327	226	340	234	352	243	365	251	377	259	390	268	403
			3	0.173	289	208	312	215	323	222	334	229	345	237	356	244	366	251	377
			4	0.259	242	198	297	204	306	210	315	216	324	222	333	228	343	234	352
			BFL	0.345	194	188	282	192	289	197	296	202	304	207	311	212	318	217	326
			6	2.05	145	175	263	179	268	182	274	186	279	189	285	193	290	197	296
			7	4.01	96.0	158	237	160	241	163	244	165	248	167	252	170	255	172	259

Efficient Use of Tables

- Using Table 3-19 for 63% composite

For 63% composite, enter Table with

$$\sum Q_{n\_63\%} = (0.63)(384 \text{ kips}) = 242 \text{ kips}$$

Note: PNA at 4.

$$\phi_b M_{n\_63\%} = 333 \text{ kft} - (333 \text{ kft} - 324 \text{ kft})\left(\frac{6 \text{ in.} - 5.6 \text{ in.}}{6 \text{ in.} - 5.5 \text{ in.}}\right) = 326 \text{ kip-ft}$$

Efficient Use of Tables

- Determine Moment Capacity using 63% Composite
  From AISC *Commentary* Section C-I3.2a and Equation C-I3-10,

Sum Fx = 0
$C + A_{sc} F_y = P_y - A_{sc} F_y$

$\mathbf{A_{sc} F_y = (P_y - C)/(2)}$

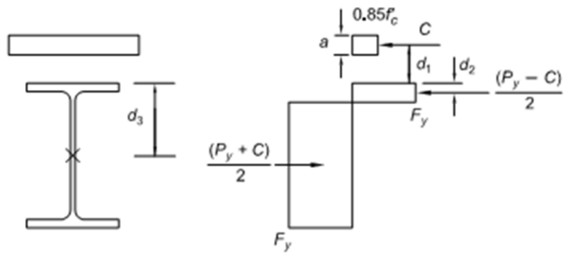


Fig. C-I3.3. Plastic stress distribution for positive moment in composite beams.

For partial composite action, there is a neutral axis in the concrete and a neutral axis in the steel.

Sum moments about $C_s/2$

$$M_n = C(d_1 + d_2) + P_y(d_3 - d_2) \qquad \text{AISC } \textit{Commentary} \text{ Eq. C-I3-10}$$

Efficient Use of Tables

- Determine Moment Capacity using 63% Composite (cont'd)

where:

$C = \sum Q_n$

$P_y = A_s F_y$

d_1 = distance from centroid of compression force, C, in concrete to top of steel, in.

d_2 = distance from centroid of the compression force in steel section to top of steel section, in. ($d_2 = 0$ for no compression in steel)

d_3 = distance from P_y to top of steel section, in.

Efficient Use of Tables

- Determine Moment Capacity using 63% Composite (cont'd)

$$P_y = 384\text{ kips} \qquad C_{63\%} = \sum Q_{n\_63\%} = 0.63(384\text{ kips}) = 242\text{ kips}$$

$$a = \frac{C_{63\%}}{0.85 f'_c b_{eff}} = \frac{242\text{ kips}}{0.85(4\text{ ksi})(90\text{ in.})} = 0.791\text{ in.}$$

$$d_1 = t_{slab} - \frac{a}{2} = 6\text{ in.} - \frac{0.791\text{ in.}}{2} = 5.60\text{ in.} \qquad d_3 = \frac{d_{bm}}{2} = \frac{15.7\text{ in.}}{2} = 7.85\text{ in.}$$

$$d_2 = \frac{x}{2} = \left(\frac{1}{2}\right)\left(\frac{A_s F_y - C_{63\%}}{2}\right)\left(\frac{1}{b_f F_y}\right) = \frac{(384\text{ kips} - 242\text{ kips})}{2(2)(5.50\text{ in.})(50\text{ ksi})} = 0.129\text{ in.}$$

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Efficient Use of Tables

- Determine Moment Capacity using 63% Composite (cont'd)

$$\begin{aligned} M_{n\_63\%} &= C_{63\%}(d_1 + d_2) + P_y(d_3 - d_2) \\ &= (242\text{ kips})(5.60\text{ in.} + 0.129\text{ in.}) + (384\text{ kips})(7.85\text{ in.} - 0.129\text{ in.}) \\ &= 4351\text{ k-in.} \end{aligned}$$

$$\phi M_{n\_63\%} = \frac{0.9(4351\text{ kip-in})}{12} = 326\text{ kip-ft}$$

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Efficient Use of Tables

- Determine Number of Studs

For 63% composite,

$$\boxed{\#\ \text{of studs} = \frac{0.63\left(384\ \text{kips}\right)}{17.2\ \text{kips/stud}} = 14.1 \quad \therefore 30\ \text{studs for beam total length}}$$

Note: Portion of steel beam will be compression.

Summary

- Be familiar with the information contained in the *Manual*
- Use the Database when possible
- Read descriptions of the tables
- Include all design information on details to facilitate review and checking
- *Manual* can facilitate efficient design.

Polling Question

PDH Certificates

Within 2 business days…

- You will receive an email on how to report attendance from: registration@aisc.org.
- Be on the lookout: Check your spam filter! Check your junk folder!
- Completely fill out online form. Don't forget to check the boxes next to each attendee's name!

PDH Certificates

Within 2 business days…

- Reporting site (URL will be provided in the forthcoming email).
- Username: Same as AISC website username.
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There's always a solution in steel.

Thank You

Please give us your feedback!
Survey at conclusion of webinar.

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