Practical Design of Bolted and Welded Steel Connections

ASCE Continuing Education Seminar

Presented by Alexander Newman, P.E.



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Agenda

- General connection issues
- Bolted connections
- Welded connections
- Simple shear connections
- Q & A



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General Connection Issues

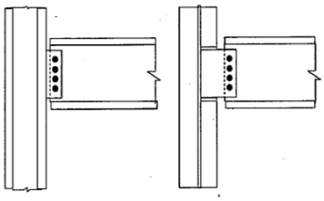
Standards: ANSI/AISC 360-10 and -05 and Manual

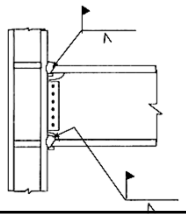
- ANSI/AISC 360-10 and -05, ***Specification for Structural Steel Buildings,*** integrated treatment of ASD and LRFD
- AISCM: Two sets of capacities for beams and columns: green for ASD, blue for LRFD load comb. of ASCE 7-10 and -05
- Others listed below

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General Connection Issues

- **Types of Connections per AISC 360 Chapter B**
  - **Simple**
    - **Flexible, can be designed for reaction shear only**
    - **Accommodate end rotations of simple beams, "some inelastic, but self-limiting deformation in the connection" OK**
  - **Moment**
    - **Transmit M across connection (…)**





FEMA

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General Connection Issues

- **Minimum Connection Capacity**
  - **6 kips, right?**
    - **No, this was removed in AISC 360-05**
    - **Now, *suggested* capacity is 6 kips**
    - **But OSHA req's 2 bolts in beams before releasing lines…**



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Bolted Connections



❑ Related Specifications

- AISC 360 Chapter J
- Research Council on Structural Connections (RCSC) 2004 ***Specification for Structural Joints using ASTM A325 or A490 Bolts*** and 2009 ***Specification for Structural Joints Using High-Strength Bolts**** (Design, installation, inspection)
- AISC Design Guide 17 – ***High Strength Bolts: A Primer for Structural Engineers***
- RCSC's ***Guide to Design Criteria for Bolted and Riveted Joints****

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*free at www.boltcouncil.org

Bolted Connections



❑ Structural Bolts

- ASTM A 325
- ASTM A 490
- "Alternative" bolts: Twist-off tension-controlled (TC) bolts
  - ASTM F1852 (strength ~ A 325 bolts)
  - ASTM F2280 (strength ~ A 490 bolts)
- Metric bolts: No exact conversion to imperial
  - E.g., 1" dia is 25.4 mm…closest is M24 (24 mm) bolt; but a hole made for 1" bolt is oversized for M24 => need SC joint! Next size, M27, would barely fit => impractical

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Bolted Connections

❑ AISC 360-10 Added 2 Groups for Bolts

- **Group A: ASTM A 325, A 325M, F 1852, A 354 Gr. BC, A 449**
- **Group B: ASTM A 490, A490M, F2280, A 354 Gr. BD**
- **Provisions apply to these groups, not to specific ASTM standards.**

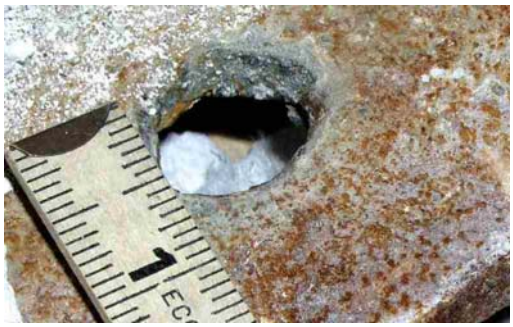


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Bolted Connections

❑ Making Holes for Bolts

- **AISC 360 Sec. M2.5 allows using any method (can punch, drill, flame cut, plasma cut) that results surface roughness ≤ 1000 microinch**



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Bolted Connections

❑ Slip-Critical (SC) vs. Bearing Joints

- **Fabrication & installation of SC joints may cost 3 times more***
- **Sec. 4.3 of RCSC Spec. states where to use SC joints:**
  - **Subjected to fatigue with reversal of load direction**
  - **Oversized holes**
  - **Slotted holes (unless load perp. to slot)**
  - **Where bolt slip is detrimental to performance**

***Source: J. E. O'Leary and S. Zoruba, "High-Strength Bolting Made Easy," *Modern Steel Construction*, May 2006**

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Bolted Connections

❑ Slip-Critical Joints

- **Check SC joints for bearing and block shear as bearing…**
- **Light rust OK at faying surfaces; galvanizing must be hand-roughened (Sec. 3.2.2 Commentary to RCSC Spec.)**
- **Available strengths values changed in AISC 360-05***
- **Further changes in 2010**

*** Reasons? See T. Schlafly and L. Muir, "Slip Critical Bolts: New Available Strength Values," *Modern Steel Construction*, August 2006**

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Bolted Connections

Surface Preparation for Slip-Critical Joints

- Two classes of faying (contact) surface preparation
  - Class A (μ = 0.35): Unpainted clean mill scale surfaces
  - Class B (μ = 0.50): Unpainted blast-cleaned surfaces
- AISC 360 Tables 7-3 and 7-4: Available shear strengths for slip as serviceability (OK for most) or strength limit states
  - Assume Class A prep.
  - For Class B, multiply tabulated values by 1.43

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Bolted Connections

Bearing Connections: Snug-Tight or Pretensioned

Snug-Tight Installation

- When no joint tension or dynamic load is expected
- RCSC: A few impacts of an impact wrench or the full effort using ordinary spud wrench "to bring the plies into firm contact"
- Standard oversized hole (1/16") OK for bearing, if larger…
  - Treat as slip critical (and prepare surfaces as SC)
  - Ream bigger holes, use larger bolts (recheck connection)

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Bolted Connections

❑ Bolt Pretensioning

- Reason for: Minimize variation of forces in the bolts
- Typ., <u>not needed</u> for bearing connections, (except per AISC 360 Sec. J1.10, for certain column splices and connections when $H > 125'$; for crane and machinery supports, etc.)
- Typ., <u>is needed</u> for SC connections
- Pretensioning force $\geq$ 70% of T_u of bolt
- No need to add pretension force to design V or T on joint (Sec. 5.1 Commentary to RCSC Spec.)
- Pretension does not affect V or T strength

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Bolted Connections

❑ Acceptable Tensioning Methods

- Turn-of-the-nut
- Calibrated torque wrench
- Tension-control bolts
- Direct tension indicators



