Superior Panel Base Anchor (PBA) - 10K Calculations

Scope: Design a site-cast (tilt-up) or plant cast precast concrete panel to footing connection to satisfy the requirements of ACI 318-19, Chapter 16, Section 16.2.4.3(b) – Connections between precast wall panels shall have at least two vertical integrity ties, with a *nominal tensile strength* of at least 10,000 lb. per tie. Section 16.3.6.1. At the base of a precast column, pedestal, or wall, the connection to the foundation shall satisfy 16.2.4.3 or 16.2.5.2.

See attached design drawing DS-1 for details

Nominal design tensile force to bolt into footing = 10,000 lbs.

Material properties: Footing concrete: f'_c = 3,000 psi

Panel concrete: f'_c = 4,000 psi

Deformed Bar Anchor: ASTM A496, f_y = 70,000 psi

Reinforcement: ASTM A706 Grade 60, f_y = 60,000 psi

Hot Rolled Steel Bars: ASTM A529 Grade 50, f_y = 50,000 psi

Calculate Component Forces:

Nominal Moment, $\mathrm{M_n}$

$$\mathrm{M_n} = 10.0 \text{ kips } (2.0 \text{ in.} + 0.125 \text{ in.} + 2.5 \text{ in.}/2) = 10.0 \text{ kips } (3.375 \text{ in.}) = 33.75 \text{ kip-in.}$$

Plain concrete bearing at the end of the 1.0 in. x 2.5 in. steel bar.

Nominal Bearing Strength, $\mathrm{B_n}$

$\mathrm{B_n} = 0.85 f'_c \mathrm{b_{eff}}\, \mathrm{l_b} = 0.85\ (4{,}000 \text{ psi})\ (2.5 \text{ in.} \times 2.5)\ \mathrm{l_b} = 21{,}250\ \mathrm{l_b}$ lbs. (ACI 318-19 Table 14.5.6.1(c))

Per PCI Design Manual, Structural Steel Corbels, $\mathrm{b_{eff}} = 2.5\ \mathrm{b_w}$

Bearing width, $\mathrm{b_w} = 2.5$ in. $\mathrm{l_b}$ = Bearing length, in.

$\text{Moment arm} = 5.25 \text{ in.} - 0.125 \text{ in.} - 2.5 \text{ in.}/2 - \mathrm{l_b}/2 = 3.875 \text{ in.} - \mathrm{l_b}/2$

$M_n = 33.75$ kip-in. $= 33{,}750$ lb-in.

Concrete force $= 33{,}750$ lb-in./ $(3.875 \text{ in.} - \frac{l_b}{2} \text{ in.})$

Set $B_n =$ Concrete force and solve for l_b

$21{,}250\ l_b$ lb. $= 33{,}750$ lb-in./ $(3.875 \text{ in.} - \frac{l_b}{2} \text{ in.})$

$33{,}750$ lb-in. $= (21{,}250\ l_b \text{ lb.})\ (3.875 \text{ in.} - \frac{l_b}{2} \text{ in.})$

$33{,}750 = 82{,}343.75\ l_b - 10{,}625\ {l_b}^2$ (rearrange terms)

$10{,}625\ {l_b}^2 - 82{,}343.75\ l_b + 33{,}750 = 0$

${l_b}^2 - 7.75\ l_b + 3.176 = 0$ (solving, $l_b = 0.434$ in.)

$B_n = 21{,}250\ (0.434)$ lb. $= 9222.5$ lb.

Resisting Moment $= 9222.5$ lb. $(3.875 \text{ in.} - 0.434 \text{ in.}/2) = 33{,}736$ lb-in. $\approx 33{,}750$ lb-in.

Force to DBAs $= 10{,}000$ lb. $(2 \text{ in.} + 5.25 \text{ in.} - 0.434 \text{ in.}/2)$ / $(5.25 \text{ in.} - 0.125 \text{ in.} - 2.5 \text{ in.}/2 - 0.434\text{in.}/2)$

$= 10{,}000$ lb. $(7.033$ in.$)$ / 3.658 in. $= 19{,}226$ lb. (Tension Capacity Required)

DBA Capacity $= 2\ (0.2 \text{ in}^2)\ (70{,}000 \text{ psi}) = 28{,}000$ lb. $> 19{,}222$ lb. required

<u>Use 2-1/2" Ø Deformed Bar Anchors (DBA's)</u>

A706 #4s Capacity $= 2\ (0.2 \text{ in}^2)\ (60{,}000 \text{ psi}) = 24{,}000$ lb. $> 19{,}222$ lb. required

