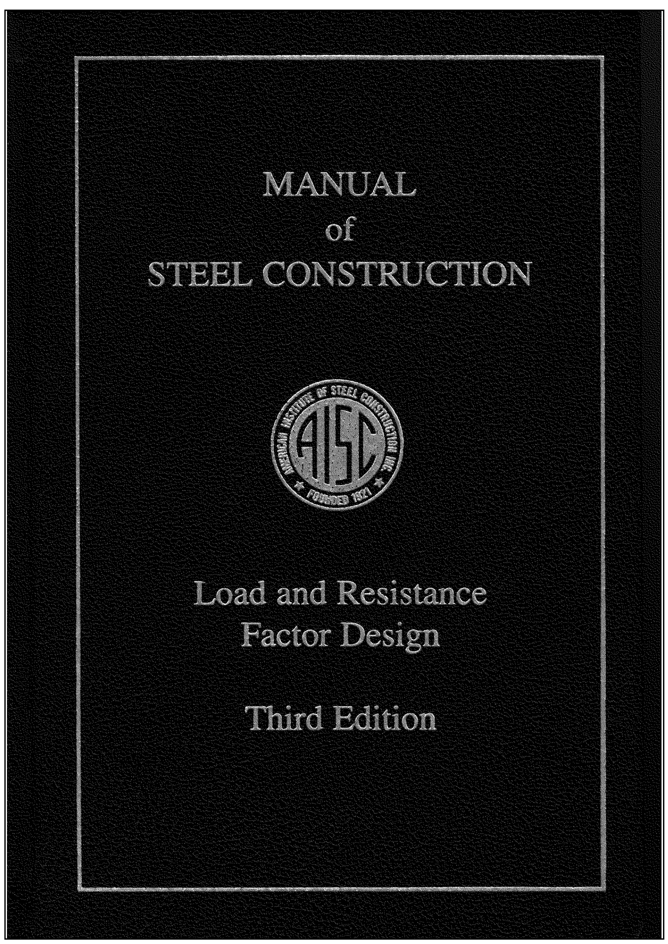
Revisions List, January 2003

Manual of Steel Construction
Load and Resistance Factor Design
3rd Edition

The following technical revisions and corrections have been made in the second printing of the Third Edition (January, 2003). To facilitate the incorporation of revisions and corrections, this booklet has been constructed using excerpts from revised pages, with corrections noted. The user may find it convenient in some cases to hand-write corrections; in others, a cut-and-paste approach may be more efficient.



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Printed in the United States of America

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REVISIONS

MANUAL OF STEEL CONSTRUCTION

LOAD AND RESISTANCE FACTOR DESIGN

Third Edition, First Printing

The follwing is a summary of revisions to the first printing of the *Manual of Steel Construction, Load and Resistance Factor Design, 3rd Edition*. All revisions have been incorporated into the second printing of the *Manual*.

Full-page replacements to the affected pages are available through the AISC website at www.aisc.org/revisions

	C4×7.25	2.13	4.00	4	0.321	5/16	3/16	1.72	1 3/4	0.296	5/16	3/4	2 1/2	1
	×5.4	1.58	↓	↓	0.184	3/16	1/8	1.58	1 5/8	↓	↓	↓	↓	–
Rev. 11/1/02	×4.5	$\boxed{1.32}$	↓	↓	0.125	1/8	1/16	1.58	1 5/8	↓	↓	↓	↓	–
	C3×6	1.76	3.00	3	0.356	3/8	3/16	1.60	1 5/8	0.273	1/4	11/16	1 5/8	–
	×5	1.47	↓	↓	0.258	1/4	1/8	1.50	1 1/2	↓	↓	↓	↓	–
	×4.1	1.20	↓	↓	0.170	3/16	1/8	1.41	1 3/8	↓	↓	↓	↓	–
Rev. 11/1/02	×3.5	$\boxed{1.03}$	↓	↓	0.132	1/8	1/16	1.37	1 3/8	↓	↓	↓	↓	–

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Rev. 11/1/02

L4×3 1/2×1/2	15/16	11.9	3.50	5.30	1.92	1.23	1.24	3.46	0.497
×3/8	13/16	9.10	2.68	4.15	1.48	1.25	1.20	2.66	0.433
×5/16	3/4	7.65	2.25	3.53	1.25	1.25	1.17	2.24	0.401
×1/4	11/16	6.18	1.82	2.89	1.01	1.26	1.14	1.81	0.368
L4×3×5/8	1 1/16	13.6	3.99	6.01	2.28	1.23	1.37	4.08	0.810
×1/2	15/16	11.1	3.25	5.02	1.87	1.24	1.32	3.36	0.747
×3/8	13/16	8.47	2.49	3.94	1.44	1.26	1.27	2.60	0.683
×5/16	3/4	7.12	2.09	3.36	1.22	1.27	1.25	2.19	0.651
×1/4	11/16	5.75	1.69	2.75	0.988	1.27	1.22	1.77	0.618

Page 1-151

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Camber	$\frac{1}{8}$ in.× $\frac{\text{(total length, ft)}}{5}$
Sweep	Due to the extreme variations in flexibility of these shapes, permitted variations for sweep are subject to negotiation between the manufacturer and purchaser for the individual sections involved.

Page 2-28, Under the heading *Washers*

larger than those for steel-to-steel structural bolting applications (see Table 14-2 for maximum anchor-rod hole sizes). Accordingly, washers used in such applications are generally larger and may require design consideration for proper force transfer, particularly when the anchorage is subject to tension. See Table 14-2 for anchor-rod washer sizes.

