Superior Panel Base Anchor PBA-S Calculations for In-Plane and Out-of-Plane Seismic Shear Capacity

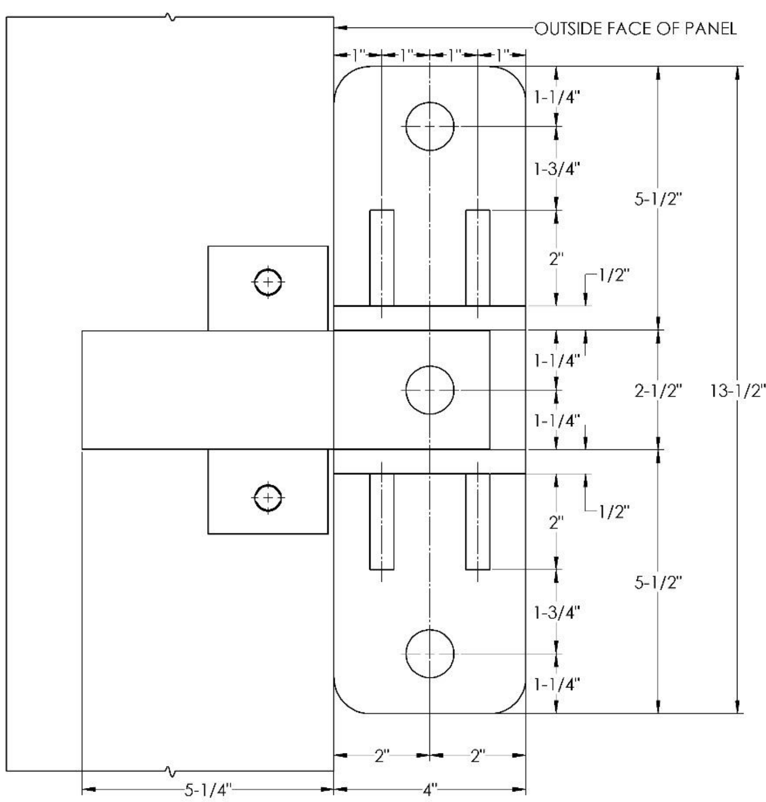
Scope: Determine in-plane shear capacity of the panel base anchor using the shear connector to meet the relevant provisions of ACI 318-19, Chapter 17, Section 17.10 – Earthquake-resistant anchor design requirements and Chapter 18, Section 18.5 – Intermediate precast structural walls and Section 18.11 – Special structural walls constructed using precast concrete. This will demonstrate that the shear anchor can be used to anchor precast concrete wall panels to the foundation to resist seismic design shear forces for structures assigned to Seismic Design Category (SDC) C, D, E or F.

See attached drawing DS-1 for details.

Material properties: Panel concrete: f'_c = 4000 psi

Anchor insert bar: ASTM A536 Grade 80-55-6, f_y = 55 ksi

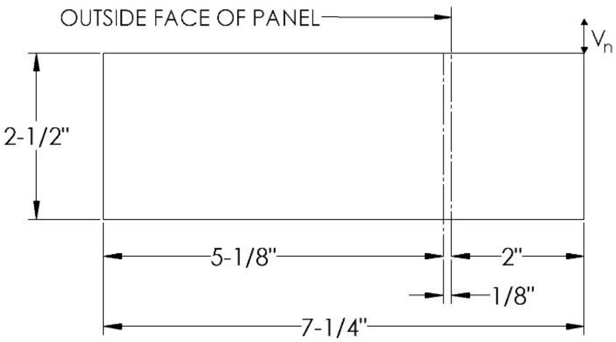
Shear Anchor: ASTM A536 Grade 80-55-6, f_y = 55 ksi



In-plane shear is resisted by the anchor insert bar bearing against the panel concrete and the shear anchor creating a moment couple.

Determine the controlling shear capacity of the connection by calculating the capacities of the individual components.

Calculate Vc = Nominal strength of the section controlled by concrete, lb.