<u>Can use 2- #4 A706 Reinforcing Bars</u>

Bearing Bar: 1 in. x 2.5 in. x 0’-6”

$\text{Load to Bearing Bar} = 19{,}226 \text{ lb.}/2.5 \text{ in.} = 7{,}690.4 \text{ lb/in.}$

$\text{Nominal Shear Capacity Required}, V_n = 19{,}226 \text{ lb.}/2 = 9613 \text{ lb.}$

$\text{Nominal Moment Capacity Required}, M_n = 9613 \text{ lb.} (1 \text{ in.} + 2.5 \text{ in.}/2) - 7{,}690.4 \text{ lb/in.} (1.25 \text{ in.})(1.25 \text{ in.}/2)$

$= (21{,}629.25 \text{ lb-in.}) - (6{,}008.13 \text{ lb-in.})$

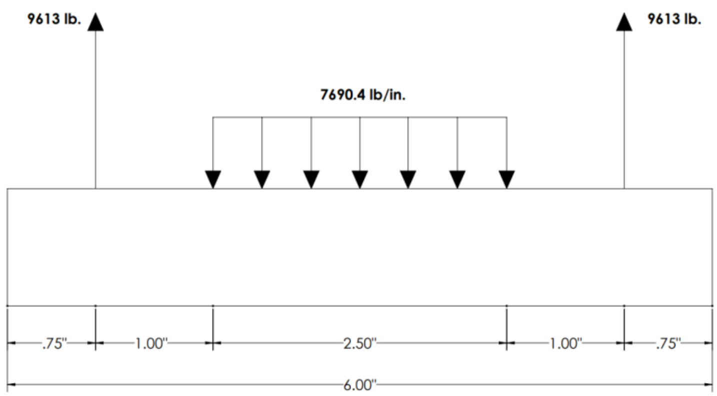
$= 15{,}621.12 \text{ lb-in.} = 15.62 \text{ kip-in.}$

$\text{Plastic Section Modulus}, Z = 2.5 \text{ in.} \times 1 \text{ in.}^2 / 4 = 0.625 \text{ in}^3$ (Steel Bar is 1 in. x 2.5 in. x 0′-6")

$f_b = 15.62 \text{ kip in.}/ 0.625 \text{ in}^3 = 25.0 \text{ ksi} < 50 \text{ ksi}$

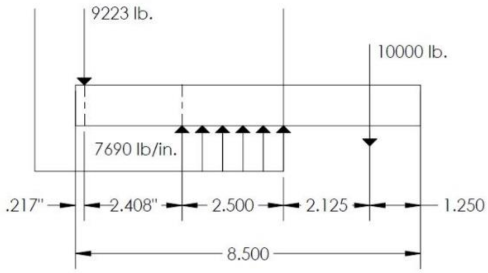
$V_n = 0.6 (50 \text{ ksi}) (1 \text{ in.} \times 2.5 \text{ in.}) = 75.0 \text{ kips} > 9.613 \text{ kips Required (AISC G2-1)}$

Use hot rolled steel bar 1 in. x 2.5 in. x 0’-6”



Load Diagram

Anchor Insert Bar: 1 in. x 2.5 in. x 0'-8-1/2"



Load Diagram

Bearing bar uniform load

$= 19{,}226 \text{ lb.}/2.5 \text{ in.} = 7{,}690.4 \text{ lb/in.} = 7.69 \text{ kips/in.}$

Nominal Moment Capacity Required,

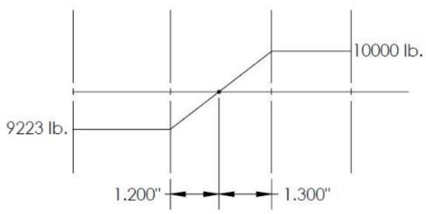
$M_n = 9.22 \text{ kips } (2.408 \text{ in.}) + 9.22 \text{ kips}(1.20 \text{ in.})0.5$ (At 0 Shear)

$= 22.2 \text{ kip-in.} + 5.53 \text{ kip-in.} = 27.73 \text{ kip-in.}$

or

$= 10.0 \text{ kips } (2.125 \text{ in.}) + 10.0 \text{ kips } (1.30 \text{ in.})0.5$

$= 21.25 \text{ kip-in.} + 6.5 \text{ kip-in.} = 27.75 \text{ kip-in.}$



Shear Diagram

Plastic Section Modulus, $Z = 2.5 \text{ in.} \times 1 \text{ in.}^2 / 4 = 0.625 \text{ in}^3$

$f_b = 27.75 \text{ kip-in.}/ 0.625 \text{ in}^3 = 44.4 \text{ ksi} < 50 \text{ ksi}$

$V_n = 0.6 \text{ (50 ksi) (1 in. x 2.5 in.)} = 75.0 \text{ kips} > 10.0 \text{ kips Required (AISC G2-1)}$

Use hot rolled steel bar 1 in. x 2.5 in. x 0'-8-1/2"

Note: Panel design professional should provide a check of the moment applied to the base of the panel due to the eccentricity of the base connection as a part of the panel design.

