2.	Determine the design tensile strength ϕN_n: a) Steel strength ϕN_{sa}:	D.5 D.5.1
	$\phi N_{sa} = \phi n A_{se} f_{uta}$	Eq. (D-3)
	where	
	$\phi = 0.75$ $n = 1$ (for one anchor)	D.4.4(a)i
	Per the ductile steel element definition in D.1, ASTM F1554 Grade 36 steel qualifies as a ductile steel element.	
	$A_{se} = 0.142$ in.^{2} (for 1/2 in. anchor bolt [Table A.2(a)])	
	$f_{uta} = 58{,}000$ psi (Table A.1)	
	Note: f_{uta} *should not be taken greater than 1.9*f_{ya} *or 125,000 psi. For ASTM F1554 Grade 36, 1.9*f_{ya} *= 1.9(36,000) = 68,400 psi; therefore, use the specified minimum* f_{uta} *of 58,000 psi.*	D.5.1.2
	Substituting:	
	$\phi N_{sa} = 0.75(1)(0.142)(58{,}000) = 6177$ lb	
	b) Concrete breakout strength ϕN_{cb}:	D.5.2
	Since no supplementary reinforcement has been provided	
	$\phi = 0.70$	D.4.4(c)ii
	In the process of calculating the pryout strength for this fastener in Example 5, Step 4, N_{cb}, for this anchor, was found to be 13,822 lb.	
	Substituting:	
	$\phi N_{cb} = 0.70(13{,}822) = 9675$ lb	
	c) Pullout strength ϕN_{pn}:	D.5.3
	$\phi N_{pn} = \phi \psi_{c,P} N_p$	Eq. (D-14)
	where	
	$\phi = 0.70$ (Condition B applies in all cases when pullout strength governs) $\psi_{c,P} = 1.0$ (cracking may occur at the edges of the foundation) $N_p = 8A_{brg} f_c'$ $A_{brg} = 0.291$ in.^{2} (for a 1/2 in. hex-head bolt, Table A.2(a))	D.4.4(c)ii D.5.3.6 Eq. (D-15)
	Pullout strength ϕN_{pn}:	
	$\phi N_{pn} = 0.7(1.0)(8)(0.291)(4000) = 6518$ lb	
	d) Concrete side-face blowout strength ϕN_{sb}:	D.5.4
	The side-face blowout failure mode should be investigated when the edge distance c_{a1} is less than $0.4h_{ef}$.	D.5.4.1
	$0.4h_{ef} = 0.4(7) = 2.80$ in. > 2.75 in.; therefore, the side-face blowout strength should be determined	
	$\phi N_{sb} = \phi(160 c_{a1} \sqrt{A_{brg}} \sqrt{f_c'})$	Eq. (D-17)
	where	
	$\phi = 0.70$ (no supplementary reinforcement has been provided) $c_{a1} = 2.75$ in. $A_{brg} = 0.291$ in.^{2} (for a 1/2 in. hex-head bolt, Table A.2(a))	D.4.4(c)ii

2. (cont.)

Substituting:

$$\phi N_{sb} = 0.7(160(2.75)\sqrt{0.291}\sqrt{4000}) = 10{,}508 \text{ lb}$$

The summary of tension design strengths and the controlling strength is shown in Table 4.10.

Table 4.10—Summary of tension design strengths

Failure mode	Anchor design strength	Calculated design strength, lb	Controlling failure mode
Steel	ϕN_{sa}	6177	← Controls
Concrete breakout	ϕN_{cb}	9675	—
Concrete pullout	ϕN_{pn}	6518	—
Concrete side-face blowout	ϕN_{sb}	10,508	—

Check if $\phi N_n \geq N_{ua}$:

6177 lb > 1600 lb – OK — Eq. (D-1)

$\therefore\ \phi N_n = 6177$ lb

3.