Per the PCI Design Handbook, 7th Edition, Structural-Steel Corbels

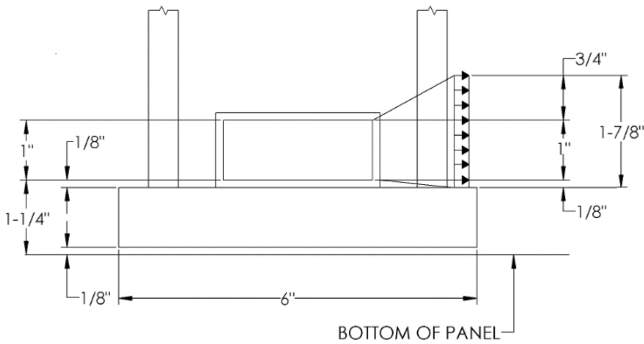
$V_c = \left(\frac{0.85 f_c' b l_e}{1+3.6e/l_e}\right) \qquad e = a + \frac{l_e}{2}$

$a = \text{shear span, in} \quad l_e = \text{embedment length, in}$

$b = \text{effective width of compression block}$

Typically $b = 2.5w$; $w = \text{width of steel corbel}$

Because the insert bar sits directly above the bearing bar, at the outside face of the panel, the effective bearing width is reduced to $(1.5/2\ in + \ 1/8\ in\ +\ 1\ in\ )\ b_w \rightarrow\ b_{eff} = 1.875(1\ in) = 1.875\ in$



$a = 2.125\ in \quad l_e = 5.125\ in \quad b = 1.875\ in$

$e = 2.125\ in + 5.125\ in\ /\ 2 = 4.6875\ in$

$V_c = \left(\frac{0.85(4000\ psi)(1.875\ in)(5.125\ in)}{1+3.6(4.6875\ in)/5.125\ in}\right) = 7{,}611\ lb$

Set $V_c = V_n = 7.6\ kips \rightarrow\ V_u = 0.75(7.61\ kips) = 5.7\ kips$

Anchor Insert Bar Strength: $z = bd^2\ /\ 4 = 1\ in\ (2.5\ in)^2 /\ 4 = 1.5625\ in^3$

Nominal steel flexural design strength: $V_n = \left(\frac{ZF_y}{a+\frac{0.5V_n}{0.85f_c'b}}\right) = \left(\frac{(1.5625\ in)(55\ ksi)}{2.125\ in+\frac{0.5(7.6\ kips)}{0.85(4\ ksi)(1.875\ in)}}\right) = \left(\frac{85.94\ kip\text{-}in}{2.72\ in}\right)$

$V_n = 31.6\ kips > 7.6\ kips; \qquad \varphi V_n = 0.9(31.6) kips = 28.4\ kips > 5.7\ kips$

$M_n = 2.72\ in\ (7.6\ kips) = 20.70\ kip\text{-}in < 85.94\ kip\text{-}in\ capacity$

Occurs at $\frac{0.5(7.61\ kips)}{0.85(4\ ksi)(1.875\ in)} + 0.125\ in = 0.72\ inches$ in to panel from face of panel

Strong axis bending stress = $20.70\ kip\text{-}in\ /\ 1.5625\ in^3 = 13.25\ ksi$

Steel shear design strength: $V_n = \left(0.6F_y\right)bd$

$V_n = (0.6\ x\ 55ksi)\ 1\ in\ x\ (2.5\ in - 1\ in\ dia\ hole) = 49.5\ kips > 7.6\ kips$

Steel bearing design strength:

Conservatively, check anchor insert bearing at just the stiffener locations.

$R_n = (1.8)\left(F_y\right)\left(A_{pb}\right) = (1.8)(55\ ksi)(1\ in\ x\ 0.5\ in\ x\ 2) = 99.0\ kips > 7.6\ kips$ AISC J7-1

$\varphi R_n = 0.75(99.0\ kips) = 74.25\ kips$

Check the insert bar combined flexural stress at two locations when connection is resisting full uplift plus full in-plane shear.

Uplift force, $N_n = 11.7\ kips$; In-plane shear force, $V_n = 7.6\ kips$

Location 1: Calculate the weak axis moment at the maximum strong axis moment location = 0.72 inches in from the face of the panel.

Weak axis moment: $M_n = 11.7\ kips\ (2.72\ in) - 9.046 \frac{kips}{in}\ (2.72\ in - 2.125\ in)^2/2 = 30.22\ kip\text{-}in$

Combined maximum flexural stress:

$f_b = 20.70\ kip\text{-}in\ /\ 1.5625\ in^3 + 30.22\ kip\text{-}in\ /\ 0.625\ in^3$

$f_b = 13.25\ ksi + 48.35\ ksi = 61.60\ ksi > 55\ ksi$

Location 2: Calculate the strong axis moment at the maximum weak axis moment location = 1.415 inches in from the face of the panel.