Determine the required length of the DBAs:

Development length required per ACI318-19

$$l_d = \left(\frac{f_y \Psi_t \Psi_e \Psi_g}{25\lambda\sqrt{f'_c}}\right) d_b \text{ for \#6 and smaller bars}$$ (ACI 318-19 Table 25.4.2.3)

$\lambda = 1.0$, $\Psi_t = 1.0$, $\Psi_e = 1.0$, $\Psi_g = 1.0$, $\Psi_s = 0.8$

$$l_d = \left(\frac{60{,}000(1.0)(1.0)(1.0)}{25(1.0)\sqrt{4{,}000}}\right) 0.5 \text{ in.} = 18.97 \text{ in.}$$

or

$$l_d = \left(\frac{3}{40}\,\frac{f_y}{\lambda\sqrt{f'_c}}\,\frac{\Psi_t \Psi_e \Psi_s \Psi_g}{\left(\frac{c_b + K_{tr}}{d_b}\right)}\right) d_b$$ (ACI 318-19 24.4.2.4a)

$c_b = 1.375$ in., $d_b = 0.5$ in., $K_{tr} = 0$, $\therefore \left(\frac{1.375 \text{ in.} + 0}{0.5 \text{ in.}}\right) = 2.75 > 2.5$ (use 2.5 per ACI)

$$l_d = \left(\frac{3}{40}\,\frac{60{,}000}{1.0\sqrt{4{,}000}}\,\frac{(1.0)(1.0)(0.8)(1.0)}{2.5}\right) 0.5 \text{ in.} = 11.38 \text{ in.} < 12 \text{ in.}$$

$$l_d = 12 \text{ in.}$$

Use class B splice w/ panel reinforcement

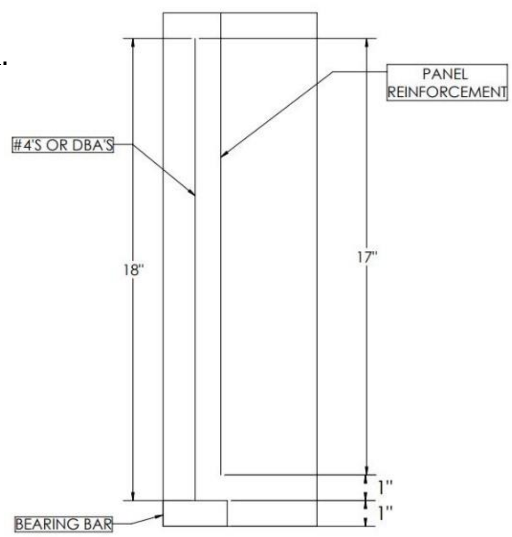
Minimum lap splice length = 1.3 (12 in.) = 15.6 in.

Minimum bar length required = 15.6 in. + 2 in. – 1 in. = 16.6 in.

Use #4 x 18 in. A706 Grade 60 reinforcing bars

or

Use 1/2" Ø x 18 in. long Deformed Bar Anchors (DBA's)



Panel Section at PBA

RELEVANT SECTIONS of ACI 318-19 CHAPTER 16 — CONNECTIONS BETWEEN MEMBERS

16.2 — Connections of precast members

16.2.1.2 Adequacy of connections shall be verified by analysis or test.

16.2.1.8 Integrity ties shall be provided in the vertical, longitudinal, and transverse directions and around the perimeter of a structure in accordance with 16.2.4 or 16.2.5.

16.2.4.3 Vertical integrity ties shall be provided at horizontal joints between all vertical precast structural members, except cladding, and shall satisfy (a) or (b):

> (b) Connections between precast wall panels shall have at least two vertical integrity ties, with a nominal tensile strength of at least 10,000 lb. per tie.

