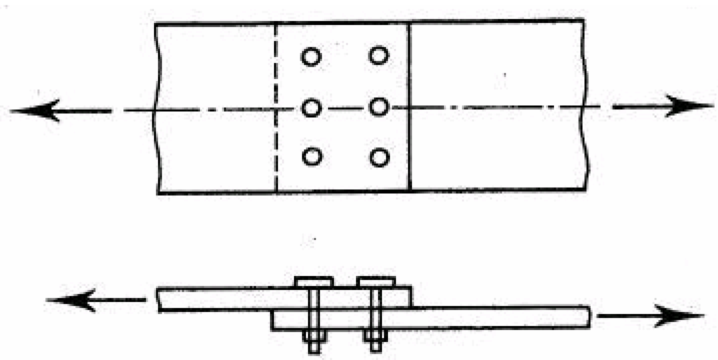
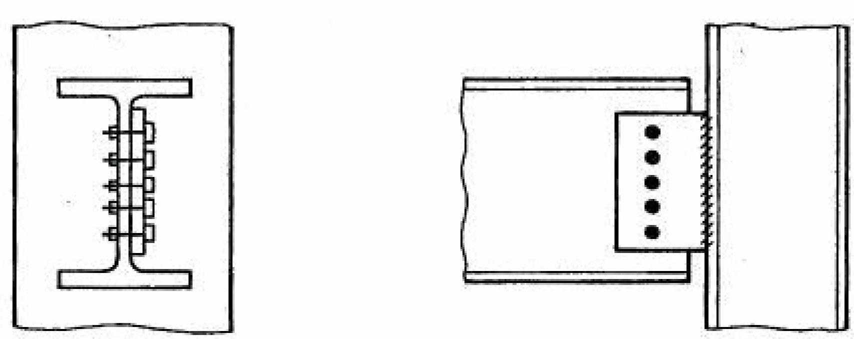
CHAPTER 5. BOLTED CONNECTION

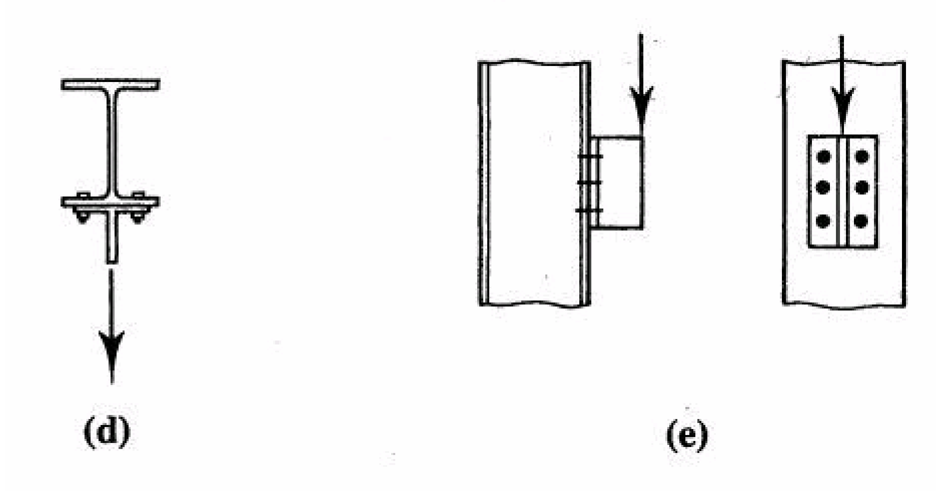
5.1 INTRODUCTORY CONCEPTS

- There are different types of bolted connections. They can be categorized based on the type of loading.
  - Tension member connection and splice. It subjects the bolts to forces that tend to shear the shank.
  - Beam end simple connection. It subjects the bolts to forces that tend to shear the shank.
  - Hanger connection. The hanger connection puts the bolts in tension



(a)

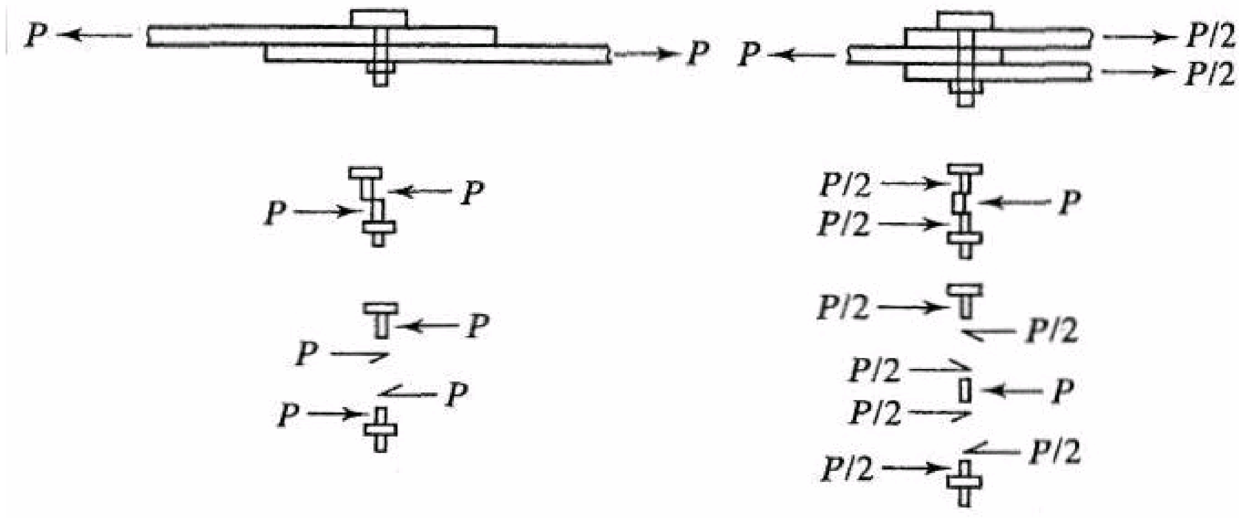




- The bolts are subjected to shear or tension loading.
  - In most bolted connection, the bolts are subjected to shear.
  - Bolts can fail in shear or in tension.
  - You can calculate the shear strength or the tensile strength of a bolt
- <u>Simple connection:</u> If the line of action of the force acting on the connection passes through the center of gravity of the connection, then each bolt can be assumed to resist an equal share of the load.
- The strength of the simple connection will be equal to the sum of the strengths of the individual bolts in the connection.
- We will first concentrate on bolted shear connections.