❑ Turn-of-the-Nut Method

- RCSC Table 8.2 shows bolt rotation turns past snug tight
- Tolerance given in footnotes…sometimes may be exceeded. If bolt doesn't fail during installation, considered OK

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Bolted Connections

- **Calibrated Torque Wrench**
  - **Some inspectors try to check bolts with it, but…**
  - **No recognized relationship between torque and pretension…variation up to 40% can exist, unless tested for each bolt lot, diameter, and fastener condition***
  - **Frequent recalibration required by RCSC**



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***Source: J. E. O'Leary and S. Zoruba, High-Strength Bolting Made Easy, *Modern Steel Construction*, May 2006**

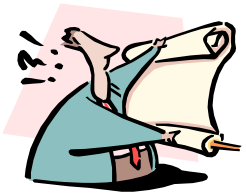
Bolted Connections

- **Are Washers Needed? (RCSC Spec. Sec. 6)**
  - **Not needed in snug-tight joints, except at sloped ones and…**
  - **Needed over slotted holes in outer plies: ASTM F 436 or 5/16" common plate. (Needed for stiffness to develop clamping force)**
  - **Table 6.1 shows more info.; e.g. for A 325 bolts:**
    - **ASTM F436 at oversized and short-slotted holes**
    - **5/16" common plate at long slots in outer ply**
    - **Can't stack thinner washers**

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Bolted Connections

- **Bolts in Seismic Load Resisting Systems**
  - **AISC-341 *Seismic Provisions*:**
    - **Design joints as pretensioned, prepare as slip-critical (Class A slip resistance)**
    - **But: Bolts need <u>not</u> be SC. Use strength in bearing if placed in standard holes (cannot prevent slip at ultimate load)**



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Bolted Connections

- **Galvanized Bolts (Plated and Hot-Dip)**
  - **Zinc does not act as lubricant => threads seizing common**
  - **Suitable lubricant should be added**
  - **If hot-dip (not for A490 bolts), threads need to be cut and chased after…nuts and bolts bought from the same supplier**
  - **Specify ASTM A153 for hot-dip galvanizing small parts (sim. to ASTM A123 for hot-dip galv. structural steel)**

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❑ Bolt Capacities for *T* and *V*

- See AISCM Table J3.2 for bolt nom. stresses (for ASD $\Omega = 2$)
- Capacities increased ~ 12.5% from 2005 to 2010, e.g.:
  - In 2005 A325N (threads included) $F_{nt} = 90$ ksi, $F_{nv} = 48$ ksi
  - In 2010 Group A (threads included) $F_{nt} = 90$ ksi, $F_{nv} = 54$ ksi
- AISCM Table 7-1, Available Shear, Table 7-2 avail. *T* strength:
  - In 2005 A325N (threads included, single shear plane) ¾" bolts, $V_{ASD} = 10.6$k and $T_{ASD} = 19.9$k
  - In 2010 Group A (threads included, single shear plane) ¾" bolts, $V_{ASD} = 11.9$k and $T_{ASD} = 19.9$k

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❑ Combined Stresses in Bolts, Bearing Connections

- AISC 360 Commentary to J3.7, for ASD:

  $(\Omega f_t/F_{nt})^2 + (\Omega f_v/F_{nv})^2 = 1$

  (elliptical relationship), or can be simplified as straight-line

  $(\Omega f_t/F_{nt}) + (\Omega f_v/F_{nv}) = 1.3$
- For ASD, A325N bolts, $\Omega = 2$, $F_{nt} = 90$ ksi, $F_{nv} = 48$ ksi, use:

  $(2f_t/90) + (2f_v/48) \leq 1.3$, or

  $(f_t/45) + (f_v/24) \leq 1.3$ ←

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❑ Combined Stresses in Bolts, Bearing Connections, Cont'd

- **No need to consider combined stress at all if (for ASD)**
  - ➢ $f_t \leq 0.2F_t$ **(or 0.2x45 = 9 ksi for A325N) or**
  - ➢ $f_v \leq 0.2F_v$ **(or 0.2x24 = 4.8 ksi for A325N in 2005 or 0.2x27 = 5.4 ksi for Group A in 2010)**

Where F_t and F_v are available stresses in tension and shear

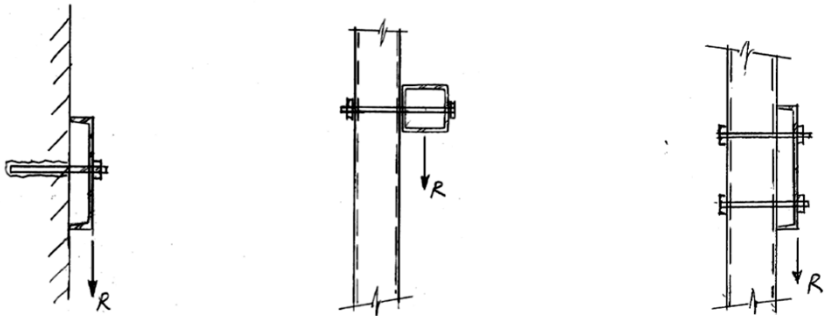


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❑ No AISC Equation for Bolt Bending!

- **Not even F_y given for HS bolts (see AISCM Table 2-5), only F_u = 120 ksi for A325 bolts from ½" to 1" dia.**
- **Clamping pretension is assumed to resist bending in steel-to-steel connections***

Source:** K. Gustafson, AISC, Answer in Steel Interchange, ***Modern Steel Construction, June 2006*



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❑ Design of Bolt Groups for Eccentric Loading

- See AISCM Tables 7-7 through 7-14

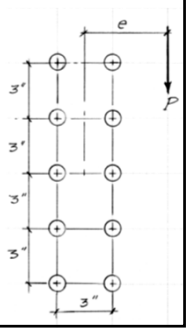
❑ Design Example 1: Eccentric Loading on Bolt Group, Using AISCM-05

- **Problem**: Determine the maximum load the following group can support, using ¾" A325N bolts (threads included), e_x = 12"
- **Solution**

From AISCM Table 7-1 available strength is:

ASD: $V_a = r_n/\Omega_v$ = 10.6 k

LRFD: $V_u = \Phi_v r_n$ = 15.9 k



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❑ Design Example 1, Cont'd

From AISCM Table 7-8

$R_n = C \times r_n$

for n = 5, s = 3" e_x = 12"

find C = 3.01

ASD: $R_n = P_a = C(r_n/\Omega_v)$ =
3.01 x 10.6 = 31.91 (k)