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Page 2-47

Concrete Framing	Interior and exterior spans of precast systems with cast-in-place joints resulting in restraint equivalent to that which would exist in [concrete framing] b(i)	restrained
	All types of prefabricated floor or roof systems where the structural members are secured to such systems and the potential thermal expansion of the floor or roof systems is resisted by the framing system or the adjoining floor or roof construction[b]	restrained

Rev. 11/1/02

Page 3-7, Given variables should include:

$$F_y = 36 \text{ ksi} \qquad A_g = 3.75 \text{ in.}^2 \qquad \boxed{r_z = 0.776 \text{ in.}}$$
$$F_u = 58 \text{ ksi} \qquad \bar{y} = 1.18$$

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Page 3-8

$$L_{\max} = 300\, r_z$$
$$= \frac{300\,(0.776 \text{ in.})}{12 \text{ in./ft}}$$
$$= 19.4 \text{ ft}$$

Rev. 11/1/02

Page 4-10

$$G_{\text{top}} = \tau \frac{\sum\left(\frac{l}{L}\right)_c}{\sum\left(\frac{l}{L}\right)_g}$$

$$\boxed{\begin{aligned} &= (1.00)\frac{\left(\dfrac{881\text{ in.}^4}{14\text{ ft}}\right)}{2\left(\dfrac{800\text{ in.}^4}{35\text{ ft}}\right)} \\ &= 1.38 \end{aligned}}$$

$$G_{\text{bottom}} = \tau \frac{\sum\left(\frac{l}{L}\right)_c}{\sum\left(\frac{l}{L}\right)_g}$$

$$\boxed{\begin{aligned} &= (1.00)\frac{2\left(\dfrac{881\text{ in.}^4}{14\text{ ft}}\right)}{2\left(\dfrac{1{,}360\text{ in.}^4}{35\text{ ft}}\right)} \\ &= 1.62 \end{aligned}}$$

Rev. 11/1/02

From LRFD Commentary Figure C-C2.2b, K ≈ 1.5.

$$(KL)_{y\,eq} = \frac{(KL)_x}{\frac{r_x}{r_y}}$$

$$= \frac{1.5(14\text{ ft})}{2.44}$$

$$= 8.61\text{ ft}$$

Page 4-11

From Table 4-2,

$$\phi_c P_n \approx \boxed{898\text{ kips}} > 250\text{ kips } \mathbf{o.k.}$$

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$$G_{\text{top}} = \tau \frac{\sum\left(\dfrac{l}{L}\right)_c}{\sum\left(\dfrac{l}{L}\right)_g}$$

$$\boxed{\begin{aligned} &= (0.85)\frac{2\left(\dfrac{881\text{ in.}^4}{14\text{ ft}}\right)}{2\left(\dfrac{1{,}360\text{ in.}^4}{35\text{ ft}}\right)} \\ &= 1.38 \end{aligned}}$$

$$G_{\text{bottom}} = 10\text{ (pinned end)}$$

From LRFD Commentary Figure C-C2.2b, K ≈ $\boxed{2.0.}$

Rev. 11/1/02

$$(KL)_{y\,eq} = \frac{(KL)_x}{\frac{r_x}{r_y}}$$

$$\boxed{\begin{aligned} &= \frac{2.0(14\text{ ft})}{2.44} \\ &= 11.48\text{ ft} \end{aligned}}$$

From Table 4-2,

$$\phi_c P_n \approx \boxed{814\text{ kips}} > 600\text{ kips } \mathbf{o.k.}$$

Thus, the W14×82 compression member is adequate.

Solution b: As determined in solution a, for the column segment between the roof and the floor,

$$\phi_c P_n \approx \boxed{898\text{ kips}}$$

As determined in solution a, for the column segment between the floor and the foundation, $G_{\text{top}} = \boxed{1.38}$ and

Page 4-11, Continued

From LRFD Commentary Figure C-C2.2b, K ≈ $\boxed{1.4.}$

Rev. 11/1/02

$$(KL)_{y\,eq} = \frac{(KL)_x}{\frac{r_x}{r_y}} = \boxed{\begin{array}{l}\dfrac{1.4(14\text{ ft})}{2.44}\\ 8.03\text{ ft}\end{array}}$$

Page 4-12

From Table 4-2,

$$\phi_c P_n \approx \boxed{913\text{ kips}}$$

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Pages 4-21 through 4-27

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I_x, in.^{4}	1380	1240	1110	999	881	795	722	640	541	484	428
I_y, in.^{4}	495	447	402	362	148	134	121	107	57.7	51.4	45.2
r_y, in.	3.74	3.73	3.71	3.70	2.48	2.48	2.46	2.45	1.92	1.91	1.89
Ratio r_x/r_y	1.67	1.67	1.66	1.66	2.44	2.44	2.44	2.44	3.07	3.06	3.08
$\boxed{P_{ex}(KL)^2/10^4}$	39500	35500	31800	28600	25200	22800	20700	18300	15500	13900	12300
$\boxed{P_{ey}(KL)^2/10^4}$	14200	12800	11500	10400	4240	3840	3460	3060	1650	1470	1290

†† For W14×99 and W14×90, flange is non-compact. For W14×43, web may be non-compact for combined axial compression and flexure; see AISC LRFD Specification Section B5.
Note: Heavy line indicates Kl/r equal to or greater than 200.

Pages 4-96 through 4-110, Table 4-12

The values presented in Table 4-12 are conservative, as they were calculated for a smaller concrete area than permitted. For values based on the full permitted concrete area, please download the replacement table at www.aisc.org/revisions

Pages 4-111 through 4-127, Table 4-13

The values presented in Table 4-13 are conservative, as they were calculated for a smaller concrete area than permitted. For values based on the full permitted concrete area, please download the replacement table at www.aisc.org/revisions

Page 5-13

From LRFD Specification Section F2, the shear yielding design strength $\phi_v V_n$ is

$$\phi_v V_n = \boxed{\begin{array}{l}\phi_v\, 0.6 F_y d t_w\\ 0.9(0.6)(50\text{ ksi})(17.9\text{ in.})(0.315\text{ in.})\\ 152\text{ kips}\end{array}}$$

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Page 5-14

Rev. 11/1/02

As determined in solution a, the shear yielding design strength $\phi_v V_n$ is

$$\phi_v V_n = \boxed{152\text{ kips}}$$

Page 5-15

For solution b, from Table 5-3, a W18×40 has $L_p = 4.49$ ft, $\phi_b M_{px} = 294$ kip-ft, $L_r = 12.0$ ft, $\phi_b M_{rx} = 205$ kip-ft and $BF = 11.7$ kips. From $\boxed{\text{solution b,}}$ $C_b = 1.46$ for the end span and 1.01 for the center span. Thus, for $L_b = 11.7$ ft,

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Page 5-18

From LRFD Specification Section F2, the shear yielding design strength $\phi_v V_n$ is

$$\phi_v V_n = \boxed{\begin{array}{l}\phi_v\, 0.6 F_y d t_w\\ 0.9(0.6)(50\text{ ksi})(20.6\text{ in.})(0.350\text{ in.})\\ 195\text{ kips}\end{array}}$$

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Page 5-20

As determined in solution a, the shear yielding design strength $\phi_v V_n$ is

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$$\phi_v V_n = \boxed{195 \text{ kips}}$$

Comments: The preceding calculations can be simplified using Tables 5-1, 5-3, 5-4 and 5-5. For solution a, from Table 5-3, a W21×48 is indicated as non-compact and provides $\phi_b M_n = 401$ kip-ft, $L'_p = 6.09$ ft, $\phi_b M_r = 279$ kip-ft and $L_r = 15.4$ ft. These values can then be used as illustrated in solution a to determine

$$\phi_b M'_n = 398 \text{ kip-ft}$$

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Also, $\phi_v V_n = \boxed{195 \text{ kips}}$ is given.