5.2 BOLTED SHEAR CONNECTIONS

- We want to design the bolted shear connections so that the factored design strength (ϕR_n) is greater than or equal to the factored load.
- So, we need to examine the various possible failure modes and calculate the corresponding design strengths.
- Possible failure modes are:
  - Shear failure of the bolts
  - Failure of member being connected due to fracture or block shear or ….
  - Edge tearing or fracture of the connected plate
  - Tearing or fracture of the connected plate between two bolt holes
  - Excessive bearing deformation at the bolt hole
- Shear failure of bolts
  - Average shearing stress in the bolt = $f_v = P/A = P/(\pi\ d_b^2/4)$
  - P is the load acting on an individual bolt
  - A is the area of the bolt and d_b is its diameter
  - Strength of the bolt = $P = f_v \times (\pi\ d_b^2/4)$ where f_v = shear yield stress = $0.6F_y$
  - Bolts can be in *single* shear or *double* shear as shown below.
  - When the bolt is in double shear, two cross-sections are effective in resisting the load. The bolt in *double shear* will have the twice the shear strength of a bolt in single shear.

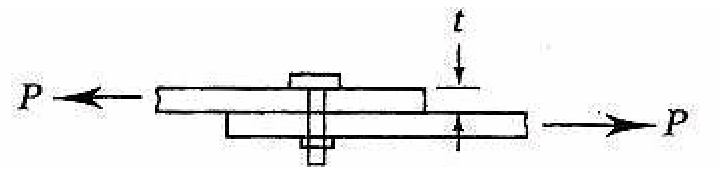


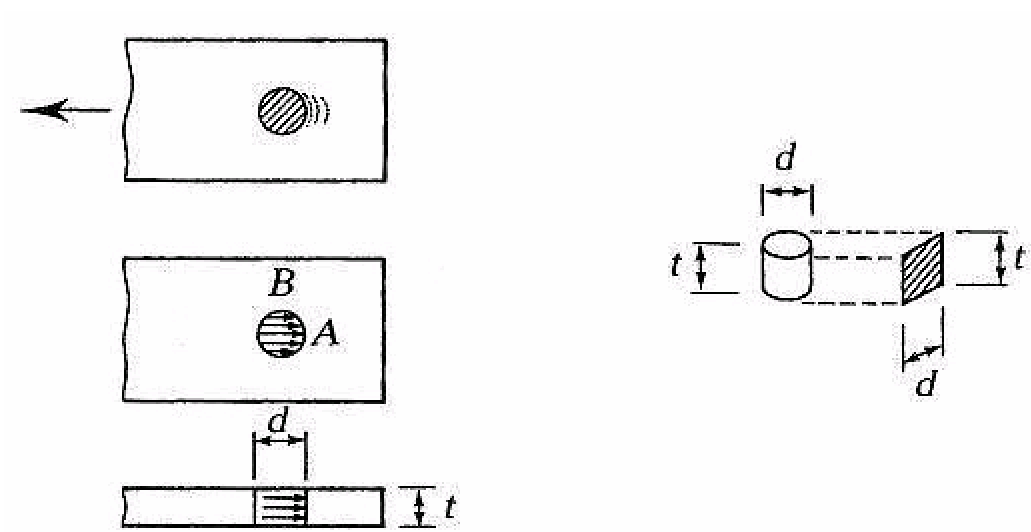
(a) Single Shear (b) Double Shear

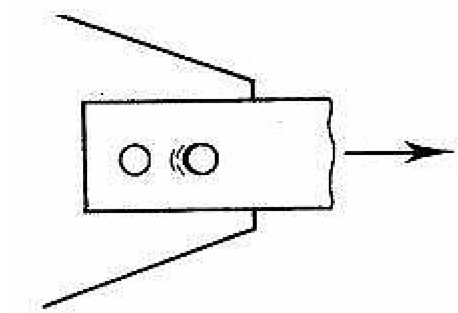
- Failure of connected member
  - We have covered this in detail in Ch. 2 on tension members
  - Member can fail due to tension fracture or block shear.
- Bearing failure of connected/connecting part due to bearing from bolt holes
  - Hole is slightly larger than the fastener and the fastener is loosely placed in hole
  - Contact between the fastener and the connected part over approximately half the circumference of the fastener
  - As such the stress will be highest at the radial contact point (A). However, the average stress can be calculated as the applied force divided by the projected area of contact
  - Average bearing stress $f_p = P/(d_b\, t)$, where P is the force applied to the fastener.
  - The bearing stress state can be complicated by the presence of nearby bolt or edge. The bolt spacing and edge distance will have an effect on the bearing str.
  - Bearing stress effects are independent of the bolt type because the bearing stress acts on the connected plate not the bolt.

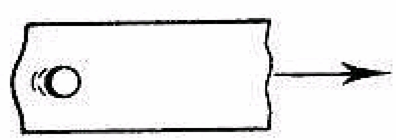
- A possible failure mode resulting from excessive bearing close to the edge of the connected element is shear tear-out as shown below. This type of shear tear-out can also occur between two holes in the direction of the bearing load.