LRFD: $R_n = P_u = C(\Phi_v r_n)$ =
3.01 x 15.9 = 47.86 (k)

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Table 7–8
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 0°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$R_n = C \times r_n$

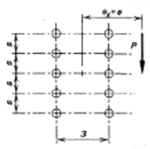
$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \frac{P_u}{\phi r_n}$	$C_{min} = \frac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)
e_x = horizontal component of e, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	0.84	2.54	4.48	6.59	8.72	10.8	12.9	15.0	17.0	19.0	21.0	23.0
	3	0.65	2.03	3.68	5.67	7.77	9.91	12.1	14.2	16.3	18.3	20.4	22.5
	4	0.54	1.67	3.06	4.86	6.84	8.93	11.1	13.2	15.4	17.5	19.6	21.7
	5	0.45	1.42	2.59	4.21	6.01	8.00	10.1	12.2	14.4	16.5	18.7	20.8
	6	0.39	1.22	2.25	3.69	5.32	7.17	9.16	11.2	13.4	15.5	17.7	19.8
	7	0.35	1.08	1.99	3.27	4.74	6.46	8.33	10.3	12.4	14.5	16.7	18.8
	8	0.31	0.96	1.78	2.93	4.27	5.86	7.60	9.50	11.5	13.6	15.7	17.8
	9	0.28	0.86	1.60	2.65	3.87	5.34	6.97	8.75	10.7	12.7	14.7	16.8
	10	0.26	0.78	1.46	2.42	3.53	4.90	6.42	8.10	9.91	11.8	13.8	15.9
	12	0.22	0.66	1.24	2.06	3.01	4.19	5.51	7.01	8.63	10.4	12.2	14.2
	14	0.19	0.57	1.08	1.78	2.62	3.[illegible]	4.82	6.15	7.61	9.19	10.9	12.7
	16	0.17	0.51	0.95	1.57	2.32	3.24	[illegible]27	5.47	6.79	8.23	9.78	11.4
	18	0.15	0.45	0.85	1.41	2.07	2.90	3.83	4.92	6.11	7.43	8.85	10.4
	20	0.14	0.41	0.77	1.27	1.88	2.63	3.48	4.47	5.55	6.76	8.07	9.48
	24	0.12	0.34	0.65	1.07	1.58	2.21	2.93	3.77	4.69	5.72	6.85	8.06
	28	0.10	0.29	0.56	0.92	1.36	1.90	2.53	3.25	4.05	4.95	5.93	7.00
	32	0.09	0.26	0.49	0.80	1.19	1.67	2.22	2.86	3.57	4.36	5.23	6.18
	36	0.08	0.23	0.43	0.72	1.06	1.49	1.98	2.55	3.18	3.90	4.67	5.52
	C'	2.94	8.33	15.8	26.0	38.7	54.2	72.2	93.1	117	143	172	204

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Welded Connections

❑ Minimum Fillet Weld Sizes in AISC 360-10,-05

- Table J2.4 of AISCM 9th ed. lists min. sizes based on <u>thicker</u> part joined, but 14th & 13th ed. base it on <u>thinner</u> part joined.
  - ➢ Reason: Past concern of small welds used in thick metal, but now low-hydrogen electrodes are common

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TABLE J2.4
Minimum Size of Fillet Welds

Material Thickness of Thinner Part Joined, in. (mm)	Minimum Size of Fillet Weld,[a] in. (mm)
To 1/4 (6) inclusive	1/8 (3)
Over 1/4 (6) to 1/2 (13)	3/16 (5)
Over 1/2 (13) to 3/4 (19)	1/4 (6)
Over 3/4 (19)	5/16 (8)

[a] Leg dimension of fillet welds. Single pass welds must be used.
Note: See Section J2.2b for maximum size of fillet welds.

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Welded Connections

❑ "Filler Metal" vs. "Weld Metal" – Same Thing?

- Basically... FM is the purchased product, WM is in final weld

❑ Min. Length of Intermittent Fillet Welds

- AISC 360-05, -10 Sec. J2.2*b*: *L*= 4x weld size, but <u>≥</u> 1.5"

❑ Strength of Fillet Welds (Changes in 2010)

- AISC 360-10 Sec. J2.4: Lower of base metal and weld
  - ➢ Nom. strength of base metal: $R_n = F_{nBM}A_{BM}$
  - ➢ Nom. strength of weld: $R_n = F_{nw}A_{we}$
  - ➢ Note "effective area of weld throat," A_{we}
  - ➢ F_{nBM} and F_{nw} are nominal stresses of base metal and weld

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❑ Strength of Fillet Welds, Cont'd

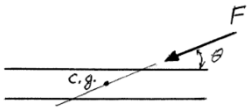
- **Alternatively, the available strength of fillet welds may be, using for ASD, $\Omega = 2$, for LRFD $\Phi = 0.75$:**
- **For linear weld group (all elements are in line or parallel) loaded in plane thru c.g. $R_n = F_{nw}A_{we}$**

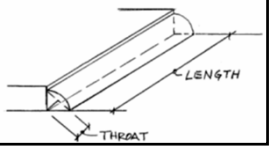
  $F_W = 0.60F_{EXX}(1 + 0.5 \sin^{1.5}\theta)$, where

  F_{EXX} = electrode class [tensile strength] (For E70xx, 70 ksi)

  θ = angle of loading measured from weld longit'l axis, deg.

  A_{we} = effective area of weld (throat x length)





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❑ Strength of Fillet Welds, Cont'd

- **E.g., for force *//* to welds ($\theta = 0^o$), ASD, ¼" weld 1" long allow. strength is 0.6x70x¼x0.707x1/2.0 = 3.71 k (same as before*)****
- **If force transverse to weld ($\theta = 90^o$), ASD, ¼" weld 1" long allow. strength is 0.6x70x(1 + 0.5sin$^{1.5}$90)¼x0.707x1/2.0 = 5.57 k**
  - **Transverse welds are 1.5 times stronger than parallel to *F*…**
  - **…but are less ductile**

*** In AISC ASD-89 for weld with F_u = 70 ksi: 0.3x70x0.707x1/4 = 3.71k**

**** Note that ASD allowable stress for E70 electrodes on effective throat is 0.6x70/2.0 = 21 (ksi)**

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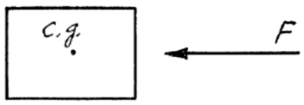
Welded Connections

