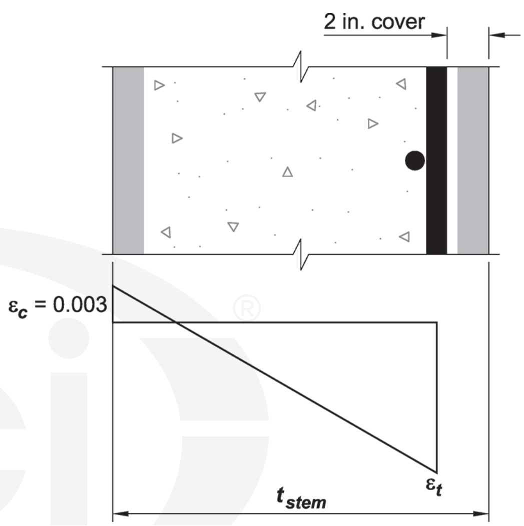
22.2.2.3	Determine the equivalent concrete compressive stress for design. The concrete compressive stress distribution is inelastic at high stress. The actual distribution of concrete compressive stress is complex and usually not known explicitly. The Code permits any stress distribution to be assumed in design if shown to result in predictions of nominal strength in reasonable agreement with the results of comprehensive tests. Rather than tests, the Code allows the use of an equivalent rectangular compressive stress distribution of $0.85f_c'$ with a depth of:	
22.2.2.4.1	$a = \beta_1 c$, where β_1 is a function of concrete compressive strength and is obtained from Table 22.2.2.4.3.	
22.2.2.4.3	For $f_c' \leq 4500$ psi	$\beta_1 = 0.85 - \dfrac{0.05(4500\text{ psi} - 4000\text{ psi})}{1000\text{ psi}} = 0.825$
22.2.1.1	Find the equivalent concrete compressive depth, a, by equating the compression force and the tension force within a unit length of the wall cross section: $C = T$	
	$C = 0.85f_c'ba$ and $T = A_sf_y$	$0.85(4500\text{ psi})(12\text{ in.})(a) = A_s(60{,}000\text{ psi})$ $a = \dfrac{A_s(60{,}000\text{ psi})}{0.85(4500\text{ psi})(12\text{ in.})} = 1.31A_s$
	Calculate required reinforcement area:	Equating design moment strength and required moment strength, A_s is:
7.5.2.1 22.3	$M_n = A_sf_y\left(d - \dfrac{a}{2}\right)$	$M_n = (60{,}000\text{ psi})A_s\left(7.68\text{ in.} - \dfrac{1.31A_s}{2}\right)$
21.2.1	Use flexure strength reduction factor:	$\phi = 0.9$
20.5.1.3.1	Assume No. 5 vertical reinforcement:	
	$d = t_{stem} - \text{cover} - d_b/2$ use 2 in. cover	$d = 10\text{ in.} - 2\text{ in.} - 0.625\text{ in.}/2 = 7.68\text{ in.}$
7.5.1.1	Substituting into: $\phi M_n \geq M_u$	üüüüüüüüüüüüA_s(üüüüüü $- \dfrac{1.31A_s}{2}$)
	$M_u = 82.3$ in.-kip calculated above. Solving for A_s (refer to Fig. E6.4):	$A_s = 0.20\text{ in.}^2$ No. 5 at 12 in. on center. $A_{s,prov'd} = 0.31\text{ in.}^2/\text{ft} > A_{s,req'd} = 0.2\text{ in.}^2/\text{ft}$