  $R_n = 2 \times 0.6\ F_u\ L_c\ t = 1.2\ F_u\ L_c\ t$









- To prevent excessive deformation of the hole, an upper limit is placed on the bearing load. This upper limit is proportional to the fracture stress times the projected bearing area

  $R_n = C \times F_u \times$ bearing area $= C\ F_u\ d_b\ t$

  If deformation is not a concern then $C = 3$, If deformation is a concern then $C=2.4$

  $C = 2.4$ corresponds to a deformation of 0.25 in.

- Finally, the equation for the bearing strength of a single bolts is ϕR_n

  where, $\phi = 0.75$ and $R_n = 1.2\ L_c\ t\ F_u < 2.4\ d_b\ t\ F_u$

  L_c is the clear distance in the load direction, from the edge of the bolt hole to the edge of the adjacent hole or to the edge of the material

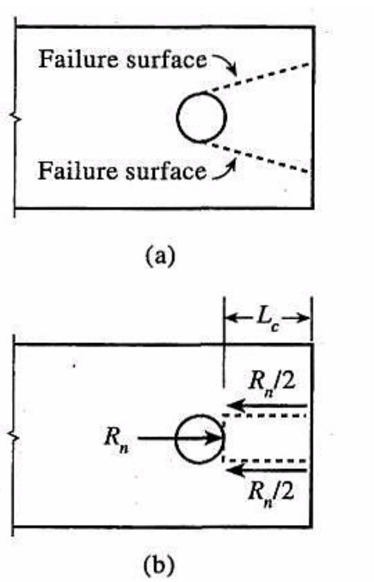
- This relationship can be simplified as follows:

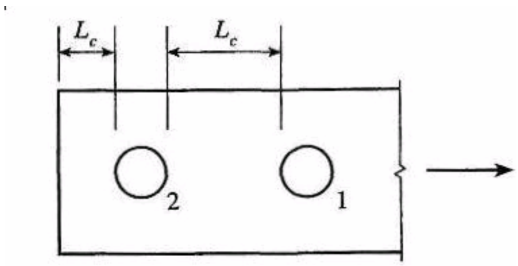
  The upper limit will become effective when $1.2\ L_c\ t\ F_u = 2.4\ d_b\ t\ F_u$

  i.e., the upper limit will become effective when $L_c = 2\ d_b$

  If $L_c < 2\ d_b$, $\quad R_n = 1.2\ L_c\ t\ F_u$

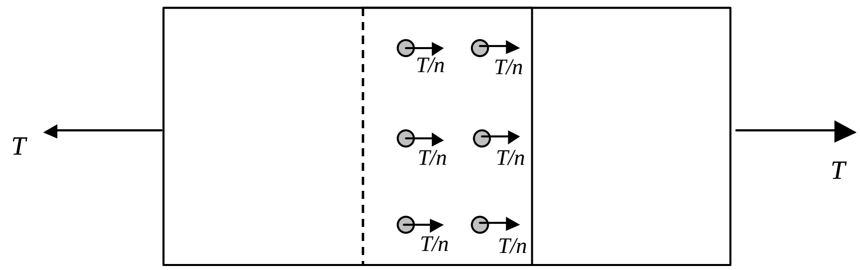
  If $L_c > 2\ d_b$, $\quad R_n = 1.4\ d_b\ t\ F_u$



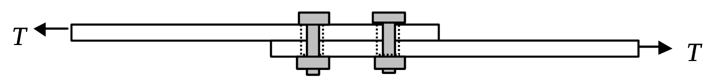


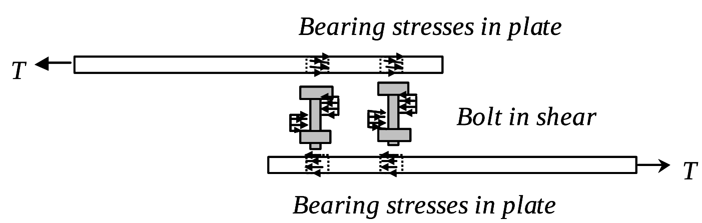
5.3 DESIGN PROVISIONS FOR BOLTED SHEAR CONNECTIONS

- In a simple connection, all bolts share the load equally.



- In a bolted shear connection, *the bolts are subjected to shear* and the connecting / connected plates are subjected to bearing stresses.





- The shear strength of all bolts = shear strength of one bolt x number of bolts
- The bearing strength of the connecting / connected plates can be calculated using equations given by AISC specifications.
- The tension strength of the connecting / connected plates can be calculated as discussed earlier in Chapter 2.

5.3.1 AISC Design Provisions

- Chapter J of the AISC Specifications focuses on connections.
- Section J3 focuses on bolts and threaded parts

- AISC Specification J3.3 indicates that the minimum distance (s) between the centers of bolt holes is $2\frac{2}{3}d_b$. A distance of $3d_b$ is preferred.
- AISC Specification J3.4 indicates that the minimum edge distance (L_e) from the center of the bolt to the edge of the connected part is given in Table J3.4 on page **16.1**-61. Table J3.4 specifies minimum edge distances for sheared edges, edges of rolled shapes, and gas cut edges.
- AISC Specification J3.5 indicates that the maximum edge distance for bolt holes is 12 times the thickness of the connected part (but not more than 6 in.). The maximum spacing for bolt holes is 24 times the thickness of the thinner part (but not more than 12 in.).
- Specification J3.6 indicates that the design tension or shear strength of bolts is $\phi F_n A_b$
  - Table J3.2 gives the values of ϕ and F_n
  - A_b is the unthreaded area of bolt.
  - In Table J3.2, there are different types of bolts A325 and A490.
  - The shear strength of the bolts depends on whether threads are included or excluded from the shear planes. If threads are included in the shear planes then the strength is lower.
  - We will always assume that threads are included in the shear plane, therefore less strength to be conservative.
- We will look at specifications J3.7 – J3.9 later.
- AISC Specification J3.10 indicates the bearing strength of plates at bolt holes.
  - The design bearing strength at bolt holes is ϕR_n
  - $R_n = 1.2\ L_c\ t\ F_u \leq 2.4\ d_b\ t\ F_u$ - deformation at the bolt holes is a design consideration
  - Where, F_u = specified tensile strength of the connected material

