Design of Anchor Reinforcement in Concrete Pedestals

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ABSTRACT

Even though the Appendix D of the ACI 318-05 permits the use of supplementary reinforcement to restrain concrete breakout, it does not provide specific guidelines in designing such reinforcement. This paper presents a method for designing anchor reinforcement in concrete pedestals, where un-reinforced concrete is insufficient to resist anchor forces. Anchor reinforcement consists of longitudinal rebar and ties to carry anchor tension forces and shear forces, respectively. The Strut-and-Tie Model is proposed to analyze shear force transfer from anchors to pedestal and to design the required amount of shear reinforcement. A proposed design procedure is illustrated in an example problem.

KEYWORDS: Anchor, Anchorage, Anchor reinforcement, Strut-and-Tie Model

1. INTRODUCTION

The Appendix D of the ACI 318-05 provides design requirements for anchors in un-reinforced concrete. It addresses only the anchor strength and the un-reinforced concrete strength:

1. Breakout strength,
2. Pullout strength,
3. Side-face blowout strength
4. Pryout strength.

Even though the Appendix D of the ACI 318-05 permits the use of supplementary reinforcement to restrain the concrete breakout (Section D.4.2.1), it does not provide specific guidelines in designing such reinforcement. Commentary of Section D.4.2.1 indicates that the designer has to rely on other test data and design theories in order to include the effects of supplementary reinforcement.

In petrochemical industry, concrete pedestals commonly support static equipment (i.e. horizontal vessels and heat exchangers) and pipe-rack or compressor building columns. In order to fully-develop the strength of anchor in un-reinforced concrete, the Appendix D of the ACI 318-05 requires the use of significantly large concrete pedestals/octagons. It is generally not economical to provide such large concrete pedestals/octagons. Therefore, the anchorage design in petrochemical industry almost always includes designing supplementary reinforcement. When supplementary reinforcement is used to transfer the full design load from the anchors, it is generally referred as anchor reinforcement. Figure 1 shows anchors of a compressor building column on a reinforced concrete pedestal.

Figure 1. Pedestal supporting a compressor-building column

This paper presents a method for designing anchorage in concrete pedestals with anchor reinforcement to anchor static equipment or columns in petrochemical facilities. The anchor tension and shear forces are assumed to be resisted by the vertical reinforcing bars and ties, respectively. The calculation for determining the required amount of vertical reinforcing bars and ties is presented. A design example of column anchorage in a reinforced concrete pedestal is given to illustrate the proposed design method.

2. DESIGN PHILOSOPHY

The following general design philosophy is used when the anchor forces are assumed to be resisted by the steel reinforcement:

1. Concrete contribution is neglected in proportioning the steel reinforcement.
2. When a non-ductile design is permitted, the reinforcement should be designed to resist the factored design load.
3. When a ductile design is required, the reinforcement should be proportioned to develop the strength of the anchor. If the anchor is sized for more than 2.5 times

factored tension design loads, it is permitted to design the reinforcement to carry 2.5 times the factored design load, where 2.5 is an overstrength factor.

4. When reinforcement is used to restraint concrete breakout, the overall anchorage design should ensure that there is sufficient strength corresponding to the three other failure modes described in the Introduction (pullout failure, side-face blowout failure, and pryout failure).

The three failure modes will be addressed as follows:

a. The pullout strength of headed anchors N_p can be estimated using the Eq. (D-15) of the ACI 318-05 (i.e. $N_p = 8A_{brg} f_c'$, where A_{brg} is the net bearing area of the anchor head).

b. The side-face blowout failure can be prevented by providing enough edge distance. Section D.5.4 of the ACI 318-05 implicitly indicates that the side-face blowout failure should be checked when the edge distance c is smaller than 0.4 times the effective embedment depth h_{ef} ($c < 0.4\ h_{ef}$). Since h_{ef} of anchors in reinforced pedestals is usually governed by the required development length for reinforcing steel (which can be significantly deeper than the $h_{ef,min}$ of 12 times anchor diameter d_o) and since the side-face blowout failure is independent of the embedment depth when the embedment depth is deeper than 12″ (Furche and Elingehausen, 1991), the minimum edge distance of $0.4 \times 12d_o = 4.8d_o$ can be used to prevent the side-face blowout failure. However, in order to satisfy the required minimum edge distance for cast-in headed anchors that will be torqued, the minimum edge distance of $6d_o$ should be used (Section D.8.2, ACI 318-05). Therefore, for simplicity and to prevent the side-face blowout failure, the minimum edge distance of $6d_o$ is recommended.

When it is impossible to provide the minimum edge distance of $6d_o$, the side-face blowout strength should be calculated using Section D.5.4 of the ACI 318-05. In addition, reinforcement may be provided to improve the behavior related to concrete side-face blowout (Fig. 2). Furche and Elingehausen (1991) found that the size of the lateral blow-out at the concrete surface was 6 to 8 times the edge distance. Cannon et al. (1981) recommended spiral reinforcement around the head. It should be emphasized that transverse reinforcement (ties) did not increase the side-face blowout capacity (DeVries et al. (1998)). Large amount of transverse reinforcement installed near the anchor head only increased the magnitude of load that was maintained after the side-face blowout failure occurred.