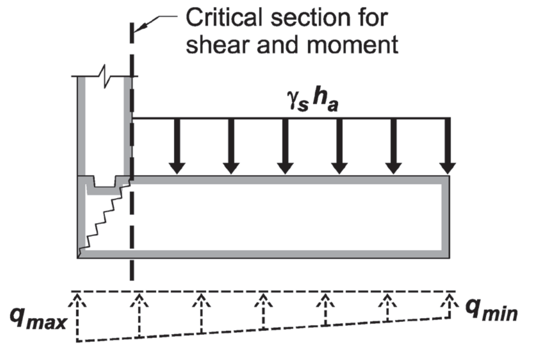
7.7.2.2 24.3.2	To provide adequate crack control, limit spacing of the vertical bars for out-of-plane bending according to Table 24.3.2. To maximize bar spacing, calculate unfactored stress in bars (f_s) using the selected 12 in. spacing of the No. 5 bars. Use the following working stress equations: $n = E_s/E_c$ $k = -n\frac{A_s}{bd} + \sqrt{n^2\left(\frac{A_s}{bd}\right)^2 + 2n\frac{A_s}{bd}}$ $j = 1 - \frac{k}{3}$ $f_s = \frac{M_s}{jdA_s}$	$E_c = 57{,}000\sqrt{4500}$ psi = 3824 ksi Use 3820 ksi $n = \frac{29{,}000 \text{ ksi}}{3820 \text{ ksi}} = 7.6$ $A_s = \frac{12 \text{ in.}}{12 \text{ in.}} 0.31 \text{ in.}^2 = 0.31 \text{ in.}^2$ $\frac{A_s}{bd} = \frac{0.31 \text{ in.}^2}{12 \text{ in.}(7.68 \text{ in.})} = 0.00336$ $k = -7.6(0.00336) + \sqrt{(7.6)^2(0.00336)^2 + 2(7.6)(0.00336)}$ $k = 0.202$ $j = 1 - \frac{0.202}{3} = 0.933$ Compute service moment at base of stem from previously calculated factored moment: M_s = 6860 ft lb/1.6 = 4287 ft lb $f_s = \frac{4290 \text{ ft lb}}{0.933(7.68 \text{ in.})(0.31 \text{ in.}^2)} = 23{,}176$ psi 15(40,000 psi/23,200 psi) in. – 2.5(2 in.) = 20.9 in. 12(40,000 psi/23,200 psi) in. = 20.7 in. **Controls**
7.7.2.3	Maximum bar spacing is limited to 3h or 18 in.	3(12 in.) = 36 in. No more than 18 in. 12 in spacing is less than 18 in. **OK**
7.6.1.1	Provide minimum reinforcement according to Code Chapter 7. This reinforcement should be placed as close as practicable to the tension face.	minimum ratio on gross section = 0.0018 Vertical bars on soil face: 0.0018(12 in.)(10 in.) = 0.216 in.^{2} < $A_{s,prov}$ = 0.31 in.^{2} **OK** Use No. 5 vert. @ 12 in. = 0.31 in.^{2}

21.2.2	Check if the tension controlled assumption and the use of $\phi = 0.9$ is correct. To answer the question, the tensile strain in reinforcement must be first calculated and compared to the values in Table 21.2.2. Assume concrete and nonprestressed reinforcement strain varying proportional to the distance from the neutral axis (refer to Fig. E6.5):	$c = \dfrac{(1.31)(0.31\text{ in.}^2)}{0.825} = 0.49\text{ in.}$
22.2.1.2	$\varepsilon_t = \dfrac{\varepsilon_c}{c}(d - c)$ where: $c = \dfrac{a}{\beta_1}$ and $a = 1.31A_s$ derived previously.	$\varepsilon_t = \dfrac{0.003}{0.49\text{ in.}}(7.68\text{ in.} - 0.49\text{ in.}) = 0.044$ $\varepsilon_t = 0.044 > 0.005$ Section is tension controlled and $\phi = 0.9$.  *Fig. E6.5—Strain distribution across stem.*

7.6.4.1 24.4	For horizontal reinforcement and vertical reinforcement on outside face use minimum shrinkage and temperature reinforcement in accordance with Code Section 24.4	
24.4.3.2	Minimum shrinkage and temperature reinforcement is 0.0018 of gross concrete area. Divide this amount equally between the two faces.	$\frac{0.0018(12\text{ in.})(10\text{ in.})}{2} = 0.108\text{ in.}^2$ in each face
24.4.3.3	Bar spacing should not exceed 5h or 18 in.	5(10 in.) = 50 in. 18 in. controls No. 4 bars at 18 in. o/c: A_s = (12 in./18 in.)0.2 in.^{2} = 0.133 in.^{2} Use No. 4 horiz. @ 18 in. each face = 0.13 in.^{2} + 0.13 in.^{2} = 0.26 in.^{2} and No. 4 vert. @ 12 in. in outside face = 0.2 in.^{2} > 0.108 in.^{2} The designer should consider increasing horizontal reinforcement or introducing vertical contraction joints, or both to control formation of vertical cracks due to to shrinkage and temperature effects. ACI 224.3R-95, ACI 224.4R-13, ACI 315R-18 are good resources for detailing joints in wall construction. If resistance to water penetration is desired, then ACI 350 and ACI 350.4R may also be of use.