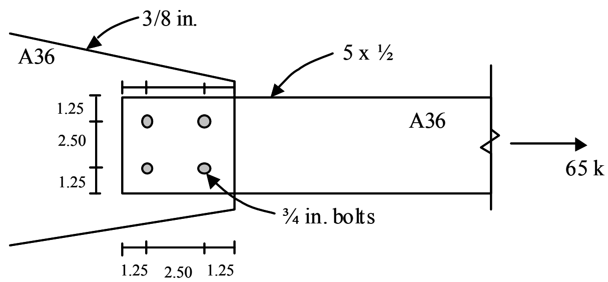
- L_c = clear distance, in the direction of the force, between the *edge* of the hole and the *edge* of the adjacent hole or edge of the material (in.).
- t = thickness of connected material

5.3.2 AISC Design Tables

- Table 7-10 on page 7-33 of the AISC Manual gives the design shear of one bolt. Different bolt types (A325, A490), thread condition (included or excluded), loading type (single shear or double shear), and bolt diameters (5/8 in. to 1-1/2 in.) are included in the Table.
- Table 7-11 on page 7-33 of the AISC Manual is an extension of Table 7-10 with the exception that it gives the shear strength of '*n*' bolts.
- Table 7-12 on page 7-34 of the AISC manual gives the design bearing strength at bolt holes for various bolt spacings.
    - These design bearing strengths are in kips/in. thickness.
    - The tabulated numbers must be multiplied by the plate thickness to calculate the design bearing strength of the plate.
    - The design bearing strengths are given for different bolt spacings ($2.67d_b$ and $3d_b$), different F_u (58 and 65 ksi), and different bolt diameters (5/8 – 1-1/2 in.)
    - Table 7-12 also includes the spacing (s_{full}) required to develop the full bearing strength for different F_u and bolt diameters
    - Table 7-12 also includes the bearing strength when $s > s_{full}$
    - Table 7-12 also includes the minimum spacing 2-2/3 d_b values
- Table 7-13 in the AISC manual on page 7-35 is similar to Table 7-12. It gives the design bearing strength at bolt holes for various edge distances.

- These design bearing strengths are in *kips/in. thickness*.
- The tabulated numbers must be multiplied by the plate thickness to calculate the design bearing strength of the plate.
- The design bearing strengths are given for different edge distances (1.25 in. and 2 in.), different F_u (58 and 65 ksi), and different bolt diameters (5/8 – 1-1/2 in.)
- Table 7-13 also includes the edge distance ($L_{e\ full}$) required to develop the full bearing strength for different F_u and bolt diameters
- Table 7-13 also includes the bearing strength when $L_e > L_{e\ full}$

Example 5.1 Calculate and check the design strength of the connection shown below. Is the connection adequate for carrying the factored load of 65 kips.



Solution

Step I. Shear strength of bolts

- The design shear strength of one bolt in shear = $\phi F_n A_b = 0.75 \times 48 \times \pi \times 0.75^2/4$
  - $\phi F_n A_b$ = 15.9 kips per bolt (See Table J3.2 and Table 7-10)
  - Shear strength of connection = 4 x 15.9 = 63.6 kips (See Table 7-11)

Step II. Minimum edge distance and spacing requirements

- See Table J3.4, minimum edge distance = 1 in. for rolled edges of plates
  - The given edge distances (1.25 in.) > 1 in. Therefore, minimum edge distance requirements are satisfied.
- Minimum spacing = 2.67 d_b = 2.67 x 0.75 = 2.0 in.
  - Preferred spacing = 3.0 d_b = 3.0 x 0.75 = 2.25 in.
  - The given spacing (2.5 in.) > 2.25 in. Therefore, spacing requirements are satisfied.

Step III. Bearing strength at bolt holes.

- Bearing strength at bolt holes in connected part (5 x ½ in. plate)
  - At edges, L_c = 1.25 – hole diameter/2 = 1.25 – (3/4 + 1/16)/2 = 0.844 in.
  - $\phi R_n = 0.75 \times (1.2 L_c t F_u) = 0.75 \times (1.2 \times 0.844 \times 0.5 \times 58)$ = 22.02 kips
  - But, $\phi R_n \leq 0.75 (2.4 d_b t F_u) = 0.75 \times (2.4 \times 0.75 \times 0.5 \times 58)$ = 39.15 kips
  - Therefore, ϕR_n = 22.02 kips at edge holes
    - *Compare with value in Table 7-13. ϕR_n = 44.0 x 0.5 = 22.0 kips*

- At other holes, s = 2.5 in, L_c = 2.5 – (3/4 +1/16) = 1.688 in.
- ϕR_n = 0.75 x (1.2 L_c t F_u) = 0.75 x (1.2 x 1.688 x 0.5 x 58) = 44.05 kips
- But, $\phi R_n \leq$ 0.75 (2.4 d_b t F_u) = 39.15 kips. Therefore ϕR_n = 39.15 kips
- Therefore, ϕR_n = 39.15 kips at other holes
    - *Compare with value in Table 7-12. ϕR_n = 78.3 x 0.5 =39.15 kips*
- Therefore, bearing strength at holes = 2 x 22.02 + 2 x 39.15 = 122.34 kips

