Superior Panel Base Anchor (PBA) - 10K Calculations
In-Plane and Out-Of-Plane Shear Capacity

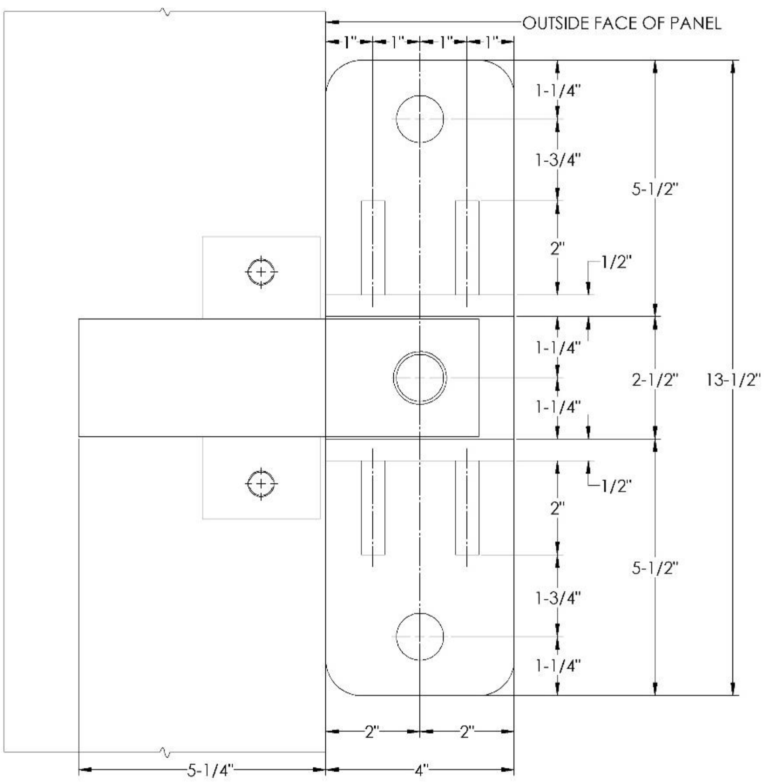
Scope: Determine in-plane shear capacity of the panel base anchor using the shear connector.

See drawing DS-1 for additional details.

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

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

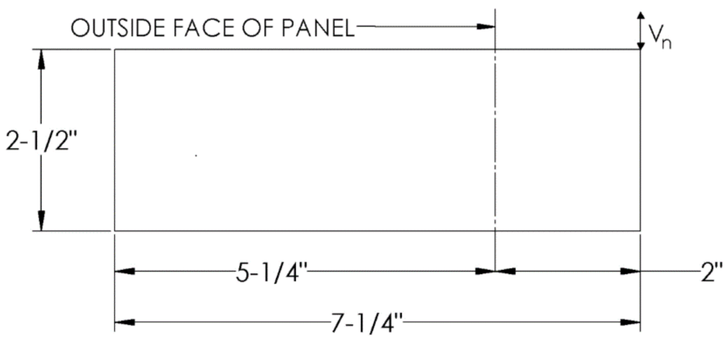
Shear Anchor: ASTM A536 Grade 80-55-6, f_y = 55 ksi



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

Calculate V_c = Nominal strength of the section controlled by concrete.



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

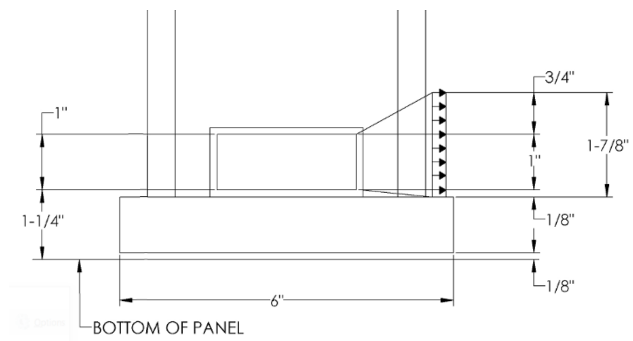
$V_c = \left(\frac{0.85 f_c' b l_e}{1+3.6e/l_e}\right) \qquad e = a + \frac{l_e}{2}$

