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 = 58 ksi 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) \( \Phi R_u = 0.75 \times \min \left\{ \frac{1.2 \times L_c \times t \times F_u}{2.4 \times d \times t \times F_u} \right\} \)

\( L_c = \) 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
By table J3.4  \( L_e \text{ min} = 1/4\text{in} \)  \( L_e \text{ max} = (1.5(d_{bolt}), L_e) \)  \( L_e = 1 \ 1/4\text{in} \)

Edge Bolts:  \( L_e = \frac{3}{4} \text{in} \)

\[
\phi R_{n,\text{edge}} = \min \left\{ (1.2) \left( \frac{1}{4} \text{in} \right) \left( \frac{3}{8} \text{in} \right) (58 \text{ksi}) = 32.63k \right\}
\]

\[
\phi R_{n,\text{edge}} = 21.21k
\]

Interior Bolts:  \( L_e = 3\text{in} - \frac{1}{4} \left( \frac{3}{4} \text{in} + \frac{1}{8} \text{in} \right) = \frac{27}{8} \text{in} \)

\[
\phi R_{n,\text{interior}} = \min \left\{ (1.2) \left( \frac{7}{8} \text{in} \right) \left( \frac{3}{8} \text{in} \right) (58 \text{ksi}) = 55.46k \right\}
\]

\[
\phi R_{n,\text{interior}} = 55.46k
\]

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

\[
\frac{\phi R_{n}}{2} = 0.75 \left[ (1)(32.63k) + (3)(55.46k) \right]
\]

\[
\phi R_{n} = 298.52k \text{ for both angles}
\]

\( P_n \leq 298.52k \text{ for Bolt Bearing} \)
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: \( F_{nv} = 48 \text{ksi} \) for A325N bolts

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

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

\[ \frac{\phi R_n}{2} = (0.75)(21.21k)(2)(4) = 86.12k \] \( P_u \leq 172.24k \) 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) \[ \Phi R_v = 0.75 * 0.6 * F_u * A_{nv} \]

\( F_u \) = ultimate strength of steel, ksi
\( A_{nv} \) = net area subject to shear, \( \text{in}^2 \)
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) \[ \Phi R_n = 0.60 \times F_y \times A_g \]

\[ F_y = \text{yield strength of steel, ksi} \]
\[ A_g = \text{gross area subject to shear, in}^2 \]

\[ \frac{\Phi R_n}{2} = 0.75 \times 38.625 \text{ kips} = 28.971 \text{ kips per angle} \]

\[ P_u \leq 57.963 \text{ kips} \]

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 \( P_u = 57.963 \text{ kip.} \)