- Bearing strength at bolt holes in gusset plate (3/8 in. plate)
    - At edges, L_c = 1.25 – hole diameter/2 = 1.25 – (3/4 + 1/16)/2 = 0.844 in.
    - ϕR_n = 0.75 x (1.2 L_c $\underline{t}$ F_u) = 0.75 x (1.2 x 0.844 x $\underline{0.375}$ x 58) = 16.52 k
    - But, $\phi R_n \leq$ 0.75 (2.4 d_b $\underline{t}$ F_u) = 0.75 x (2.4 x 0.75 x $\underline{0.375}$ x 58) = 29.36 kips
    - Therefore, ϕR_n = 16.52 kips at edge holes
        - *Compare with value in Table 7-13. ϕR_n = 44.0 x 3/8 = 16.5 kips*
    - At other holes, s = 2.5 in, L_c = 2.5 – (3/4 +1/16) = 1.688 in.
    - ϕR_n = 0.75 x (1.2 L_c $\underline{t}$ F_u) = 0.75 x (1.2 x 1.688 x $\underline{0.375}$ x 58) = 33.04 kips
    - But, $\phi R_n \leq$ 0.75 (2.4 d_b t F_u) = 29.36 kips
    - Therefore, ϕR_n = 29.36 kips at other holes
        - *Compare with value in Table 7-12. ϕR_n = 78.3 x 0.375 = 29.36 kips*
    - Therefore, bearing strength at holes = 2 x 16.52 + 2 x 29.36 = 91.76 kips

- Bearing strength of the connection is the smaller of the bearing strengths = 91.76 kips

<u>Connection Strength</u>
Shear strength = 63.3 kips
Bearing strength (plate) = 122.34 kips
Bearing strength (gusset) = 91.76 kips

Connection strength (ϕR_n) > applied factored loads (γQ). Therefore ok.

Example 5.2 Design a double angle tension member and a gusset plated bolted connection system to carry a factored load of 100 kips. Assume A36 (36 ksi yield stress) material for the double angles and the gusset plate. Assume A325 bolts. Note that you have to design the double angle member sizes, the gusset plate thickness, the bolt diameter, numbers, and spacing.

Solution

Step I. Design and select a trial tension member

- See **Table 3-7** on page 3-33 of the AISC manual.
    - Select 2*L* 3 x 2 x 3/8 with ϕP_n = 113 kips (yielding) and 114 kips (fracture)
    - While selecting a trial tension member check the fracture strength with the load.

Step II. Select size and number of bolts

The bolts are in double shear for this design (may not be so for other designs)

- See **Table 7-11** on page 7-33 in the AISC manual

  Use four 3/4 in. A325 bolts in double shear

  ϕR_n = 127 kips - shear strength of bolts from Table 7-11

Step III. Design edge distance and bolt spacing

- **See Table J3.4**
    - The minimum edge distance = 1 in. for 3/4 in. diameter bolts in rolled edges.
    - Select edge distance = 1.25 in.
- **See specification J3.5**
    - Minimum spacing = 2.67 d_b = 2.0 in.
    - Preferred spacing = 3.0 d_b = 2.25 in.
    - Select spacing = 3.0 in., which is greater than preferred or minimum spacing

Step IV. Check the bearing strength at bolt holes in angles

- Bearing strength at bolt holes in angles
  - Angle thickness = 3/8 in.
  - See **Table 7-13** for the bearing strength per in. thickness at the edge holes
  - Bearing strength at the edge holes (L_e = 1.25 in.) = ϕR_n = 44.0 x 3/8 = 16.5 k
  - See **Table 7-12** for the bearing strength per in. thickness at non-edge holes
  - Bearing strength at non-edge holes (s = 3 in.) = ϕR_n = 78.3 x 3/8 = 29.4 k
  - *Bearing strength at bolt holes in each angle = 16.5 + 3 x 29.4 = 104.7 kips*
  - *Bearing strength of double angles = 2 x 104.7 kips = 209.4 kips*

Step V. Check the fracture and block shear strength of the tension member

- *This has been covered in the chapter on tension members and is left to the students.*

Step VI. Design the gusset plate

- See **specification J5.2** for designing gusset plates. These plates must be designed for the limit states of yielding and rupture
  - Limit state of yielding
    - $\phi R_n = 0.9\ A_g\ F_y > 100$ kips
    - Therefore, $A_g = L \times t > 3.09\ in^2$
    - Assume t = ½ in; Therefore L > 6.18 in.
    - Design gusset plate = 6.5 x ½ in.
    - Yield strength = ϕR_n = 0.9 x 6.5 x 0.5 x 36 = 105.3 kips
  - Limit state for fracture

- $A_n = A_g - (d_b+1/8) \times t$
- $A_n = 6.5 \times 0.5 - (3/4 + 1/8) \times 0.5 = 2.81\ in^2$
- **But, $A_n \leq 0.85\ A_g = 0.85 \times 3.25 = 2.76\ in^2$**
- $\phi R_n = 0.75 \times A_n \times F_u = 0.75 \times \mathbf{2.76} \times 58 = 120$ kips

- Design gusset plate = 6.5 x 0.5 in.

- **Step VII.** Bearing strength at bolt holes in gusset plates

Assume $L_e = 1.25$ in. (same as double angles)

- Plate thickness = 3/8 in.
- Bearing strength at the edge holes = $\phi R_n = 44.0 \times 1/2 = 22.0$ k
- Bearing strength at non-edge holes = $\phi R_n = 78.3 \times 1/2 = 39.15$ k
- *Bearing strength at bolt holes in gusset plate = 22.0 + 3 x 39.15 = 139.5 kips*