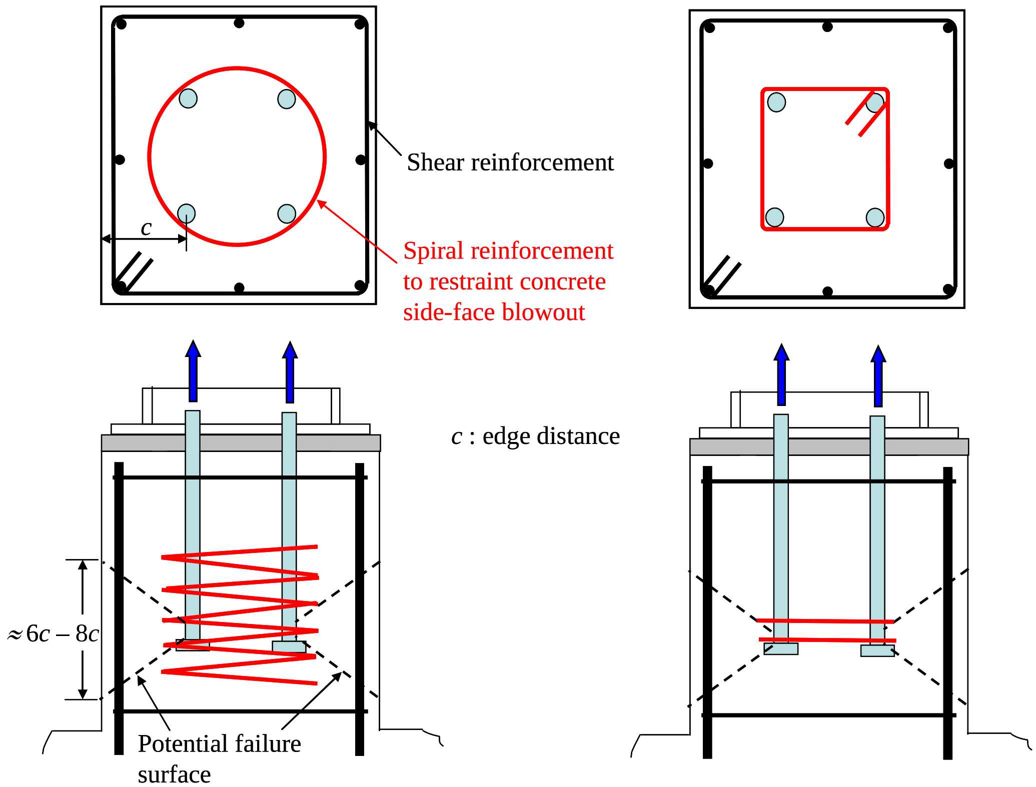


Figure 2. Reinforcement around the head to improve the behavior related to concrete side-face blowout

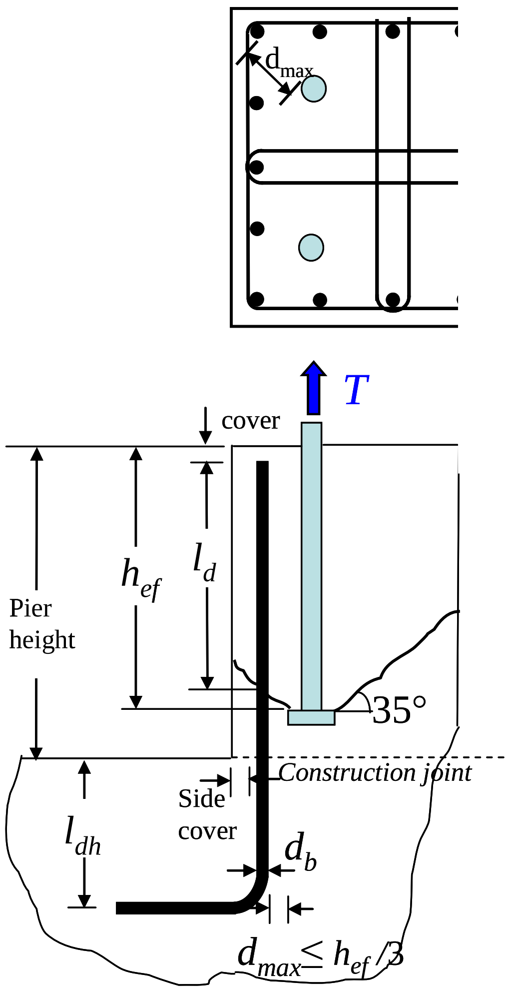
When the reinforcement is used to restraint concrete side-face blowout, it should be designed to carry the lateral force causing the side-face blowout. Cannon et al. (1981) indicated that for conventional anchor heads, the lateral force causing side-face blowout may be conservatively taken as ¼ of the tensile capacity of the anchor steel (based on the Poisson effect in the lateral direction). A more complex procedure to calculate the lateral force is given in Furche and Elingehausen (1991). In general, the Furche and Elingehausen's procedure gives a smaller lateral load than that recommended by Cannon et al. (1981).

c. The pryout failure is only critical for short and stiff anchors. It is reasonable to assume that for general cast-in place headed anchors with $h_{ef,min}$ = 12 d_o, the pryout failure will not govern.

3. DESIGNING STEEL REINFORCEMENT TO CARRY TENSION FORCES

The vertical reinforcement intersects potential crack planes adjacent to the anchor head thus transferring the tension load from the anchor to the reinforcement as long as proper development length is provided to develop the required strength, both above and below the intersection between the assumed failure plane and reinforcement (Fig. 3). The development length may be reduced when excess reinforcement is provided per section 12.2.5 of the ACI 318-05 (but cannot be less than 12″). Reduction in the development length cannot be applied in the areas of moderate or high seismic risk. In order to limit the embedment length of anchor, a larger number of smaller-size reinforcing bars is preferred over fewer, larger-size reinforcing bars.

To be considered effective, the distance of the reinforcement from the embedded anchor head or nut should not exceed one-third of the embedment length of the anchor h_{ef}, as shown in Fig. 3 (Cannon et al., 1981).



Note:

To be considered effective for resisting anchor tension, the maximum distance from anchor head to the reinforcement, d_{max}, shall be not more than $h_{ef}/3$.

Figure 3. Reinforcement for carrying anchor tension force

When a non-ductile failure is permitted, the required area of steel reinforcement A_{st} can be determined as follows:

$$A_{st} \geq \frac{T_u}{\phi \; f_y} \tag{1}$$

When a ductile failure is required:

$$A_{st} \geq \frac{A_{se}\ f_{uta}}{f_y} \tag{2}$$

However, the anchor is sized for more than 2.5 times factored tension design loads T_u, it is permitted to design the reinforcement to carry 2.5 times T_u to satisfy IBC 2006 and ASCE 7-05 requirements for Seismic Design Categories C and above where ductility cannot be achieved. The required area of steel reinforcement A_{st} can be determined as follows:

$$A_{st} \geq \frac{2.5\,T_u}{\phi\ f_y} \tag{3}$$

where:

A_{se} = effective cross-sectional area of anchor
T_u = factored tension design load per anchor
ϕ = 0.90, strength reduction factor (Chapter 9 of the ACI 318-05)
f_y = specified minimum yield strength of reinforcement
f_{uta} = specified minimum tensile strength of anchor steel

Design for anchor ductility requires that the necessary conditions for elongation over a reasonable gage length are fulfilled (i.e., that strain localization will not limit the yield strain). This may involve the use of upset threads or other detailing methods to avoid strain localization.