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

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

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

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



$a = 2.0\ in \qquad l_e = 5.25\ in \qquad b = 1.875\ in$

$e = 2.0\ in + 5.25\ in\ /\ 2 = 4.625\ in$

$V_c = \left(\frac{0.85(4000\ psi)(1.875\ in)(5.25\ in)}{1+3.6(4.625\ in)/5.25\ in}\right) = 8{,}023\ lb$

Set $V_c = V_n = 8.02\ kips \rightarrow\ V_u = 0.75(8.02\ kips) = 6.02\ kips$ Use $V_u = 6.0\ kips$

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

For flexural design strength: $\varphi V_n = \left(\frac{\varphi Z F_y}{a + \frac{0.5 V_u}{0.85 f_c' b}}\right) = \left(\frac{0.9(1.5625\ in)(55\ ksi)}{2\ in + \frac{0.5(6.0\ kips)}{0.85(4\ ksi)(1.875\ in)}}\right) = \left(\frac{77.34\ kip\text{-}in}{2.47\ in}\right)$

$\varphi V_n = 31.31\ kips > 6.0\ kips$

$M_u = 2.47\ in\ (6.0\ kips) = 14.82\ kip\text{-}in < 77.34\ kip\text{-}in$

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

$\varphi V_n = 0.9(0.6\ x\ 55ksi)\ 1\ in\ x\ (2.5\ in - 1\ in\ dia\ hole) = 44.55\ kips > 6.0\ kips$

Bearing design strength:

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

$\varphi R_n = \varphi(1.8)(F_y)(A_{pb}) = 0.75(1.8)(55\ ksi)(1\ in\ x\ 0.5\ in\ x\ 2) = 74.25\ kips > 6.0\ kips$

Check combined flexural stress when connection is resisting full uplift plus in-plane shear.

Uplift force, $P_u = 0.9(10.0\ kips) = 9.0\ kips$

Weak axis bending stress = $0.9(44.4\ ksi)^* = 39.96\ ksi$ *from previous calculations

