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Connection Design for Moment Frames and Braced Frames
Session 4: Bracing Connections
March 11, 2020

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Course Description

Bracing Connections
March 11, 2020

This session will present information on light and heavy bracing connections, and discuss the differences and similarities between the two. The applicable limit states for each will be identified, and demonstrated in design examples. The examples will apply the Uniform Force Method for analysis and design.

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Learning Objectives

- Identify the Whitmore Section and explain how it is applied to the safe design of bracing connections both in tension and compression.
- Describe how to design bolts and welds that are subjected to a combination of tension and shear.
- List the limit states to consider in the design of a heavy bracing connection.
- Explain the Uniform Force Method and use it to design the connection between a gusset plate and the beam and column of a bracing connection.

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Connection Design for Moment Frames and Braced Frames

Session 4: Bracing Connections

March 11, 2020

Brad Davis, PhD, SE
Associate Professor, University of Kentucky
Owner, Davis Structural Engineering

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SCHEDULE

- February 19, 2020 — Moment Connections Part I
- February 26, 2020 — Moment Connections Part II
- March 4, 2020 — Introduction to Seismic Connections
- March 11, 2020 — Bracing Connections

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BRACING CONNECTIONS

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TOPICS

- Basic Bracing Connections
  - Limit States
  - Example
- Heavy Bracing Connections
  - Internal Force Distribution
  - Limit States
  - Example

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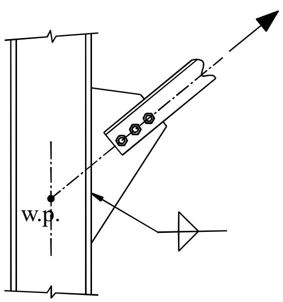
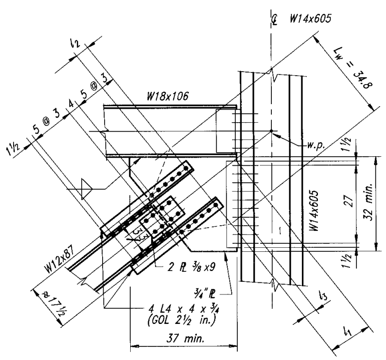
Bracing Connections





Basic and Heavy Bracing Connections

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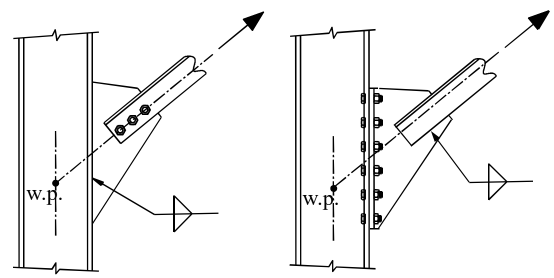
Basic Bracing Connections



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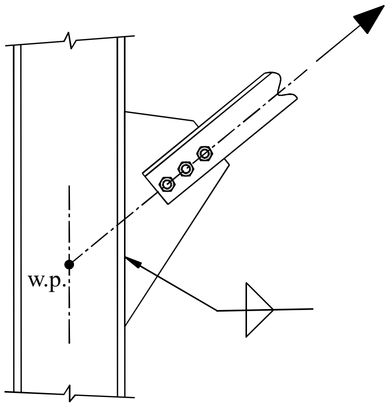
Basic Bracing Connections



Welded to Column Flange

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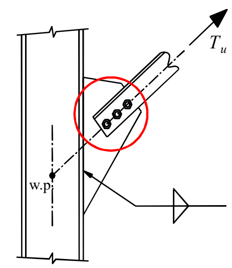
Bolted-Welded Basic Bracing Connections

Limit States

- Angles
  - Tensile Yielding
  - Tensile Rupture
  - Block Shear
- Shear Transfer
  - Bolt Shear Rupture
  - Bearing (Angle and Plate)
  - Tearout (Angle and Plate)



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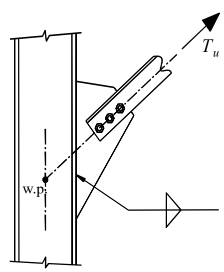
Bolted-Welded Basic Bracing Connections

Limit States

- Plate
  - Tensile Yielding on Whitmore Section
  - Tensile Rupture on Whitmore Section
  - Compressive Yielding / Buckling on Whitmore Section
    - Brace in Compression
  - Block Shear
    - Multiple Rows of Bolts
  - Base Metal Strength



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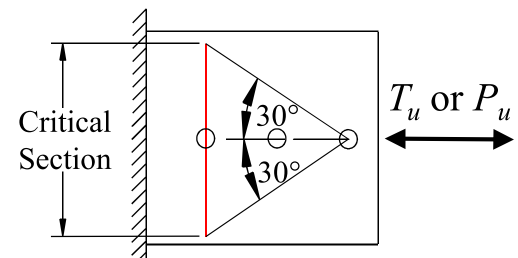
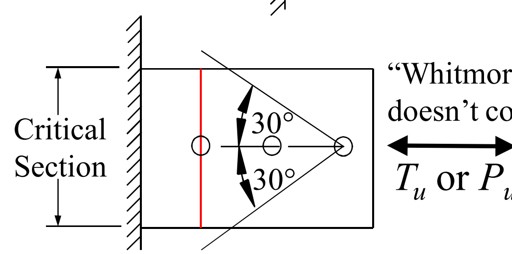
Bolted-Welded Basic Bracing Connections

Limit States

Whitmore Section

(*Manual* Figure 9-1)





"Whitmore Section doesn't control."

T_u or P_u

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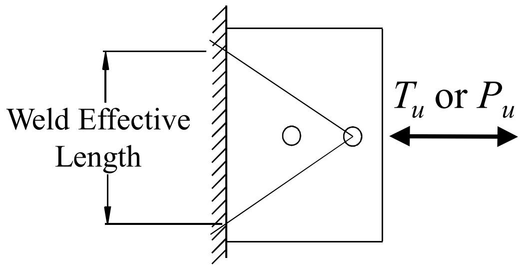
Bolted-Welded Basic Bracing Connections

Limit States

- Whitmore Section



Engineering Judgment

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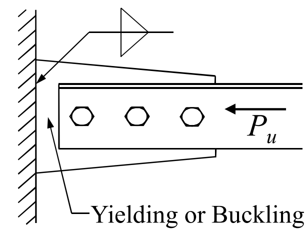
Bolted-Welded Basic Bracing Connections

Limit States

Plate Compression per *Spec.* J4.4

$P_u \leq \phi P_n \quad \phi = 0.90$



If $L_c/r \leq 25$, then

$\phi P_n = \phi F_y A_g$

Otherwise

ϕP_n per Chapter E

$L_c = KL$ → See Design Guide 29.

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Bolted-Welded Basic Bracing Connections

Limit States

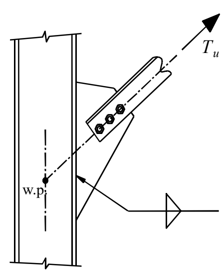
- Weld Rupture Design Strength

$\phi R_n = \phi F_{nw} A_{we}$ (*Spec.* Eq. J2-3)

$F_{nw} = 0.60 F_{EXX}(1.0 + 0.50 \sin^{1.5}\theta)$

Or, for 70 ksi welds:

$\phi R_n = \underbrace{1.392 D l}_{\textit{Manual}\text{ Eq. 8-1}}(1.0 + 0.50 \sin^{1.5}\theta)$



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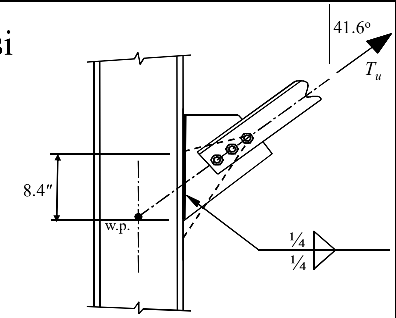
Bolted-Welded Basic Bracing Connections

Ex. Weld Strength, 70 ksi

Effective Weld Length

- Whitmore 30° Spread
- Concentric with line of action of brace force.