Determine the shear design strength ϕV_n: — D.6

The summary of steel strength, concrete breakout strength, and pryout strength for anchor in shear from Example 5 is replicated for convenience here in Table 4.11:

Table 4.11—Summary of shear design strengths from Example 5

Failure mode	Anchor design strength	Calculated design strength, lb	Controlling failure mode
Steel	ϕV_{sa}	3212	—
Concrete breakout	ϕV_{cb}	1514	← Controls
Concrete pryout	ϕV_{cp}	19,350	—

Check $\phi V_n \geq V_{ua}$:

1514 lb > 960 lb – OK — Eq. (D-2)

$\therefore \phi V_n = 1514$ lb

4.

Check tension and shear interaction: — D.7

If $V_{ua} \leq 0.2\phi V_n$, then the full tension design strength is permitted — D.7.1

$V_{ua} = 960$ lb

$0.2\phi V_n = 0.2(1514) = 302$ lb < 960 lb

V_{ua} exceeds $0.2\phi V_n$, so the full tension design strength is not permitted.

If $N_{ua} \leq 0.2\phi N_n$ then the full shear design strength is permitted — D.7.2

$N_{ua} = 1600$ lb

$0.2\phi N_n = 0.2(6177) = 1235$ lb < 1600 lb

N_{ua} exceeds $0.2\phi N_n$, so the full shear design strength is not permitted.

The interaction equation should be used — D.7.3

$$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2 \qquad \text{Eq. (D-31)}$$

$$\frac{1600}{6177} + \frac{960}{1514} = 0.26 + 0.63 = 0.89 < 1.2 - \text{OK}$$

5.	Required edge distances, spacings, and thickness to preclude splitting failure: Since a headed bolt used to attach wood frame construction is not likely to be torqued significantly, the minimum cover requirements of ACI 318-05 Section 7.7 apply. The minimum clear cover for a 1/2 in. bar is 1-1/2 in. when exposed to earth or weather. The clear cover provided for the bolt is 2-1/2 in. (2-3/4 in. to bolt centerline less one-half bolt diameter). *Note: The bolt head will have less cover (2-3/16-in. for a hex head). Cover is considered adequate, that is, 2-3/4 in. to the centerline of the bolt results in the tip of the hex head close enough to the 1-1/2 in. clear clover to necessitate not moving the bolt axis farther away from the edge.*	D.8
6.	*Summary:* A single 1/2 in. diameter cast-in hex-headed bolt installed with a 7 in. embedment depth and a 2-3/4 in. edge distance in a concrete foundation is adequate for a service tension load from wind of 1000 lb and reversible service shear load from wind of 600 lb.	

4.7—Example 7: Single post-installed anchor in tension and shear near an edge

Determine if a single 1/2 in. diameter, post-installed, torque-controlled expansion anchor shown in Fig. 4.11, with a minimum 5-1/2 in. effective embedment, installed 3 in. from the edge of a continuous normalweight concrete footing (formed outer surface) is adequate for a service tension load of 1000 lb for wind and a reversible service shear load of 350 lb for wind. The anchor will be installed in the tension zone and the concrete is assumed to be cracked.

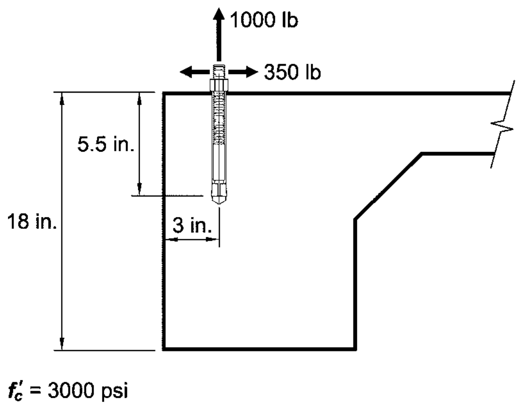

Fig. 4.11—Example 7: Single post-installed anchor in tension and shear near an edge. (Note: reinforcement not shown for clarity.)

Refer to Table A.3 for a sample table of post-installed anchor data from manufacturer (fictitious for example purposes only), as determined from testing in accordance with ACI 355.2.

