Superior Panel Base Anchor (PBA-S) Seismic Calculations

Scope: Design a connection for the PBA-10K to anchor the wall panels to the wall footing that will meet the provisions of ACI 318-19, Chapter 17, Section 17.10 Earthquake-resistant anchor design requirements. This will allow the PBA-10K to be used for structures assigned to Seismic Design Category (SDC) C, D, E, or F to anchor wall panels to the footing for design tension (uplift) forces while still meeting the requirements of ACI 318-19, Chapter 16, Section 16.2.4.3 (b).

Use ½ inch diameter F1554 Grade 55 threaded rod epoxied into the footing to resist seismic design tension (uplift) forces.

Material properties: $F_y = 55\ ksi; F_u = 75\ ksi\ to\ 95\ ksi$

Nominal unthreaded body area of threaded part:

$$A_b = \frac{\pi d^2}{4} = \frac{\pi (0.5)^2}{4} = 0.1963\ in^2$$

Per AISC, nominal tensile stress, $F_{nt} = 0.75 F_u$ AISC Table J3.2

Design tensile strength, $\Phi R_n = F_{nt} A_b$ (AISC J3.1)

$\Phi = 0.75 \rightarrow \Phi R_n = 0.75\ (0.75\ x\ 75\ ksi)(0.1963\ in^2) = 8.28\ kips$

Nominal tensile strength, $R_n = 8.28\ kips\ /\ 0.75 = 11.04\ kips$

Note: $F_u = 75\ ksi$ was used to determine the minimum value of ΦR_n for design. The nominal tensile strength of 11.04 kips exceeds the 10.0 kip requirement of ACI Section 16.2.4.3 (b).

Use ACI 318-19, Chapter 17 to determine the required anchorage design force for embedment of the threaded rod into the footing.

Nominal steel strength of an anchor in tension, $N_{sa} = A_{se,N} f_{uta}$ (ACI 17.6.1.2)

$A_{se,N}$ is the effective cross-sectional area of an anchor in tension, in^2

f_{uta} is the specified tensile strength of anchor steel, psi

Use $f_{uta} = 1.5\,(F_y) = 1.5\,(55\,ksi) = 82.5\,ksi = 82{,}500\,psi$ per ACI Section R17.10.1

$A_{se,N} = \frac{\pi}{4}(d_a - \frac{0.9743}{n_t})^2$ (ASME B1.1)

n_t = the number of threads per inch = 13 for coarse thread – UNC ½ in. diameter threaded rods

$$A_{se,N} = \frac{\pi}{4}(0.5\,in - \frac{0.9743}{13})^2 = 0.1419\ \mathrm{in}^2$$

$$N_{sa} = (0.1419\,in^2)\,(82{,}500\,psi) = 11{,}707\,lbs$$

$\Phi = 0.75$ per ACI Table 17.5.3 (a) anchor strength governed by steel. Ductile steel element in tension.

Anchor design tensile strength; $\Phi N_{sa} = 0.75\,(11{,}707\,lbs) = 8780\,lbs$

Load rate connection: Use average of AISC and ACI values

$\Phi N_{sa} = N_{ua}$ = Design tensile strength = $(8.28\,kips + 8.78\,kips)/2 = 8.5\,kips$

Required Anchor Embedment:

ACI R17.10.1 – Ideally, for tension, anchor strength should be governed by yielding of the ductile steel element of the anchor.

If ductility requirements of 17.10.5.3 (a) are satisfied, then any attachments to the anchor should be designed not to yield.

ACI 17.10.3 Post-installed anchors shall be qualified for earthquake-induced-forces in accordance with ACI 355.2 or ACI 355.4.

ACI 17.10.5.3 Anchors and their attachments shall satisfy (a), (b), (c), or (d).

ACI 17.10.5.3 (a) For single anchors, the concrete-governed strength shall be greater than the steel strength of the anchor.

ACI 17.10.5.3 (a) (i) The steel strength shall be taken as 1.2 times the nominal steel strength of the anchor.

ACI 17.10.5.3 (a) (iii) Anchors shall transmit the tensile loads via a ductile steel element with a stretch length of at least $8d_a$ unless otherwise determined by analysis.