In-plane shear force, $V_u = 6.0\ kips$

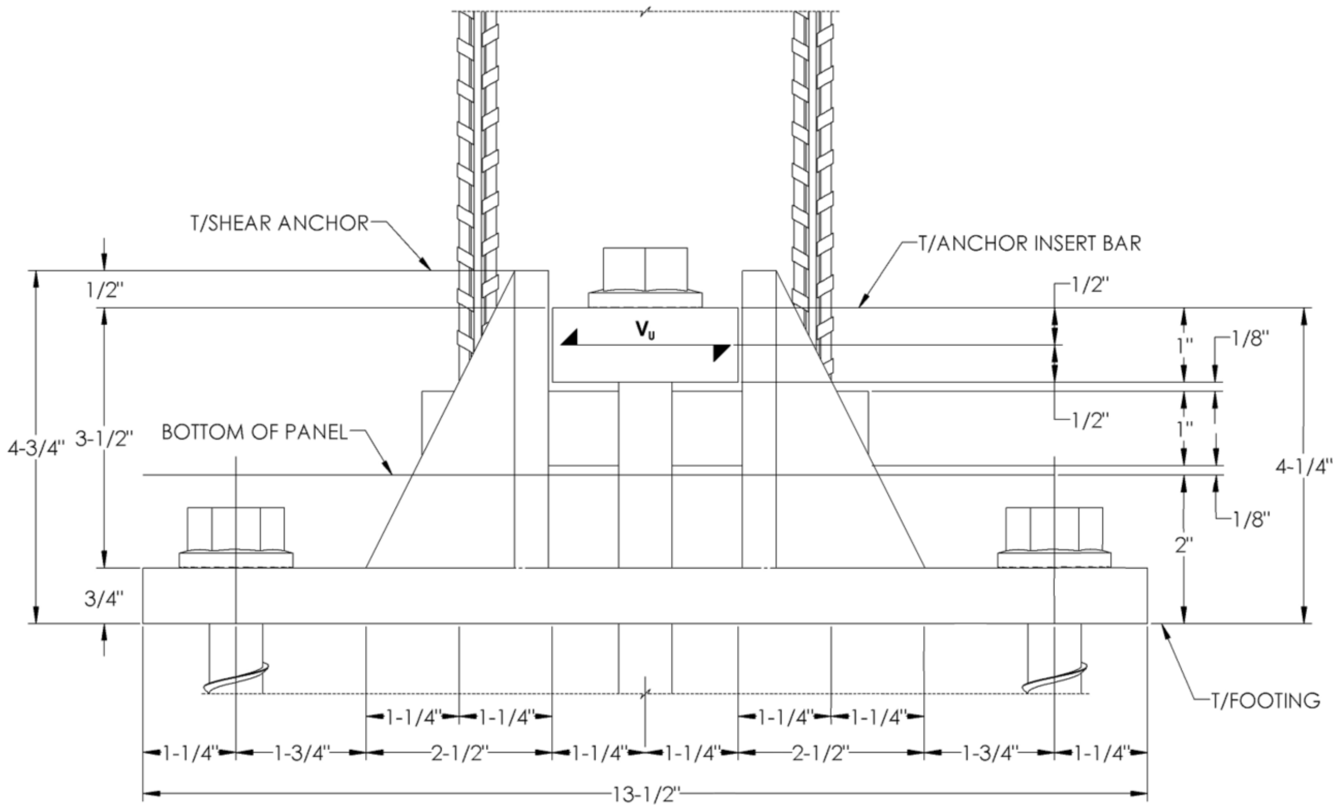
Strong axis bending stress = $(6.0\ kips\ x\ 2.47\ in)\ /\ 1.5625\ in^3 = 9.48\ ksi$

Maximum combined flexural stress (conservatively)

$39.96\ ksi + 9.48\ ksi = 49.44\ ksi < 0.9(55\ ksi) = 49.5\ ksi$

*Note: Maximum strong axis and weak axis moments don't occur at the exact same location, but are within $(2\ in\ /\ 2\ + 1/8\ in - 0.47\ in = 0.655\ in)$.

<u>IN-PLANE SHEAR ANCHOR:</u> Design shear $V_u = 6.0\ kips$



Shear Anchor strength:

Anchor bolt forces:

In-plane shear, $V_u = 6.0\ kips\ /\ 2 = 3.0\ kips\ /\ bolt$

Tension from eccentric loading:

Tension, $N_u = 6.0\ kips\ (4.25\ in - 1\ in\ /\ 2)\ /\ (13.5\ in - 2(1.25\ in)) = 22.50\ kip\text{-}in\ /\ 11\ in$

$= 2.05\ kips\ /\ bolt$

Design anchor bolts for: Shear, $V_u = 3.0\ kips\ /\ bolt$ and Tension, $N_u = 2.05\ kips\ /\ bolt$

Acting simultaneously to develop the shear anchor in-plane shear rated capacity.

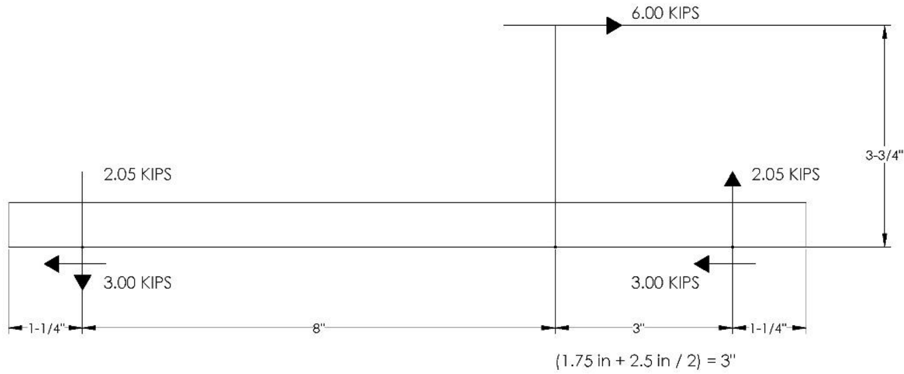
IN-PLANE SHEAR CAPACITY OF THE SHEAR ANCHOR IS $V_u = 6.0\ kips$ CONTROLLED BY THE INSERT BAR BEARING ON THE PANEL CONCRETE