	Lap splice length with dowels Dowels must extend above the critical section (interface between stem and footing) by the lap splice length, which is a function of straight bar development length. Determine the development length of the No. 5 dowel.	
25.4.2.1	Determine required development length for No. 5 bars using detailed formula from 25.4.2.1	d_b = 0.625 in. Diameter of No. 5 bar K_{tr} = 0 No transverse reinforcement present
25.4.2.4a	$\ell_d \geq \left(\frac{3}{40}\frac{f_y}{\sqrt{f'_c}}\frac{\psi_t\psi_e\psi_s\psi_g}{\left(\frac{c_b+K_{tr}}{d_b}\right)}\right)d_b$	c_b = 2.3 in. Least of cover to center of bar or one-half center-to-center bar spacing. Consider stem reinf.
	$(c_b + K_{tr})/d_b \leq 2.5$	(2.3 in. + 0)/0.625 in. = 3.68 Use a value no more than 2.5.
25.4.2.4b	$K_{tr} = 40A_{tr}/sn$	
25.4.2.1(b)	$\ell_d \geq$ 12 in.	Bars are cast with less than 12 in. of fresh concrete below the bars. $\psi_t = 1.0$
25.4.2.5	ψ_t – Casting position ψ_e – Epoxy coating ψ_s – Bar size ψ_g – Reinforcement grade	Bars are uncoated. $\psi_e = 1.0$ Bars are No. 5 (No. 6 and smaller) $\psi_s = 0.8$ Bars are Grade 60. $\psi_g = 1.0$ Required development length for No. 5: $\frac{3}{40}\frac{(60{,}000\text{ psi})}{[(1.0)\sqrt{4500}\text{ psi}]}\frac{(1.0)(1.0)(0.8)(1.0)}{2.5}0.625\text{ in.} = 13.4\text{ in.}$ Use development length of 1 ft 2 in. minimum.
25.5.2.1	Lap splice length type is based on the ratio of the provided area of steel to required area of steel and the maximum percentage of steel spliced within the lap splice length. Use Table 25.5.2.1 to determine required lap splice length.	$A_{s,provided}/A_{s,required} < 2.0$ Max A_s spliced = 100% All bars are spliced at the critical section, which is located at the base of the wall. Class B splice is required regardless of area of steel spliced Lap splice length = 1.3(13.4 in.) = 17.5 in. Use lap splice length of 1 ft 6 in.

	Shear	
11.5.5	Out-of-plane shear is checked in accordance with Code Section 22.5 (similar to one-way slabs).	$h_{stem} = h - t_{base} = 10.5\text{ ft} - 1.25\text{ ft} = 9.25\text{ ft}$ $V_u = 1.6(32.5\text{ pcf})(9.25\text{ ft})(9.25\text{ ft}/2) = 2225\text{ lb} \sim 2.3\text{ kip}$
22.5.5.1c	Use appropriate equation from Table 22.5.5.1c $V_c = \left[8\lambda_s\lambda(\rho_w)^{1/3}\sqrt{f_c'} + \frac{N_u}{6A_g}\right]b_w d$	
21.2.1	Strength reduction factor for shear from Table 21.2.1b	$\phi = 0.75$
	Effective depth to centroid of reinforcement	$d = 7.68$ in.
	Use No. 5 at 12 in. spacing, which is the flexural reinforcement for out-of-plane bending. Area of steel per foot of width.	$A_s = (12\text{ in.}/12\text{ in.})0.31\text{ in.}^2 = 0.31\text{ in.}^2$
	Unit slab width of 12 in.	$b_w = 12$ in.
2.2	Reinforcement ratio of flexural reinforcement relative to a unit width of 12 in. $\rho_w = A_s/b_w d$	$\rho_w = \frac{0.31\text{ in.}^2}{12\text{ in.}(7.68\text{ in.})} = 0.00336$
	Assume that axial load is zero.	$N_u = 0$
22.5.5.1.3	Size effect factor. $\lambda_s = \sqrt{\frac{2}{1+0.1d}}$	$\lambda_s = \sqrt{\frac{2}{1+0.1(7.68)}} = 1.064$ use 1.0 $\lambda = 1.0$ $V_c = 8(1.0)(1.0)(0.00336)^{1/3}\sqrt{4500}\text{ psi}(12\text{ in.})(7.68\text{ in.})$ $V_c = 7.41$ kip $\phi V_c = 0.75(7.41\text{ kip}) = 5.56\text{ kip} > V_u = 2.3\text{ kip}$ **OK** Design shear strength from concrete contribution exceeds the factored shear. Wall thickness is adequate for out-of-plane shear.

