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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 ¾ in diameter
7. Assume 1/8 in tolerance
8. Assume no deformation for bolt bearing
9. Assume L4X4X3/8 controls for bolt bearing with Fu =58ksi for A36 steel versus Fu = 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.

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

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

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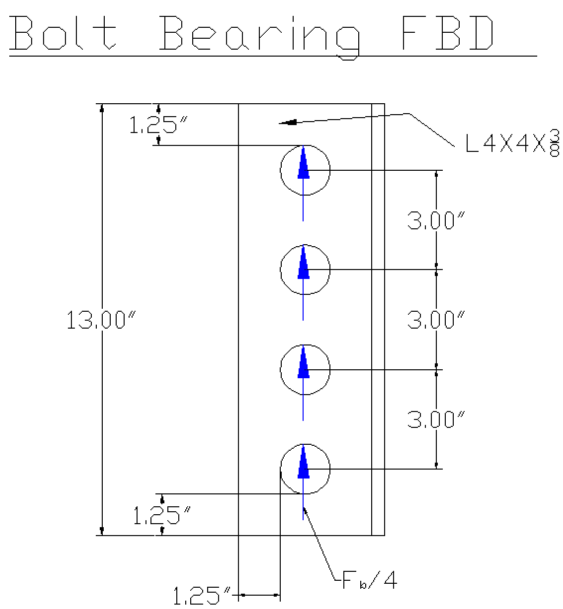

By table J3.4 $L_{e\,min}$=1 1/4in $L_{e\,max}$= (1.5(d_{bolt}), L_e) Le = 1 1/4in

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

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

$\phi R_{n,edge} = 21.21$k

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

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

$\phi R_{n,interior} = 55.46$k

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

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

$\phi R_n = 298.52k$ for both angles **$P_u \leq$ 298.52k for Bolt Bearing**

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To calculate bolt shear, we followed equation (J3-1). The bolts used in the connection are ¾" 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.

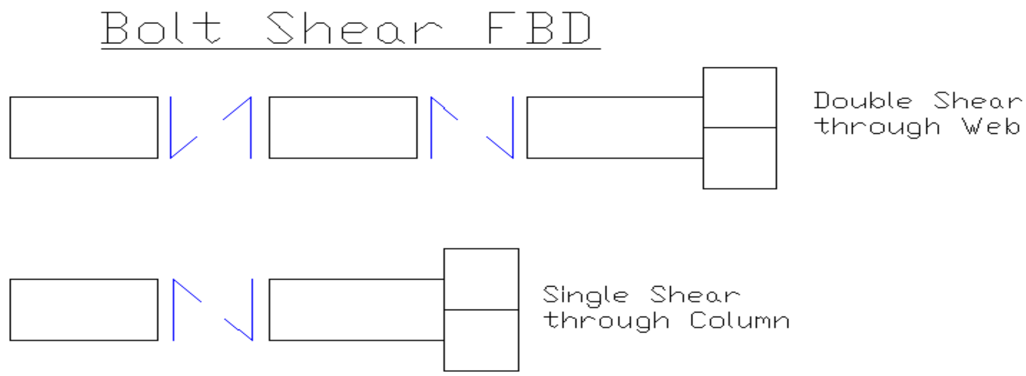
(Equation J3-1) $\Phi R_n = 0.75 * F_{nv} * A_b * n * N$

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} = 48ksi for A325N bolts

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

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

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

$$\frac{\phi R_n}{2} = (0.75)(21.21k)(1)(4) = 63.63k$$ **$P_u \leq$ 127.26k 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, in^2

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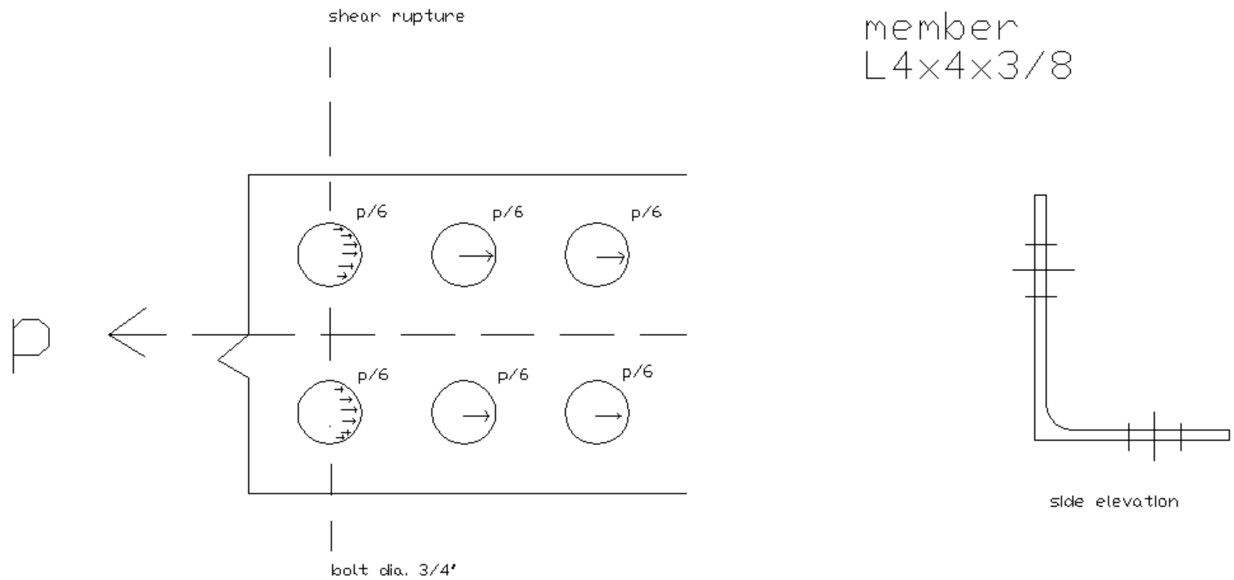

$\frac{\Phi R_n}{2} = 0.75*38.625\ kips = 28.971\ kips$ per angle

$$\mathbf{P_u \leq 57.963}\ \boldsymbol{kips}$$

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.

(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, in^2

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

$$\mathbf{P_u \leq 123.550}\ \boldsymbol{kips}$$

The maximum capacity of the connection is controlled by the shear rupture of the angle section because is 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 kip.