Required concrete-governed nominal strength for determining the required embedment = $1.2\ (11{,}707\ lbs) = 14{,}048\ lbs$

Specify 14,000 lbs for required embedment design

Required stretch length $= 8\ (0.5\ in) = 4\ in$

Use the PBA-S Design Tension Capacity as:

$\boldsymbol{N_u = 8.5\ kips}$

$\boldsymbol{N_n = 11.4\ kips}$

Use a ½ inch diameter F1554 Grade 55 galvanized threaded rod epoxied into the footing and a ¼ inch x 2 inch x 2 inch galvanized plate washer under the heavy hex nut.

Per ACI R17.10.1, since the anchor is governed by yielding of the ½ inch diameter F1554 Grade 55 threaded rod and the ductility requirements of ACI 17.10.5.3 (a) are satisfied by the anchor design, verify that the attachments to the anchor are designed not to yield.

<u>See attached design drawing DS-1 for details</u>

Nominal tensile force to threaded anchor bolt into the footing, $N_{sa} = 11{,}700\ lbs.$

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 or A572 Grade 50, f_y = 50,000 psi

Ductile Iron Bars: ASTM A536 Grade 80-55-6, f_y = 55,000 psi

Calculate Component Forces:

Nominal Moment, $\mathrm{M_n}$

$$\mathrm{M_n} = 11.7\ \text{kips}\ (2.0\ \text{in.} + 0.125\ \text{in.} + 2.5\ \text{in.}/2) = 11.7\ \text{kips}\ (3.375\ \text{in.}) = 39.5\ \text{kip-in.}$$

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

Nominal Bearing Strength, $\mathrm{B_n}$

$\mathrm{B_n} = 0.85 f'_c \mathrm{b_{eff}}\ \mathrm{l_b} = 0.85\ (4{,}000\ \text{psi})\ (2.5\ \text{in. x } 2.5)\ \mathrm{l_b} = 21{,}250\ \mathrm{l_b}\ \ \text{lbs.}$ (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, $\mathrm{b_w} = 2.5\ \text{in.}$ $\mathrm{l_b}$ = Bearing length, in.

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

$M_n = 39.5 \text{ kip-in.} = 39{,}500 \text{ lb-in.}$

$\text{Concrete force} = 39{,}500 \text{ lb-in.} / \left(3.875 \text{ in.} - \frac{l_b}{2} \text{ in.}\right)$

Set B_n = Concrete force and solve for l_b

$21{,}250\ l_b \text{ lb.} = 39{,}500 \text{ lb-in.} / \left(3.875 \text{ in.} - \frac{l_b}{2} \text{ in.}\right)$

$39{,}500 \text{ lb-in.} = (21{,}250\ l_b \text{ lb.})\left(3.875 \text{ in.} - \frac{l_b}{2} \text{ in.}\right)$

$39{,}500 = 82{,}343.75\ l_b - 10{,}625\ l_b^2$ (rearrange terms)

$10{,}625\ l_b^2 - 82{,}343.75\ l_b + 39{,}500 = 0$

$l_b^2 - 7.75\ l_b + 3.718 = 0$ (solving, $l_b = 0.5138$ in.)

$B_n = 21{,}250\ (0.5138) \text{ lb.} = 10{,}918.25 \text{ lb.}$

$\text{Resisting Moment} = 10{,}918.25 \text{ lb.}\ (3.875 \text{ in.} - 0.5138 \text{ in.}/2) = 39{,}503 \text{ lb-in.} \approx 39{,}500 \text{ lb-in.}$

$\text{Force to DBAs} = 11{,}700 \text{ lb.}\ (2 \text{ in.} + 5.25 \text{ in.} - 0.5138 \text{ in.}/2) / (5.25 \text{ in.} - 0.125 \text{ in.} - 2.5 \text{ in.}/2 - 0.5138 \text{in.}/2)$

$= 11{,}700 \text{ lb.}\ (6.993 \text{ in.}) / 3.618 \text{ in.} = 22{,}614 \text{ lb.}$ (Tension Capacity Required)