Page 5-21

From Table 5-2,

$$\phi_b M_{px} = 398 \text{ kip-ft} > 250 \text{ kip-ft } \mathbf{o.k.}$$
$$\phi_v V_n = \boxed{252 \text{ kips}} > 40 \text{ kips } \mathbf{o.k.}$$

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Thus, the W24×55 flexural member is **o.k.**

From Table 5-2,

$$\phi_v V_n = \boxed{118 \text{ kips}} > 20 \text{ kips } \mathbf{o.k.}$$

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Thus, the W16×31 flexural member is **o.k.**

Pages 5-40 and 5-41 and Pages 5-46 through 5-48

The values in Tables 5-2 and 5-3 on these pages have changed based on the note below. For full-page replacements to these tables, please visit www.aisc.org/revisions.

†† Indicates flange is non-compact.
* Tabulated value is $\phi_b M'_{px}$ to account for non-compact flange.
** Tabulated value is L'_p to account for non-compact flange.

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Beam Properties										
Z_x, in.^{3}	115	102	87.1	78.4	69.6	61.5	54.6	47.3	40.2	33.2
$\phi_b W_c$, kip-ft	3450	3060	2610	2350	2090	1830	1630	1410	1200	984
$\phi_v V_n$, kips	157	141	139	127	113	118	108	101	95.7	85.1

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Table 5-13.
Shear Stud Connectors
Unreduced Nominal Shear Strength Q_n, kips[a,b]

Specified Compressive Strength of Concrete f'_c, ksi	Light-Weight Concrete (115 lb/ft^3)				Normal-Weight Concrete (145 lb/ft^3)			
	Nominal Shear Stud Connector Diameter, in.				Nominal Shear Stud Connector Diameter, in.			
	$^1/_2$	$^5/_8$	$^3/_4$	$^7/_8$	$^1/_2$	$^5/_8$	$^3/_4$	$^7/_8$
3	7.86	12.3	17.7	24.1	9.35	14.6	21.0	28.6
3.5	8.82	13.8	19.8	27.0	10.5	16.4	23.6	32.1
4	9.75	15.2	21.9	29.9	11.6	18.1	26.1	35.5
4.5	10.7	16.6	24.0	32.6	11.8	18.4	26.5	36.1
5	11.5	18.0	25.9	35.3	11.8	18.4	26.5	36.1
Minimum Stud Length, in.	**2**	**2 $^1/_2$**	**3**	**3 $^1/_2$**	**2**	**2 $^1/_2$**	**3**	**3 $^1/_2$**

[a] Applicable only to concrete made with ASTM C33 aggregates.
[b] Values do not reflect the strength reduction required for shear connections embedded in a slab on a formed steel deck, outlined in Section I3 of the Specification.

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Pages 5-147 through 159

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[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.
[b] $\boxed{Y2}$ = distance from top of the steel beam to concrete flange force.
[c] See Figure 5-4c for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

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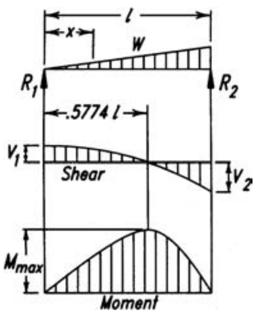
2. SIMPLE BEAM—LOAD INCREASING UNIFORMLY TO ONE END



Total Equiv. Uniform Load .... $= \frac{16W}{9\sqrt{3}} = 1.03W$

$R_1 = V_1$ .... $= \frac{W}{3}$

$R_2 = V_2 = V_{max}$ .... $= \frac{2W}{3}$

V_x .... $= \frac{W}{3} - \frac{Wx^2}{l^2}$

M_{max} (at $x = \frac{l}{\sqrt{3}} = .577l$) .... $= \frac{2Wl}{9\sqrt{3}} = .128Wl$

M_x .... $= \frac{Wx}{3l^2}(l^2 - x^2)$

Δ_{max} (at $x = l\sqrt{1 - \sqrt{\frac{8}{15}}} = .519l$) .... $= \boxed{0.0130\,\frac{Wl^3}{EI}}$ — Rev. 11/1/02

Δ_x .... $= \frac{Wx}{180EIl^2}(3x^4 - 10l^2x^2 + 7l^4)$

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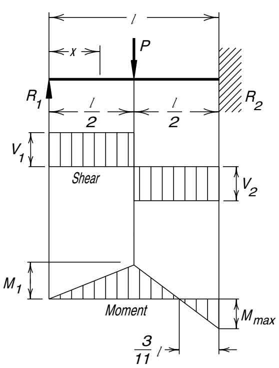


$R_1 = V_1$ .... $= \boxed{\frac{5P}{16}}$ — Rev. 11/1/02

$R_2 = V_{2\,max}$ .... $= \frac{11P}{16}$

M_{max} (at fixed end) .... $= \frac{3Pl}{16}$

M_1 (at point of load) .... $= \frac{5Pl}{32}$

M_x $\left(\text{when } x < \frac{l}{2}\right)$ .... $= \frac{5Px}{16}$

M_x $\left(\text{when } x > \frac{l}{2}\right)$ .... $= P\left(\frac{l}{2} - \frac{11x}{16}\right)$

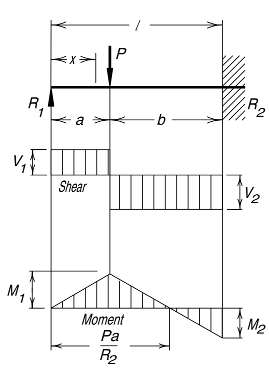
Δ_{max} $\left(\text{at } x = l\sqrt{\frac{1}{5}} = 0.447l\right)$ .... $= \frac{Pl^3}{48EI\sqrt{5}} = 0.00932\frac{Pl^3}{EI}$