$\phi R_n = 1.392 D l(1.0 + 0.50 \sin^{1.5}\theta)$

$= 1.392(4)(8.4" - 0.25")(2 \text{ welds})(1.0 + 0.50 \sin^{1.5} 41.6°)$

$= 115 \text{ kips} \quad \geq T_u$

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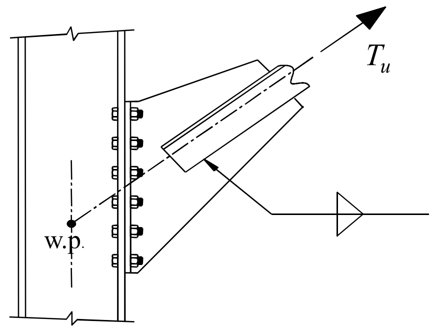
Welded-Bolted Basic Bracing Connections



Bolted to Column Flange

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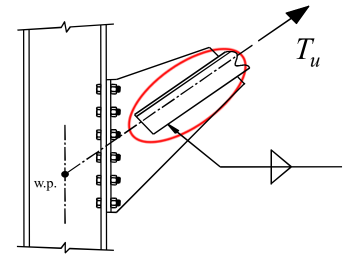
Welded-Bolted Basic Bracing Connections

Limit States

- Angles
  - Tensile Yielding
  - Tensile Rupture
  - Base Metal Strength
- Angle-to-Stem Weld
  - Weld Rupture



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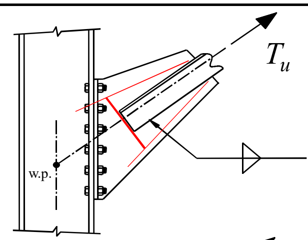
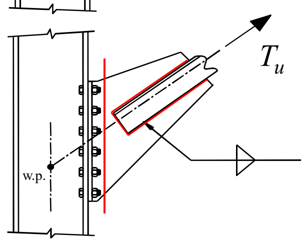
Welded-Bolted Basic Bracing Connections

Limit States

- Tee Stem
  - Tensile Yielding on Whitmore Section
  - Tensile Rupture on Whitmore Section
  - Compression on Whitmore Section
    - Brace in Compression
  - Block Shear
  - Shear Yielding





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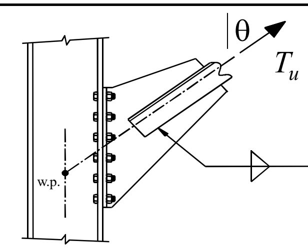
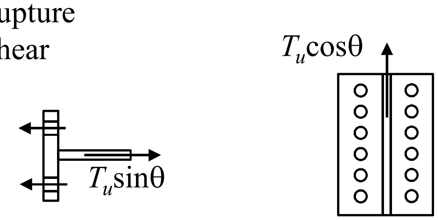
Welded-Bolted Basic Bracing Connections

Limit States

- Tee Flange
  - Flange Bending / Prying Action, *Manual* Page 9-10
  - Shear Yielding
  - Shear Rupture
  - Block Shear





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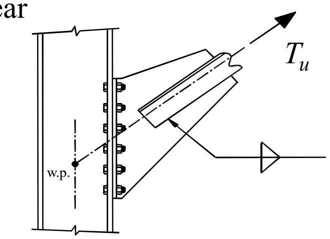
Welded-Bolted Basic Bracing Connections

Limit States

- Bolts
  - Combined Tension and Shear
- Shear Transfer
- Slip Resistance (SC Only)
- Column Flange
  - Flange Bending / Prying Action
- Column Web
  - Web Local Yielding
  - Web Local Crippling
    - Brace in Compression



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Welded-Bolted Basic Bracing Connections

Example. Evaluate the slip resistance of the connection.

T_u = 150 kips, θ = 50°, 3/4 in. Gr. A325-SC Bolts, Class B, OVS Holes.

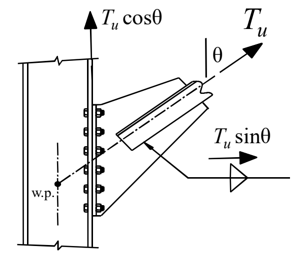
Spec. Eq. J3-4 & J3-5a:

$$\phi R_n = \phi \mu D_u h_f T_b n_s k_{sc} \text{ / bolt}$$

$\phi = 0.85$ $\mu = 0.5$ $D_u = 1.13$
$h_f = 1.0$ $T_b = 28$ kips $n_s = 1$

Spec. Eq. J3-5a:

$$k_{sc} = 1 - \frac{T_u \sin\theta}{D_u T_b n_b} = 1 - \frac{(150 \text{ kips}) \sin 50^\circ}{(1.13)(28 \text{ kips})(12)} = 0.697$$



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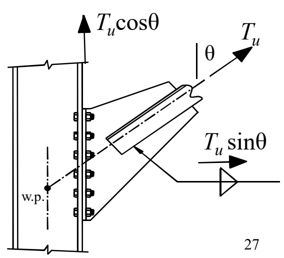
Welded-Bolted Basic Bracing Connections

Example. Check Slip Resistance.

T_u = 150 kips $\phi = 1.0$ $\mu = 0.5$ $D_u = 1.13$
$h_f = 1.0$ $T_b = 28$ kips $n_s = 1$ $k_{sc} = 0.697$

$$\phi R_n = \phi \mu D_u h_f T_b n_s k_{sc} \text{/bolt}$$
$$= (0.85)(0.50)(1.13)(1.0)(28 \text{ kips})(1)(0.697)$$
$$= 9.37 \text{ kips/bolt}$$

$\sum \phi R_n = (9.37 \text{ kips})(12) = 112$ kips
$T_u \cos\theta = (150 \text{ kips})(\cos 50^\circ)$
$= 96.4 \text{ kips} < 112$ kips OK



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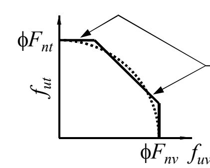
Welded-Bolted Basic Bracing Connections

Example. Check Bolt Design Strength ($V_u + T_u$)

¾ in. Gr. A325-N Bolts, Thick Plate Design: $q_u = 0.0$



$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{uv} \le F_{nt}$$

Shear: $R_u \le \phi R_n = \phi F_{nv} A_b \Rightarrow f_{uv} \le \phi F_{nv}$ (*Spec.* J3-1)

Tension + Shear: $R_u \le \phi R_n = F'_{nt} A_b$ (*Spec.* J3-2 & J3-3a)

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Welded-Bolted Basic Bracing Connections

Example, continued.

Spec. Table J3.2
$F_{nv} = 54$ ksi
$F_{nt} = 90$ ksi

Shear:

$$f_{uv} = \frac{(150\text{ kips})\cos 50^\circ}{(12\text{ bolts})(0.442\text{ in.}^2)} = 18.2\text{ ksi} < \phi F_{nv} = 40.5\text{ ksi, OK}$$

(Bearing and tearout should be considered also.)