$\text{DBA Capacity} = 2\ (0.2 \text{ in}^2)\ (70{,}000 \text{ psi}) = 28{,}000 \text{ lb.} > 22{,}614 \text{ lb. required}$

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

A706 #4s Capacity $= 2\ (0.2 \text{ in}^2)\ (60{,}000 \text{ psi}) = 24{,}000 \text{ lb.} > 22{,}614 \text{ lb. required}$

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

Steel Bearing Bar: 1 in. x 2.5 in. x 0'-6"

$\text{Load to Bearing Bar} = 22{,}614\ \text{lb.}/2.5\ \text{in.} = 9{,}045.6\ \text{lb/in.}$

$\text{Nominal Shear Capacity Required}, V_n = 22{,}614\ \text{lb.}/2 = 11{,}307\ \text{lb.}$

$\text{Nominal Moment}, M_n = 11{,}307\ \text{lb.}\,(1\ \text{in.} + 2.5\ \text{in.}/2) - 9{,}045.6\ \text{lb/in.}\,(1.25\ \text{in.})(1.25\ \text{in.}/2)$

$= (25{,}441\ \text{lb-in.}) - (7{,}067\ \text{lb-in.})$

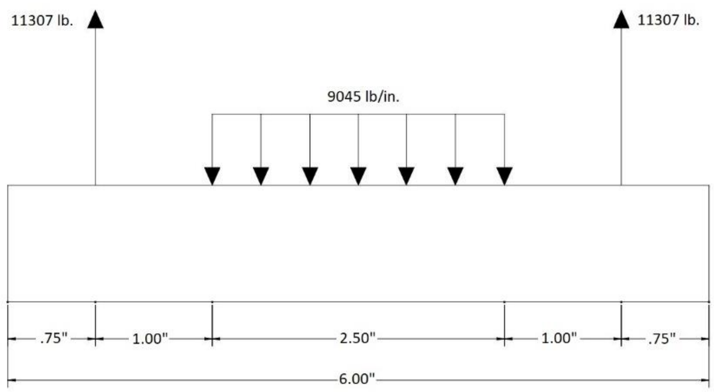
$= 18{,}374\ \text{lb-in.} = 18.37\ \text{kip-in.}$

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

$f_b = 18.37\ \text{kip in.}/0.625\ \text{in}^3 = 29.4\ \text{ksi} < 50\ \text{ksi}$ (Does not yield)

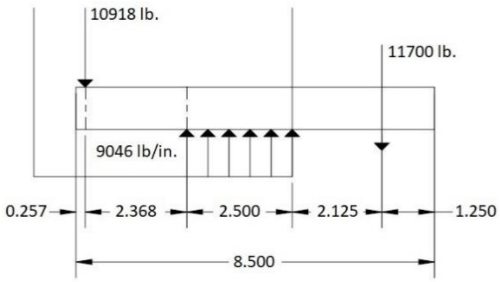
$V_n = 0.6\ (50\ \text{ksi})\ (1\ \text{in.} \times 2.5\ \text{in.}) = 75.0\ \text{kips} > 11.3\ \text{kips Required}$ (AISC G2-1)

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"



Bearing bar uniform load

$= 22{,}614\ \text{lb.}/2.5\ \text{in.} = 9{,}045.6\ \text{lb/in.} = 9.05\ \text{kips/in.}$

Nominal Moment,

$M_n = 10.918\ \text{kips}\ (2.368\ \text{in.}) + 10.918\ \text{kips}(1.21\ \text{in.})0.5$

(At 0 Shear)

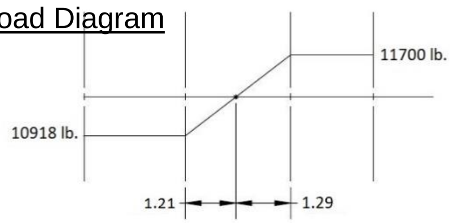
$= 25.85\ \text{kip-in.} + 6.61\ \text{kip-in.} = 32.46\ \text{kip-in.}$

or

$= 11.7\ \text{kips}\ (2.125\ \text{in.}) + 11.7\ \text{kips}\ (1.29\ \text{in.})0.5$