Step 8: Heel design		
	<u>Shear</u>	
13.3.6.3 7.4.3.2 9.4.3.2	The Code indicates that the critical section for both shear and moment in the stem is located at the interface between the stem and footing. This is due to the opening moment that is applied. Although not specifically addressed in the foundations chapter of the Code, the heel of the footing has an opening moment that is applied to it as well, which will then result in a diagonal crack through the joint. Thus, the critical section for both shear and moment should be located at the face of the stem wall as shown in Fig. E6.6. Check shear strength of footing without shear reinforcement to ensure that the footing has adequate thickness. Ignore contribution of upward soil pressure due to the uncertainty of its distribution	*Fig. E6.6—Critical section for shear and moment.* V_u = 1.2(150 pcf)(4.84 ft)(1.25 ft) + 1.6(120 pcf)(9.25 ft)(4.84 ft) V_u = 9690 lb ~9.7 kip
7.5.3.1	Out-of-plane shear is to be checked in accordance with Code Section 22.5 (similar to one-way slabs).	
22.5.5.1c	Use appropriate equation from Table 22.5.5.1c $V_c = \left[8\lambda_s\lambda(\rho_w)^{1/3}\sqrt{f_c'} + \frac{N_u}{6A_g}\right]b_w d$	
21.2.1	Strength reduction factor for shear from Table 21.2.1b.	$\phi = 0.75$
	Effective depth to centroid of reinforcement assuming No. 6 bar.	d = 15 in. – 12 in. – 0.5(0.625 in.) = 12.69 in.
	Try No. 6 at 6 in. spacing, which is a multiple of the stem bar spacing. This spacing will allow the stem wall dowels to be tied to alternate top footing bars, which will simplify inspection and placement. This quantity of reinforcement is well beyond what will be required for flexure, but provides improved shear performance to avoid an excessively thick footing.	A_s = (12 in./6 in.)(0.44 in.^{2}) = 0.88 in.^{2}
2.2	Reinforcement ratio of flexural reinforcement relative to the slab unit width of 12 in.	b_w = 12 in.
	$\rho_w = A_s/b_w d$	$\rho_w = \frac{0.88\text{ in.}^2}{12\text{ in.}(12.69\text{ in.})} = 0.00578$
	Assume that axial load is zero.	$N_u = 0$
13.2.6.2 22.5.5.1.3	Size effect factor can be neglected.	$\lambda_s = 1.0$ $\lambda = 1.0$ $V_c = 8(1.0)(1.0)(0.00578)^{1/3}\sqrt{4500}$ psi(12 in.)(12.69 in.) V_c = 14.67 kip ϕV_c = 0.75(14.6 kip) = 11.0 kip > V_u = 9.7 kip **OK** Use No. 6 at 6 in. spacing in top of footing.