❑ Strength of Fillet Welds, Cont'd

- **For fillet weld groups of same leg size w/some elements *//* and some transverse to load, take the combined strength as the <u>greater</u> of**

$$R_n = R_{nwl} + R_{nwt} \quad \textit{or}$$

$$R_n = 0.85R_{nwl} + 1.5R_{nw}$$



See Table J2.5 for determination of R_{nwl} and R_{nwt}

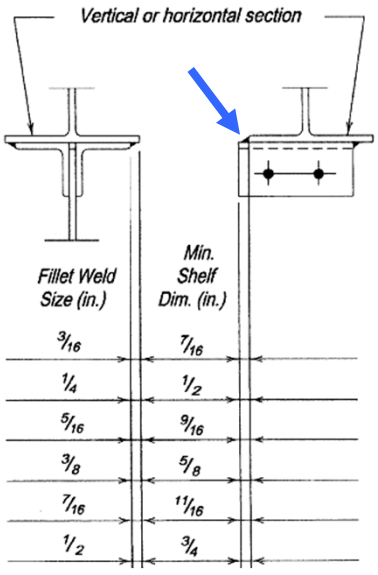
- **A group of various welds (fillet, plug, groove…) is a sum of capacities computed separately**

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Welded Connections

❑ Min. Shelf Dimensions

- **Recommended for SMAW fillet welds**



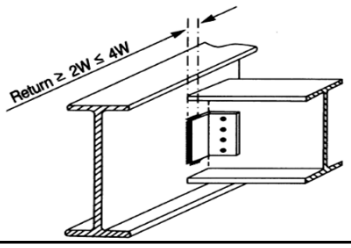
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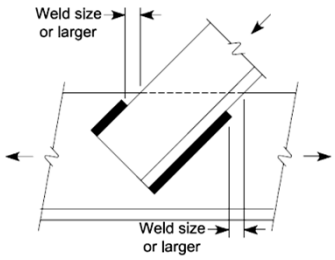
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Welded Connections

❑ Fillet Weld Terminations: AISC 360 Sec. J2.2*b*

- Can either stop short or extend to ends/sides or be boxed except if:
  - ➢ Tension in lap joints where one part extend past another, stop distance ≤ weld size from edge (helps avoid notching)
  - ➢ For conn. w/ flexibility of outstanding parts and end returns, return $L \leq 4x$ weld size and ≤ ½ width of part (…)





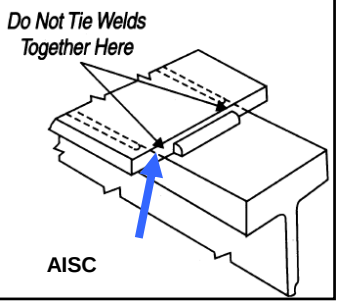
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Welded Connections

❑ Fillet Weld Terminations per Sec. J2.2*b*, Cont'd

- Welds for plate-girder web stiffeners (webs ≤ ¾") shall end ≥ $4t_w$ but ≤ $6t_w$ from flange-web toe, unless welded to flange (minor flange movement causes fatigue)
- Fillet welds on opp. sides of a common plane shall be interrupted at common corner (helps avoid a gouge)



But welds terminating at ends of joints (ex. plate-girder web stiff.) need not be fixed

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AISC

❑ Eccentrically Loaded Fillet Weld Groups

- Two approaches in AISCM:
  - ➢ Instantaneous center of rotation (more precise, but req's using tables)
  - ➢ Elastic method
- AISCM Tables 8-5 through 8-11: various config., load angles
- Most tables compute available strength of group as

  $R_n = CC_1DL$, where

  C = coeff. from tables

  C_1 = electrode strength coeff. (1.00 for E70, 0.857 for E60)

  D = weld size in 1/16" and L = length

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❑ Design Example 2: Eccentrically Loaded Weld Group (Same Tables in 2010 and 2005)

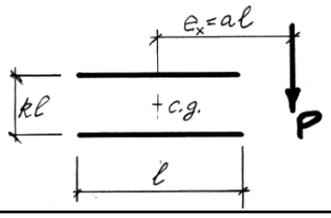
- <u>Problem</u>: Determine the maximum load the group shown will support if e_x = 12", L = 8", ¼" weld, E70 electrodes, welds spaced 4" apart.
- <u>Solution</u>

  Use AISCM Table 8-5. Compute k = 4/8 = 0.5 and a = 12/8 = 1.5

  Using L = 8", k = 0.5, C_1 = 1.00, D = 4

  find C interpolating

  between a = 1.4 and 1.6.



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Table 8–5 (continued)

Coefficients C for Eccentrically Loaded Weld Groups

Angle = 0°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with $R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

or

LRFD	ASD
$C_{min} = \frac{P_u}{\phi C_1 Dl}$ $D_{min} = \frac{P_u}{\phi C C_1 l}$ $l_{min} = \frac{\Omega P_u}{\phi C C_1 D}$	$C_{min} = \frac{\Omega P_a}{C_1 Dl}$ $D_{min} = \frac{\Omega P_a}{C C_1 l}$ $l_{min} = \frac{\Omega P_a}{C C_1 D}$

where

P = required force, P_u or P_a, kips

D = number of sixteenths-of-an-inch in the fillet weld size

l = characteristic length of weld group, in.

$a = e_x / l$

e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.

C = coefficient tabulated below

C_1 = electrode strength coefficient from Table 8-4 (1.0 for E70XX electrodes)