$= 24.86\ \text{kip-in.} + 7.55\ \text{kip-in.} = 32.41\ \text{kip-in.}$

Load Diagram



Shear Diagram

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

$f_b = 32.46\ \text{kip-in.} / 0.625\ \text{in}^3 = 51.94\ \text{ksi} < 55\ \text{ksi}$ (Does not yield)

$V_n = 0.6\ (55\ \text{ksi})\ (1\ \text{in.} \times 2.5\ \text{in.}) = 82.5\ \text{kips} > 11.7\ \text{kips}$ Required (AISC G2-1)

Use ductile iron bar 1 in. x 2.5 in. x 0'-8-1/2"

Note: The 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.

<u>Note to panel design professional</u>:

When using the PBA-S to resist uplift loads due to seismic forces in structures assigned to Seismic Design Category (SDC) C, D, E or F, a ½ inch diameter F1554 Grade 55 galvanized threaded rod epoxied into the footing should be used in order to develop the rated capacity of the connection. The panel design engineer is responsible for determining and specifying the threaded rod embedment depth into the footing and the epoxy to be used, based on the specific requirements for the project, such as footing concrete strength and anchor edge distance necessary to develop the required concrete anchorage tensile strength of 14,000 pounds.

Relevant ACI 318-19 Code Sections 16 & 17 References:

Chapter 16 – Connections

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):

(a) Connections between precast columns shall have vertical integrity ties, with a nominal tensile strength of at least **200A_g lb**, where A_g is the gross area of the column. For columns with a larger cross section than required by consideration of loading, a reduced effective area based on the cross section required shall be permitted. The reduced effective area shall be at least one-half the gross area of the column.

(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.

16.2.5 *Integrity tie requirements for precast concrete bearing wall structures three stories or more in height*

16.2.5.2 Vertical integrity ties shall satisfy (a) through (c):
(a) Integrity ties shall be provided in all wall panels and shall be continuous over the height of the building.
(b) Integrity ties shall provide a nominal tensile strength of at least 3000 lb per horizontal foot of wall.
(c) At least two integrity ties shall be provided in each wall panel.

16.3.3.1 Design strengths of connections between columns, walls, or pedestals and foundations shall satisfy Eq. (16.3.3.1) for each applicable load combination. For connections between precast members and foundations, requirements for vertical integrity ties 16.2.4.3 or 16.2.5.2 shall be satisfied.

$$\phi S_n \geq U \qquad (16.3.3.1)$$

where S_n is the nominal flexural, shear, axial, torsional, or bearing strength of the connection.

16.3.3.6 At the base of a precast column, pedestal, or wall, anchor bolts and anchors for mechanical connections shall be designed in accordance with **Chapter 17**. Forces developed during erection shall be considered.

16.3.3.7 At the base of a precast column, pedestal, or wall, mechanical connectors shall be designed to reach their design strength before anchorage failure or failure of surrounding concrete.

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.

Chapter 17 – Anchoring to Concrete

17.4—Required strength

17.4.2 For anchors in structures assigned to SDC C, D, E, and F, the additional requirements of 17.10 shall apply.

17.5.2.5 Anchors in structures assigned to Seismic Design Category C, D, E, or F shall satisfy the additional requirements of 17.10.

17.6—Tensile strength

17.6.1 *Steel strength of anchors in tension,* $\mathbf{N_{sa}}$

17.6.1.1 Nominal steel strength of anchors in tension as governed by the steel, $\boldsymbol{N_{sa}}$, shall be evaluated based on the properties of the anchor material and the physical dimensions of the anchors.

17.6.1.2 Nominal steel strength of an anchor in tension, $\boldsymbol{N_{sa}}$, shall be calculated by:

$$N_{sa} = A_{se,N} f_{uta} \tag{17.6.1.2}$$

where $\boldsymbol{A_{se,N}}$ is the effective cross-sectional area of an anchor in tension, in.^{2}, and $\boldsymbol{f_{uta}}$ used for calculations shall not exceed either $\mathbf{1.9}\boldsymbol{f_{ya}}$ or 125,000 psi.