Δ_x (at point of load) .... $= \boxed{\frac{7Pl^3}{768EI}}$ — Rev. 11/1/02

Δ_x $\left(\text{when } x < \frac{l}{2}\right)$ .... $= \frac{Px}{96EI}(3l^2 - 5x^2)$

Δ_x $\left(\text{when } x > \frac{l}{2}\right)$ .... $= \frac{P}{96EI}(x - l)^2(11x - 2l)$

14. BEAM FIXED AT ONE END, SUPPORTED AT OTHER—CONCENTRATED LOAD AT ANY POINT



$R_1 = V_1$ .... $= \frac{Pb^2}{2l^3}(a + 2l)$

$R_2 = V_2$ .... $= \frac{Pa}{2l^3}(3l^2 - a^2)$

M (at point of load) .... $= R_1 a$

M_2 (at fixed end) .... $= \frac{Pab}{2l^2}(a + l)$

M_x (when $x < a$) .... $= R_1 x$

M_x (when $x > a$) .... $= R_1 x - P(x - a)$

Δ_{max} $\left(\text{when } a < 0.414l \text{ at } x = l\frac{(l^2 + a^2)}{(3l^2 - a^2)}\right) = \frac{Pa(l^2 + a^2)^3}{3EI(3l^2 - a^2)^2}$

Δ_{max} $\left(\text{when } a > 0.414l \text{ at } x = l\sqrt{\frac{a}{2l + a}}\right)$ .. $= \frac{Pab^2}{6EI}\sqrt{\frac{a}{2l + a}}$

Δ_a (at point of load) .... $= \boxed{\frac{Pa^2b^3}{12EIl^3}(3l + a)}$ — Rev. 11/1/02

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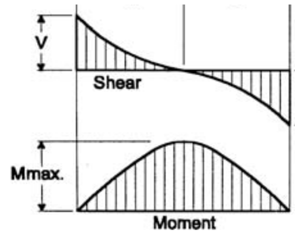

M_{max} (at center) .... $= \frac{\quad}{12}$

$M_x \quad \left(\text{when } x < \frac{l}{2}\right)$ .... $= \frac{W}{2}\left(x - \frac{2x^2}{l} + \frac{4x^3}{3l^2}\right)$

Δ_{max} (at center) .... $= \frac{3\,Wl^3}{320\,EI}$

$\Delta_x \quad \left(\text{when } x < \frac{l}{2}\right)$ .... $= \frac{W}{12\,EI}\left(x^3 - \frac{x^4}{l} + \boxed{\frac{2\,x^5}{5\,l^2}} - \frac{3\,l^2 x}{8}\right)$

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Page 6-6

EXAMPLE 6.2. W-Shape subject to combined axial compression and flexure (braced frame).

Given: Check the adequacy of an ASTM A992 W14×176 with $L_x = L_y = 14.0$ ft in a symmetric braced frame subject to the loading $P_u = 1{,}400$ kips, $M_{ux} = 200$ kip-ft, $M_{uy} = 70$ kip-ft. Assume reverse-curvature bending with equal end moments about both axes and no loads along the member.

$$F_y = 50\,\text{ksi} \qquad A = 51.8\,\text{in.}^2 \qquad r_x = 6.43\,\text{in.}$$
$$F_u = 65\,\text{ksi} \qquad Z_x = 320\,\text{in.}^3 \qquad r_y = 4.02\,\text{in.}$$
$$Z_y = 163\,\text{in.}^3 \qquad \boxed{I_x = 2{,}140\,\text{in.}^4}$$

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Page 6-7

For reverse curvature bending and equal end moments:

$$M_1/M_2 = +1.0$$
$$C_m = 0.6 - 0.4(1.0) = 0.2$$
$$\boxed{P_{e1} = \frac{\pi^2 EI}{(KL)^2}}$$

Rev.
11/1/02

Page 7-19, Table 7-1 - Correct the row label as follows:

Rev.
11/1/02

F436 Circular Washers[c]											
	Nom. Outside Diameter, *OD*		1 1/16	1 5/16	1 15/32	1 3/4	2	2 1/4	2 1/2	2 3/4	3
	Diameter, *ID*		17/32	11/16	13/16	15/16	1 1/8	1 1/4	1 3/8	1 1/2	1 5/8
	Thckns., *T*	**Min.**	0.097	0.122	0.122	0.136	0.136	0.136	0.136	0.136	0.136
		Max.	0.177	0.177	0.177	0.177	0.177	0.177	0.177	0.177	0.177
	Min. Edge Distance, *E*[d]		7/16	9/16	21/32	25/32	7/8	1	1 3/32	1 7/32	1 5/16

Page 7-35

Edge distance for full bearing strength $L_{e\,full}$[b], in.	**STD, SSLT, LSLT**	1 5/8	1 15/16	2 1/4	2 9/16	2 7/8	3 3/16	3 1/16	3 13/16
	OVS	1 11/16	2	2 5/16	2 5/8	3	3 5/16	3 5/8	3 15/16
	SSLP	1 11/16	2	2 5/16	2 11/16	3	3 5/16	3 5/8	3 15/16
	LSLP	2 1/16	2 7/16	2 7/8	3 1/4	3 11/16	4 1/16	4 1/2	4 7/8

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11/1/02

Page 7-37

[1] For design slip resistance using factored loads, refer to Table 7-15.
[2] For Class B faying surfaces, multiply the tabled design resistance by 1.52.
For Class C faying surfaces, multiply the tabled design resistance by 1.06.

Rev.
11/1/02

Page 9-3

Whitmore Section (Effective Width)

When connection elements are large in comparison to the bolted or welded joints within them, the Whitmore section may limit the gross and net areas of the connection element to less than the full area (Whitmore, 1952). As illustrated in Figure 9-1, the width of the Whitmore section is determined at the end of the joint by spreading the force from the start of the joint 30 degrees to each side in the connection element along the line of force. The Whitmore section may spread across the joint between connection elements (see Figure 13-11), but cannot spread beyond an unconnected edge.