Tension in the Presence of Shear:

$$F'_{nt} = 1.3(90\text{ ksi}) - \frac{90\text{ ksi}}{(0.75)(54\text{ ksi})}(18.2\text{ ksi}) = 76.6\text{ ksi} < 90\text{ ksi}$$

$$\phi R_n = (0.75)(76.6)(0.442) = 25.4\text{ kips}$$

$$R_u = \frac{(150\text{ kips})\sin 50^\circ}{12\text{ bolts}} = 9.58\text{ kips} < 25.4\text{ kips, OK}$$

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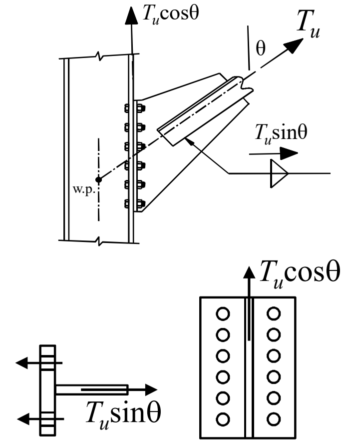
Welded-Bolted Basic Bracing Connections

Example.

Notes

- Tee flange limit states that also must be checked:
  - Shear Yielding
  - Shear Rupture
  - Block Shear
  - Shear Transfer
  - Flange Bending



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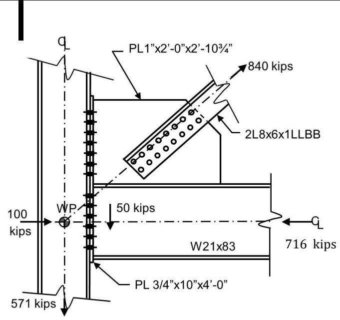
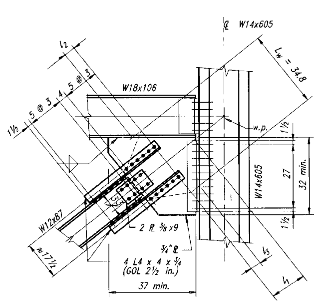
Heavy Bracing Connections





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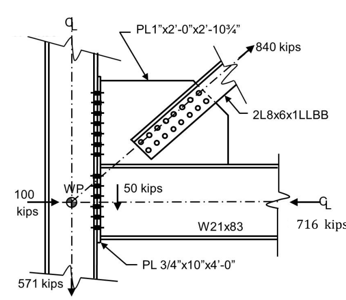
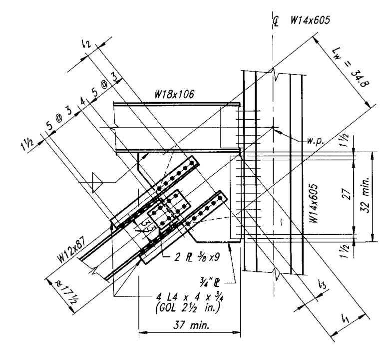
Heavy Bracing Connections



Brace Connected Directly to Gusset Plate



Connection with "Claw" Angles

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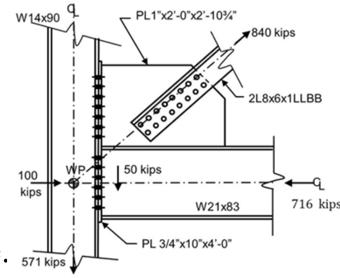
Heavy Bracing Connections

- Gusset plate usually welded to beam.
- Gusset plate usually bolted to column.
  - Single Plate Connection
  - Double Angles
- Forces at beam and column are indeterminate.
  - Recommended: Uniform Force Method in *Manual* Part 13 and DG29.



Heavy Bracing Connections

Internal Force Distribution

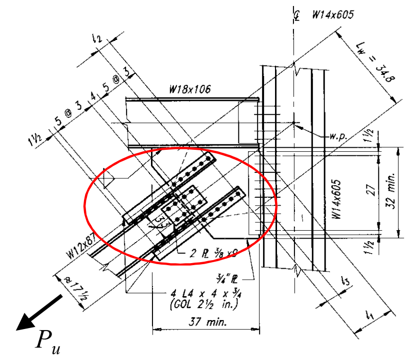
- Brace-to-Gusset Plate Forces (Claw Angles)

ht_w

$$R_{u,web} = P_u(A_{web}/A)$$

$$R_{u,angles} = P_u(A_{flanges}/A)$$

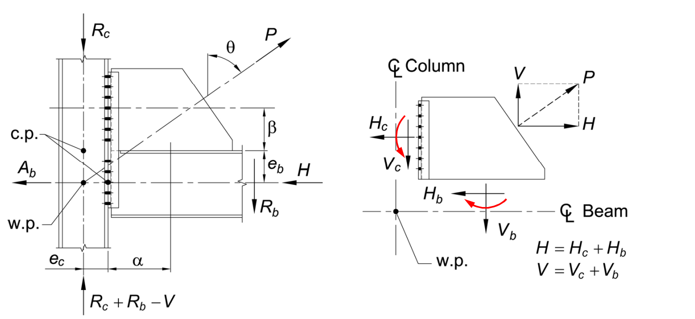
$(A\text{-}ht_w)/A$



Heavy Bracing Connections

Internal Force Distribution

- Gusset Plate Forces at Column and Beam (*Manual* Part 13)

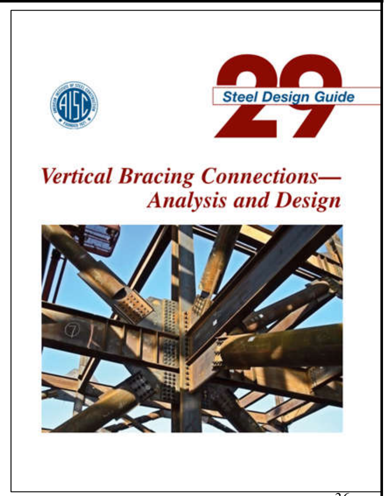


(a) External Forces (b) FBD of Gusset Plate

Heavy Bracing Connections

Lower Bound Theorem

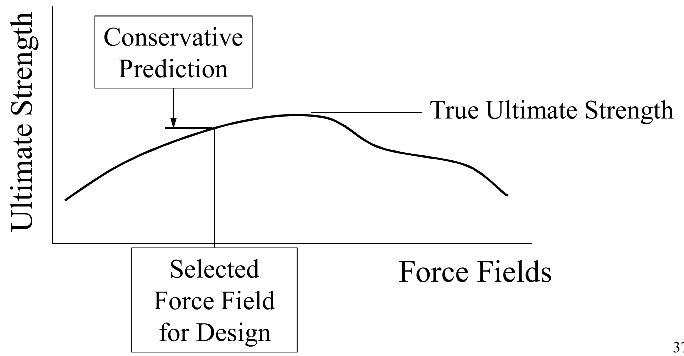
- DG29 Chapter 4
  - Corner connections are statically indeterminate.
  - Infinite number of force fields (combinations of H_c, V_c, M_c, H_b, V_b, and M_b).
  - The force field that produces the maximum ultimate strength is closest to the true solution.
  - Choice of forces affects economy of the connection.



Heavy Bracing Connections

Lower Bound Theorem

- The force field that produces the maximum ultimate strength is closest to the true solution.



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Heavy Bracing Connections

Lower Bound Theorem

- Requirements of the LBT
  - The force field must be statically admissible.
    - $\Sigma F_H = 0$
    - $\Sigma F_V = 0$
    - $\Sigma M = 0$
  - No limit state can be exceeded.
    - $R_u \leq \phi R_n$ for all limit states.
  - Ductile limit states to allow some redistribution of forces.

Statically admissible and **safe** force field.