Strong axis moment:

$M_n = \left((7.25\ in - 2.72\ in) - (3.415\ in - 2.72\ in)\right) \div (7.25\ in - 2.72\ in) \times 20.7\ kip\text{-}in$

$M_n = (4.53\ in - 0.695\ in) \div 4.53\ in \times 20.7\ kip\text{-}in = 17.52\ kip\text{-}in$

Combined maximum flexural stress:

$f_b = 17.52\ kip\text{-}in\ /\ 1.5625\ in^3 + 32.46\ kip\text{-}in\ /\ 0.625\ in^3$

$f_b = 11.21\ ksi + 51.94\ ksi = 63.15\ ksi > 55\ ksi$

Since the combined maximum flexural stress exceeds the yield stress of the insert bar, reduce the nominal uplift capacity of the connection to lower the combined maximum flexural stress equal to the yield stress.

$N_n = (55.0\ ksi - 13.25\ ksi) \div 48.35\ ksi \times 11.7\ kips = 10.1\ kips$ for Location 1

$N_n = (55.0\ ksi - 11.21\ ksi) \div 51.94\ ksi \times 11.7\ kips = 9.9\ kips$ for Location 2

ACI 18.5.2.1 states, “In connections between wall panels, or between wall panels and the foundation, yielding shall be restricted to steel elements or reinforcement.”

ACI 18.5.2.2 states, “For elements of the connection that are not designed to yield, the required strength shall be based on **1.5*Sy*** of the yielding portion of the connection.

ACI 18.5.2.2 is referenced in R17.10.5.3 for option (b).

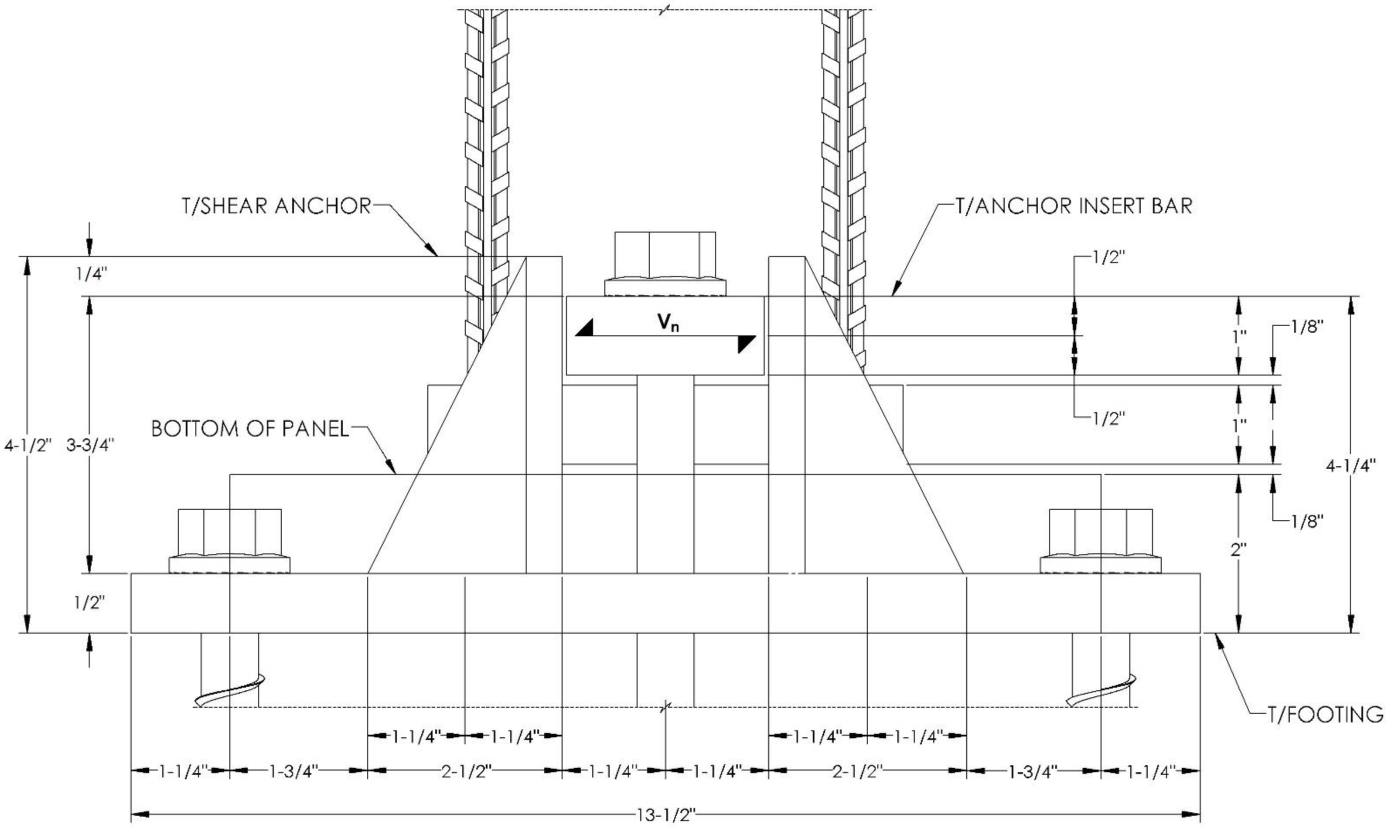
Therefore, design the base plate of the shear anchor attachment to be the yielding steel element. To meet the requirement of ACI 18.5.2.2, design the shear anchor attachment for:

$V_n = 7.6\ kips \div 1.5\ =\ 5.1\ kips$

The balance of the connection is designed for:

$V_n = 7.6\ kips$

IN-PLANE SHEAR ANCHOR: Nominal shear, $V_n = 7.6\ kips$ for the anchor bolts and $V_n = 5.1\ kips$ for the steel attachment.



Shear anchor attachment bolt forces:

In-plane shear, $V_n = 7.6\ kips\ /\ 2 = 3.8\ kips\ /\ bolt$

Tension from eccentric loading:

$N_n = 7.6\ kips\ (4.25\ in - 0.50\ in)\ \div\ (13.5\ in - 1.25\ in) = 28.50\ kip\text{-}in\ \div\ 12.25\ in$

$= 2.3\ kips\ /\ bolt$

Design both anchor bolts for: $V_n = 3.8\ kips\ /\ bolt + N_n = 2.3\ kips\ /\ bolt$

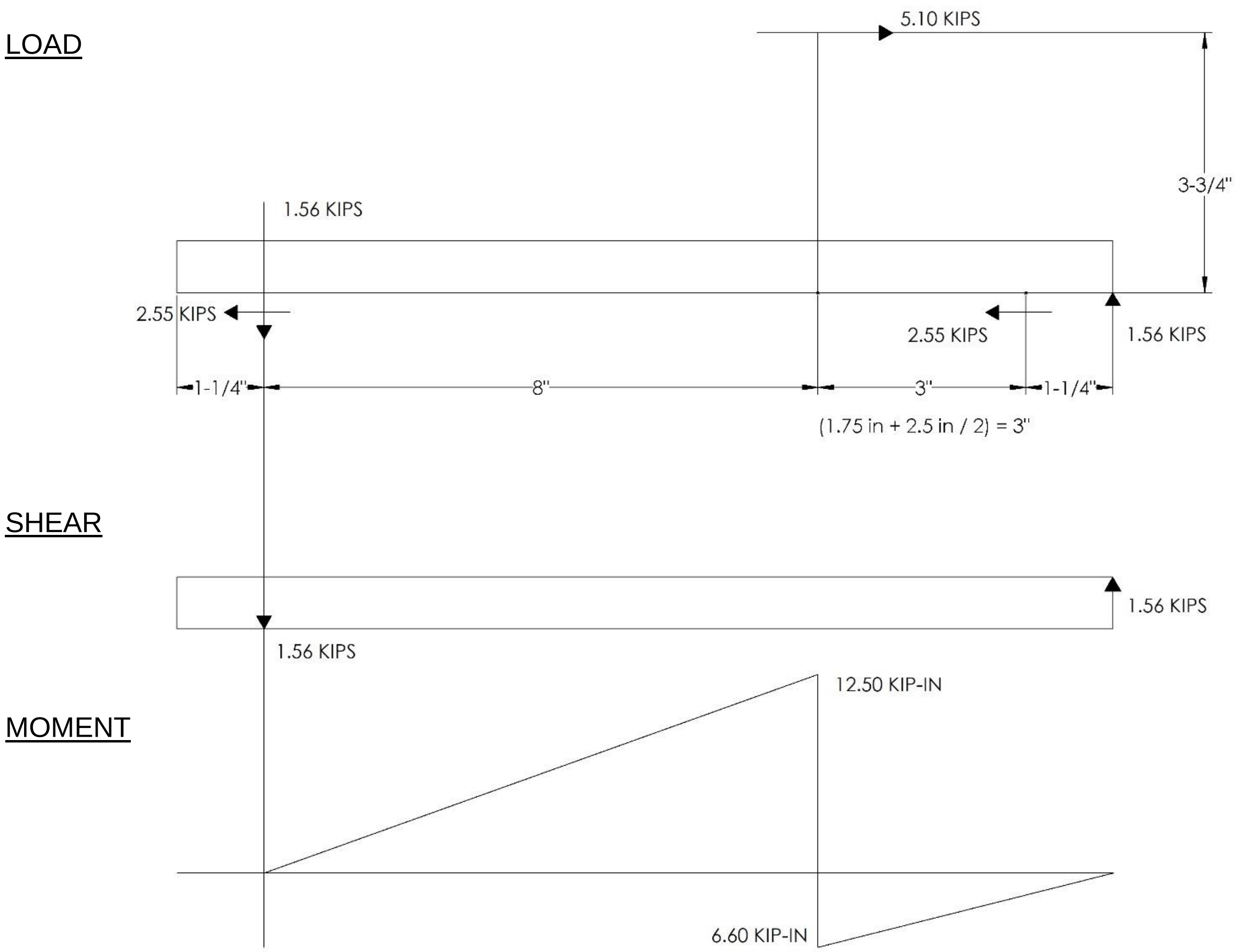
Design the shear anchor attachment base plate to yield and provide the ductility required by ACI 318-19.