Step	Calculations and discussion	ACI 318-05 Section
1.	Determine the factored tension and shear design loads: $N_{ua} = 1.6W = 1.6 \times 1000 = 1600$ lb $V_{ua} = 1.6W = 1.6 \times 350 = 560$ lb	9.2
2.	Design considerations: This is a tension/shear interaction problem where values for both ϕN_n and ϕV_n need to be determined. ϕN_n is the lesser of the tension design strength controlled by steel (ϕN_{sa}), concrete breakout (ϕN_{cb}), concrete side-face blowout (ϕN_{sb}), or concrete pullout (ϕN_{pn}). ϕV_n is the lesser of the shear design strength controlled by steel (ϕV_{sa}), concrete breakout (ϕV_{cb}), or pryout (ϕV_{cp}). Concrete side-face blowout requirements apply to cast-in and undercut anchors (RD.5.4) and are not considered here.	D.4.1.2
3.	Evaluate anchor material: For this example, consider that the post-installed, torque-controlled expansion anchor is manufactured from carbon steel material conforming to the material requirements of ASTM F1554 Grade 55, which is a headed bolt ASTM specification. The data from anchor prequalification testing according to ACI 355.2 are shown in Table A.3. For ASTM F1554 Grade 55 material (Table A.1): $f_{uta} = 75{,}000$ psi $f_{ya} = 55{,}000$ psi Elongation at 2 in. = 21% minimum with a reduction of area = 30% minimum. Appendix D requires 14% minimum elongation and 30% minimum reduction of area for an anchor to be considered as a ductile steel element. $\therefore$ Anchor steel is ductile.	D.1

4.	Steel strength under tension loading: $\phi N_{sa} \geq N_{ua}$ $N_{sa} = nA_{se}f_{uta}$ $\phi nA_{se}f_{uta} \geq N_{ua} = 1600$ lb For ductile steel as controlling failure mode: $\phi = 0.75$ $n = 1$ (single anchor) Calculating for ϕN_{sa}: $\phi N_{sa} = 0.75 \times 1 \times 0.142 \times 75{,}000 = 7988$ lb > 1600 lb – OK $\therefore$ 1/2 in. diameter anchor steel strength is adequate under tension loading.	D.5.1 D.4.1.1 Eq. (D-3) D.4.4(a)i
5.	Minimum edge distance requirements: The minimum edge distance for post-installed anchors should be based on the greater of the minimum cover requirements in ACI 318 Section 7.7 or minimum edge distance requirements for the products as determined by tests in accordance with ACI 355.2 (Table A.3), and should not be less than two times the maximum aggregate size. $c_{a,min}$ = (2 in. – maximum cover requirements for concrete exposed to earth; 2.5 in. – product requirement; or 2(0.75) = 1.5 in.) – assuming 3/4 in. maximum aggregate size) $\therefore$ $c_{a,min}$ = 3 – 1/2(0.5 in.) = 2-3/4 in. – OK	D.8.3