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Heavy Bracing Connections

Example – KISS Method (DG29)

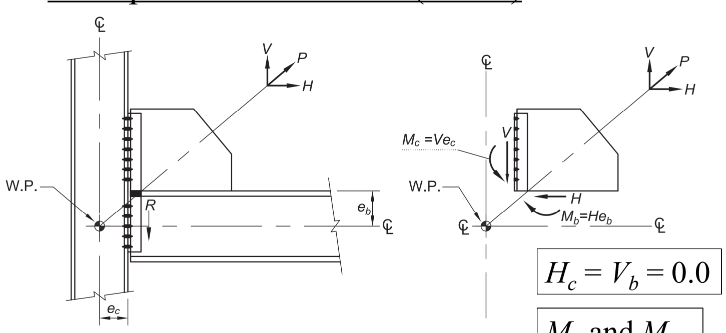

Fig. 4-1. KISS method admissible force field.

$H_c = V_b = 0.0$

M_b and M_c required for $\Sigma M = 0$.

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Heavy Bracing Connections

Uniform Force Method (DG29 and *Manual* Ch. 13)

- Eliminates moments on gusset plate.
- Complies with LBT.
- Shown to produce safe and economical designs.
- Emphasized in DG29 and *Manual* Chapter 13.

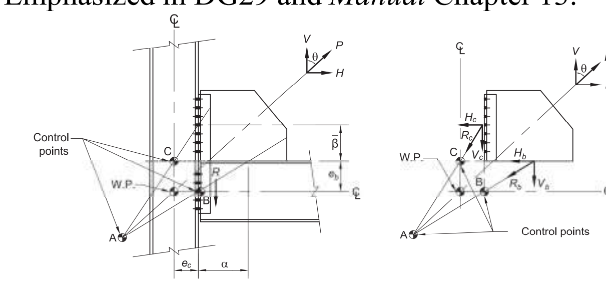

Fig. 4-4a. Uniform force method admissible force field at gusset plate interface.

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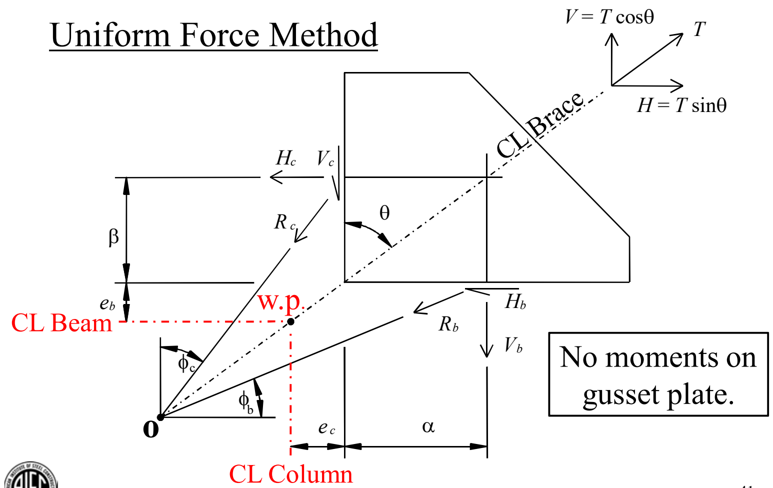
Heavy Bracing Connections

Uniform Force Method



No moments on gusset plate.

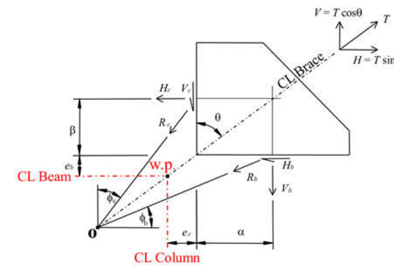
Heavy Bracing Connections

Uniform Force Method

- e_b = beam d / 2
- e_c = column d / 2
- α = face of column to centroid of gusset-to-beam connection.
- β = face of beam to centroid of gusset-to-column connection.
- No gusset moments if:

$$\alpha - \beta \tan\theta = e_b \tan\theta - e_c \qquad (\textit{Manual}\ \text{Eq. 13-1})$$

- Select α or β and compute the other.



Heavy Bracing Connections

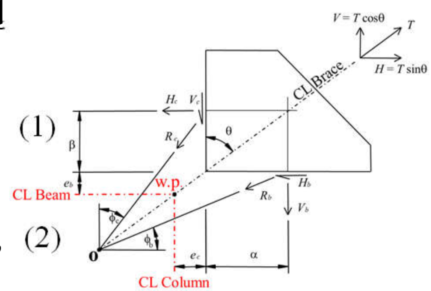
Uniform Force Method

$$\Sigma F_H = 0$$

$$T\sin\theta = R_c\sin\phi_c + R_b\cos\phi_b \quad (1)$$

$$\Sigma F_V = 0$$

$$T\cos\theta = R_c\cos\phi_c + R_b\sin\phi_b \quad (2)$$



Combine (1) and (2) and use angle relationships to obtain…

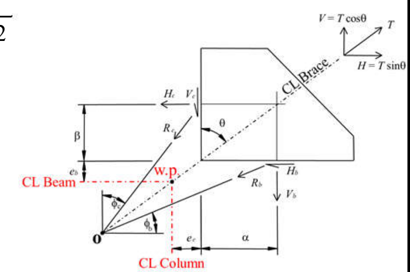
Heavy Bracing Connections

Uniform Force Method

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2}$$

$$V_c = \frac{\beta}{r}T \qquad H_c = \frac{e_c}{r}T$$

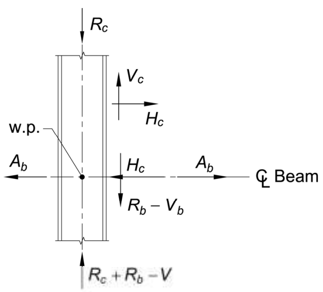
$$H_b = \frac{\alpha}{r}T \qquad V_b = \frac{e_b}{r}T$$



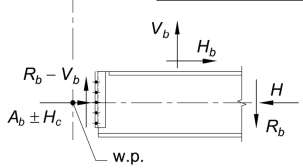
Heavy Bracing Connections

Forces at Beam-to-Column Connection

Notes
- $V_b + R_b$ can be large.
- No horizontal slots at the beam web.
- EOR must give A_b and R_b, not just brace force.
- If HSS, EOR must give R_c for Q_f calculation.



(c) Column free-body diagram



(d) Beam free-body diagram

Manual Figure 13-2

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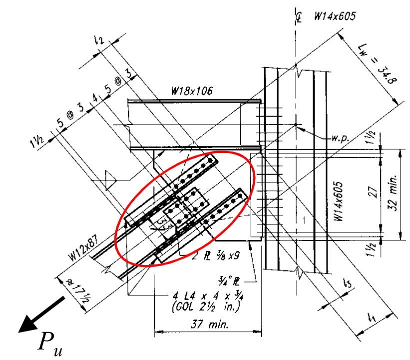
Heavy Bracing Connections

Limit States – Brace-to-Gusset Connection

- Brace, Web Plate, Claw Angles
  - Tension Yielding
  - Tension Rupture
  - Block Shear
  - Shear Transfer



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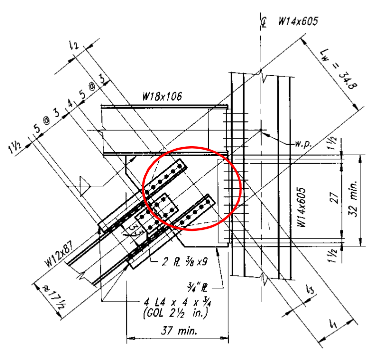
Heavy Bracing Connections

Limit States – Gusset Plate at Brace-to-Gusset Connection

- Shear Transfer
- Whitmore Section
  - Tensile Yielding
  - Tensile Rupture
  - Compressive Yielding or Buckling
- Block Shear



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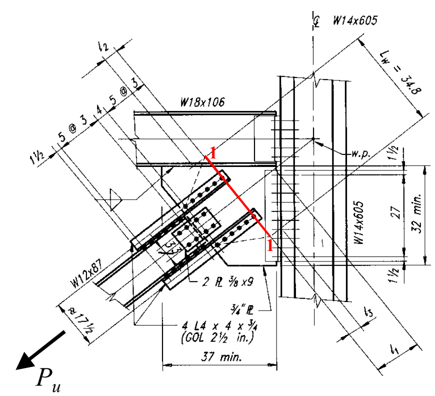
Heavy Bracing Connections

Limit States – Gusset Plate at Brace-to-Gusset Connection

- Whitmore Section
  - Tensile yielding and rupture along 1-1; 2 bolt holes.