4. DESIGNING STEEL REINFORCEMENT TO CARRY SHEAR FORCES

Where allowed by Code, shear may be transferred by friction between the base plate and the concrete with the anchors are used for transferring tension force only. For large shear forces, where the shear friction is insufficient, shear lugs or anchors can be used to transfer the load. The shear forces must be transferred to concrete pedestal. *Strut-and-tie models* can be used to analyze shear transfer to concrete pedestal.

4.1. What is the strut-and-tie models (STM)?

A strut-and-tie model (STM) is an ultimate strength design method based on the formation of a hypothetical truss that transmits forces from loading points to supports. The STM utilizes concrete struts to resist compression and reinforcing ties to carry tension. Design using STM involves calculating the required amount of reinforcement to serve as the tension ties and then checking that the compressive struts and nodal zone (joints) are sufficiently large enough to support the forces. A key advantage of design using STM is that the designer can visualize the flow of stresses in the member. A common application of the STM is to design "disturbed" regions (i.e. at concentrated loads and reactions, and at geometric discontinuity), where the flow of stresses cannot be predicted by normal "beam theory" (i.e. linear strain distribution).

The most important assumptions in the STM are:

1. Failure is due to the formation of a mechanism resulting from yielding of one or more ties.
2. Crushing of the concrete struts should not occur prior to yielding of the ties. This is prevented by limiting the stress levels in the concrete.
3. Only uniaxial forces are present in the struts and ties
4. The reinforcement is properly detailed to prevent local bond or anchorage failure.

Since the STM satisfies force equilibrium and ensures that the yield criterion is nowhere exceeded in the structure, the STM satisfies the requirements of a lower bound solution in the theory of plasticity. This implies that the failure load computed by the STM underestimates the actual failure load.

ACI Design provision using STM was first introduced in the Appendix A of the ACI 318-02. Several important guidelines of using STM as a design tool according to the ACI 318-05 are:

1. The STM shall be in equilibrium with the applied loads and the reactions
2. Ties shall be permitted to cross struts and struts shall cross or overlap only at nodes
3. The angle between the axes of any strut and any tie entering a single node shall not be taken as less than 25 degrees.
4. The tie force shall be developed at the point where the centroid of the reinforcement in a tie leaves the extended nodal zone.

4.2 Advantages and assumptions for shear transfer analysis in concrete pedestals using STM

The advantage of using STM for analyzing shear transfer and designing shear reinforcement on pedestal anchorages is the elimination of "questionable" assumptions related to the size and shape of concrete breakout cone, the crack location (whether the shear cracks propagate from the middle of pedestals, front-row anchors, or back-row anchors), and the amount of shear reinforcement that is effective to restraint concrete breakout cone.

While the STM is a conceptually simple design tool, it requires an assumption for the following parameters:

1. Capacity of struts and nodes
2. Geometry of struts and nodal zones
3. Anchorage of tie reinforcement

In order to shed a light in the lack of guidelines, the following assumptions are suggested in order to proceed with the use of STM for shear transfer analysis on pedestal anchorage and for designing the anchor shear reinforcement:

1. Concrete strength for struts and bearings f_{cu} is 0.85 $f_c{}'$ based on the Appendix A of the ACI 318-05. This assumption is conservative considering significant amount of confinement in pedestals.
2. The concrete struts from anchors to vertical rebars are shown in Fig. 4. Section D.6.2.2 of the ACI 318-05 indicates that the maximum load bearing length of the anchor for shear is $8d_o$. Therefore, the bearing area of the anchor is assumed $(8d_o)d_o = 8d_o^2$. The compressive force from the anchor to rebar is assumed to spread with a slope of 1.5 to 1.
   When the internal ties are not required (in the case where axial force in the pedestal is so small that Section 7.10.5.3 of the ACI 318-05 does not apply), the STM shown in Fig. 5 can be used. For a given anchor shear V, the tension tie force T in Fig. 5 is larger than T_1 in Fig. 4.