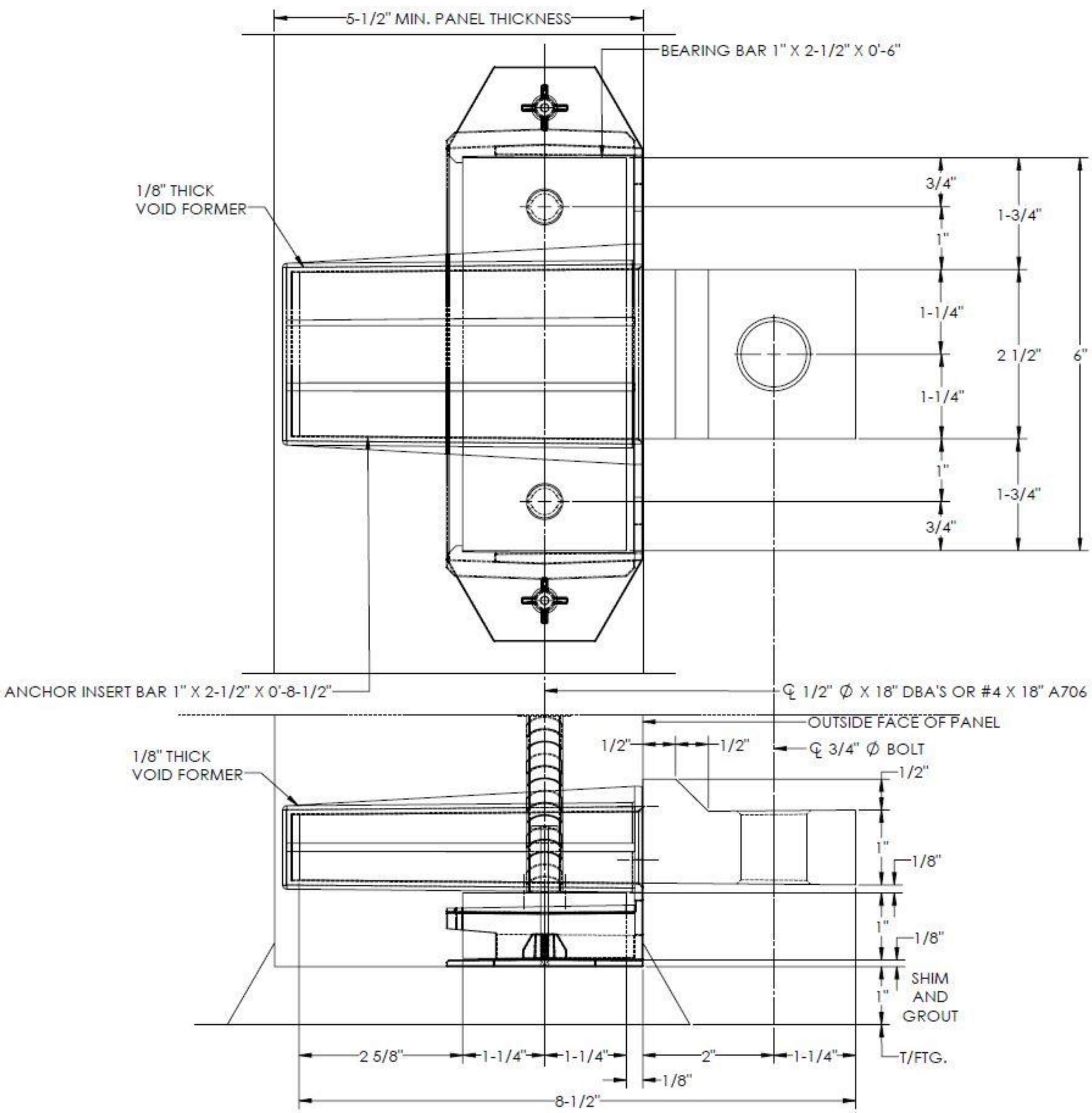
16.3 — Connections to foundations

16.3.6.1 At the base of a precast column, pedestal, or wall, the connection to the foundation shall satisfy 16.2.4.3 or 16.2.5.2.

2.3 — Terminology

strength, nominal — strength of a member or cross section calculated in accordance with provisions and assumptions of the strength design method of this Code before application of any strength reduction factors.

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"
1-1/4"
1-1/4"
1"
3/4"
1-3/4"
2 1/2"
1-3/4"
6"
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"
℄ 3/4" Ø BOLT
1/2"
1"
1/8"
1"
1/8"
1"
SHIM AND GROUT
T/FTG.
2 5/8"
1-1/4"
1-1/4"
2"
1-1/4"
1/8"
8-1/2"