17.10—Earthquake-resistant anchor design requirements

17.10.1 Anchors in structures assigned to Seismic Design Category (SDC) C, D, E, or F shall satisfy the additional requirements of this section.

17.10.5.3 Anchors and their attachments shall satisfy (a), (b), (c), or (d).

(a) For single anchors, the concrete-governed strength shall be greater than the steel strength of the anchor. For anchor groups, the ratio of the tensile load on the most highly stressed anchor to the steel strength of that anchor shall be equal to or greater than the ratio of the tensile load on anchors loaded in tension to the concrete-governed strength of those anchors. In each case:

(i) The steel strength shall be taken as 1.2 times the nominal steel strength of the anchor.

(ii) The concrete-governed strength shall be taken as the nominal strength considering pullout, side-face blowout, concrete breakout, and bond strength as applicable. For consideration of pullout in groups, the ratio shall be calculated for the most highly stressed anchor.

In addition, the following shall be satisfied:

(iii) Anchors shall transmit tensile loads via a ductile steel element with a stretch length of at least $\mathbf{8}d_a$ unless otherwise determined by analysis.

(iv) Anchors that resist load reversals shall be protected against buckling.

(v) If connections are threaded and the ductile steel elements are not threaded over their entire length, the ratio of f_{uta}/f_{ya} shall be at least 1.3 unless the threaded portions are upset. The upset portions shall not be included in the stretch length.

(vi) Deformed reinforcing bars used as ductile steel elements to resist earthquake-induced forces shall be in accordance with the anchor reinforcement requirements of 20.2.2

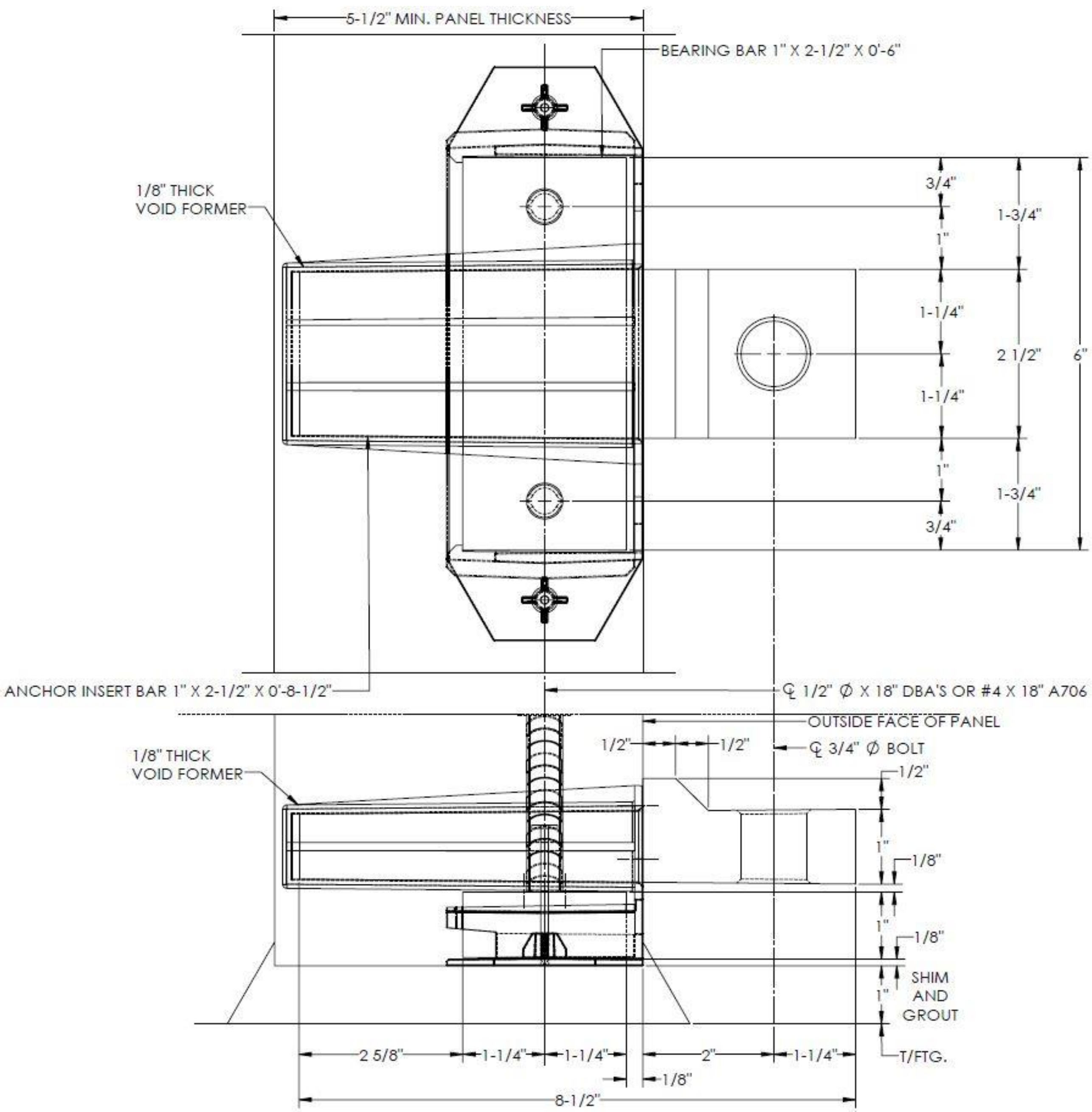
17.10.5.4 The anchor design tensile strength shall be calculated from (a) through (e) for the failure modes given in Table 17.5.2 assuming the concrete is cracked unless it can be demonstrated that the concrete remains uncracked.