6.	Concrete breakout strength under tension loading:	D.5.2
	$\phi N_{cb} \geq N_{ua}$	D.4.1.1
	$N_{cb} = \frac{A_{Nc}}{A_{Nco}} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$	Eq. (D-4)
	where	
	$N_b = k_c \sqrt{f'_c} h_{ef}^{1.5}$	Eq. (D-7)
	Substituting:	
	$\phi N_{cb} = \phi \frac{A_{Nc}}{A_{Nco}} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} k_c \sqrt{f'_c} h_{ef}^{1.5} \geq N_{ua} = 1600$ lb	
	where	
	$k_c = k_{cr} = 17$ (Table A.3)	
	$\psi_{ed,N} = 0.7 + 0.3 \frac{c_{a,min}}{1.5 h_{ef}}$ when $c_{a,min} < 1.5 h_{ef}$	Eq. (D-11)
	by observation, $c_{a,min} < 1.5 h_{ef}$	
	$\psi_{ed,N} = 0.7 + 0.3 \frac{3}{1.5(5.5)} = 0.81$	
	$\psi_{c,N} = 1.0$ (assuming cracking at service loads, $f_t > f_r$)	D.5.2.6
	$\psi_{cp,N} = 1.0$ (assuming cracking at service loads, $f_t > f_r$)	D.5.2.7
	$\phi = 0.55$ (for a Category 2 anchor, no supplemental reinforcement provided)	D.4.4(c)ii
	$A_{Nco} = 9 h_{ef}^2 = 9(5.5)^2$	Eq. (D-6)
	$\therefore A_{Nco} = 272.25$ in.^{2}	D.5.2.1
	$A_{Nc} = (c_{a1} + 1.5 h_{ef})(2 \times 1.5 h_{ef}) = (3.0 + 1.5(5.5))(2 \times (1.5)(5.5))$	Fig. RD. 5.2.1(b)
	$\therefore A_{Nc} = 185.63$ in.^{2}	
	$\frac{A_{Nc}}{A_{Nco}} = \frac{185.63}{272.25} = 0.68$	
	Calculating ϕN_{cb}:	
	$N_{cb} = 0.68 \times 0.81 \times 1.0 \times 17 \times \sqrt{3000} \times (5.5)^{1.5} = 6615$ lb	
	$\phi N_{cb} = 0.55(6615 \text{ lb}) = 3638 \text{ lb} > N_{ua} = 1600$ lb – OK	
7.	Pullout strength:	
	Pullout strength N_p for post-installed anchors is established by reference tests in cracked and uncracked concrete in accordance with ACI 355.2. Data from the anchor prequalification testing should be used.	D.5.3.2
	$N_p = N_{p,cr} = 7544$ lb (Table A.3).	
	$\phi N_{pn} \geq N_{ua}$	D.4.1.1
	$N_{pn} = \psi_{c,P} N_p$	Eq. (D-14)
	$\phi = 0.55$ (for a Category 2 anchor, Condition B applies in all cases when pullout strength governs)	D.4.4.(c)ii
	$\psi_{c,P} = 1.0$ (assuming cracking at service loads, $f_t > f_r$)	D.5.3.6
	$\phi N_{pn} = 0.5 \times 7544 = 4149 > 1600$ lb – OK	

8. Check all failure modes under tension loading: — D.4.1.2

Summary: Table 4.12 shows that concrete breakout strength controls

$\phi N_n = 3638$ lb

Table 4.12—Summary of tension design strengths

Failure mode	Anchor design strength	Calculated design strength, lb	Controlling failure mode
Steel	ϕN_{sa}	7988	
Concrete breakout	ϕN_{cb}	3638	← Controls
Concrete pullout	ϕN_{pn}	4149	—

9. Steel strength under shear loading: — D.6.1

$\phi V_{sa} \geq V_{ua}$ — D.4.1.1

$V_{sa} = n0.6A_{se}f_{uta}$ (anchor does not have sleeve to extend through shear plane) — Eq. (D-20)

$\phi n0.6A_{se}f_{uta} \geq V_{ua} = 560$ lb

For ductile steel as controlling failure mode:

$\phi = 0.65$ — D.4.4(a)ii

$n = 1$ (single anchor)

Calculating for ϕV_{sa}:

$\phi V_{sa} = 0.65 \times 0.6 \times 0.142 \times 75{,}000 = 4153$ lb > 560 lb – OK

∴ 1/2 in. diameter anchor steel strength is adequate under shear loading.