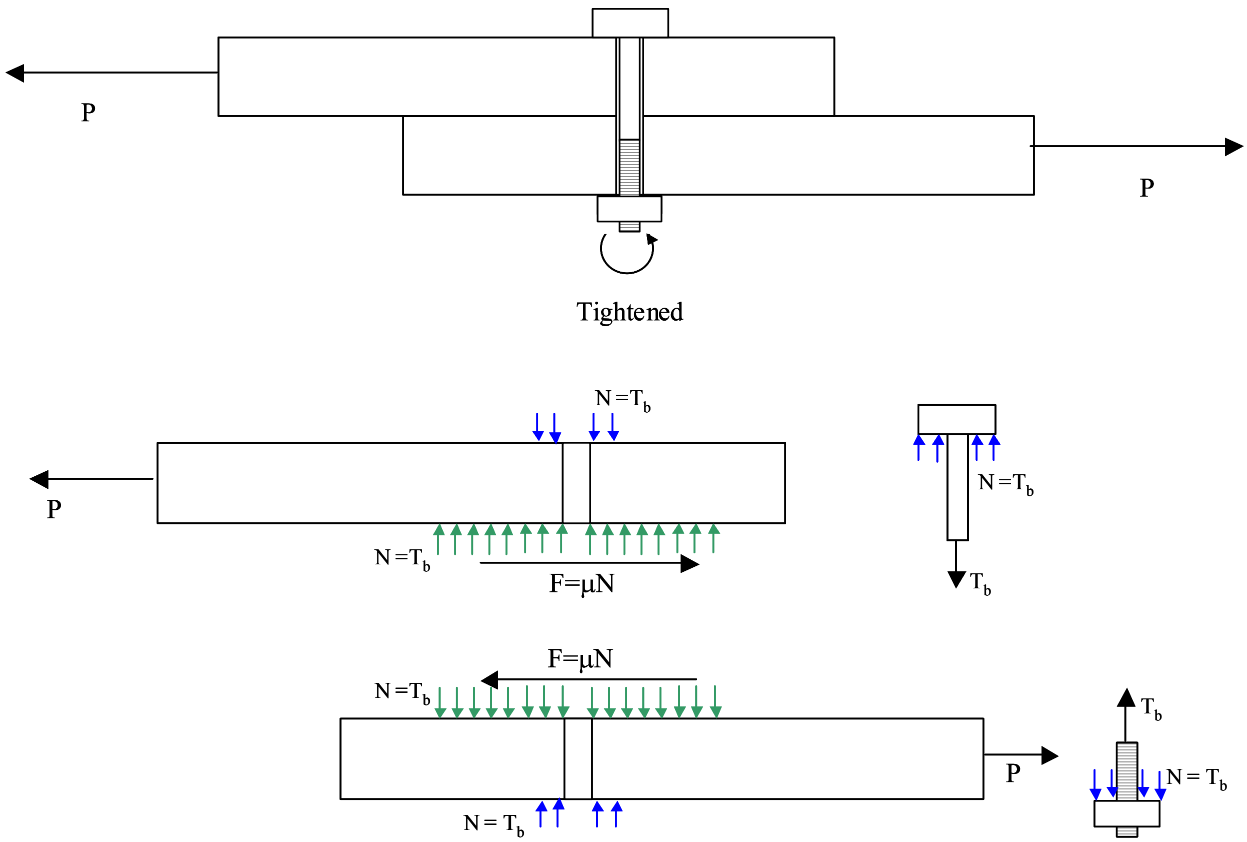
Summary of Member and Connection Strength

Connection	Member	Gusset Plate
Shear strength = 127 kips	Yielding = 113 kips	Yielding = 105.3 kips
Bearing strength = 209.4 kips (angles)	Fracture = ?	Fracture = 120 kips
Bearing Strength = 139.5 (gusset)	Block Shear = ?	

- Overall Strength is the smallest of all these numbers = 105.3 kips
- Gusset plate yielding controls
- Resistance > Factored Load (100 kips).
- Design is acceptable

5.4 SLIP-CRITICAL BOLTED CONNECTIONS

- High strength (A325 and A490) bolts can be installed with such a degree of tightness that they are subject to large tensile forces.
- These large tensile forces in the bolt clamp the connected plates together. The shear force applied to such a tightened connection will be resisted by friction as shown in the Figure below.



- Thus, *slip-critical bolted connections* can be designed to resist the applied shear forces using friction. If the applied shear force is less than the friction that develops between the two surfaces, then no slip will occur between them.
- However, slip will occur when the friction force is less than the applied shear force. After slip occurs, the connection will behave similar to the bearing-type bolted connections designed earlier.

- Table J3.1 summarizes the minimum bolt tension that must be applied to develop a slip-critical connection.

- The shear resistance of fully tensioned bolts to slip at factored loads is given by AISC Specification J3.8 a

  Shear resistance at factored load = $\phi R_n = 1.13\ \mu\ T_b\ N_s$

  where, ϕ = 1.0 for standard holes

  μ = 0.33 (Class A surface with unpainted clean mill scale surface: CE 405)

  T_b = minimum bolt tension given in Table J3.1

  N_s = number of slip planes

  - See Table 7-15 on page 7-36 of the AISC manual. This Table gives the shear resistance of fully tensioned bolts to slip at factored loads on class A surfaces.
  - For example, the shear resistance of 1-1/8 in. bolt fully tensioned to 56 kips (Table J3.1) is equal to 20.9 kips (Class A faying surface).
  - When the applied shear force exceeds the ϕR_n value stated above, slip will occur in the connection.

- The shear resistance of fully tensioned bolts to slip at service loads is given by AISC Specification J3.8 b.

  - Shear resistance at service load = $\phi R_n = \phi\ F_v\ A_b$
  - Where, ϕ = 1.0 for standard holes

    F_v = slip-critical resistance to shear at service loads, see **Table A-J3.2** on page 16.1-116 of the AISC manual
  - See Table 7-16 on page 7-37 of the AISC manual. This Table gives the shear resistance of fully tensioned bolts to slip at service loads on class A surfaces.

- For example, the shear resistance of 1-1/8 in. bolt fully tensioned to 56 kips (Table J3.1) is equal to 16.9 kips (Class A faying surface).
- When the applied shear force exceeds the ϕR_n value stated above, slip will occur in the connection.