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11/1/02

Page 9-8

$$
\begin{aligned}
k &= \text{plate buckling coefficient} \\
&= 2.2\left(\frac{h_o}{c}\right)^{1.65} \quad \text{when } \boxed{\frac{c}{h_o} \leq 1.0} \\
&= \frac{2.2h_o}{c} \quad \text{when } \boxed{\frac{c}{h_o} > 1.0}
\end{aligned}
$$

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Page 9-9

$$\boxed{\lambda = \frac{\sqrt{F_y}}{167}\frac{1}{\sqrt{K}}\frac{h_o}{2t_w}}$$

Rev. 11/1/02

Page 9-13

$$\boxed{d_{b\ \min} = 0.163\, t_f \sqrt{\frac{F_y}{b}\left(\frac{b^2}{L^2} + 2\right)}}$$

Rev. 11/1/02

Pages 9-39 through 9-43

Revise the ϕR_1 values in Table 9-5. Full-page replacements are available at www.aisc.org/revisions

Page 10-81

End Plate	Beam		1-in.-Diameter Bolts
$F_y = 36$ ksi	$F_y = 50$ ksi		12 Rows
$F_u = 58$ ksi	$F_u = 65$ ksi		W44

Table 10-4 (cont.).
Bolted/Welded Shear End-Plate Connections

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End Plate	Beam		1-in.-Diameter Bolts
$F_y = 36$ ksi	$F_y = 50$ ksi		10 Rows
$F_u = 58$ ksi	$F_u = 65$ ksi		W44,40, 36

Table 10-4 (cont.).
Bolted/Welded Shear End-Plate Connections

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$$
\begin{aligned}
N_{\min} &= \frac{R_u - \phi R_1}{\phi R_2} \geq k \\
&\boxed{\begin{aligned} &= \frac{55\text{ kips} - 48.9\text{ kips}}{19\text{ kips/in.}} \geq 1.03\text{ in.} \\ &= 0.321\text{ in.} \geq 1.03\text{ in.} \end{aligned}} \\
&= 1.03\text{ in.}
\end{aligned}
$$

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For web crippling,

When $\frac{N}{d} \leq 0.2$

$$
\begin{aligned}
N_{\min} &= \frac{R_u - \phi_r R_3}{\phi_r R_4} \\
&\boxed{= \frac{55\text{ kips} - 67.2\text{ kips}}{5.79\text{ kips/in.}}}
\end{aligned}
$$

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which results in a negative quantity.

When $\frac{N}{d} > 0.2$

$$
\begin{aligned}
N_{\min} &= \frac{R_u - \phi_r R_5}{\phi_r R_6} \\
&\boxed{= \frac{55\text{ kips} - 60.9\text{ kips}}{7.72\text{ kips/in.}}}
\end{aligned}
$$

which results in a negative quantity.

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For local web yielding,

$$N_{\min} = \frac{R_u - \phi R_1}{\phi R_2} \geq k$$
$$\boxed{\begin{aligned} &= \frac{55\text{ kips} - 56\text{ kips}}{20.0\text{ kips/in.}} \geq 1.12\text{ in.} \\ &= \text{-0.05 in.} \geq 1.12\text{ in.} \end{aligned}}$$
$$= 1.12\text{ in.}$$

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For web crippling,

When $\dfrac{N}{d} \leq 0.2$

$$N_{\min} = \frac{R_u - \phi_r R_3}{\phi_r R_4}$$
$$\boxed{= \frac{55\text{ kips} - 71.7\text{ kips}}{5.37\text{ kips/in.}}}$$

which results in a negative quantity.

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When $\dfrac{N}{d} > 0.2$

$$N_{\min} = \frac{R_u - \phi_r R_5}{\phi_r R_6}$$
$$\boxed{= \frac{55\text{ kips} - 64.2\text{ kips}}{7.02\text{ kips/in.}}}$$

which results in a negative quantity.

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Thus, $N_{\min} = 1.12$ in.

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For local web yielding, use constants ϕR_1 and ϕR_2 from Table 9-5. Rev. 11/1/02

$$W_{\min} = \frac{R_u - \phi R_1}{\phi R_2} + \text{ setback}$$
$$\boxed{\begin{aligned} &= \frac{125\text{ kips} - 64\text{ kips}}{21.5\text{ kips/in.}} + {}^3\!/_4\text{-in.} \\ &= 3.59\text{ in.} \end{aligned}}$$

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$$\boxed{t_{p\,\min} = \frac{L}{234}\sqrt{\frac{F_y}{K}} \geq {}^1\!/_4\text{ in.}}$$

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where K is the plate buckling coefficient tabulated in Part 9 for local buckling of beams coped at both the top and bottom flanges. To use the table in Part 9, calculate the plate aspect ratio as $\boxed{2a/L}$. Rev. 11/1/02

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Design Checks

The design strength of a single-angle connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8) and connection elements (see Part 9). In Rev. 11/1/02

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The angle of skew A appears in Figure 10-38a and represents the horizontal bevel to which the fittings must be bent or set, or the direction of gage lines on a seated connection.

Rev. 11/1/02 When the skew angle is less than 5° (1-in-12 slope), a pair of double angles can be bent inward or outward to make the connection as shown in Figure 10-39. While bent angle

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Determine required weld size for fillet welds to supporting column flange.

$$\boxed{\begin{aligned} D_{\min} &= \frac{P_{uf}}{1.5 \times 2 \times 1.392\, l} \\ &= \frac{144 \text{ kips}}{1.5 \times 2 \times 1.392(6\tfrac{1}{4} \text{ in.})} \\ &= 5.52 \rightarrow 6 \text{ sixteenths} \\ &\text{Use } \tfrac{3}{8}\text{-in. fillet welds.} \end{aligned}}$$

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Determine required weld size for fillet welds to supporting column flange

$$\boxed{\begin{aligned} D_{\min} &= \frac{P_{uf}}{1.5 \times 2 \times 1.392\, l} \\ &= \frac{144 \text{ kips}}{1.5 \times 2 \times 1.392(8\tfrac{3}{4} \text{ in.})} \\ &= 3.94 \rightarrow 4 \text{ sixteenths} \\ &\text{Use } \tfrac{1}{4}\text{-in. fillet welds.} \end{aligned}}$$

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Given: Design a welded flange-plated FR moment connection for a W18×50 beam to W14×99 column flange connection. For structural members, $F_y = 50$ ksi; for connecting material $F_y = 36$ ksi. Use 70 ksi electrodes and ASTM A325-N bolts.

$$\begin{aligned} R_u &= 45.0 \text{ kips} \\ M_u &= 250 \boxed{\text{ft-kips}} \end{aligned}$$

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From Figure $\boxed{\text{12-4,}}$ assume a shelf dimension of $\tfrac{5}{8}$ in. on both sides of the plate. The plate width, then, is 7.495 in.−2($\tfrac{5}{8}$ in.) = 6.245. Try a 1 in.×6$\tfrac{1}{4}$-in. flange plate.