Shear anchor base plate:

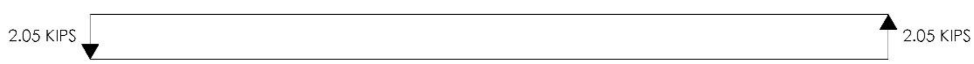
Maximum moment, $M_u = 6.0\ kips\ (3.75\ in) = 22.50\ kip\text{-}in$

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

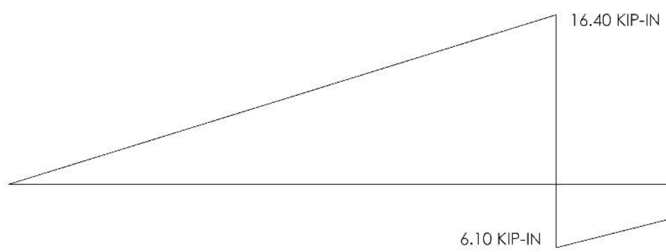
LOAD



SHEAR



MOMENT



Moment at vertical face of the stiffener, $M_u = 2.05\ kips\ (8\ in - 2.5\ in\ /\ 2) = 13.84\ kip\text{-}in$

Moment at centerline of shear anchor, $M_u = 2.05\ kips\ (11\ in\ /\ 2) = 11.28\ kip\text{-}in$

Baseplate capacity at vertical face of stiffener:

$$z = \frac{bd^2}{4} = \frac{(4\ in)(0.75\ in)^2}{4} = 0.5625\ in^3$$

$$\varphi M_n = 0.9\ (0.5625\ in^3)\ (55\ ksi) = 27.84\ kip\text{-}in > 13.84\ kip\text{-}in$$

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

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

$$z = \frac{bd^2}{4} = \frac{(2.5\ in)(0.75\ in)^2}{4} = 0.3516\ in^3$$

$$\varphi M_n = 0.9\ (0.3516\ in^3)\ (55\ ksi) = 17.4\ kip\text{-}in > 11.28\ kip\text{-}in$$