(a) ϕN_{sa} for a single anchor, or for the most highly stressed individual anchor in an anchor group
(b) $0.75\phi N_{cb}$ or $0.75\phi N_{cbg}$, except that N_{cb} or N_{cbg} need not be calculated if anchor reinforcement satisfying 17.5.2.1(a) is provided
(c) $0.75\phi N_{pn}$ for a single anchor or for the most highly stressed individual anchor in an anchor group
(d) $0.75\phi N_{sb}$ or $0.75\phi N_{sbg}$
(e) $0.75\phi N_{a}$ or $0.75\phi N_{ag}$

R17.10.1 Unless 17.10.5.1 or 17.10.6.1 apply, all anchors in structures assigned to Seismic Design Categories (SDC) C, D, E, or F are required to satisfy the additional requirements of 17.10.2 through 17.10.7, regardless of whether earthquake-induced forces are included in the controlling load combination for the anchor design. In addition, all post-installed anchors in structures assigned to SDC C, D, E, or F must meet the requirements of ACI 355.2 or ACI 355.4 for prequalification of anchors to resist earthquake-induced forces. **Ideally, for tension, anchor strength should be governed by yielding of the ductile steel element of the anchor.** If the anchor cannot meet the specified ductility requirements of 17.10.5.3(a), then the attachment should be designed to yield if it is structural or light gauge steel, or designed to crush if it is wood. If ductility requirements of 17.10.5.3(a) are satisfied, then any attachments to the anchor should be designed not to yield. In designing attachments using yield mechanisms to provide adequate ductility, as permitted by 17.10.5.3(b) and 17.10.6.3(a), the ratio of specified yield strength to expected strength for the material of the attachment should be considered in determining the design force. The value used for the expected strength should consider both material overstrength and strain hardening effects. For example, the material in a connection element could yield and, due to an increase in its strength with strain hardening, cause a secondary failure of a sub-element or place extra force or deformation demands on the anchors. For a structural steel attachment, if only the specified yield strength of the steel is known, the expected strength should be taken as approximately 1.5 times the specified yield strength. If the actual yield strength of the steel is known, the expected strength should be taken as approximately 1.25 times the actual yield strength.

Under earthquake conditions, the direction of shear may not be predictable. The full shear should be assumed in any direction for a safe design.

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
1/2"
1/2"
℄ 3/4" Ø BOLT
1/2"
1/8" THICK
VOID FORMER
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"