10.	Concrete breakout strength under shear loading: $\phi V_{cb} \geq V_{ua}$ $V_{cb} = \frac{A_{Vc}}{A_{Vco}} \psi_{ed,V} \psi_{c,V} V_b$ where $V_{cb} = 7\left(\frac{\ell_e}{d_o}\right)^{0.2} \sqrt{d_o} \sqrt{f_c'} (c_{a1})^{1.5}$ Substituting: $\phi V_{cb} = \phi \frac{A_{Vc}}{A_{Vco}} \psi_{ed,V} \psi_{c,V} 7\left(\frac{\ell_e}{d_o}\right)^{0.2} \sqrt{d_o} \sqrt{f_c'} (c_{a1})^{1.5} \geq V_{ua} \geq 560$ lb where $\phi = 0.7$ (for anchors governed by concrete breakout due to shear; Condition B where no supplemental reinforcement is provided); $\frac{A_{Vc}}{A_{Vco}} = 1.0$; and $c_{a1} = 3$ in. (edge distance) c_{a2} is the distance from the center of the anchor to the edge of concrete in the direction orthogonal to c_{a1} (not specified in this example, but consider this distance greater than $1.5h_{ef}$) $\psi_{ed,V} = 1.0$ (since $c_{a2} \geq 1.5c_{a1}$) $\psi_{c,V} = 1.0$ (assuming cracking at service loads, $f_t > f_r$) $d_o = 0.5$ in. $\ell_e = h_{ef} = 5.5$ in., but $\ell_e \leq 8d_o$; $8d_o = 8(0.5) = 4$ in. $\therefore \ell_e = 4$ in. Substituting: $\phi V_{cb} = 0.7 \times 1.0 \times 1.0 \times 1.0 \times 7\left(\frac{4}{0.5}\right)^{0.2} \times \sqrt{0.5} \times \sqrt{3000} \times (3)^{1.5} = 1495 \text{ lb} > 560 \text{ lb} – \text{OK}$	D.6.2 D.4.1.1 Eq. (D-21) Eq. (D-24) D.4.4(c)ii Eq. (D-27) D.6.2.7
11.	Concrete pryout strength: $\phi V_{cp} \geq V_{ua}$ $V_{cp} = k_{cp} N_{cb}$ where $k_{cp} = 2.0$ (since $h_{ef} = 5.5$ in. > 2.5 in.) $N_{cb} = 6615$ lb (refer to Step 6 of this design example) $\phi = 0.7$ (Condition B applies) $\therefore \phi V_{cb} = 0.7 \times 2.0 \times 6615 = 9261 \text{ lb} > 560 \text{ lb} – \text{OK}$	D.6.3 D.4.1.1 Eq. (D-29) D.6.3.1 D.4.4(c)i

12. Check all failure modes under shear loading: D.4.1.2

Table 4.13 shows that concrete breakout strength controls.

Table 4.13—Summary of shear design strengths

Failure mode	Anchor design strength	Calculated design strength, lb	Controlling failure mode
Steel	ϕV_{sa}	4153	—
Concrete breakout	ϕV_{cb}	1495	← Controls
Concrete pryout	ϕV_{cp}	9261	—

13. Check interaction of tension and shear forces: D.7

If $V_{ua} \le 0.2\phi V_n$, then the full strength in tension is permitted:

$\phi N_n \ge N_{ua}$ D.7.1

$0.2\phi V_n = 0.2(1495 \text{ lb}) = 299 \text{ lb} < 560 \text{ lb}$ – Requirement not met

If $N_{ua} \le 0.2\phi N_n$, then the full strength in shear is permitted:

$\phi V_n \ge V_{ua}$ D.7.2

$0.2\phi N_n = 0.2(3638 \text{ lb}) = 728 \text{ lb} < 1600 \text{ lb}$ – Requirement not met

Since $V_{ua} > 0.2\phi V_n$ and $N_{ua} > 0.2\phi N_n$, then: D.7.3

$$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \le 1.2 \qquad \text{Eq. (D-31)}$$

$$\frac{1600}{3638} + \frac{560}{1495} = 0.44 + 0.37 = 0.81 < 1.2 \text{ – OK}$$

14. *Summary:*

The post-installed, torque-controlled expansion anchor, 1/2-in. diameter at a 5-1/2 in. effective embedment depth, is adequate to resist the applied service tension and shear loads of 1000 lb and 350 lb, respectively.

4.8—Example 8: Group of cast-in anchors in tension and shear with two free edges and supplemental reinforcement

Check the capacity of a fastener group with four 3/4 in. diameter, ASTM F1554 Grade 55, cast-in anchor rods embedded 12 in. with hex nuts into the thickened slab as shown in Fig. 4.12(a) through (d). The concrete is 24 in. thick, $f_c' = 3000$ psi, and is normalweight. The anchor support is a combined factored load of 4000 lb shear and 12,000 lb tension. The plate is symmetrically placed at the corner. Seismic forces are not a consideration. Reinforcement is 60 ksi. Because both shear and tension are to be considered, the capacity of this detail will be based on the interaction equation given in D.7.