USE BASEPLATE 3/4 IN X 4 IN X 13-1/2 IN

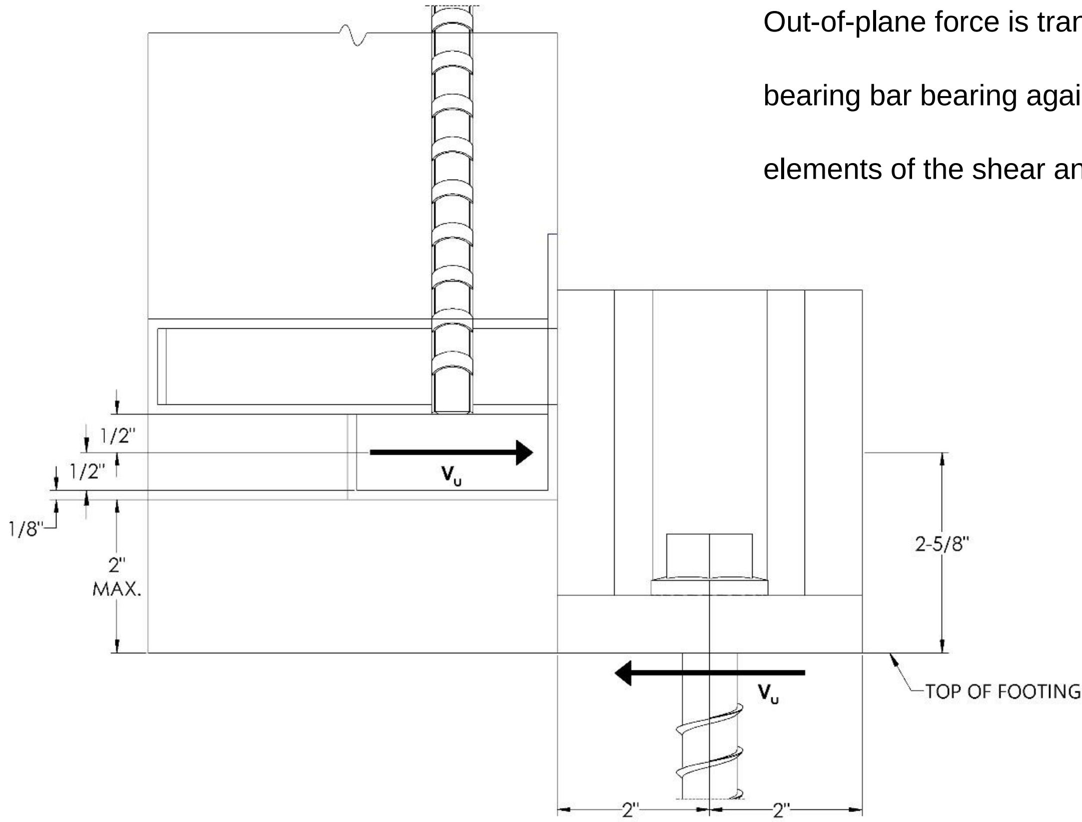
RATE THE PANEL BASE ANCHOR WITH THE SHEAR ANCHOR FOR A FACTORED SHEAR CAPACITY, $V_u = 6.0\ \text{KIPS}$, FOR IN-PLANE SHEAR DEMAND.

TO DEVELOP THE RATED IN-PLANE SHEAR CAPACITY OF $V_u = 6.0\ \text{KIPS}$ FOR THE SHEAR ANCHOR, THE TWO ANCHOR BOLTS EACH MUST BE DESIGNED WITH A MINIMUM SHEAR CAPACITY OF $V_u = 3.0\ \text{KIPS/BOLT}$ ACTING IN COMBINATION WITH A MINIMUM TENSION CAPACITY OF $N_u = 2.05\ \text{KIPS/BOLT}$.

THIS CAPACITY CAN BE DEVELOPED IN COMBINATION WITH A FACTORED UPLIFT (TENSION) DEMAND OF 9.0 KIPS.

SCOPE: Determine out-of-plane shear capacity of the panel base anchor using the shear connector.

Use minimum panel thickness = 5-1/2”



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

Steel Bearing design strength, $\varphi R_n = \varphi(1.8)(F_y)(A_{pb})$

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

$\varphi R_n = 0.75\ (1.8)\ (55\ ksi)\ (2\ x\ 0.5\ in\ x\ 1\ in) = 74.25\ kips$

Force transfer from panel concrete to bearing bar.

Concrete bearing design strength, $B_n = 0.85\ f_c'\ b_{eff}\ l_b$

$\varphi = 0.60$ (consider as plain concrete)

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

$\varphi B_n = 0.60\ (0.85)\ (4\ ksi)\ (9.06\ in^2) = 18.48\ kips$

Shear friction: $V_n = \mu\ A_{vf}\ F_y$ $\mu = 0.7$ for concrete against structural steel

$V_n = 0.7\ (2\ x\ 0.2\ in^2)\ (60\ ksi) = 16.8\ kips$ $\varphi = 0.75$

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

Total force transfer $= 18.48\ kips + 12.6\ kips = 31.08\ kips = V_u$

Baseplate design strength:

Force to two anchor bolts:

$\text{Shear} = V_n\ /\ 2$ $\text{Tension} = V_n\ /\ 2\ (2.625\ in\ /\ 2\ in)$

Torsional moment to steel baseplate:

$= (V_n\ /\ 2)\ (2.625\ in) = V_n\ (1.3125\ in) = T_n, kip\text{-}in$

(The torsional moment, T_n, is resisted by the base plate each side of the vertical bearing bars)

