

Sample calculations – connection 35

All the following calculations and assumptions were based on estimates from the field example.

Assumptions:

1. Use W18X46 for beam
2. W18X46 uses A992 steel by table 2-3
3. Use L4X4X3/8 for angle
4. L4X4X3/8 uses A36 steel by table 2-3
5. Assume A325N bolts
6. Assume bolts are 3/4 in diameter
7. Assume 1/8 in tolerance
8. Assume no deformation for bolt bearing
9. Assume L4X4X3/8 controls for bolt bearing with $F_u = 58$ ksi for A36 steel versus $F_u = 65$ ksi for A992 steel

To calculate bolt bearing, we followed equation (J3-6a). The angles were assumed to be A36 steel with a yield strength of 36 ksi, and an ultimate strength of 58 ksi. The angles used in the connections are L4 x 4 x 3/8. Deformation around the bolt hole was used as a design consideration. The following calculations were used to determine the bearing capacity of the angles.

$$\text{(Equation J3-6a) } \Phi R_n = 0.75 * \min \left\{ \begin{array}{l} 1.2 * L_c * t * F_u \\ 2.4 * d * t * F_u \end{array} \right\}$$

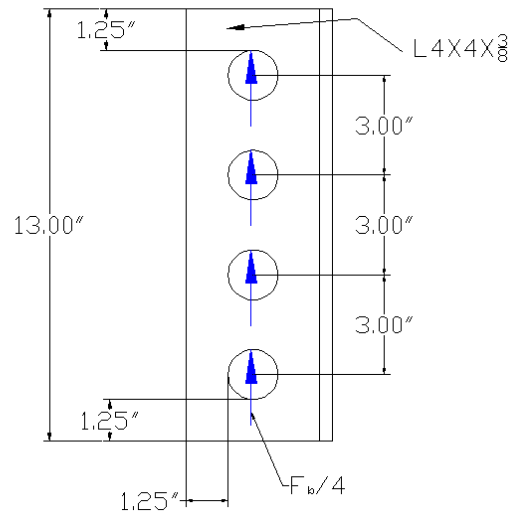
L_c = Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, in inches.

t = thickness of the connected material, in inches

F_u = ultimate tensile strength of the connected material, in inches

d = nominal bolt diameter

Bolt Bearing FBD



By table J3.4 $L_{e \min} = 1 \frac{1}{4} \text{ in}$ $L_{e \max} = (1.5(d_{\text{bolt}}), L_e)$ $L_e = 1 \frac{1}{4} \text{ in}$

Edge Bolts: $L_c = 1 \frac{1}{4} \text{ in}$

$$\phi R_{n, \text{edge}} = \min \begin{cases} (1.2) \left(1 \frac{1}{4} \text{ in} \right) \left(\frac{3}{8} \text{ in} \right) (58 \text{ ksi}) = 32.63 \text{ k} \\ (2.4) \left(\frac{3}{4} \text{ in} \right) \left(\frac{3}{8} \text{ in} \right) (58 \text{ ksi}) = 39.15 \text{ k} \end{cases} \quad \phi R_{n, \text{edge}} = 21.21 \text{ k}$$

Interior Bolts: $L_c = 3 \text{ in} - 1 \left(\frac{3}{4} \text{ in} + \frac{1}{8} \text{ in} \right) = 2 \frac{1}{8} \text{ in}$

$$\phi R_{n, \text{interior}} = \min \begin{cases} (1.2) \left(2 \frac{1}{8} \text{ in} \right) \left(\frac{3}{8} \text{ in} \right) (58 \text{ ksi}) = 55.46 \text{ k} \\ (2.4) \left(2 \frac{1}{8} \text{ in} \right) \left(\frac{3}{8} \text{ in} \right) (58 \text{ ksi}) = 110.93 \text{ k} \end{cases} \quad \phi R_{n, \text{interior}} = 55.46 \text{ k}$$

$$\phi R_n = \phi [(\# \text{ edges})(R_{n, \text{edge}}) + (\# \text{ interiors})(R_{n, \text{interior}})]$$

$$\frac{\phi R_n}{2} = 0.75 [(1)(32.63 \text{ k}) + (3)(55.46 \text{ k})]$$

$\phi R_n = 298.52 \text{ k}$ for both angles

$P_u \leq 298.52 \text{ k}$ for Bolt Bearing

To calculate bolt shear, we followed equation (J3-1). The bolts used in the connection are 3/4" A325N bolts. The A325N bolts have a yield strength of 48 ksi. The following calculations were used to determine the shear rupture capacity of the bolts.