Check tension yielding of the flange plate:

$$\begin{aligned} \phi R_n &= \phi F_y A_g \\ &= 0.9 \times 36 \text{ ksi} \times 6\tfrac{1}{4} \text{ in.} \times 1 \text{ in.} \\ &= 202.5 \text{ kips} \quad \textbf{o.k.} \end{aligned}$$

Determine required weld size and length for fillet welds to beam flange. Try a $\tfrac{5}{16}$ in. fillet weld. The minimum length of weld $l_{\min}$ is:

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$$\boxed{\begin{aligned} l_{\min} &= \frac{P_{uf}}{2\text{x}1.392 D} \\ &= \frac{167 \text{ kips}}{2\text{x}1.392(5 \text{ sixteenths})} \end{aligned}}$$
$$= 12.0 \text{ in.}$$

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Determine number of 1-in. diameter A325-SC bolts required for slip resistance. From Table $\boxed{\text{7-15}}$

$$\begin{aligned} n_{\min} &= \frac{R_u}{\phi r_n} \\ &= \frac{45 \text{ kips}}{19.0 \text{ kips/bolt}} \end{aligned}$$

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Determine required fillet weld for beam-web-to-end-plate connection.

From LRFD Specification Table J2.4, the minimum size is $^1/_4$ in. Determine size required to develop web flexural strength near tension bolts:

$$D_{\min} = \frac{0.9 F_y t_w}{2 \times 1.392}$$
$$= \frac{0.9 \times 50 \text{ ksi} \times 0.355 \text{ in.}}{2 \times 1.392}$$
$$= 5.74 \rightarrow 6 \text{ sixteenths}$$

Use $^3/_8$ in. fillet weld on both sides of the beam web from the inside face of the beam flange to the centerline of the inside bolt holes plus two bolt diameters.

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$$D_{\min} = \frac{R_u}{2 \times 1.392\, l}$$
$$= \frac{45 \text{ kips}}{2 \times 1.392(8.43 \text{ in.})}$$
$$= 1.92 \rightarrow 5 \text{ sixteenths (minimum size)}$$

Use $^1/_4$ in. fillet weld on both sides of the beam web below the tension-bolt region.

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Page 13-4

For the force distribution shown in the free-body diagrams of Figures 13-2b, 13-2c, and 13-2d to remain free of moments on the connection interfaces, the following expression must be satisfied.

$$\alpha - \beta \tan\theta = e_b \tan\theta - e_c \qquad \boxed{(13\text{-}1)}$$

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Analysis of Existing Diagonal Bracing Connections

A combination of α and β which provides for no moments on the three interfaces can usually be achieved when a connection is being designed. However, when analyzing an existing connection or when other constraints exist on gusset dimensions, the values of α and β may not satisfy Equation 13-1. When this happens, uniform interface forces will not satisfy equilibrium

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$H_{ub} = H_u$

$V_{ub} = V_u$

$\boxed{M_{ub} = V_{ub} \times (\bar{\alpha} - \alpha)}$

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(b) Gusset free-body diagram

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From LRFD Specification Sections B3.2,

$$U = 1 - \frac{\bar{x}}{l} \leq 0.9$$
$$= 1 - \frac{1.27 \text{ in.}}{15 \text{ in.}} \leq 0.9$$

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In the above equation, $\phi_c F_{cr}$ may be determined from $\frac{kl_1}{r}$ with LRFD Specification Table 3-36, where l_1 is the perpendicular distance from the Whitmore section to the interior corner of the gusset. Alternatively, the

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of $1.2D + 1.3W$. Refer to AISC (1992) for the unfactored loads and complete designs. For the given values of α and β, determine the interface forces on the gusset-to-column and gusset-to-beam connections for

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From Table $\boxed{9\text{-}5,}$

$$N_{req} = \frac{R_u - \phi R_1}{\phi R_2}$$

$$\boxed{\begin{aligned} &= \frac{85 \text{ kips} - 43.1 \text{ kips}}{17.8 \text{ kips/in.}} \\ &= 2.35 \text{ in.} < 10 \text{ in.} \qquad \textbf{o.k.} \end{aligned}}$$

Check web crippling

$$\frac{N}{d} = \frac{10 \text{ in.}}{18.0 \text{ in.}} = 0.556$$

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Since $\frac{N}{d} > 0.2$, from Table $\boxed{9\text{-}5,}$

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For lifting devices, see Figure $\boxed{14\text{-}11.}$

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$\boxed{\text{Table 14-3 (cont.).}}$ Rev. 11/1/02

Typical Column splices

Case III:

All-bolted flange-plated and butt-plated column splices between columns with depth d_u nominally two inches less than depth d_l.

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

and Quenched-and-Tempered Alloy Structural Steel Plates for Bridges, ASTM A709/A709M

Quenched and Tempered Low-Alloy Structural Steel Plate with 70 ksi (485 MPa) Minimum Yield Strength to 4 in. (100 mm) Thick, ASTM A852/A852M

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Webs in combined flexural and axial compression	h/t_w	for $P_u/\phi_b P_y \le 0.125$ [c],[g] $3.76\sqrt{\frac{E}{F_y}}\left(1-\frac{2.75P_u}{\phi_b P_y}\right)$ for $P_u/\phi_b P_y > 0.125$ [c][g] $1.12\sqrt{\frac{E}{F_y}}\left(2.33-\frac{P_u}{\phi_b P_y}\right) \ge 1.49\sqrt{\frac{E}{F_y}}$	[h] $5.70\sqrt{\frac{E}{F_y}}\left(1-0.74\frac{P_u}{\boxed{\phi_b P_y}}\right)$

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[a] For hybrid beams, use the yield strength of the flange F_{yf} instead of F_y.
[b] Assumes net area of plate at widest hole.
[c] ~~Assumes an inelastic rotation capacity of 3 radians. For structures in zones of high seismicity, a greater rotation capacity may be required.~~
[d] For plastic design use $0.045E/F_y$.
[e] F_L = smaller of ($F_{yf} - F_r$) or F_{yw}, ksi (MPa)
F_r = compressive residual stress in flange
= 10 ksi (69 MPa) for rolled shapes
= 16.5 ksi (114 MPa) for welded shapes
[f] $k_c = \frac{4}{\sqrt{h/t_w}}$ and $0.35 \le k_c \le 0.763$
[g] For members with unequal flanges, use h_p instead of h when comparing to λ_p.
[h] For members with unequal flanges, see Appendix B5.1.