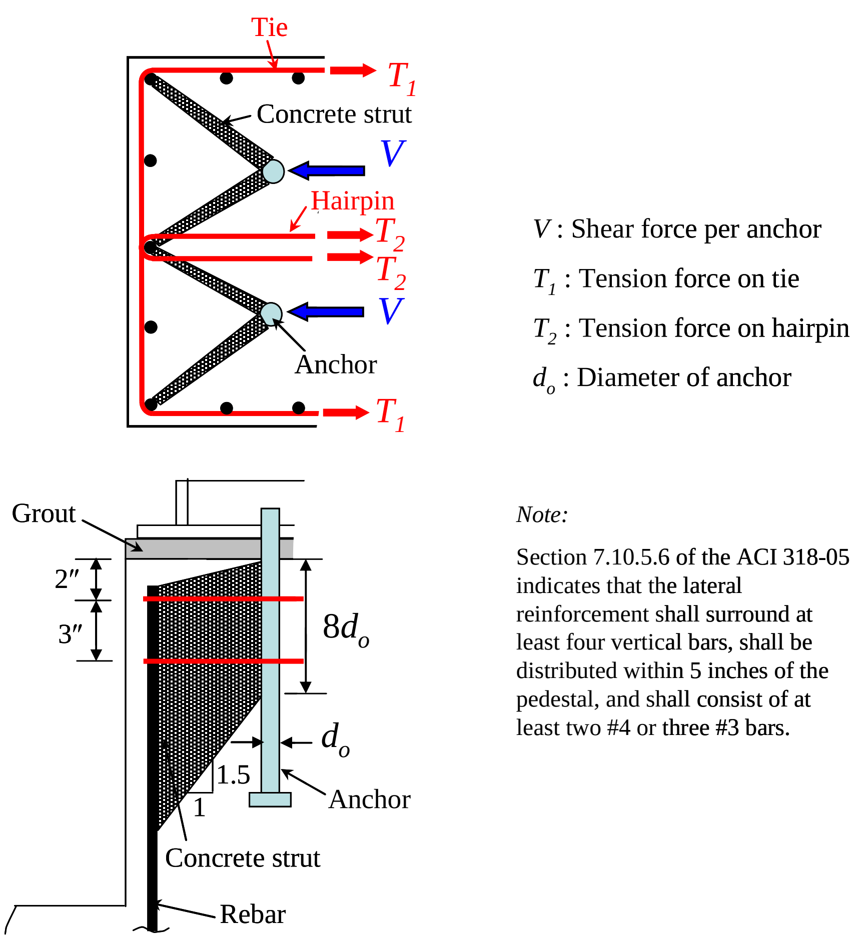


Figure 4. Concrete struts and tension ties for carrying anchor shear force

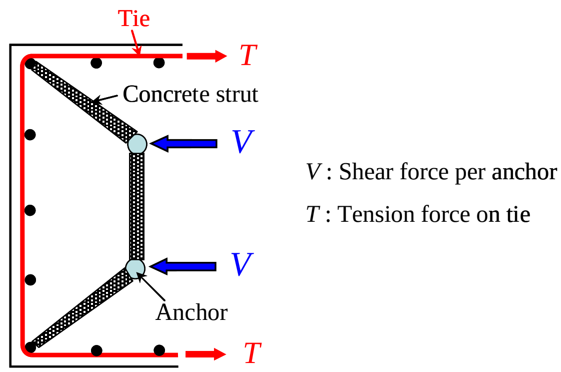


Figure 5. STM without internal ties

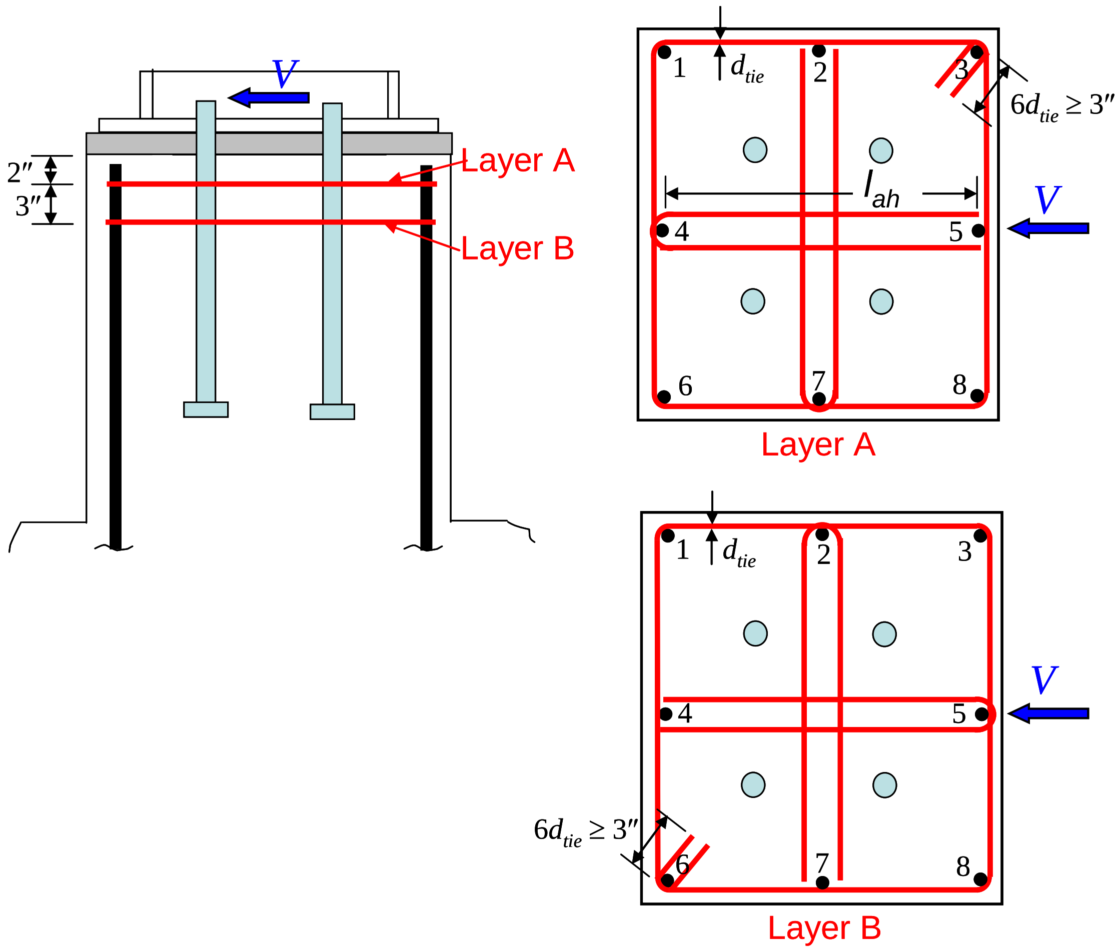


Figure 6. Alternated direction of hooks and hairpins for the top most two layers of ties

3. For tie reinforcement, the following assumptions are suggested:
   a. Only the top most two layers of ties (Assume 2-#4 within 5″ of top of pedestal as required by Section 7.10.5.6 of the ACI 318-05), shown in Fig. 6, are effective.

b. Tie reinforcement should consist of tie with seismic hooks. If internal ties are required, hairpins could be used. As an alternative, diamond-shaped ties can also be used.
c. The location of hooks and the direction of hairpins should be alternated as shown in Fig. 6.
d. If the available length of hairpin l_{ah} (Fig. 6) is shorter than the required straight development length for a fully developed hairpin l_d, the maximum strength that can be developed in hairpin is $f_y \times \frac{l_{ah}}{l_d}$, where f_y is the yield strength of hairpin. If l_{ah} is shorter than 12″ (i.e. the minimum development length based on Section 12.2.1 of the ACI 318-05), hairpin should not be used.
e. At the nodes away from the hook, the tie is assumed to be fully developed. For example, under the shear force V, the tie on layer A can develop f_y at the nodes 1 and 6 (Fig. 6).
f. At the node where the hook is located, the tie cannot develop f_y. For example, under the shear force V, while the tie on layer A (Fig. 6) can develop f_y at the node 6, the tie on layer B cannot develop f_y because the hook of tie B is located at the node 6. In order to calculate the contribution from tie B to the tension tie at the node 6, the stiffness of “Case 1” shown in Fig. 7 (smooth rebar with 180° hook bearing in concrete (Fabbrocino et al., 2005)) is compared to the stiffness of “Case 2” (the conventional single-leg stirrup with reinforcing bars inside the bends (Leonhardt and Walther, 1965 as cited in Ghali and Youakim, 2005)). Even though the capacity of “Case 2” may be higher than the capacity of “Case 1” due to bearing on the heavier rebar, the contact will not always present because of common imprecise workmanship. When the contact is not present, the “Case 2” is assumed to behave as “Case 1”.