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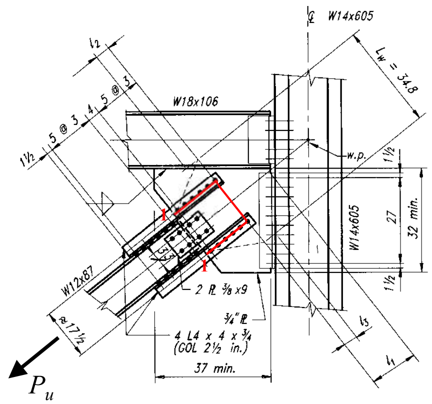
Heavy Bracing Connections

Limit States – Gusset Plate at Brace-to-Gusset Connection

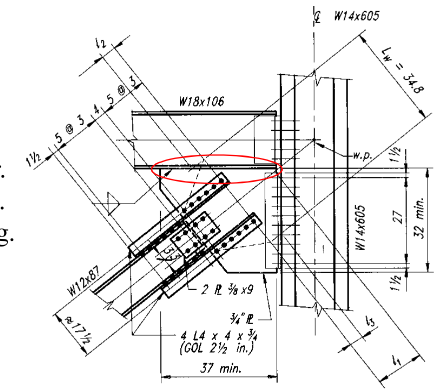
- Block shear along 1-1.



Heavy Bracing Connections

Limit States – Gusset Plate-to-Beam Connection

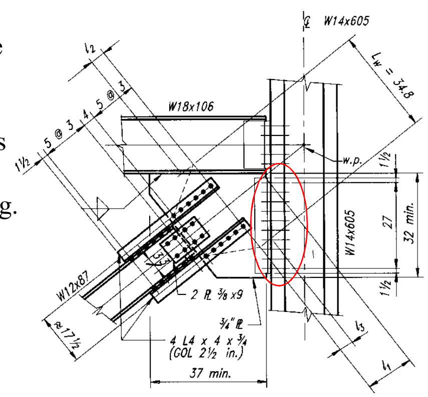
- Fillet or CJP welds. Increase by 25%. Non-uniform distribution of forces. Hewitt and Thornton, 2004 EJ Paper.
- Beam web local yielding.
- Beam web local crippling.
- If HSS, then beam wall punching shear and bending (*Manual* Part 9, pages 9-14 – 9-17).
- Basic limit states also apply.



Heavy Bracing Connections

Limit States – Gusset Plate-to-Column Connection

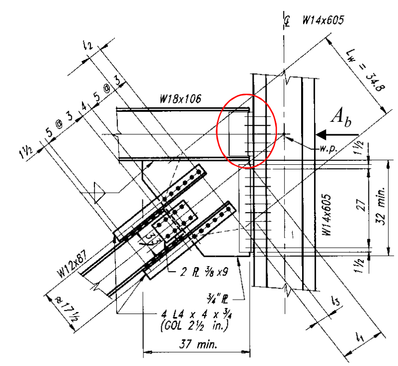
- Angles, end plate, or plate with shear plus tension.
  - If angles or EP → prying forces in angles and/or column.
- Column web local yielding.
- Column web local crippling.
- If HSS column, then punching shear and wall bending.
- Basic limit states also apply.



Heavy Bracing Connections

Limit States – Beam-to-Column Connection

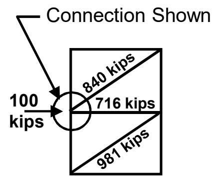
- Beam shear yielding.
- Angles, end plate, or plate with shear plus tension.
  - If angles or EP → prying forces
  - If plate → ~~horizontal slots~~
- Column web local yielding
- Column web local crippling
- Column web compression buckling (A_b)
- If HSS column, then punching shear and wall bending.
- Typical limit states apply.

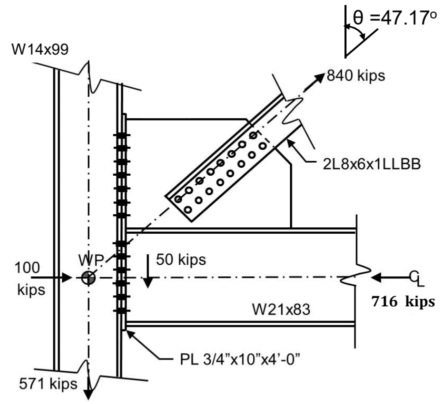


Heavy Bracing Connection Example

Objectives

- Forces by UFM
- Select plate dimensions.
- Evaluate selected limit states.

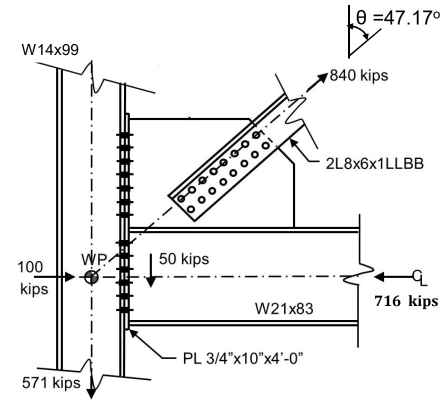




Heavy Bracing Connection Example

Materials

Bolts: 7/8 in. A490-X
Holes: 15/16 in. STD
Beam / Column: A992
Welds: E70xx
Plate: A572 Gr 50



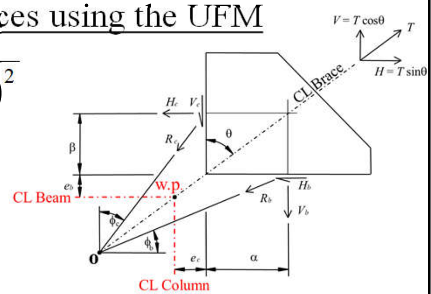
Heavy Bracing Connection Example

Determine Gusset Forces using the UFM

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2}$$

$$V_c = \frac{\beta}{r}T \qquad H_c = \frac{e_c}{r}T$$

$$H_b = \frac{\alpha}{r}T \qquad V_b = \frac{e_b}{r}T$$

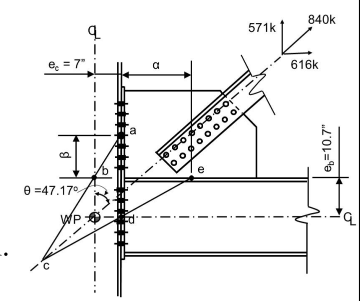


Column W14x99 $d = 14.2$ in. → $e_c = 7.10$ in.
Beam W21x83 $d = 21.4$ in. → $e_b = 10.7$ in.
$V_u = T_u \cos\theta = (840 \text{ kips}) \cos 47.17° = 571$ kips
$H_u = T_u \sin\theta = (840 \text{ kips}) \sin 47.17° = 616$ kips

Heavy Bracing Connection Example

Select Trial α and β

α:
Try 7/16 in. fillet welds.
Select length of weld to resist 616 kips.
$L_w = 616 \text{ kips} / [(1.392)(7)(2)]$
$= 31.6$ in. → try $\alpha = 17.5$ in.