Assumes an inelastic ductility ratio (ratio of strain at fracture to strain at yield) of 3. When the seismic response modification factor R is taken greater than 3, a greater rotation capacity may be required.

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H2. UNSYMMETRIC MEMBERS AND MEMBERS UNDER TORSION AND COMBINED TORSION, FLEXURE, SHEAR, AND/OR AXIAL FORCE

The design strength, $\phi \boxed{F_n}$, of the member shall equal or exceed the required strength expressed in terms of the normal stress f_{un} or the shear stress f_{uv}, determined by elastic analysis for the factored loads:

(a) For the limit state of yielding under normal stress:

$$f_{un} \leq \phi \boxed{F_n} \qquad \text{(H2-1)}$$

$$\phi = 0.90$$

$$\boxed{F_n = F_y}$$

(b) For the limit state of yielding under shear stress:

$$f_{uv} \leq 0.6\phi \boxed{F_n} \qquad \text{(H2-2)}$$

$$\phi = 0.90$$

$$\boxed{F_n = F_y}$$

(c) For the limit state of buckling:

$$f_{un} \text{ or } f_{uv} \leq \phi_c \boxed{F_n}, \text{ as applicable} \qquad \text{(H2-3)}$$

$$\phi_c = 0.85$$

$$\boxed{F_n = F_{cr}}$$

Some constrained local yielding is permitted adjacent to areas which remain elastic.

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TABLE J2.2
Effective Throat Thickness of Flare Groove Welds

Type of Weld	Radius (R) of Bar or Bend	Effective Throat Thickness
Flare bevel groove	All	$\frac{5}{16}R$
Flare V-groove	All	$\boxed{\frac{1}{2}R \text{ [a]}}$
[a] Use $\frac{3}{8}R$ for Gas Metal Arc Welding (except short circuiting transfer process) when $R \geq 1$ in. (25 mm)		

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TABLE J3.5
Nominal Tension Stress (F_t), ksi (MPa)
Fasteners in Bearing-type Connections

Description of Fasteners	Threads Included in the Shear Plane	Threads Excluded from the Shear Plane
A307 bolts	$59 - 2.5 f_v \leq 45$ $\boxed{(407 - 2.5 f_v \leq 310)}$	

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$$R_n = 6.0(F_y - 13)l\sqrt{d} / 20 \qquad \text{(J8-3)}$$

$$\boxed{\text{(Metric: } R_n = 30.2(F_y - 90)l\sqrt{d} / 20\text{)}} \qquad \text{(J8-3M)}$$

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when $b/t \boxed{\geq} 0.91\sqrt{E/F_y}$:

$$Q_s = 0.53E / \left[F_y (b/t)^2\right] \qquad \text{(A-B5-4)}$$

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(d) For stems of tees:

when $0.75\sqrt{E/F_y} < \boxed{d/t} < 1.03\sqrt{E/F_y}$:

$$Q_s = 1.908 - 1.22\boxed{(d/t)}\sqrt{F_y/E} \qquad \text{(A-B5-9)}$$

when $\boxed{d/t} \geq 1.03\sqrt{E/F_y}$:

$$Q_s = 0.69\boxed{E}/\left[F_y \boxed{(d/t)}^2\right] \qquad \text{(A-B5-10)}$$

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where

d = width of unstiffened compression element as defined in Section B5.1, in. (mm)

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NOTE: LTB applies only for strong axis bending.
[a] Excluding double angles and tees.
[b] Computed from fully plastic stress distribution for hybrid sections.

[c] $X_1 = \frac{\pi}{S_x}\sqrt{\frac{EGJA}{2}}$ $\boxed{X_2 = 4\frac{C_w}{I_y}\left(\frac{S_x}{GJ}\right)^2}$

[d] $\lambda_r = \frac{X_1}{F_L}\sqrt{1+\sqrt{1+X_2F_L^2}}$

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h_s = factor equal to $1.0 + \boxed{0.023\gamma}\sqrt{Ld_o/A_f}$

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Tension field action is not permitted for end-panels in non-hybrid plate girders, all panels in hybrid and web-tapered plate girders, and when a/h exceeds 3.0 or $[260/\boxed{(h/t_w)]^2}$. For these cases, the nominal strength is:

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For $P_u/P_y \leq \boxed{0.4,}$ $a = 0.06$, and $b = 1.0$;

For $P_u/P_y > \boxed{0.4,}$ $a = 0.15$, and $b = 2.0$:

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A490 bolts (A490M bolts)	$\sqrt{113^2 - 6.31f_v^2}$ $\boxed{(\sqrt{779^2 - 6.31f_v^2})}$	$\sqrt{113^2 - 4.04f_v^2}$ $\boxed{(\sqrt{779^2 - 4.04f_v^2})}$
Threaded parts A449 bolts over 1½ in. (38 mm)	$\sqrt{(0.75F_u)^2 - 6.25f_v^2}$	$\sqrt{(0.75F_u)^2 - 4.00f_v^2}$
A502 Gr. 1 rivets (Metric)	$\sqrt{45^2 - 5.76f_v^2}$ $(\sqrt{310^2 - 5.76f_v^2})$	
A502 Gr. 2 rivets (Metric)	$\sqrt{60^2 - 5.86f_v^2}$ $(\sqrt{414^2 - 5.86f_v^2})$	

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TABLE A-J3.2
Slip-Critical Resistance to Shear at Service Loads, F_v, ksi (MPa), of High-Strength Bolts[a]

	Resistance to Shear at Service Loads, ksi (MPa)		
		Oversized and	Long-slotted Holes