	k																
a	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	
0.00	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	
0.100	4.31	4.36	4.48	4.65	4.82	4.98	5.11	5.21	5.29	5.35	5.40	5.45	5.49	5.51	5.52	5.53	
0.150	3.90	3.94	4.04	4.20	4.39	4.58	4.75	4.90	5.02	5.12	5.20	5.31	5.38	5.43	5.46	5.48	
0.200	3.53	3.57	3.66	3.80	3.99	4.20	4.40	4.57	4.73	4.86	4.97	5.14	5.24	5.32	5.37	5.41	
0.250	3.21	3.25	3.34	3.47	3.64	3.84	4.06	4.26	4.43	4.59	4.72	4.93	5.08	5.19	5.27	5.32	
0.300	2.93	2.97	3.06	3.18	3.34	3.52	3.74	3.95	4.14	4.32	4.47	4.72	4.91	5.05	5.15	5.22	
0.400	2.48	2.51	2.59	2.71	2.85	3.01	3.19	3.40	3.61	3.81	3.99	4.29	4.54	4.73	4.87	4.99	
0.500	2.13	2.16	2.24	2.34	2.47	2.62	2.78	2.95	3.14	3.35	3.54	3.88	4.16	4.39	4.58	4.73	
0.600	1.86	1.89	1.96	2.05	2.17	2.31	2.45	2.61	2.77	2.95	3.15	3.50	3.81	4.06	4.28	4.46	
0.700	1.65	1.67	1.74	1.82	1.93	2.05	2.19	2.33	2.48	2.64	2.81	3.17	3.48	3.75	3.99	4.19	
0.800	1.48	1.50	1.56	1.64	1.74	1.85	1.97	2.10	2.24	2.38	2.53	2.87	3.19	3.46	3.71	3.93	
0.900	1.34	1.36	1.41	1.49	1.58	1.68	1.79	1.91	2.04	2.17	2.31	2.61	2.92	3.20	3.45	3.68	
1.00	1.22	1.24	1.29	1.36	1.44	1.54	1.64	1.75	1.87	1.99	2.12	2.39	2.69	2.97	3.22	3.45	
1.20	1.04	1.05	1.09	1.15	1.23	1.31	1.40	1.50	1.60	1.71	1.82	2.05	2.29	2.56	2.81	3.03	
1.40	0.899	0.913	0.951	1.00	1.07	1.14	1.22	1.31	1.40	1.49	1.59	1.79	2.00	2.24	2.47	2.69	
1.60	0.792	0.805	0.839	0.887	0.945	1.01	1.08	1.16	1.24	1.32	1.41	1.59	1.78	1.98	2.19	2.40	
1.80	0.708	0.721	0.751	0.793	0.847	0.907	0.972	1.04	1.11	1.19	1.27	1.43	1.59	1.77	1.96	2.16	
2.00	0.641	0.652	0.679	0.717	0.765	0.821	0.881	0.944	1.01	1.08	1.15	1.29	1.45	1.61	1.77	1.95	
2.20	0.585	0.595	0.620	0.656	0.699	0.749	0.805	0.863	0.924	0.987	1.05	1.19	1.33	1.47	1.62	1.78	
2.40	0.539	0.547	0.569	0.603	0.644	0.689	0.740	0.795	0.851	0.908	0.969	1.09	1.22	1.35	1.49	1.64	
2.60	0.499	0.505	0.528	0.559	0.596	0.639	0.685	0.736	0.789	0.843	0.899	1.01	1.13	1.25	1.38	1.51	
2.80	0.464	0.471	0.491	0.520	0.555	0.595	0.639	0.685	0.735	0.785	0.837	0.945	1.06	1.17	1.29	1.41	
3.00	0.433	0.440	0.459	0.485	0.519	0.556	0.597	0.641	0.688	0.736	0.784	0.885	0.989	1.10	1.21	1.32	

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Design Example 2, Cont'd

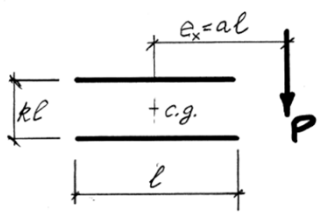
- C = (1.14 + 1.01)/2 = 1.075

  Available strength $R_n = CC_1DL$

  $\Omega = 2.00$, $\Phi = 0.75$

ASD: $R_n = P_a = CC_1DL/\Omega$ = (1.075 x 1 x 4 x 8)/2.0 = 17.2 (k)

LRFD: $R_n = P_u = \Phi CC_1DL$ = 0.75 x 1.075 x 1 x 4 x 8 = 25.8 (k)



❑ Out-of-Plane Weld Loading: AISCM Table 8-4

- **Tables for various *L*'s to vertical**
- **Similar tables in 2010 and 2005**

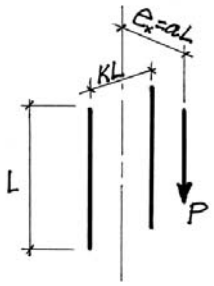
❑ Design Example 3: Out-of-Plane Loading

- **<u>Problem</u>: Determine the maximum load the group shown will support. Assume e_x = 12", *L* = 10", ¼" weld, E70 electrodes, welds spaced 5" apart; use AISCM 13th ed.**
- **<u>Solution</u>**

  Use AISCM Table 8-4.

  For *L* = 10" compute *a* = 12/10 = 1.2

  ***k* = 0 for out-of plane loading, C_1 = 1.00, *D* = 4**



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Available strength

$R_n = CC_1DL$

$\Omega = 2.00$,

$\Phi = 0.75$

Table 8–4

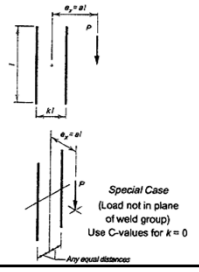
Coefficients *C* for Eccentrically Loaded Weld Groups

Angle = 0°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \frac{P_u}{\phi C_1 Dl}$ $D_{min} = \frac{P_u}{\phi C C_1 l}$ $l_{min} = \frac{\Omega P_u}{\phi C C_1 D}$	$C_{min} = \frac{\Omega P_a}{C_1 Dl}$ $D_{min} = \frac{\Omega P_a}{C C_1 l}$ $l_{min} = \frac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-4 (1.0 for E70XX electrodes)



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❑ Design Example 3, Cont'd

ASD: $R_n = P_a = CC_1DL/\Omega$ = (1.07 x 1 x 4 x 10)/2.0 = 21.4 (k)

LRFD: $R_n = P_u = \phi CC_1DL$ = 0.75 x 1.07 x 1 x 4 x 10 = 32.1 (k)