β: $\alpha - \beta\tan\theta = e_b \tan\theta - e_c$

$17.5 \text{ in.} - \beta\tan 47.17° = (10.7 \text{ in.})\tan 47.17° - 7.10 \text{ in.}$

$\beta = 12.1$ in.

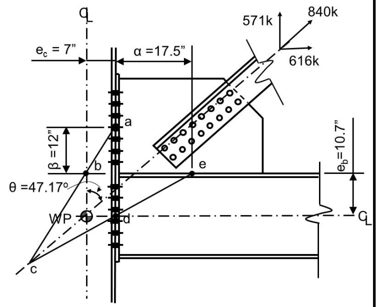
Try 7 rows of bolts.
$\beta = 3\text{-}1/8 \text{ in.} + (3)(3 \text{ in.}) = 12\text{-}1/8$ in.

Heavy Bracing Connection Example

Compute Interface Forces

$\alpha = 17.5$ in. $\beta = 12.1$ in.

$e_c = 7.10$ in. $e_b = 10.7$ in.



$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2}$$
$$= \sqrt{(17.5\text{ in.} + 7.10\text{ in.})^2 + (12.1\text{ in.} + 10.7\text{ in.})^2}$$
$$= 33.5\text{ in.}$$

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Heavy Bracing Connection Example

Compute Interface Forces

$$H_c = \frac{e_c}{r}P = \frac{7.10\text{ in.}}{33.5\text{ in.}}(840\text{ kips}) = 178\text{ kips}$$

$$V_c = \frac{\beta}{r}P = \frac{12.1\text{ in.}}{33.5\text{ in.}}(840\text{ kips}) = 303\text{ kips}$$

$$H_b = \frac{\alpha}{r}P = \frac{17.5\text{ in.}}{33.5\text{ in.}}(840\text{ kips}) = 438\text{ kips}$$

$$V_b = \frac{e_b}{r}P = \frac{10.7\text{ in.}}{33.5\text{ in.}}(840\text{ kips}) = 268\text{ kips}$$

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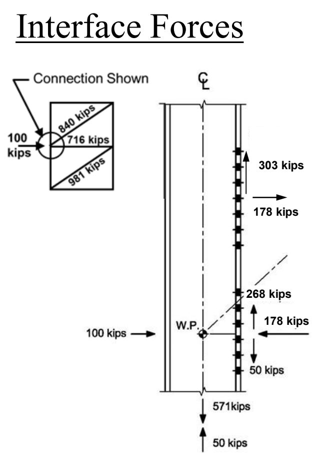
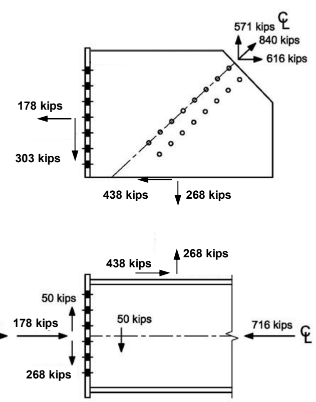
Heavy Bracing Connection Example

Interface Forces





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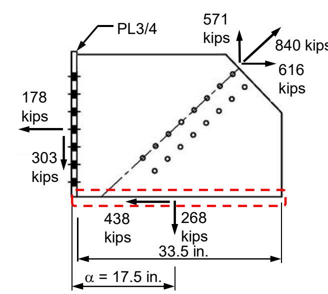
Heavy Bracing Connection Example

Gusset-to-Beam Weld

$$R_u = \sqrt{(438\text{ kips})^2 + (268\text{ kips})^2} = 513\text{ kips}$$

$$\theta = \text{atan}(268\text{ kips} / 438\text{ kips}) = 31.5^\circ$$



$D_{reqd} = (513\text{ kips}) / [(1.392)(33.5\text{ in.})(1+0.5\sin^{1.5}31.5^\circ)(2)]$
$= 4.63$

Increase 25%: (1.25)(4.63) = 5.78 → 3/8 in. D.S. fillet weld.

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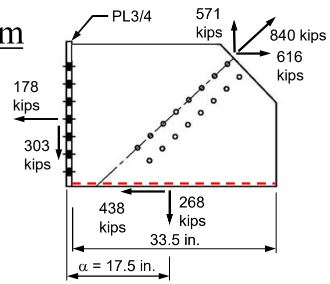
Heavy Bracing Connection Example

Gusset Plate Strength at Beam

Try 1 in. thick, A572 Gr 50

$T_u = 268$ kips

$V_u = 438$ kips



$\phi T_n = \phi F_y A_g = (0.9)(50 \text{ ksi})(1 \text{ in.})(33.5 \text{ in.})$

$= 1510 \text{ kips} > T_u = 268 \text{ kips}$ OK

$\phi V_n = \phi 0.6 F_y A_{gv} = (1.0)(0.6)(50 \text{ ksi})(1 \text{ in.})(33.5 \text{ in.})$

$= 1010 \text{ kips} > V_u = 438 \text{ kips}$ OK

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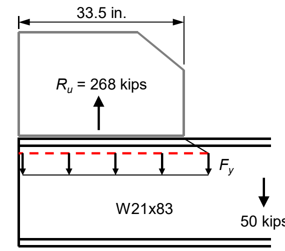
Heavy Bracing Connection Example

Beam Web Local Yielding

W21x83 A992

$t_w = 0.515$ in.

$k_{des} = 1.34$ in.



$\phi R_n = \phi F_{yw} t_w (2.5k + l_b)$ (*Spec*. Eq. J10-3)

$= (1.00)(50 \text{ ksi})(0.515 \text{ in.})\left[2.5(1.34 \text{ in.}) + 33.5 \text{ in.}\right]$

$= 949 \text{ kips} > 268 \text{ kips}$, OK

Note: Beam-to-column connection must be designed for a shear of $V_u = \pm$ 268 kips + 50 kips. (Checks not shown.)

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Heavy Bracing Connection Example

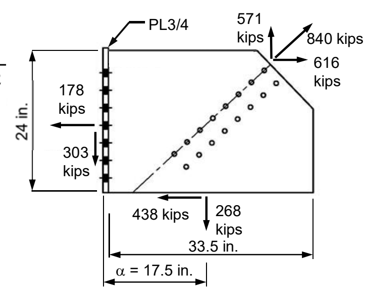
Gusset-to-End-Plate Weld

$R_u = \sqrt{(178 \text{ kips})^2 + (303 \text{ kips})^2}$

$= 351$ kips

$\theta = \text{atan}(178 \text{ kips} / 303 \text{ kips})$

$= 30.4^\circ$



$D_{reqd} = (351 \text{ kips}) / [(1.392)(24.0 \text{ in.})(1+0.5\sin^{1.5}30.4^\circ)(2)]$

$= 4.45$ (Min. is 1/4 in. from *Spec*. Table J2.3)

Use 5/16 in. double-sided fillet welds.

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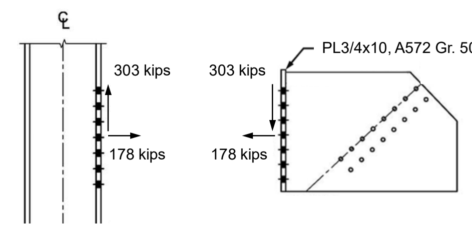
Heavy Bracing Connection Example

Check End-Plate-to-Column Connection

W14x99 A992
$b_f = 14.6$ in.
$t_f = 0.780$ in.
$g = 5$ in.