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F_v = ~~nominal~~ slip-critical shear resistance tabulated in Table A-J3.2, ksi (MPa). The values for F_v in Table A-J3.2 are based on Class A surfaces with slip coefficient $\mu = 0.33$. When specified by the designer, the ~~nominal~~ slip resistance for connections having special faying surface conditions is permitted to be adjusted to the applicable values in the RCSC Load and Resistance Factor Design Specification. Rev. 9/4/01

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For any combination of primary and secondary framing, the stress index is computed as

$$U_p = \left[\frac{F_y - f_o}{f_o}\right]_p \quad \text{for the primary member} \qquad \text{(A-K2-1)}$$

$$U_s = \left[\frac{F_y - f_o}{f_o}\right]_s \quad \text{for the secondary member} \qquad \text{(A-K2-2)}$$

where

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$$\left(\text{Metric: } F_{SR} = \left(\frac{C_f \times \boxed{329}}{N}\right)^{0.333} \geq F_{TH}\right) \qquad \text{(A-K3.1M)}$$

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$$\boxed{\left(\text{Metric: } F_{SR} = R_{PJP}\left(\frac{14.4 \times 10^{11}}{N}\right)^{0.333}\right)} \qquad \text{(A-K3.3M)}$$

where:

R_{PJP} = reduction factor for reinforced or non-reinforced transverse partial-joint-penetration (PJP) joints. Use Category C if $R_{PJP} = 1.0$.

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$$= \left(\frac{0.65 - 0.59\left(\dfrac{2a}{t_p}\right) + 0.72\left(\dfrac{w}{t_p}\right)}{t_p^{0.167}}\right) \leq 1.0 \qquad \boxed{= \left(\frac{1.12 - 1.01\left(\dfrac{2a}{t_p}\right) + 1.24\left(\dfrac{w}{t_p}\right)}{t_p^{0.167}}\right) \leq 1.0 \text{ (Metric)}}$$

$2a$ = the length of the non-welded root face in the direction of the thickness of the tension-loaded plate, in. (mm)

w = the leg size of the reinforcing or contouring fillet, if any, in the direction of the thickness of the tension-loaded plate, in. (mm)

t_p = thickness of tension loaded plate, in. (mm)

Based upon crack initiation from the roots of a pair of transverse fillet welds on opposite sides of the tension loaded plate element the design stress range, F_{SR}, on the cross section at the toe of the welds shall be determined by Equation A-K3.4 or A-K3.4M, Category C″ as follows:

$$F_{SR} = R_{FIL}\left(\frac{44 \times 10^8}{N}\right)^{0.333} \qquad \text{(A-K3.4)}$$

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$$\boxed{\left(\text{Metric: } F_{SR} = R_{FIL}\left(\frac{14.4 \times 10^{11}}{N}\right)^{0.333}\right)} \qquad \text{(A-K3.4M)}$$

where

R_{FIL} = reduction factor for joints using a pair of transverse fillet welds only. Use Category C if $R_{FIL} = 1.0$.

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$$= \left(\frac{0.06 + 0.72(w/t_p)}{t_p^{0.167}}\right) \leq 1.0 \qquad \boxed{= \left(\frac{0.10 + 1.24(w/t_p)}{t_p^{0.167}}\right) \leq 1.0 \text{ (Metric)}}$$

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SECTION 7 – BASE METAL AT SHORT ATTACHMENTS[1]				
7.1 Base metal subject to longitudinal loading at details attached by fillet welds parallel or transverse to the direction of stress where the detail embodies no transition radius and with detail length in direction of stress, *a*, and attachment height normal to surface of member, *b*:				In the member at the end of the weld

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$$1.2D + 1.6L \qquad \text{(C-A4-4)}$$

$$1.2D + 0.5L + \boxed{1.6}W \qquad \text{(C-A4-5)}$$

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practical designs. If the column end is rigidly attached to a properly designed footing, G may be taken as 1.0. Smaller values may be used if justified by analysis.

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factors for the serviceability limit state in Equation H1-1a or H1-1b with $\boxed{M_{nx}} = S_x F_y$ and $\boxed{M_{ny}} = S_y F_y$. Appendix H3 also provides interaction equations for rectangular box-shaped beam-columns. These equations are taken from Zhou and Chen (1985). Rev. 11/1/02

H2. UNSYMMETRIC MEMBERS AND MEMBERS UNDER TORSION AND COMBINED TORSION, FLEXURE, SHEAR, AND/OR AXIAL FORCE

This section deals with types of cross sections and loadings not covered in Section H1, especially where torsion is a consideration. For such cases it is recommended to perform an elastic analysis based on the theoretical numerical methods available from the literature for the determination of the maximum normal and shear stresses, or for the elastic buckling stresses. In the buckling calculations an equivalent slenderness parameter is determined for use in Equation E2-2 or E2-3, as follows:

$$\lambda_e = \sqrt{F_y / F_e}$$

where F_e is the elastic buckling stress determined from a stability analysis. This procedure is similar to that of Appendix E3.

For the analysis of members with open sections under torsion refer to Seaburg and Carter (1997). Rev. 11/1/02

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I_{lb} = lower bound moment of inertia, in.^{3} (mm^3)

$\boxed{Y_{ENA} = [(A_s d_3 + (\Sigma Q_n/F_y)(2d_3 + d_1))/(A_s + (\Sigma Q_n/F_y))]}$ Rev. 11/1/02

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should not be less than 3*d*, to ensure maximum design strength in bearing. Plotting of numerous test results indicates that the critical bearing strength is directly proportional to the above defined distances up to a maximum value of 3*d*, above which no additional bearing strength is achieved (Kulak et al., 1987). ~~Table J3.7 lists the increments that must be added to adjust the spacing upward to compensate for an increase in hole dimension parallel to the line of force.~~ Section J3.10 gives the bearing strength criteria as a function of spacing. Rev. 9/4/01

Nomenclature, Page 3

F_y''' The theoretical maximum yield stress (ksi) based on the web depth-thickness ratio (h / t_w) above which the web of a column is considered a slender element (See LRFD Specification Table B5.1) Rev. 11/1/02

$$= \left(\frac{253}{h / t_w}\right)^2$$

Nomenclature, Page 7

Y1 Distance from top of steel beam to the plastic neutral axis, in. Errata 11/1/02

Y2 Distance from top of steel beam to the concrete flange force in a composite beam. in.

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