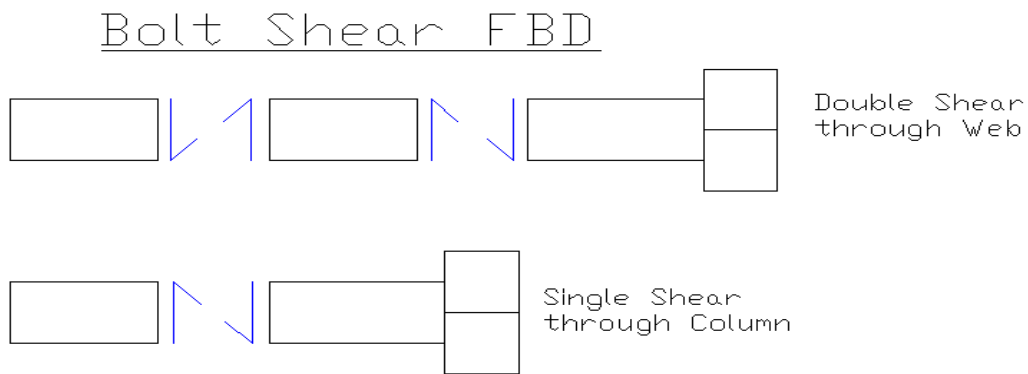
(Equation J3-1) $\Phi R_n = 0.75 * F_{nv} * A_b * n * N$

F_n = nominal tensile or shear stress from table J3.2, ksi

A_b = nominal area of each bolt

n = number of shear planes

N = number of bolts



By table J3.2: $F_{nv} = 48\text{ksi}$ for A325N bolts

For W18X46 Web: $N = 4$ per angle $n = 2$ for web

For Column: $N = 4$ per angle $n = 1$ for column

$$\frac{\phi R_n}{2} = (0.75)(21.21k)(2)(4) = 86.12k \quad \mathbf{P_u \leq 172.24k \text{ for Bolt Shear for Web}}$$

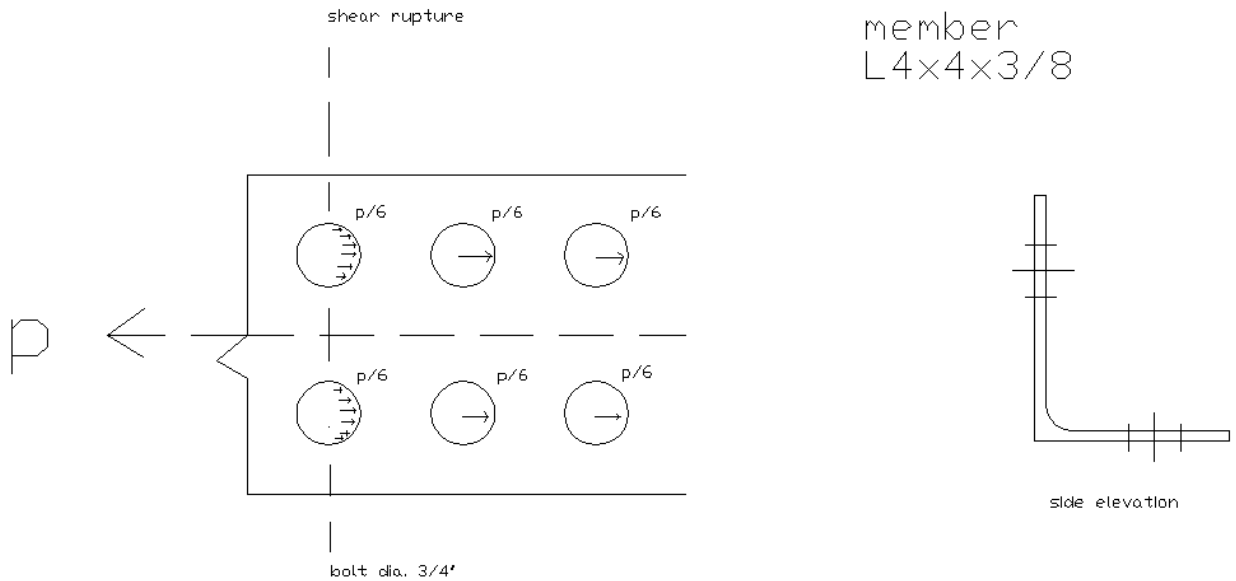
$$\frac{\phi R_n}{2} = (0.75)(21.21k)(1)(4) = 63.63k \quad \mathbf{P_u \leq 127.26k \text{ for Bolt Shear in Column}}$$

To calculate shear rupture, we followed equation (J4-4). The angles were assumed to be A36 steel with a yield strength of 36 ksi, and an ultimate strength of 58 ksi. The angles used in the connections are L4 x 4 x 3/8. The following calculations were used to determine the bearing capacity of the angles.

(Equation J4-4) $\Phi R_n = 0.75 * 0.6 * F_u * A_{nv}$

F_u = ultimate strength of steel, ksi

A_{nv} = net area subject to shear, in²



$$\frac{\Phi R_n}{2} = 0.75 * 38.625 \text{ kips} = 28.971 \text{ kips per angle}$$

$$P_u \leq 57.963 \text{ kips}$$

To calculate shear yielding, we followed equation (J4-3). The angles were assumed to be A36 steel with a yield strength of 36 ksi, and an ultimate strength of 58 ksi. The angles used in the connections are L4 x 4 x 3/8. The following calculations were used to determine the bearing capacity of the angles.

$$\text{(Equation J4-3) } \Phi R_n = 0.60 * F_y * A_g$$

F_y = yield strength of steel, ksi

A_g = gross area subject to shear, in²

$$\frac{\Phi R_n}{2} = 0.6 * (36 \text{ ksi}) * (2.86 \text{ in}^2) = 61.776 \text{ kips per angle}$$

$$P_u \leq 123.550 \text{ kips}$$

The maximum capacity of the connection is controlled by the shear rupture of the angle section because it has the lowest capacity (P_u) compared to the other limit states. This estimated maximum capacity of the field connection at the Bresnan Arena is $P_u = 57.963 \text{ kip}$.