14 - 7/8 in.
Gr. A490-X
Manual Tables 7-1 and 7-2:
$\phi r_{nv} = 37.9$ kips
$\phi r_{nt} = 51.0$ kips



$r_{uv} = 303 \text{ kips} / 14 = 21.6 \text{ kips} < \phi r_{nv} = 37.9$ kips

$r_{ut} = 178 \text{ kips} / 14 = 12.7 \text{ kips} < \phi r_{nt} = 51.0$ kips

OK so far.

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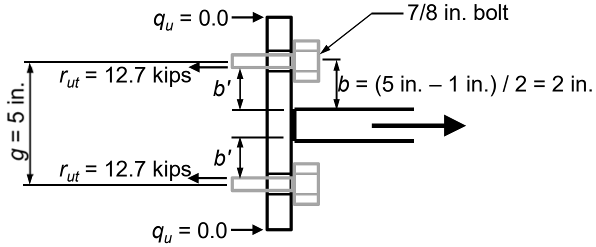
Heavy Bracing Connection Example

End-Plate Bending / Prying Action

Assumed no prying action.

Manual Part 9

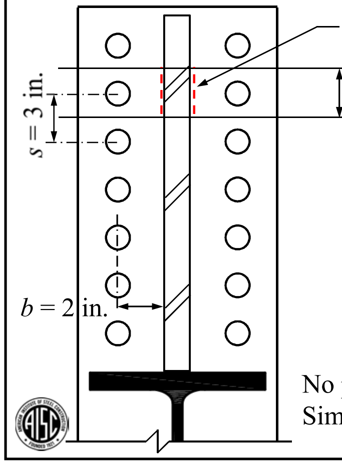
3/4 in. x 10 in. end plate and 1 in. gusset plate



$b' = 2\text{ in.} - (7/8\text{ in.})/2 = 1.56\text{ in.}$

Heavy Bracing Connection Example

End-Plate Bending / Prying Action



$$p = \min \begin{cases} s = 3\text{ in.} \\ 2(1.75b) = 7\text{ in.} \end{cases}$$

$$t_{np} = \sqrt{\frac{4r_{ut}b'}{\phi p F_u}}$$

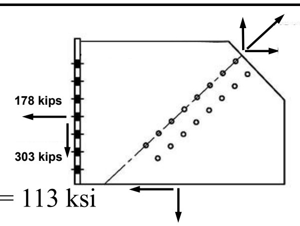
$$= \sqrt{\frac{(4)(12.7\text{ kips})(1.56\text{ in.})}{(0.9)(3\text{ in.})(65\text{ ksi})}}$$

$$= 0.672\text{ in.} < 3/4\text{ in.}$$

No prying action due to end plate bending.
Similar at column flange → no prying.

Heavy Bracing Connection Example

End-Plate to Column Bolts



14 – 7/8 in. Gr. A490-X Bolts

r_{uv} = 303 kips / 14 = 21.6 kips

r_{ut} = 178 kips / 14 = 12.7 kips

Spec. Table J3.2: F_{nv} = 84 ksi and F_{nt} = 113 ksi

f_{uv} = 21.6 kips / 0.601 in.^{2} = 35.9 ksi < ϕF_{nv} = 63.0 ksi, OK
(Bearing and tearout must be considered also.)

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{uv} = 1.3(113\text{ ksi}) - \frac{113\text{ ksi}}{63.0\text{ ksi}}(35.9\text{ ksi})$$

$$= 82.5\text{ ksi } (<113\text{ ksi}) \quad (\textit{Spec}.\text{ Eq. J3-3a})$$

$$\phi R_n = \phi F'_{nt} A_b = (0.75)(82.5\text{ ksi})(0.601\text{ in.}^2) \quad (\textit{Spec}.\text{ Eq. J3-2})$$

$$= 37.2\text{ kips} > r_{ut} = 12.7\text{ kips, OK}$$

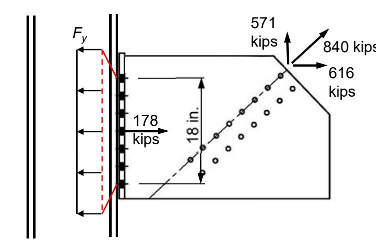
Heavy Bracing Connection Example

Check Column Local Web Yielding

W14x99 A992

t_w = 0.485 in.

k_{des} = 1.38 in.



$$\phi R_n = \phi F_{yw} t_w (5k + l_b) \quad (\textit{Spec}.\text{ Eq. J10-2})$$

$$= (1.00)(50\text{ ksi})(0.485\text{ in.})[5(1.38\text{ in.}) + 18\text{ in}]$$

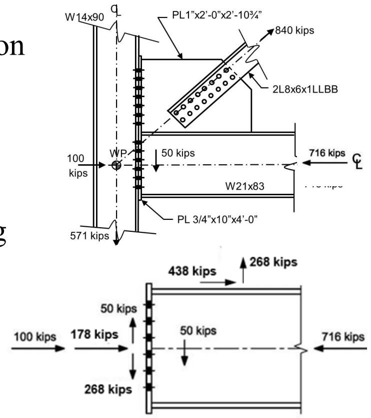
$$= 604\text{ kips} > 178\text{ kips, OK}$$

Heavy Bracing Connection Example

Beam-to-Column Connection

This Load Case: Compression

Limit States
- Web Local Yielding
- Web Local Crippling
- Web Compression Buckling
- Shear (High)
  - Beam shear yielding
  - Beam-to-EP Weld
  - Shear Transfer



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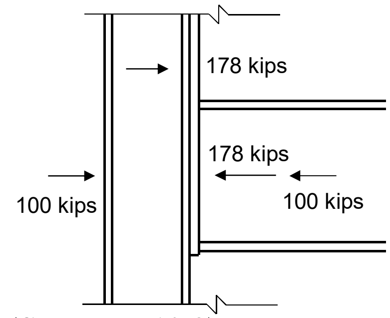
Heavy Bracing Connection Example

Check Column Web Local Yielding

W14x99 A992
- $t_w = 0.485$ in.
- $k_{des} = 1.38$ in.

Conservatively,
$l_b = d_b = 21.4$ in.



$$\phi R_n = \phi F_{yw} t_w (5k + l_b) \quad (\textit{Spec.}\ \text{Eq. J10-2})$$
$$= (1.00)(50\text{ ksi})(0.485\text{ in.})[5(1.38\text{ in.}) + 21.4\text{ in}]$$
$$= 686\text{ kips} > 278\text{ kips, OK}$$

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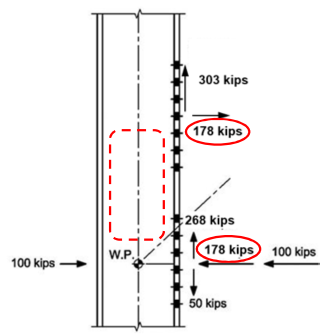
Heavy Bracing Connection Example

Column Shear

W14x99 A992
- $t_w = 0.485$ in.
- $d = 14.2$ in.
- $h/t_w = 23.5$

Spec. G2.1(a) applies.



$$\phi V_n = \phi 0.6 F_y A_w C_{v1}$$
$$= (1.00)(0.6)(50\text{ ksi})(0.485\text{ in.})(14.2\text{ in.})(1.00)$$
$$= 207\text{ kips} > 178\text{ kips, OK}$$

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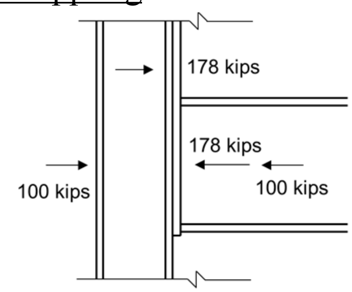
Heavy Bracing Connection Example

Check Column Web Local Crippling

W14x99 A992
- $t_w = 0.485$ in.
- $t_f = 0.780$ in.
- $d = 14.2$ in.