Figure 7. Bearing of J-shape bars on concrete and bearing of conventional stirrup on rebar

Leonhardt and Walther (1965) found that in order to develop f_y on the bends of 90°, 135°, and 180° hooks when engaging heavier bars lodged inside the bends (“Case 2” in Fig. 7), there was a slip about 0.2 mm. Based on the test results of Fabbrocino et al. (2005), the stress at the hook that was developed at the smooth rebar with 180° hook bearing in concrete when it slipped 0.2 mm was about 20 ksi. Therefore, it is assumed that the tie can only develop 20 ksi at the node where the hook is located. It is also reasonable to assume that the maximum force that can be developed at the

hook is the same as the pullout strength of a single hooked bolt (Eq. (D-16) of the ACI 318-05).
In summary, at the node where the hook is located, the contribution of the ties to the tension ties T is the lesser of Eqs. (4) and (5), where Eq. (5) is based on the Eq. (D-16) of the ACI 318-05.

$$T = A_{tie} \times f_s \tag{4}$$

$$T = 0.9 f_c' e_h d_{tie} \tag{5}$$

where:
A_{tie} is the area of the tie, f_s is the stress on the tie (≈ 20 ksi), e_h is the length of the extension of the hook, and d_{tie} is the diameter of the tie.

5. EXAMPLE PROBLEM

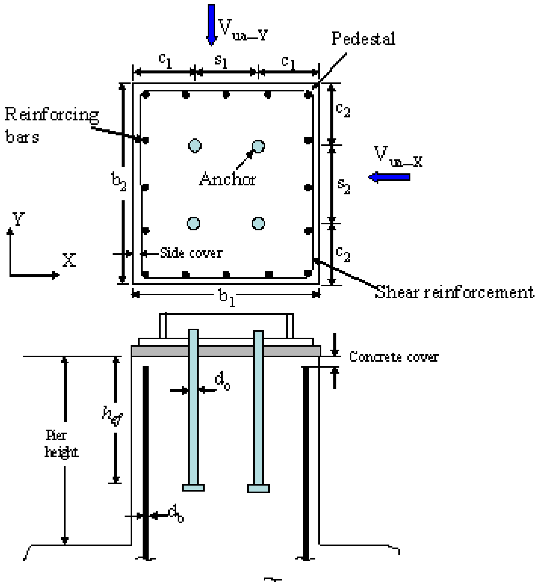


Figure 8

Design the anchor for the steel column located at the top of concrete pedestals shown in Figure 8. Anchors resist tension and shear forces.

Maximum total factored loads:

Tension: $N_{ua\_total} := 80kip$

Maximum shear in the X-direction: $V_{ua\_total\_X} := 20kip$

Maximum shear in the Y-direction: $V_{ua\_total\_Y} := 20kip$

Assumptions:

1. Untorqued, cast-in anchors
2. No sleeve is used
3. Low seismic risk and capacity design is not considered
4. Tension force is distributed equally among all anchors
5. Shear force is assumed to be carried by two anchors because of oversize holes in the base plate

Note: All code section numbers referred in this example are in the ACI 318-05

Pier / Pedestal Data:

Specified compressive strength of concrete: $\boxed{f_c := 4000psi}$ *(Blocks are input data)*

Height: $\boxed{Pier\_height := 28in}$ Concrete_cover := 1.5in $\boxed{Side\_cover := 2in}$

Note: In many cases, the height of the pier is a design constraint.

Cross-section dimensions: $\boxed{b_1 := 24in}$ $\boxed{b_2 := 26in}$

Edge Distance: $\boxed{C_1 := 8 \cdot in}$ $\boxed{C_2 := 8in}$

Anchor Spacing: $\boxed{S_1 := 8in}$ $\boxed{S_2 := 10in}$

Anchors:

Specification: ASTM F1554, A36 $\boxed{f_{ya} := 36ksi}$ $\boxed{f_{uta} := 58ksi}$

ASTM F1554, A36 is a ductile steel (Table 2.1). Therefore: $\phi_T := 0.75$ (tension loads) $\phi_V := 0.65$ (shear loads) (D.4.4.a)

Note: Load combinations shall be per Chapter 9 (or ASCE 7-05, Chapter 2)

Reinforcing bars: Grade 60 steel: $\boxed{f_{y\_rebar} := 60ksi}$

Vertical (longitudinal rebars): #6 $\boxed{d_b := 0.75in}$ $As_b := 0.44in^2$

Shear reinforcement: #4 $\boxed{d_{tie} := 0.5in}$ $As_{tie} := 0.2in^2$

Design assumptions:

1. The tension and the shear forces in the anchors are transfered to the longitudinal rebars and shear reinforcement, respectively, which will restrain the concrete failure prism. Therefore, the concrete breakout strength in tension and shear (D.5.2 and D.6.2) is not checked. The concrete pryout strength in shear (D.6.3) is assumed OK by inspection because it is usually critical for short and stiff anchors.
2. If the edge distance is larger than $6d_o$, the concrete side-face blowout resistance is assumed to be sufficient.
3. When welded washers are not used, it is not likely that all anchors are effective in resisting shear due to oversize holes in the base plate. For this case, it is conservative to assume that only the bolts on the critical face are engaged. For this example, only two anchors are assumed to be effective for resisting shear.

1. Determine the size of anchors

The size of anchors is determined based on the steel strength of anchor in tension and shear. Since the tension force is assumed to be distributed equally, each anchor carries 80 kip / 4 = 20 kip. There are two anchors in both X and Y directions (i.e. half of the total number of anchors) are effective in resisting shear, the maximum shear force carried by one anchor is 20 kip / 2 = 10 kip. If there is any shear in the X-direction acting simultaneously, it may be added here.

Try 1.25-in. anchor: $\boxed{d_o := 1.25in}$ (Threads per inch: $n_t := 7$)

Effective cross-sectional area: $A_{se} := \frac{\pi}{4}\cdot\left(d_o - \frac{0.9743\cdot in}{n_t}\right)^2$

$A_{se} = 0.969\,in^2$

The steel strength of one anchor in tension: $N_{sa} := A_{se}\cdot f_{uta}$ (D.5.1.2) $\phi_T\cdot N_{sa} = 42.156\,kip > N_{ua} := 20kip$ (OK !)

The steel strength of one anchor in shear: $V_{sa} := 0.8\cdot 0.6A_{se}\cdot f_{uta}$ (D.6.1.2.b) $\phi_V\cdot V_{sa} = 17.537\,kip > V_{ua} := 10kip$ (OK !)