Use C = 1.07

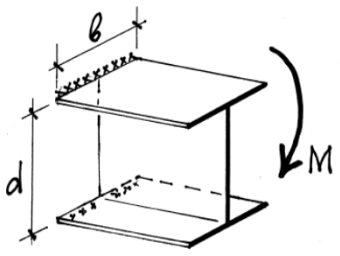
	k																
a	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.7
0.100	3.72	3.73	3.72	3.71	3.70	3.69	3.67	3.65	3.63	3.61	3.60	3.56	3.52	3.48	3.45	3.7
0.150	3.67	3.66	3.65	3.64	3.62	3.60	3.58	3.56	3.54	3.52	3.50	3.47	3.43	3.40	3.37	3.3
0.200	3.51	3.51	3.50	3.49	3.47	3.46	3.44	3.43	3.41	3.40	3.38	3.36	3.33	3.30	3.28	3.2
0.250	3.31	3.31	3.31	3.30	3.29	3.28	3.28	3.27	3.26	3.26	3.25	3.23	3.22	3.20	3.18	3.1
0.300	3.09	3.09	3.09	3.10	3.10	3.10	3.11	3.11	3.11	3.11	3.11	3.11	3.11	3.10	3.09	3.0
0.400	2.66	2.66	2.68	2.70	2.73	2.75	2.78	2.80	2.82	2.83	2.85	2.87	2.89	2.90	2.90	2.9
0.500	2.29	2.30	2.32	2.35	2.40	2.44	2.48	2.52	2.55	2.58	2.61	2.65	2.68	2.71	2.73	2.7
0.600	2.00	2.00	2.03	2.07	2.12	2.18	2.23	2.28	2.32	2.36	2.39	2.45	2.50	2.54	2.57	2.5
0.700	1.76	1.76	1.79	1.84	1.90	1.96	2.02	2.07	2.12	2.16	2.21	2.28	2.33	2.38	2.42	2.4
0.800	1.56	1.57	1.60	1.65	1.71	1.77	1.84	1.90	1.95	2.00	2.04	2.12	2.19	2.24	2.29	2.3
0.900	1.41	1.41	1.44	1.49	1.56	1.62	1.69	1.75	1.80	1.85	1.90	1.98	2.05	2.11	2.16	2.2
1.00	1.28	1.28	1.31	1.37	1.43	1.49	1.56	1.62	1.67	1.73	1.77	1.86	1.94	2.00	2.05	2.1
1.20	1.07	1.08	1.11	1.16	1.22	1.28	1.35	1.41	1.46	1.51	1.57	1.66	1.73	1.80	1.86	1.9
1.40	0.927	0.939	0.965	1.01	1.07	1.13	1.19	1.24	1.30	1.35	1.40	1.49	1.57	1.64	1.70	1.7
1.60	0.815	0.821	0.851	0.893	0.944	1.00	1.06	1.11	1.16	1.21	1.26	1.35	1.43	1.50	1.57	1.6
1.80	0.725	0.733	0.760	0.800	0.847	0.899	0.952	1.00	1.05	1.10	1.15	1.24	1.32	1.39	1.45	1.5
2.00	0.655	0.661	0.687	0.723	0.768	0.816	0.867	0.916	0.964	1.01	1.06	1.14	1.22	1.29	1.35	1.4
2.20	0.596	0.603	0.627	0.660	0.701	0.747	0.795	0.841	0.887	0.932	0.975	1.06	1.13	1.20	1.26	1.3
2.40	0.547	0.553	0.575	0.607	0.645	0.688	0.733	0.777	0.821	0.864	0.905	0.984	1.06	1.12	1.18	1.2
2.60	0.505	0.512	0.532	0.561	0.597	0.637	0.680	0.723	0.764	0.805	0.845	0.921	0.991	1.05	1.11	1.1
2.80	0.469	0.476	0.495	0.523	0.556	0.595	0.635	0.675	0.715	0.753	0.792	0.865	0.932	0.995	1.05	1.1
3.00	0.439	0.444	0.463	0.488	0.520	0.556	0.595	0.632	0.671	0.708	0.745	0.815	0.881	0.941	0.997	1.0

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❑ Another Approach: Treating Weld As a Line

- **Instead of computing <u>stress</u> in the weld $\sigma = M/S$ (psi), can compute <u>force</u> in the weld $f_b = M/S_w$ (lbs/in)**
- **Blodgett* and other sources provide tables for S_w for various weld configurations**
- **Find forces from bending and shear, add as vectors (shear is vertical, forces from M horizontal)**

$$f_r = \sqrt{f_b^{\,2} + f_v^{\,2}}$$



***Blodgett, Design of Welded Structures**
Lincoln Electric, Cleveland, OH, various editions

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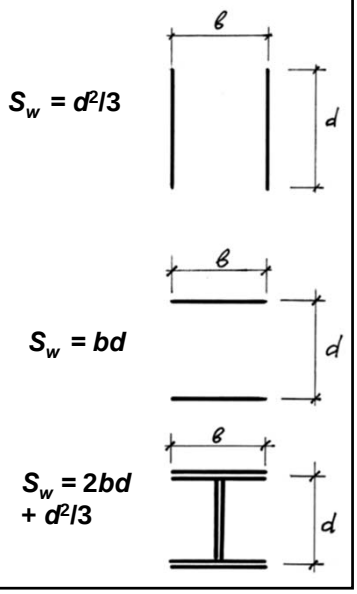
Welded Connections

- **Treating Weld As a Line, Cont'd**
  - Use weld capacities as above: ASD allowable weld stress, force *//* to welds is 21 ksi (on effective throat area)
  - E.g., allowable force on ¼" weld 1" long is 21x0.25x0.707 = 3.71 k

    (force transverse to weld, 5.57 k)

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Some Values of S_w



Welded Connections

- **Design Example 4: Treating Weld As a Line**
  - Compare results of Example 3 for b = 5", d = 10" P = 25.6k
  - <u>Solution</u>

    M = 25.6k x 12" = 307.2 k" $S_w = d^2/3 = 100/3 = 33.33$ (in^2)

    $f_b = M/S_w = 307.2/33.33 = 9.22$ (k)

    $f_v = P/2d = 25.6/(2x10) = 1.28$ (k)

    $f_r = \sqrt{f_b^2 + f_v^2} = \sqrt{9.22^2 + 1.28^2} = 9.31$(k)

    The min. effective throat size is 9.31/21 = 0.44" and min. weld size = 0.44/0.707 = 0.626 ~ 5/8"

    => AISCM table method is more accurate…

    …but tables cannot cover all possible situations.

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Welded Connections

- **Tips for Better Welding**
  - **Surface preparation, low-H**
  - **Weld lengthwise, avoid transverse welding and notches**
  - **Weld in flat or horizontal position (e.g., cover plate)**
  - **Intermittent fillet vs. groove**
  - **Ease transitions**
  - **Less expensive: smaller but longer welds**

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A great source: David T. Ricker, "Field Welding to Existing Steel Structures," *AISC Engineering Journal*, V. 25, No. 4 (1988)

Welded Connections

- **Tips for Better Welding, Cont'd**
  - **Preheat thick sections to reduce weld cracking**
    - **Slows down cooling, allows H to escape**
  - **See AWS D1.1 Table 3.2, "Minimum Prequalified Preheat and Interpass Temperature" (depends on grade, welding process, thickness)**
  - **AWS D1.1 Section 3.5.1: For joints made of materials with different preheat temps use the highest min. preheat**

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Welded Connections

❑ Tips for Better Welding, Cont'd

- For CJP welds stressed in tension perp. to weld area, use matching filler metal (AWS D1.1). Others may use ≤ matching
- Tension splices in heavy sections (Groups 4, 5; plates > 2" thk) & CJP joints in tension req. CVN material impact testing, weld access hole geometry, grinding
- Surface prep.: Grind off oil, paint, grease, rust

Source: David T. Ricker, "Field Welding to Existing Steel Structures" and AISC Design Guide 12

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Welded Connections

❑ Welding in Winter?