$l_b = d_b = 21.4$ in.

Use Eq. J10-4 since @ > $d/2$.



$$\phi R_n = \phi 0.80 t_w^2 \left[1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}Q_f$$
$$= 693\text{ kips} > R_u = 278\text{ kips, OK}$$

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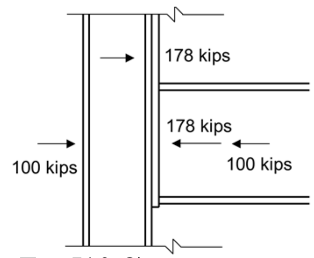
Heavy Bracing Connection Example

Check Column Web Compression Buckling

W14x99 A992

$t_w = 0.485$ in.

$h/t_w = 23.5 \rightarrow h = 11.4$ in.



$$\phi R_n = \phi \frac{24 t_w^3 \sqrt{E F_{yw}}}{h} Q_f \quad (\textit{Spec}. \text{ Eq. J10-8})$$

$$= (0.90) \frac{(24)(0.485 \text{ in.})^3 \sqrt{(29000 \text{ ksi})(50 \text{ ksi})}}{11.4 \text{ in.}} (1.0)$$

$= 260$ kips $< R_u = 278$ kips, NG, Add Stiffeners

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Heavy Bracing Connection Example

Notes

- All limit states have not been checked.
- See DG29 for complete design examples illustrating all limit states.
- If the brace force is compressive:
  - Beam-to-column connection is in tension. Limit states same as those shown in this example for the gusset-to-column connection.
  - Beam web local crippling applies.
  - Gusset plate flexural buckling applies. $K = 0.5$ for this case (DG29).

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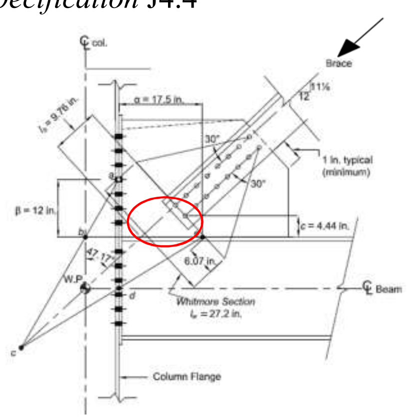
Heavy Bracing Connection Example

Gusset Plate Compression Yielding / Buckling

Specification J4.4



$K = 0.5$

See DG29 for other cases. Some have $K = 1.2$.

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THE END!!
Thank You for Attending!!

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- 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!

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- Password: Same as AISC website password.

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- You have two options to receive credit for a given session.
    - Option 1: Watch the live session. Credit for live attendance will be displayed on the Course Resources table within two days of the session.
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Course Resources

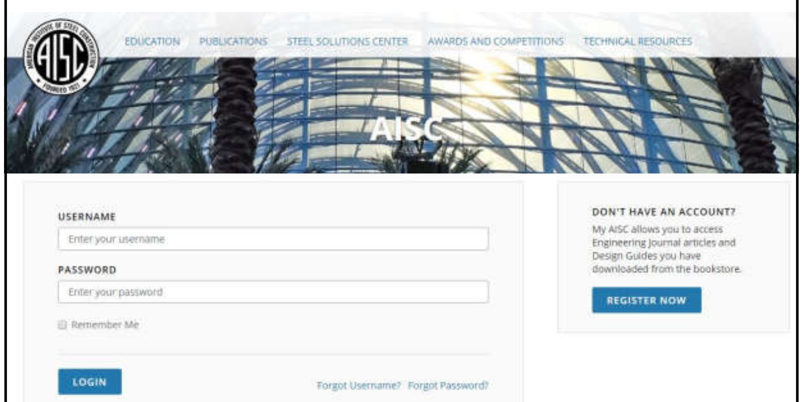
Find all your handouts, quizzes and quiz scores, recording access, and attendance information in one place!

Smarter. Stronger. Steel.

4-Session Registrants

Course Resources

Go to www.aisc.org and sign in.



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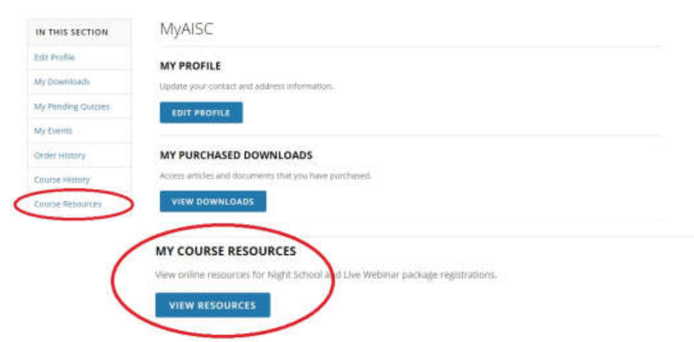

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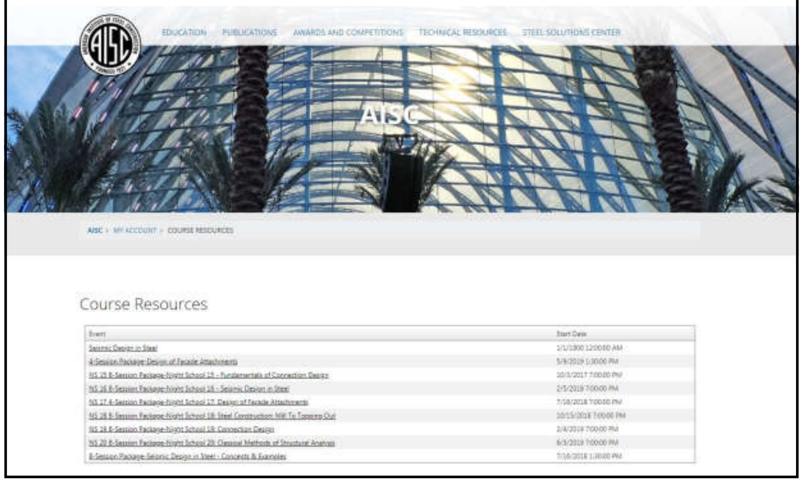
Course Resources



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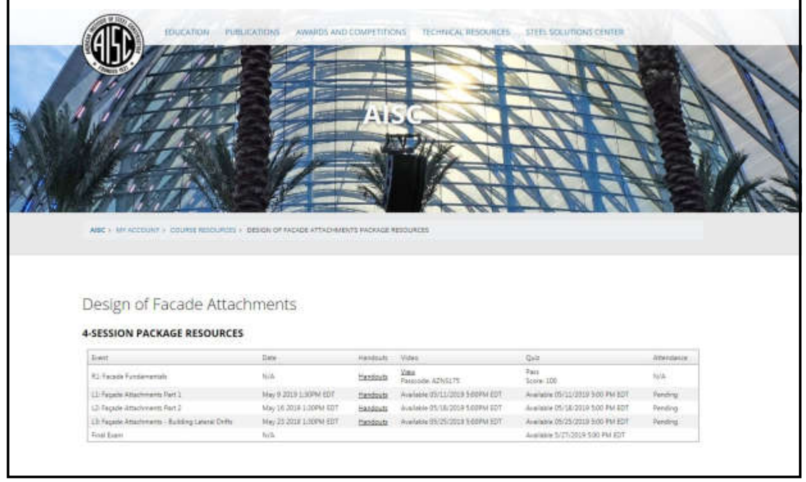

AISC | Thank you.

Smarter.
Stronger.
Steel.