* The final strength of the connection will depend on the shear strength of the bolts calculated using the values in Table 7-11 and on the bearing strength of the bolts calculated using the values in Table 7-12, 7-13. This is the same strength as that of a bearing type connection.

Example 5.3 Design a slip-critical splice for a tension member subjected to 300 kips of tension loading. The tension member is a W8 x 28 section made from A992 (50 ksi) material. The unfactored dead load is equal to 50 kips and the unfactored live load is equal to 150 kips. Use A325 bolts. The splice should be slip-critical at service loads.

<u>Solution</u>

Step I. <u>Service and factored loads</u>

- Service Load = D + L = 200 kips.
- Factored design load = 1.2 D + 1.6 L = 300 kips
- Tension member is W8 x 28 section made from A992 (50 ksi) steel. The tension splice must be slip critical (i.e., it must not slip) at service loads.

Step II. <u>Slip-critical splice connection</u>

- ϕR_n of one fully-tensioned slip-critical bolt = $\phi\ F_v\ A_b$ (See Spec. **A-J3.8 b**) *page 16.1-117 of AISC*
- If d_b = 3/4 in.

  ϕR_n of one bolt = $1.0 \times 17 \times \pi \times 0.75^2/4 = 7.51$ kips

  Note, F_v = 17 ksi from Table A-J3.2

  From **Table 7-16** on page **7-37** ϕRn = 7.51 kips

  ϕR_n of n bolts = 7.51 x n > 200 kips (splice must be slip-critical at service)

  Therefore, n > 26.63

- If d_b = 7/8 in.

  ϕR_n of one bolt = 10.2 kips -from **Table 7-16**

  ϕR_n of n bolts = 10.2 x n > 200 kips (splice must be slip-critical at service)

  Therefore, n > 19.6 bolts

Example 5.3

Step I: Service and Factored Loads

$D := 50$ Kips $\qquad L := 150$ Kips

- Service Loads $\qquad P_s := D + L \qquad\qquad P_s = 200$ Kips

- Factored Loads $\qquad P_u := 1.2 \cdot D + 1.6 \cdot L \qquad\qquad P_u = 300$ Kips

Step II: Slip Critical connection

- In Service loads consideration, ϕR_n of one fully tenstioned slip-critical bolt = $\phi\, F_v\, A_b$

  (As given in Spec. A-J3.8b - page 16.1-117) $\qquad \phi := 1.0 \qquad F_v := 17$ Ksi - A325 - Table A-J3.2

- If diameter of the bolt = $d_b := \frac{3}{4}$ in $\qquad A_b := \frac{\pi}{4}\left(d_b\right)^2$

  for one bolt $\qquad \phi R_n := \phi \cdot F_v \cdot A_b \qquad\qquad \phi R_n = 7.51$ Kips

  Number of bolts required $\qquad n := \frac{P_s}{\phi R_n} \qquad\qquad n = 26.63$ (min. reqd.)

- If diameter of the bolt = $d_b := \frac{7}{8}$ in $\qquad A_b := \frac{\pi}{4}\left(d_b\right)^2$

  for one bolt $\qquad \phi R_n := \phi \cdot F_v \cdot A_b \qquad\qquad \phi R_n = 10.22$ Kips

  Number of bolts required $\qquad n := \frac{P_s}{\phi R_n} \qquad\qquad n = 19.565$ (min. reqd.)

  say we provide **24 bolts** on either side of the center line, 6 on either side of the flanges, top + bottom

Step III: Connection Details and spacings for 24 bolts on each W8 x 28

- Note that there are 24 bolts on either side of the center line. In all there are 48 number - 7/8 in dia bolts used in the connection.

- Minimum pretension applied to the bolts = 39.0 Kips from Table J3.1

- Minimum Edge distance from Table J3.4 = $L_{e\text{-}min}$ = 1.125 in

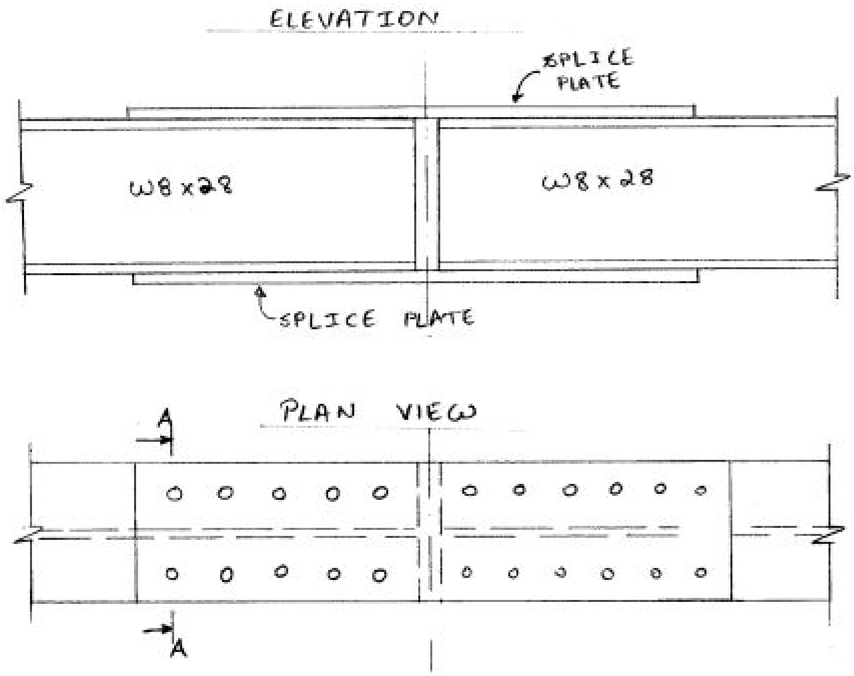
- Provide Edge Distance = $L_e := 1.25$ in

- Minimum spacing (Spec. J3.3) = $s := 2.67 \cdot d_b \qquad\qquad s = 2.336$ in

Preferred spacing = $s := 3 \cdot d_b$ $\qquad s = 2.625$ in

From table 7-12 $\quad s_{full} := 2.6875$ in

For design provide spacing = $s := 3$ in



DETAILS OF CONNECTION USING 7/8 DIA BOLTS.