Note: Shear strength of anchors with grout pads shall be multiplied by 0.8 (D.6.1.3).

Since $N_f > 0.2\phi_T N_{sa}$ and $V_f > 0.2\phi_V V_{sa}$, check interaction equation based on D.7.3:

$$\frac{N_{ua}}{\phi_T\cdot N_{sa}} + \frac{V_{ua}}{\phi_V\cdot V_{sa}} = 1.045 \qquad < 1.2 \text{ , OK !}$$

The minimum effective embedment depth of the non-sleeve 1.25-in anchor: $h_{ef,min}=12d_o=15$ in

Note: Since the pier height is 28 inches, try h_{ef}=24 in. This effective embedment depth will be checked if it is sufficient for the required development length of vertical reinforcing bars.

2. Check the pullout resistance of anchor in tension (D.5.3.4)

Section D.5.3.4 indicates the load at which the concrete above the anchor head begins to crush. Since the local crushing above the head will greatly reduce the stiffness of the connection, and generally will be the beginning of a pullout failure. The pullout resistance of anchor in tension must be ensured to be larger than the factored tension load (N_{ua}). If the capacity design (which is not considered herein) is performed, the pullout resistance of anchor in tension should be larger than the tensile capacity of the anchor ($\phi_T N_{sa}$).

Use the heavy hex nut (on the anchor head) with the flat-to-flat dimension of 2 inches.

Bearing area: $A_{brg} := 0.866\cdot 2^2 - \frac{5}{4}\cdot 1.25^2$ $A_{brg} := 1.51in^2$

The pullout resistance in tension of a single headed bolt: $N_{pn} = \psi_{c\_P}\, 8\, A_{brg}\, f_c'$ (D.5.3.1 and D.5.3.4)

Assume concrete cracks: $\psi_{c\_P} := 1$ N_{pn} = (1)8(3.68)(4) = 117.8 kip

Strength reduction factor for anchor governed by pullout, assuming condition A (supplementary reinforcement is provided to tie the failure prism): $\phi = 0.75$ (D.4.4.(c))

Therefore: $\phi\, N_{pn}$ = (0.75)(117.8) = 88.4 kip $> N_{ua}$ OK !

3. Check the side-face blowout resistance of anchor in tension (D.5.3.4)

The minimum recommended edge distance = $6d_o$ $6\cdot d_o = 7.5\,in$

The minimum edge distance: $C_{min} := \min(C_1, C_2)$ $C_{min} = 8\,in$ $> 6d_o$, the side-face blowout resistance is OK !

4. Transfer of Anchor Load to Vertical Rebars

4.1 Amount of vertical reinforcing steel

In order to be considered effective for resisting anchor tension, vertical reinforcing steel must be located within $h_{ef} / 3 = 8$ inches from the anchor head or edge of washer. As shown in Figure 9, the number of pier vertical rebars that are effective for resisting anchor tension is 3.

Since capacity design is not considered (for a low seismic risk), determine the required number of vertical rebar to resist N_{ua} :

$$n\_required := \frac{N_{ua}}{\phi_s \cdot f_{y\_rebar} \cdot As_b}$$

For capacity that is governed by yielding of rebars: $\phi_s := 0.9$ (9.3.2.1)

$$n\_required := \frac{20}{(0.9) \cdot (60) \cdot (0.44)}$$

$n\_required := 0.8$ < provided effective number of rebar, OK !

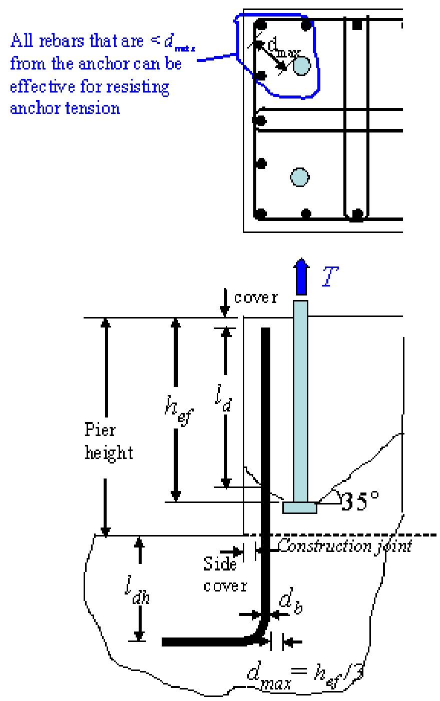


Figure 9

4.2 Development length

The vertical rebar should be developed on either side of the potential failure plane. The part of the rebar above the failure surface is commonly straight and the part of the rebar that goes into the mat is commonly bent (as shown in Figure 9). Therefore, the development length for straight bar applies to the part of the rebar above the failure surface and the development length for the 90-degree hooked bar can be applied to the part of the rebar below the failure surface. Since the development length for the 90-degree hooked bar (below the failure surface) is part of the pier/foundation design, it is not considered in this calculation.

Development length for straight bars above the failure surface:

The minimum development length, l_d (> 12 in), is determined based on the 12.2.2 and 12.2.4 as follows:

Bar location factor: $\psi_t := 1$ (for vertical bar)

Coating factor: $\psi_e := 1$ (for uncoated bar)

Concrete density factor: $\lambda := 1$ (for normal concrete)

For #6 and smaller bars, use: $l_d := \left[\dfrac{f_{y\_rebar}\cdot(\psi_t\cdot\psi_e\cdot\lambda)}{25\sqrt{f'_c}}\right]\cdot d_b$ $\qquad l_d = 28.5\ \text{in}$

Available development length based on the pier height and the embedment depth of the anchor bolt:

$\text{Available\_length} := h_{ef} - \text{Side\_cover} - d_{max}\cdot\tan(35\text{deg})$

From Figure 2, $d_{max} = 5.7$ in $\qquad$ Available_length = 24 - 1.5 - 5.7 tan(35deg) = 18.5 in $\qquad < l_d$

However, since the provided number of effective rebar is significantly more than the required number of rebars and for low seismic risks, l_d can be reduced using the excess reinforcement factor per 12.2.5 but cannot be less than 12 in per 12.2.1.

$l_{d\_reduced} := l_d\cdot\dfrac{A_{s\_required}}{A_{s\_provided}}$ $\qquad l_{d\_reduced} := 28.5\cdot\dfrac{0.8}{3} = 7.6\ \text{in}$ $\qquad$ (12 in. governs in this case)

$<$ Available_length $\qquad$ OK !