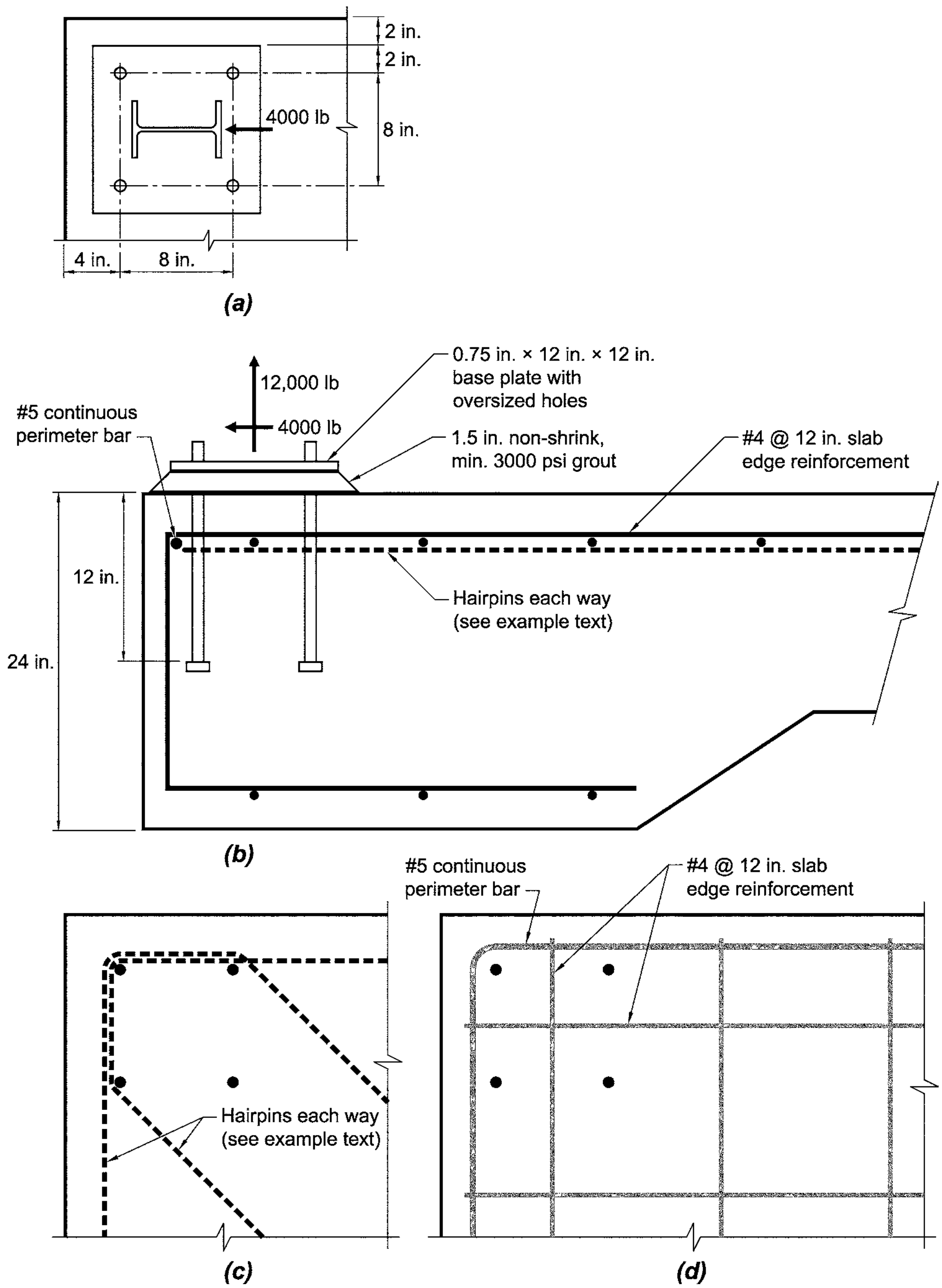

Fig. 4.12—(a) Example 8: Plan of column base plate and anchor rods; (b) section through foundation and anchors; (c) section (plan) looking down at hairpin supplemental reinforcement; and (d) section (plan) looking down at slab edge reinforcement.