Step IV: Connection Strength at factored loads

- The splice connection should be designed as a normal shear / bearing connection beyong this point for the factored load = 300 kips
- The shear strength of the bolts (Table 7-10) = 21.6 kips/bolt x 24 bolts = 518.4 Kips
- Bearing strength of 7/8 in. bolt at edge holes = $B_e := 45.7$ Kip / in thickness $\quad$ Table 7-13
- Bearing strength of 7/8 in. bolt at other holes = $B_o := 102$ Kip / in thickness $\quad$ Table 7-12
- Total bearing strength of the bolt holes in wide flange section $\quad B_t := 4 \cdot B_e + 20 \cdot B_o$

$$B_t = 2.223 \times 10^3 \text{ Kips}$$

Step V: Design the splice plate $\quad F_y := 50$ Ksi $\quad F_u := 65$ Ksi $\quad P_u = 300$ Kips

- Tension Yielding = $0.9\ A_g\ F_y > P_u$ $\quad minA_g := \dfrac{P_u}{0.9 \cdot F_y}$ $\quad minA_g = 6.667 \quad in^2$

- Tension Fracture = $0.75\ A_n\ F_u > P_u$ $\quad minA_n := \dfrac{P_u}{0.75 \cdot F_u}$ $\quad minA_n = 6.154 \quad in^2$

We know, flange width of W 8 x 28 = 6.54 in. This is the limiting width of the splice plate. The unknown quantity which is the thickness of each splice plate is calculated as shown.

Net area = Gross area - area of the bolts $\quad A_n := minA_n$

$A_n := A_g - 4 \cdot \left(\frac{7}{8} + \frac{1}{8}\right) \cdot t$ $\quad$ Here, $\quad A_g := 6.54 \cdot t$ $\quad A_n := 6.154 \quad in^2$

Solving for t, we get $\quad t_{min} := 2.42 \ \ in$ $\quad$ (This is the total thickness of the plate at the top and bottom)

Assume each plate of the dimensions $\quad \boxed{b := 6.54}\ \boxed{in} \quad \boxed{t_p := 1.25}\ \boxed{in}$

therefore, total plate thickness = $\quad t := 2 \cdot t_p$ $\quad t = 2.5 \quad in \quad > \quad t_{min} = 2.42 \quad in$

Check for A_n and A_g:

$A_g := b \cdot t$ $\quad A_g = 16.35 \quad in^2 \quad > \quad minA_g = 6.667 \quad in^2$

$A_n := A_g - 4 \cdot \left(\frac{7}{8} + \frac{1}{8}\right) \cdot t$ $\quad A_n = 6.35 \quad in^2 \quad > \quad minA_n = 6.154 \quad in^2$

Check $\quad A_n = 6.35 \quad in^2 \quad < \quad 0.85 \cdot A_g = 13.898 \quad in^2$

- Strength of the splice plate in

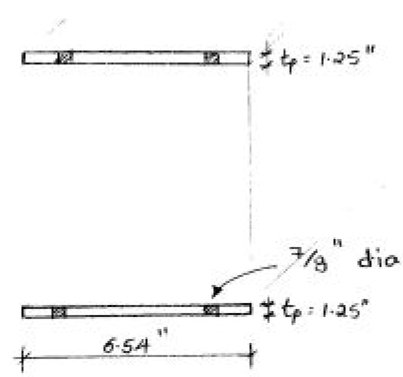
  yielding = $\quad 0.9 \cdot A_g \cdot F_y = 735.75 \quad$ Kips

  $> \quad P_u = 300 \quad$ Kips

  fracture = $\quad 0.75 \cdot A_n \cdot F_u = 309.563 \quad$ Kips

- Check for bearing strength of the splice plates

- Check for block shear rupture



<u>Step VI : Check member yield, fracture and block shear....</u>

Example 4.4 Modify Example 4.2 so that the connection system is slip critical for the factored load of 100 kips.

Solution

Step I. Design and select a trial tension member (same as **example 4.2**)

- Select 2*L* 3 x 2 x 3/8 with ϕP_n = 113 kips (yielding) and 114 kips (fracture)

Step II. Select size and number of bolts (modified step)

- The connection must be designed to be slip-critical at the factored loads
    - ϕR_n for one bolt = 1.0 x 1.13 x μ x T_b x N_s (T_b from **Table J3.1**)
    - ϕR_n for one 3/4 in. bolt = 1.0 x 1.13 x 0.33 x 28 x 2 = 20.9 kips
    - ϕR_n for one 7/8 in. bolt = 1.0 x 1.13 x 0.33 x 39 x 2 = 29.1 kips
    - **See Values in Table 7-15**.

      ϕR_n for ¾ and 7/8 in. bolts in double slip = 20.9 and 29.1 kips, respectively.
    - We need at least five ¾ in. bolts to have strength ϕR_n = 5 x 20.9 = 104.5 k > 100 k
    - We need at least four 7/8 in. bolts to have strength ϕR_n = 4 x 29.1 = 116.4 k> 100
- **Use five ¾ in. fully tightened bolts**. Bolts must be tightened to 28 kips.
- Compare with solution for example 4.2 where only four snug-tight ¾ in bolts design.

For the remaining steps III to VII follow Example 4.2

Step III. Design edge distance and bolt spacing

Step IV. Check the bearing strength at bolt holes in angles

Step V. Check the fracture and block shear strength of the tension member

Step VI. Design the gusset plate

Step VII. Bearing strength at bolt holes in gusset plates

Summary of Member and Connection Strength