5. Design of shear reinforcement

Assumptions:

1. Strut-and-tie modeling (Figure 10) is used to analyze shear transfer to concrete pedestal and to design the required amount of shear reinforcement.
2. Since the shear forces in both directions are the same and the total number of anchors resisting the total shear forces in both directions are the same, only the shear in the X-direction is presented in this example problem.

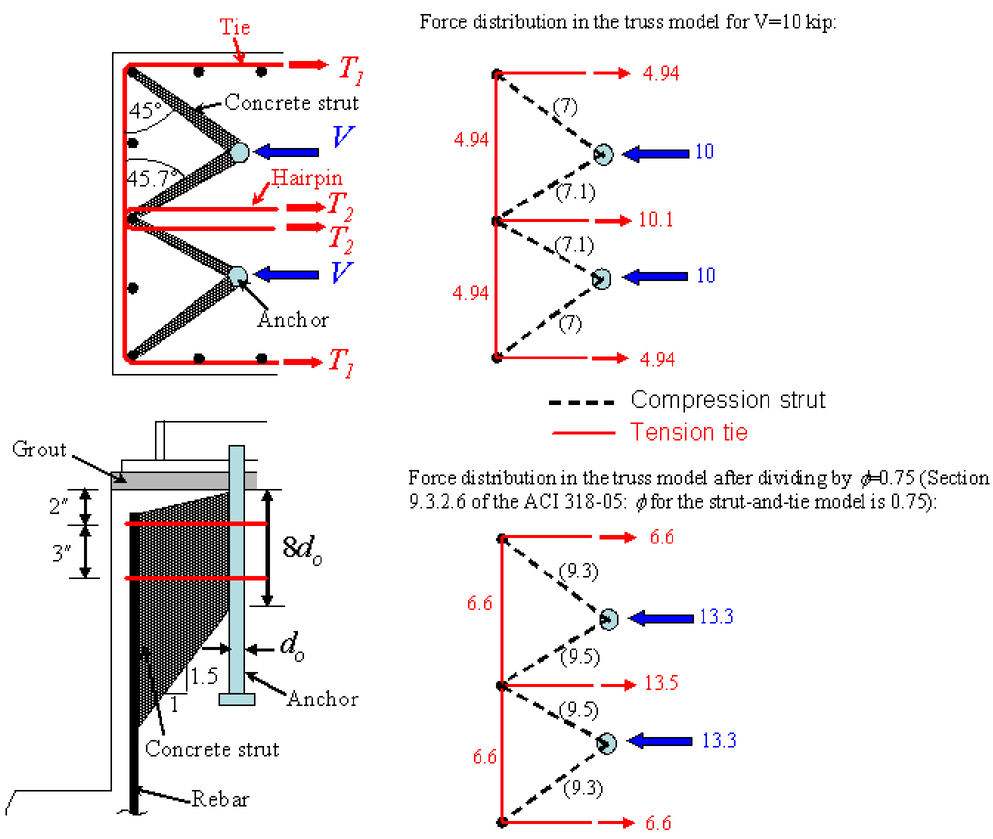


Figure 10

5.1 Check a geometry of the truss model to see if a direct strut can develop

Since the angles between the axes of all struts and ties entering a single node is larger than 25 degrees, direct struts can develop (Section A.2.5 of the ACI 318-05).

5.2 Develop a truss model and calculate member forces

The truss model and member forces are shown in Figure 10.

5.3 Check strength of bearing

Assume concrete strength for checking the strength of bearing and compression struts: $f_{cu} = 0.85\, f'_c$

5.3.a. Bearing of the anchor

Bearing area: $A_{brg\_anc} := 8 \cdot d_o \cdot d_o$

$A_{brg\_anc} = 12.5\,in^2$

Strength: $f_{cu} := 0.85 \cdot (f'_c)$

$f_{cu} = 3400\,psi \quad > \quad \dfrac{13.3kip}{A_{brg\_anc}} = 1064\,psi \qquad$ OK !

5.3.b. Bearing of the reinforcing bars

By inspection, bearing on the rebar at the node D (Fig. 11) governs (shorter length and larger force):

The clear distance between the nodes B and D, l_{BD} : $\quad l_{bd} := \sqrt{(5 \cdot in)^2 + (5.125 \cdot in)^2} - \dfrac{d_o}{2} - \dfrac{d_b}{2}$

$l_{bd} = 6.16\,in$

Bearing area: $A_{brg\_rebar} := (8 \cdot d_o + 1.5 \cdot l_{bd} - Concrete\_cover) \cdot d_b$

$A_{brg\_rebar} = 13.305\,in^2$

Strength: $f_{cu} := 0.85 \cdot (f'_c)$

$f_{cu} = 3400\,psi \quad > \quad \dfrac{9.5kip}{A_{brg\_rebar}} = 714.017\,psi \qquad$ OK !

Force distribution in the truss model after dividing by ϕ=0.75 :

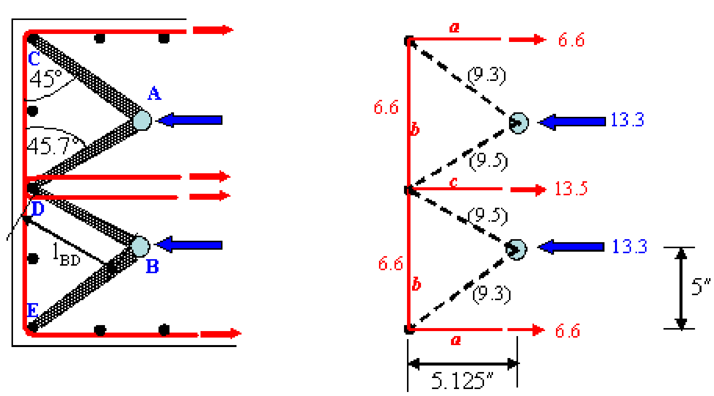


Figure 11

5.4. Check strength of struts

Since it is assumed that the strength of strut is the same as the bearing strength (f_{cu} = 0.85 f_c') and the available area for struts is typically larger than the available area for bearing, the bearing strength governs over the strength of struts. Therefore, if the bearing strengths at the anchor and rebar are OK, the strength of struts does not need to be checked.