Design the shear anchor base plate:

Verify a base plate thickness, t = 0.5 inches

Maximum applied moment, $M_n = 5.1\ kips\ (3.75\ in) = 19.1\ kip\text{-}in$

Apply maximum moment at centerline of shear anchor stiffener where it attaches to the base plate.



Moment at vertical face of the stiffener, $M_n = 1.56\ kips\ (8\ in - 2.50/2\ in) = 10.5\ kip\text{-}in$

Moment at centerline of connector, $M_n = 1.56\ kips\ (11\ in\ /\ 2) = 8.6\ kip\text{-}in$

Baseplate capacity at vertical face of stiffener:

$$z = \frac{bd^2}{4} = \frac{(4\ in)(0.50\ in)^2}{4} = 0.25\ in^3$$

$M_n = (0.25\ in^3)\ (55\ ksi) = 13.75\ kip\text{-}in > 10.50\ kip\text{-}in$ demand

Baseplate capacity at centerline of connector with 1.50” long slot for PBA-S tension anchor bolt.

Baseplate effective width $= 4\ in - 1.5\ in = 2.5\ in$

$$z = \frac{bd^2}{4} = \frac{(2.5\ in)(0.50\ in)^2}{4} = 0.1563\ in^3$$

$M_n = (0.1563\ in^3)\ (55\ ksi) = 8.6\ kip\text{-}in = 8.6\ kip\text{-}in$ demand

Base plate will yield and be the ductile element of the attachment.

Use Base Plate 1/2 inch X 4 inches X 13-1/2 inches

For in-plane shear demand, rate the panel base anchor (PBA-S) with the shear anchor attachment for a nominal shear capacity, $\boldsymbol{V_n = 5.1}$ kips and a factored shear capacity of $\boldsymbol{V_u = 4.6}$ kips

The in-plane shear capacity stated can be developed in combination with a reduced nominal uplift (tension) capacity, $N_n = 10.0$ kips and a factored uplift (tension) capacity, $N_u = 8.5$ kips, based on the capacity of the ½ inch diameter F1554 Grade 55 galvanized threaded anchor rod used with the PBA-S anchor.

For in-plane shear, design both anchor bolts attaching the shear anchor attachment to the footing for:

Bolt design shear, $V_n = 3.8\ kips/bolt$

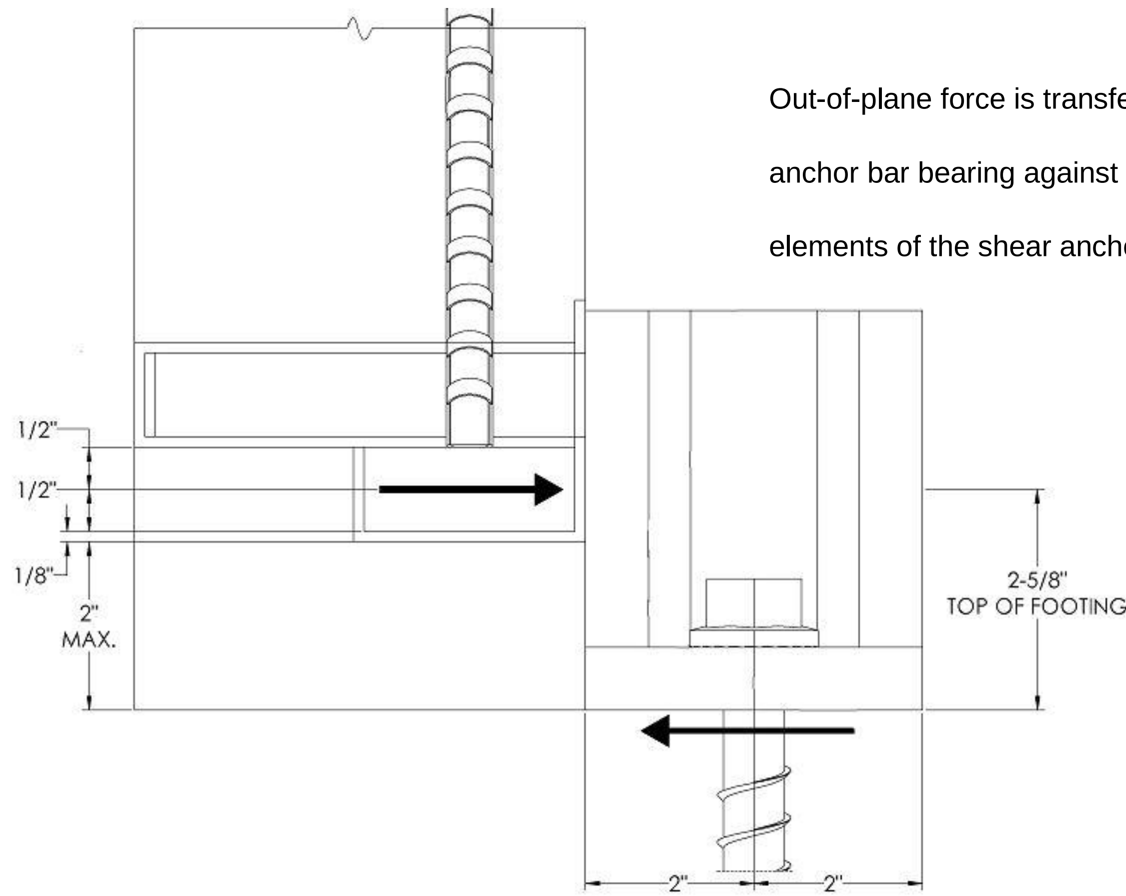
In combination with

Bolt design tension, $N_n = 2.3\ kips\ /\ bolt$

SCOPE: Determine the out-of-plane shear capacity of the panel base anchor using the shear anchor attachment as the ductile element.

Use minimum panel thickness = 5-1/2”

Out-of-plane force is transferred by the anchor bar bearing against the vertical elements of the shear anchor attachment.



Steel nominal bearing design strength, $R_n = (1.8)(F_y)(A_{pb})$ AISC J7-1

$A_{pb} = 2\ (0.5\ in)(1\ in)$ $\varphi = 0.75$

$\varphi R_n = 0.75\ (1.8)\ (55\ ksi)\ (2\ x\ 0.5\ in\ x\ 1\ in) = 74.3\ kips$

Force transfer from panel concrete to bearing bar.

Concrete bearing design strength, $B_n = 0.85\ f_c'\ b_{eff}\ l_b$ ACI Table 14.5.6.1 (c)

$\varphi = 0.60$ (Consider as plain concrete) ACI Table 21.2.1 (d)

$b_{eff} = 1.875\ in\ (6\ in - 2.5\ in) + 1.0\ in\ (2.5\ in) = 6.56\ in^2 + 2.5\ in^2 = 9.06\ in^2$

$\varphi B_n = 0.60\ (0.85)\ (4\ ksi)\ (9.06\ in^2) = 18.5\ kips$

Shear friction: $V_n = \mu\ A_{vf}\ F_y$

$\mu = 0.7$ for concrete against structural steel ACI Table 22.9.4.2 (d)

$V_n = 0.7\ (2\ x\ 0.2\ in^2)\ (60\ ksi) = 16.8\ kips$

$\varphi = 0.75$ ACI Table 21.2.1 (b)

$\varphi V_n = 0.75\ (16.8\ kips) = 12.6\ kips$

Total force transfer $= 18.5\ kips + 12.6\ kips = 31.1\ kips =\ V_u$

Baseplate design strength:

Force to two anchor bolts:

$Shear = V_n\ /\ 2$ $\quad$ $Tension = (V_n\ /\ 2)\ (2.625\ in\ /\ 2\ in)$

Torsional moment to steel baseplate:

$= (V_n\ /\ 2)\ (2.625\ in) = V_n\ (1.3125\ in) = T_n\ kip - in$

$F_n = 0.6\ F_y$ $\quad$ $\varphi_T = 0.9$ $\quad$ $b = 4\ in$ $\quad$ $t = 0.50\ in$ $\quad$ $b\ /\ t = 4\ in\ /\ 0.50\ in = 8.0 < 10$