- Not below 0°F!
- For 0 < T < 32°F, preheat to & maintain ≥ 70°F
- No welding on wet or frost-covered surfaces; protect from wind and moisture
- Post-heating may be needed to slow down cooling, allow gases to escape

Source: David T. Ricker, "Field Welding to Existing Steel Structures" and AISC Design Guide 12

❑ Welding Through Coatings?

- Yes, if they "do not adversely affect weld quality nor create objectionable fumes." (AISC 360-05, -10 Comm. Sec. M3.5)
  - ➢ E.g., galvanizing… See AWS D19.0 on welding to galv. steel

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Welded Connections

❑ One-Sided Fillet Welds?

- **Common in MBS**
- **AISCM: Can result in severe notch from unwelded side if lateral deformation is not otherwise prevented. (Same for PJP welds.)**



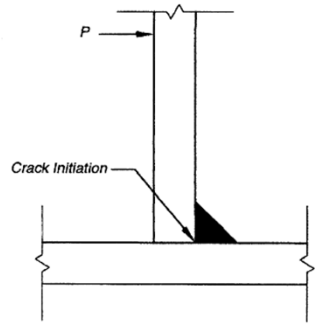


Figure 8–8. Notch effect at one-sided weld.

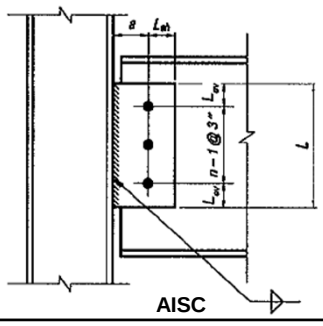
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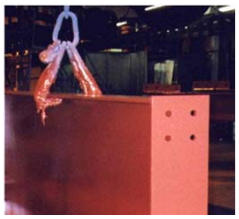
Simple Shear Connections

❑ Common Simple Connections

- **Framing (*L*, double *L*, shear plate)**
- **Seated (stiffened and not)**
- **End-plate (partial depth of beam – otherwise moment)**







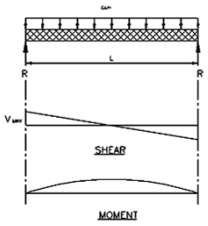
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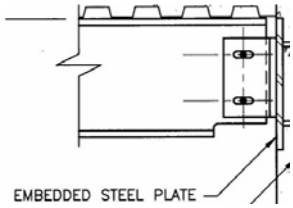
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Simple Shear Connections

❑ Single-Plate Connections

- **Most common, difficult to model (inelastic deformations thru "bolt plowing" into hole edges)… new procedure in 13th ed.**
- **Tabulated values for welded to support, bolted to beam AISCM 14th ed. Table 10-10 (AISCM 13th ed. Table 10-9), with weld sizes = 5/8t_p (was 3/4t_p before). Can use for 2.5" ≤ *a* ≤ 3.0"**



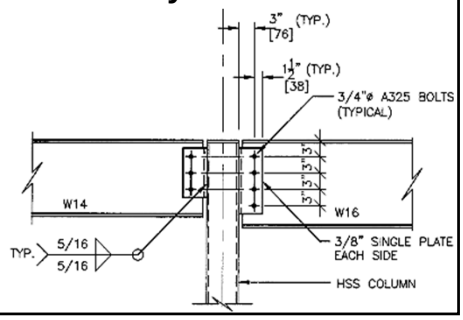


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Simple Shear Connections

❑ Assuring Ductility in Single-Plate Connections

- **To enable bolt plowing:**
  - **Either plate or web must be thin to deform before bolts fail**
  - **Achieved by limiting edge distance L_{eh} ≥ 2 bolt dia. and weld sizes = 5/8t_p**
  - **This tends to relieve connection eccentricity…**

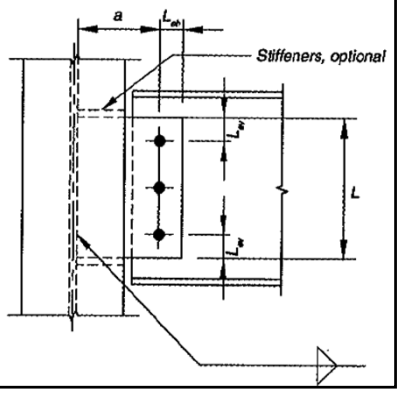


Source: Christopher M. Hewitt, Simpler Shear Connections, *Modern Steel Construction*, October 2006

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Simple Shear Connections

- **Single-Plate Connections, Cont'd**
  - **Use new "extended configuration" procedure (Fig. 10-12) if *a* > 3" and for cases not tabulated**



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Simple Shear Connections

- **Tabulated Capacity of Single-Plate Connections**
  - **Support rigidity does not affect results in AISCM 14th &13th ed.**
  - **Connection capacities change between editions**
  - **E.g., for (3) ¾" A325 bolts in 3/8" single plate threads included, single shear plane, the tabulated values of available strengths are:**
  - **9th ed.: 16.3k.**
  - **13th ed.: 31.8 k.**
  - **14th ed.: 28.8 k.**

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Simple Shear Connections

- **Tabulated Capacity of Single-Plate Connections, Cont'd**
  - **Example page from 13th ed.**

Table 10–9a (continued)