5.5. Select tie reinforcement

Assumptions:

1. Only the top most two layers of ties (within 5" of pedestal as required by Section 7.10.5.6 of the ACI 318-05), shown in Fig. 12, are effective.
2. Tie reinforcement consists of tie with seismic hooks. Hairpins are used as internal ties.
3. The location of hooks and the direction of hairpins are alternated as shown in Fig. 12.
4. At the nodes away from the hook, the tie is assumed to be fully developed.
5. At the node where the hook is located, the contribution of the hoop to the tension ties T is the lesser of T_1 and T_2 where $T_1 = As_{tie}$*(20 ksi) and $T_2 = 0.9*f_c*e_h*d_{tie}$ (based on the Eq.(D-16) of the ACI 318-05).

$T_1 := As_{tie} \cdot 20ksi$ $\qquad$ $T_1 = 4\,kip$

The equation based on the Eq. (D-16) of the ACI 318-05:

$e_h := \min(4.5 \cdot d_{tie}, \max(6d_{tie}, 3in))$ $\qquad$ $e_h = 2.25\,in$

$T_2 := 0.9 \cdot f_c \cdot e_h \cdot d_{tie}$ $\qquad$ $T_2 = 4.05\,kip$

$T_{hook} := \min(T_1, T_2)$ $\qquad$ $T_{hook} = 4\,kip$ $\qquad$ Note: (As_{tie}*(20 ksi) governs

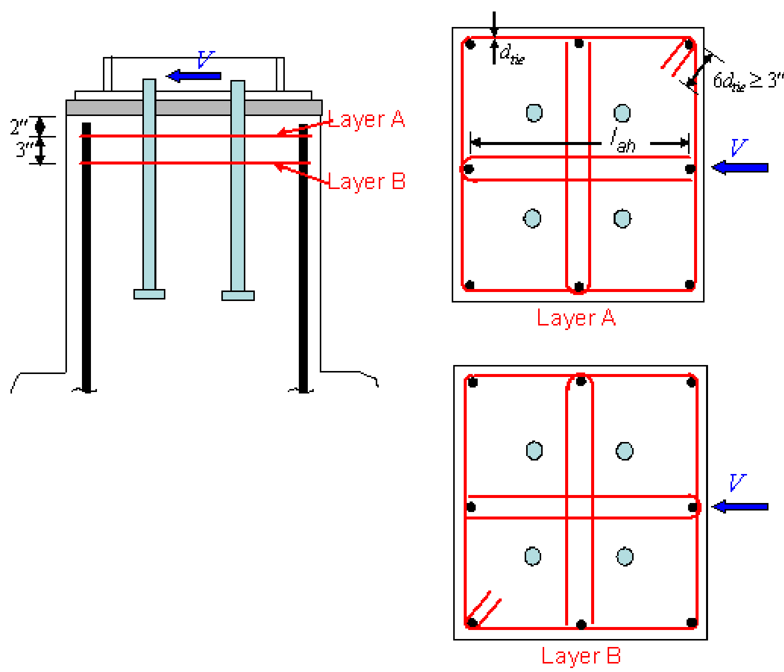


Figure 12

Ties a and b (see Figure 11):

Ties a and b are resisted by exterior ties.

Assuming that one layer of the exterior tie can develop f_y and the other layer can provide T_{hook} :

Total resistance: $R_{tot\_ab} := As_{tie} \cdot f_{y\_rebar} + T_{hook}$

$R_{tot\_ab} = 16\,kip \quad > \; 6.6\ kip \qquad$ OK !

Tie c (see Figure 11):

Tie c is resisted by a hairpin. Diameter of hairpin: $d_{hairpin} := 0.5in \qquad As_{hairpin} := \frac{\pi}{4} \cdot d_{hairpin}^{\;2}$

Yield stress of hairpin: $f_{y\_hairpin} := 60ksi$

Check the stress that can be developed in the hairpin:

Check available length of the hairpin: $l_{a\_hairpin} := 26in - 2 \cdot Side\_cover - 2 \cdot d_{tie}$

$l_{a\_hairpin} = 21\ in$

Required straight development length for a fully developed hairpin:

Bar location factor: $\psi_t := 1.3$

Coating factor: $\psi_e := 1$ (for uncoated bar)

Concrete density factor: $\lambda := 1$ (for normal concrete)

For #6 and smaller bars, use: $l_{d\_hairpin} := \left[\frac{f_{y\_rebar} \cdot (\psi_t \cdot \psi_e \cdot \lambda)}{25\sqrt{f'_c}}\right] \cdot d_{hairpin}$

$l_{d\_hairpin} := 25in$ cannot be less than 12 in per Section 12.2.1 of the ACI 318-05.

The stress that can be developed in the hairpin:

$f_{s\_hairpin} := \frac{l_{a\_hairpin}}{l_{d\_hairpin}} \cdot f_{y\_hairpin} \qquad f_{s\_hairpin} = 50.4\,ksi$

Since the direction of hairpin is alternated, only one layer of hairpin can be accounted as tie reinforcement.

Total resistance: $R_{tot\_c} := 2As_{hairpin} \cdot f_{s\_hairpin}$ (Note: 2 legs per hairpin)

$R_{tot\_c} = 19.792\,kip \quad > \; 13.5\ kip \quad$ OK !

6. Check the minimum distance requirements to preclude splitting failure

The following minimum distances for anchors shall be satisfied unless reinforcement is provided to control splitting.

I. Center-to-center spacing (D.8.1): $S_{min\_untorqued} := 4 \cdot d_o \; = 5\ in \quad < \min(S_1, S_2)$ OK !

II. Minimum edge distance (D.8.2):

For untorqued cast-in anchors, the minimum edge distances shall be based on minimum cover requirements.

$C_{min\_untorqued} = cover = 1.5\ in \quad < \min(C_1, C_2)$ OK !

6. CONCLUDING REMARKS

Since the ACI 318-05 does not provide specific guidelines in designing supplementary reinforcement to carry anchor forces, a design procedure for anchorage in reinforced-concrete pedestal is proposed. The anchor tension and shear forces are assumed to be resisted by the vertical reinforcing bars and ties, respectively. The Strut-and-Tie Model is proposed to analyze shear force transfer from anchors to pedestal and to design the required amount of anchor shear reinforcement. The proposed design procedure is illustrated in an example problem. It can be seen that designing anchorage in reinforced-concrete pedestals is simpler than that in un-reinforced concrete pedestals using complex design equations shown in the Appendix D of the ACI 318-05.

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