Torsional constant, $J = \left(1/3 - 0.2\ (t/b)\right) bt^3 = \left(1/3 - 0.2\ (0.50\ in\ /\ 4\ in)\right)\ (4\ in)\ (0.50\ in)^3$

$J = 0.154\ in^4$

Shear modulus, $G = 9.3\ x\ 10^6\ psi$ for ASTM Grade 80-55-6 Ductile Iron

Torsional shear stress, $\tau_t = \frac{T_n t}{J} = \frac{T_n(0.50\ in)}{0.154\ in^4} = 3.247\ T_n$

Shear stress; set $\tau_t = F_n = 0.6\ F_y = 0.6\ (55\ ksi) = 33\ ksi$ AISC H3-8

$\varphi_T = 0.9$ Solving for max $T_n = F_n\ /\ 3.247$

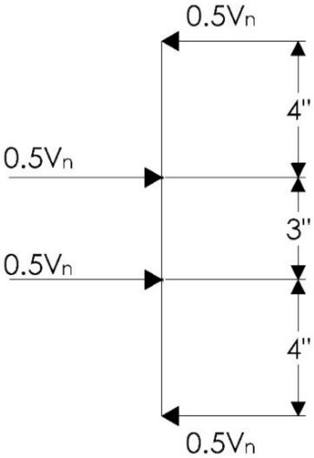
$$T_n = \frac{33\ ksi}{3.247} = 10.163\ kip\text{-}in = 1.3125\ in\ (V_n)$$

Maximum $V_n = 10.163\ kip\text{-}in\ /\ 1.3125\ in\ = 7.8\ kips$

$\varphi V_n = V_u = 0.9\ (7.8\ kips) = 7.0\ kips$

Strong axis bending of the baseplate:

Flexural design strength:



Maximum $M_n = 0.5\ V_n\ (4\ in) = 2\ V_n\ kip\text{-}in$

Occurs at mid-span of the baseplate at the location of the 1.5 in long slot

$z = \frac{b}{4}\left(d^2 - {d_1}^2\right)$ $b = 0.5\ in$ $d = 4\ in$ $d_1 = 1.5\ in$

$$z = \frac{0.50\ in}{4}\left((4\ in)^2 - (1.5\ in)^2\right) = 1.719\ in^3$$

$\varphi M_n = 0.9\ (55\ ksi)\ (1.719\ in^3) = 85.09\ kip\text{-}in \rightarrow \varphi V_n = 85.09\ kip\text{-}in\ /\ 2$

$V_u = 85.09\ kip\text{-}in\ /\ 2 = 42.6\ kips$

Shear design strength: $\varphi V_n = \varphi\ (0.6\ Fy)\ (bd)$

$\varphi V_n = 0.9\ (0.6\ x\ 55\ ksi)\ (0.50\ in)\ (4\ in) = 59.4\ kips$

$V_u = 2\ (59.4\ kips) = 118.8\ kips$

Out-of-plane force capacity summary of V_u for the shear anchor attachment.

Steel Bearing: $V_u = 74.3\ kips$

Panel to Bearing Bar: $V_u = 31.1\ kips$

Base Plate Torsion: $V_u = 7.0\ kips$ **(Controls)**

Base Plate Flexure: $V_u = 42.6\ kips$

Base Plate Shear: $V_u = 118.8\ kips$

The factored out-of-plane shear capacity of the shear anchor attachment is, $\boldsymbol{V_u = 7.0\ kips}$. The nominal out-of-plane shear capacity of the shear anchor attachment is, $\boldsymbol{V_n = 7.8\ kips}$.

The out-of-plane shear capacity of the shear anchor attachment is controlled by the torsional yield strength of the base plate. The two anchor bolts connecting the shear anchor attachment to the footing are designed for 1.5 times the base plate capacity to ensure that the base plate provides the required ductility.