$F_n = 0.6\ F_y$ $\varphi_t = 0.9$ $b = 4\ in$ $t = 0.75\ in$ $b\ /\ t = 4\ in\ /\ 0.75\ in = 5.333 < 10$

Torsional constant, $J = \left(1/3 - 0.2\ (t/b)\right)bt^3 = \left(1/3 - 0.2\ (0.75\ in\ /\ 4\ in)\right)\ (4\ in)(0.75\ in)^3 = 0.499\ in^4$

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

Torsional shear stress, $\tau_t = \frac{T_n t}{J} = \frac{T_n(0.75\ in)}{0.499\ in^4} = 1.503\ T_n$

Shear stress: $f_n \leq 0.6\ F_y = 0.6\ (55\ ksi) = 33\ ksi$

$\varphi_T = 0.9$ Solving for max $T_n = \tau_t\ /\ 1.53 = T_n$

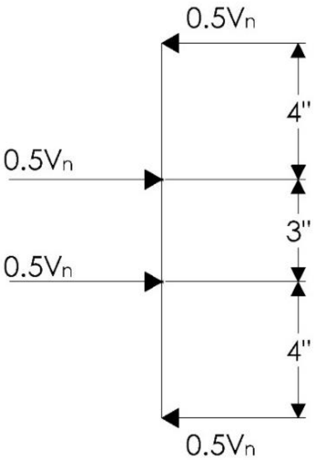
$$T_n = \frac{33\ ksi}{1.503} = 21.956\ kip\text{-}in = 1.3125\ in\ (V_n)$$

Maximum $V_n = 21.956\ kip\text{-}in\ /\ 1.3125\ in\ = 16.73\ kips$

$\varphi V_n = 0.9\ (16.73\ kips) = 15.06\ kips = V_u$

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

$z = \frac{b}{4}(d^2 - {d_1}^2)$ $b = 3/4\ in$ $d = 4\ in$ $d_1 = 1.5\ in$

$$z = \frac{0.75\ in}{4}((4\ in)^2 - (1.5\ in)^2) = 2.578\ in^3$$

$\varphi M_n = 0.9\ (55\ ksi)\ (2.578\ in^3) = 127.61\ kip\text{-}in \rightarrow \varphi V_n = 127.61\ kip\text{-}in\ /\ 2$

$V_u = 127.61\ kip\text{-}in\ /\ 2 = 63.81\ kips$

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

$\varphi V_n = 0.9\ (0.6\ x\ 55\ ksi)\ (0.75\ in)\ (4\ in) = 89.1\ kips$

$V_u = 2\ (89.1\ kips) = 178.2\ kips$

Out-of-plane force capacity - summary V_u for the shear connector

Steel Bearing: $V_u = 74.25\ kips$

Panel to Bearing Bar: $V_u = 31.08\ kips$

Base Plate Torsion: $V_u = 15.06\ kips$

Base Plate Flexure: $V_u = 63.81\ kips$

Base Plate Shear: $V_u = 178.2\ kips$

Bolt Combined Loading

Bolt Shear $= 0.5\ V_u$ normal to free edge of footing

In combination with

Bolt Tension $= (1.3125\ in\ /\ 2)\ V_u = 0.656\ V_u$

With V_u Maximum $= 15.06\ kips$ where V_u = the out-of-plane shear capacity of the shear anchor

NOTE: The out-of-plane shear capacity of the shear anchor will be controlled by the capacity of the two anchor bolts into the footing, with a maximum shear, V_u = 15.06 kips.

EXAMPLE: If the calculated factored out-of-plane shear to the shear anchor is

$V_u = 5.0\ kips\ (< 15.06\ kips)$, the forces to each of the two anchor bolts are:

Bolt Shear force (normal to the free edge of the footing), $V_u = 0.5\ (5.0\ kips) = 2.5\ kips$

acting in combination with

Bolt tension force, $N_u = 0.656\ (5.0\ kips) = 3.28\ kips$