Single-Plate Connections

Plate F_y = 36 ksi

Bolt, Weld, and Single-Plate Available Strengths, kips

3/4-in. diameter bolts

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	7/16 ASD	7/16 LRFD	1/2 ASD	1/2 LRFD	9/16 ASD	9/16 LRFD
4 (L = 11 1/2)	A325 F1852	N	STD	34.8	52.2	42.4	63.6	42.4	63.6	42.4	63.6	—	—	—	—
			SSLT	34.7	52.0	42.4	63.6	42.4	63.6	42.4	63.6	—	—	—	—
		X	STD	34.8	52.2	43.5	65.3	52.2	78.3	53.0	79.5	—	—	—	—
			SSLT	34.7	52.0	43.4	65.1	52.0	78.1	53.0	79.5	—	—	—	—
	A490	N	STD	34.8	52.2	43.5	65.3	52.2	78.3	53.0	79.5	—	—	—	—
			SSLT	34.7	52.0	43.4	65.1	52.0	78.1	53.0	79.5	—	—	—	—
		X	STD	34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	—	—	—	—
			SSLT	34.7	52.0	43.4	65.1	52.0	78.1	60.7	91.1	—	—	—	—
3 (L = 8 1/2)	A325 F1852	N	STD	25.6	38.3	31.8	47.7	31.8	47.7	31.8	47.7	—	—	—	—
			SSLT	25.6	38.3	31.8	47.7	31.8	47.7	31.8	47.7	—	—	—	—
		X	STD	25.6	38.3	31.9	47.9	38.3	57.5	39.8	59.6	—	—	—	—
			SSLT	25.6	38.3	31.9	47.9	38.3	57.5	39.8	59.6	—	—	—	—
	A490	N	STD	25.6	38.3	31.9	47.9	38.3	57.5	39.8	59.6	—	—	—	—
			SSLT	25.6	38.3	31.9	47.9	38.3	57.5	39.8	59.6	—	—	—	—
		X	STD	25.6	38.3	31.9	47.9	38.3	57.5	44.7	67.1	—	—	—	—
			SSLT	25.6	38.3	31.9	47.9	38.3	57.5	44.7	67.1	—	—	—	—
2 (L = 5 1/2)	A325 F1852	N	STD	16.3	24.5	20.4	30.6	21.2	31.8	21.2	31.8	—	—	—	—
			SSLT	16.3	24.5	20.4	30.6	21.2	31.8	21.2	31.8	—	—	—	—
		X	STD	16.3	24.5	20.4	30.6	24.5	36.7	26.5	39.8	—	—	—	—
			SSLT	16.3	24.5	20.4	30.6	24.5	36.7	26.5	39.8	—	—	—	—
	A490	N	STD	16.3	24.5	20.4	30.6	24.5	36.7	26.5	39.8	—	—	—	—
			SSLT	16.3	24.5	20.4	30.6	24.5	36.7	26.5	39.8	—	—	—	—
		X	STD	16.3	24.5	20.4	30.6	24.5	36.7	28.5	42.8	—	—	—	—
			SSLT	16.3	24.5	20.4	30.6	24.5	36.7	28.5	42.8	—	—	—	—
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
– indicates that the plate thickness is greater than $d_h/2$
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

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AISC

Simple Shear Connections

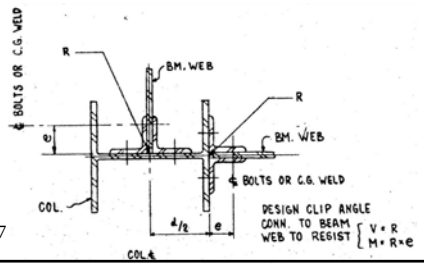
- **Why Tabulated Capacity Increased from 9th ed.**
  - **Testing allowed significant reduction or elimination of eccentricity. Also, design procedure was simplified.***
  - **But: Watch out for beam web capacity – block shear or bolt bearing can control. (How to check? See bottom part of Table 10-1 for the beam in question)**
  - **Shear tab vertical edge distance: Now 1 ¼" (was 1 ½")**
- **Multiple-Bolt (Long) Connections**
  - **See discussion on reduction factors in AISC 360 Comm. Sec. C-J3.6**

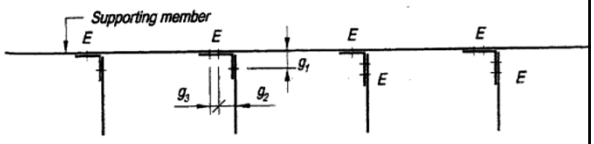
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***Source: C. Carter, Shear Connection Capacities, Answer in *Modern Steel Construction*, June 2006**

Simple Shear Connections

- **When to Consider Eccentricity in Connections?**
  - **In single-angle <u>bolted</u> connections, consider eccentricity:**
    - **in the leg attached to support**
    - **if double rows of bolts used,**
    - **if e > 3" (2 ¾" gage + ¼" half web)**
  - **Eccentricity in <u>welded</u> connections is always considered**





E indicates that eccentricity must be considered in this leg. Gages g_1, g_2, and g_3 are workable gages as shown in Table 1-7

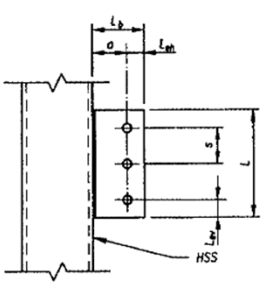
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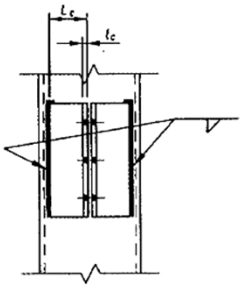
Simple Shear Connection

- **Shear Connections to HSS & Pipe Columns**
  - **Options…For round HSS, can use single-pl, thru-pl, stiff. seat**
  - **For rotational flexibility, use only vertical welds w/ small returns, but no returns at single-pl, thru-pl. (Returns OK at stiff. seat & across bottom of single *L*.)**

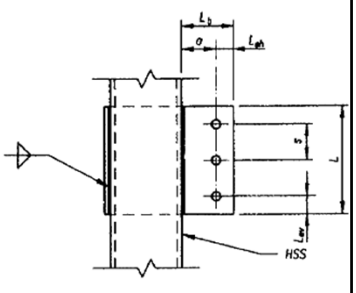
MOST ECONOMICAL → MORE EXPENSIVE → MOST EXPENSIVE



Single plate connection (shear tab)



Double angle shear connection



Through-plate shear connection

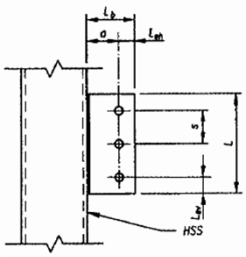
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Source: Donald R. Sherman, Connecting HSS, *Modern Steel Construction*, July 2005

Simple Shear Connection

❑ Shear Connections to HSS & Pipe Col's, Cont'd

- **Connection design is same as for W columns, except limit states of HSS wall**
- **No beam coping needed except at double *L*s**
- **Min. HSS column width = 12" for double and single *L*s**
- **See design checks in Chapter K of AISCM 14th ed. to preclude shear yielding (punching) of HSS wall**



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Q & A

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