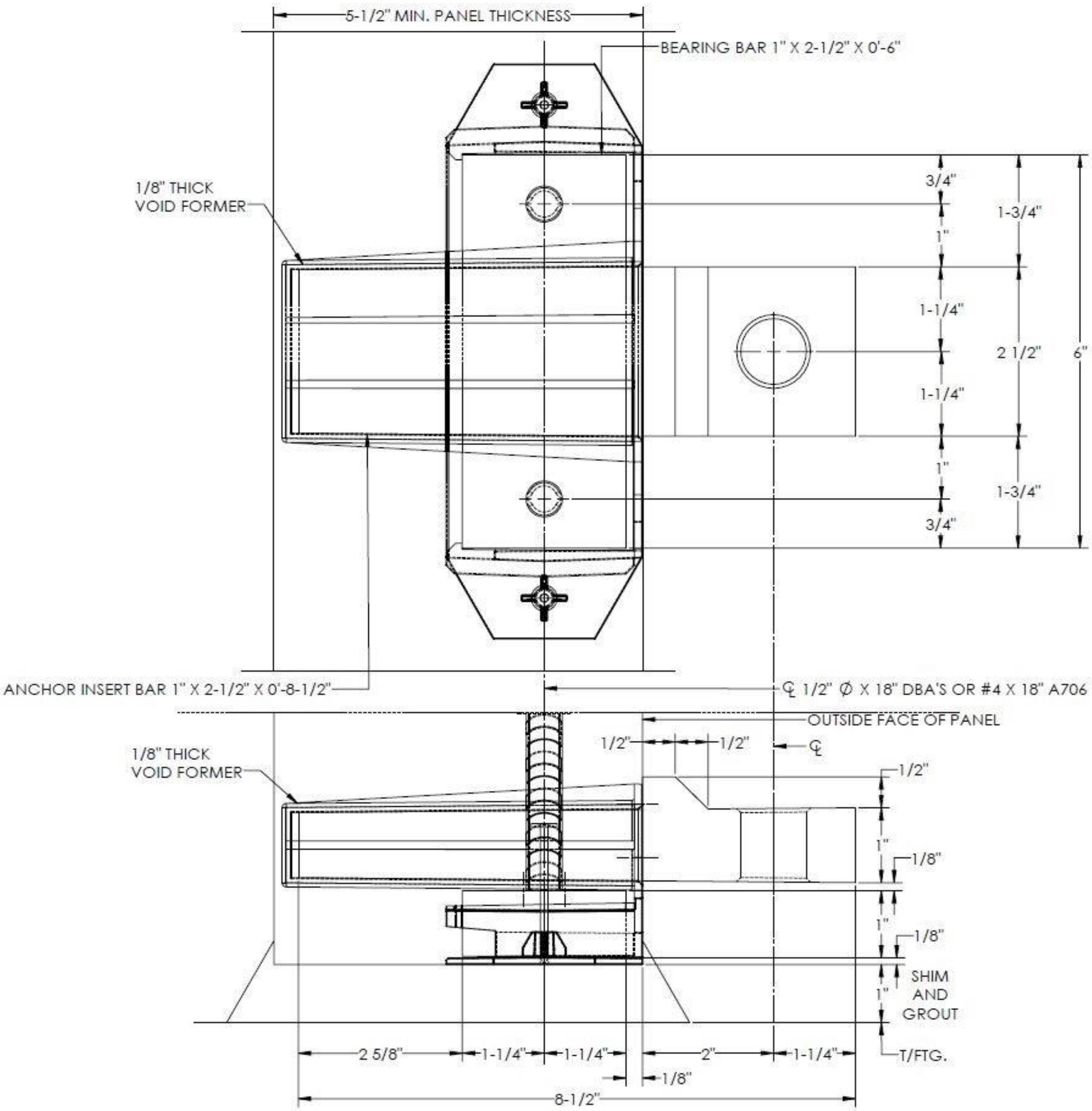
Bolt combined factored design forces for the base plate capacity, $V_u = 7.0\ kips$

Bolt design shear, normal to the free edge of footing $= (1.5)(0.5)V_u = (0.75)7.0\ kips = \boldsymbol{5.3\ kips/bolt}$

In combination with

Bolt design tension $= (1.5)(2.625\ in/2\ in)(0.5)V_u = (0.9844)7.0\ kips = 6.9\ kips/bolt$

Drawing DS-1


5-1/2" MIN. PANEL THICKNESS
BEARING BAR 1" X 2-1/2" X 0'-6"
1/8" THICK
VOID FORMER
3/4"
1-3/4"
1"
1-1/4"
2 1/2"
6"
1-1/4"
1"
1-3/4"
3/4"
ANCHOR INSERT BAR 1" X 2-1/2" X 0'-8-1/2"
℄ 1/2" Ø X 18" DBA'S OR #4 X 18" A706
OUTSIDE FACE OF PANEL
1/8" THICK
VOID FORMER
1/2"
1/2"
℄
1/2"
1"
1/8"
1"
1/8"
SHIM
AND
GROUT
1"
T/FTG.
2 5/8"
1-1/4"
1-1/4"
2"
1-1/4"
1/8"
8-1/2"