	Flexure The heel is subject to flexure caused by the superimposed weight of soil and self-weight of heel. The soil pressure counteracting the applied gravity loads is neglected as the soil pressure may not be linear as assumed. Therefore, it is not included in the calculation of flexure.	
6.6.1.2	The cantilever retaining wall maximum moment and shear in the heel and toe of the base occur at the stem face. Redistribution of moments cannot occur.	
5.3.1 5.3.8	A load factor of 1.2 is used for the concrete self-weight and 1.6 for soil backfill.	M_{u1} = 1.2(150 pcf)(1.25 ft)(4.84 ft)^2/2 + 1.6(120 pcf)(9.25 ft)(4.84 ft)^2/2 = (2634 ft-lb) + (20,802 ft-lb) = 23,347 ft-lb ≅ 281,000 in.-lb
22.2.1.1	Setting $C = T$	0.85(4500 psi)(12 in.)(a) = A_s(60,000 psi) $a = \dfrac{A_s(60{,}000\text{ psi})}{0.85(4500\text{ psi})(12\text{ in.})} = 1.31A_s$
7.5.2.1	$M_n = f_y A_s\left(d - \dfrac{a}{2}\right)$	$M_n = (60{,}000\text{ psi})A_s\left(12.69\text{ in.} - \dfrac{1.31A_s}{2}\right)$
21.2.1	Strength reduction factor for flexure:	$\phi = 0.9$
7.5.1.1	$\phi M_n \geq M_u$ M_u = 281 in.-kip, calculated above. Solving for A_s:	$0.9(60\text{ ksi})A_s\left(12.69\text{ in.} - \dfrac{1.31A_s}{2}\right) \geq 281\text{ in.-kip}$ A_s = 0.42 in.^{2}

13.3.2.1	Design footing according to the applicable provisions of Code Chapters 7 and 9.	Lesser of:
7.7.2.2 24.3.2	To provide adequate crack control, limit spacing of the primary bars for out-of-plane bending according to Table 24.3.2.	15(40,000/40,000) in. – 2.5(2 in.) = 10 in. controls 12(40,000/40,000) in. = 12 in.
7.7.2.3	Maximum bar spacing is limited to 3*h* or 18 in.	3(15 in.) = 45 in. No more than 18 in. Limit bar spacing to 10 in.
7.6.1.1	Provide minimum flexural reinforcement according to Code Chapter 7. This reinforcement should be placed as close as practicable to the tension face.	minimum flexural reinforcement ratio = 0.0018 0.0018(12 in.)(15 in.) = 0.324 in.^{2} Shear strength controls area of reinforcement required. Use No. 6 bars at 6 in. spacing. $A_{s,prov'd} = 0.88$ in.^{2} $> A_{s,min} = 0.324$ in.^{2} $> A_{s,req'd} = 0.42$ in.^{2} **OK**
7.6.4.1 24.4	For footing longitudinal reinforcement, provide minimum shrinkage and temperature reinforcement in accordance with Code Section 24.4.	
24.4.3.2	Minimum shrinkage and temperature reinforcement is 0.0018 of gross concrete area.	0.0018(15 in.)(5.67 ft) 12 in./ft = 1.8 in.^{2}
24.4.3.3	Bar spacing should not exceed 5*h* or 18 in.	5(15 in.) = 75 in. No more than 18 in. 1.8 in.^{2}/0.31 in.^{2} = 5.806 Use seven No. 5 bars distributed in top and bottom of footing. $A_{s,prov} = 7(0.31 \text{ in.}^2) = 2.17 \text{ in.}^2$
21.2.2	Check if the tension controlled assumption and the use of $\phi = 0.9$ is correct. To answer the question, the tensile strain in reinforcement must be first calculated and compared to the values in Table 21.2.2. Concrete and nonprestressed reinforcement strain is assumed to vary proportionally from the neutral axis. From similar triangles (refer to Fig. E6.5): $\varepsilon_t = \frac{\varepsilon_c}{c}(d-c)$ where: $c = \frac{a}{\beta_1}$ and $a = 1.31A_s$ derived previously.	$c = \frac{(1.31)(0.88 \text{ in.}^2)}{0.825} = 1.40 \text{ in.}$ $\varepsilon_t = \frac{0.003}{0.49 \text{ in.}}(12.69 \text{ in.} - 1.40 \text{ in.}) = 0.024$ $\varepsilon_t = 0.024 > 0.005$ Section is tension controlled and $\phi = 0.9$.

Step 9: Stem-footing joint detailing		
	Eliminating the footing toe due to the proximity of the property line makes development of the reinforcement and maintaining stability of the joint more difficult because the heel and stem reinforcement must be developed within the bounds of the joint region. Code Chapter 15 covers beam-column joints, but does not directly address wall-slab joints where the use of transverse reinforcement is impractical. The joint geometry restricts the use of 90-degree hooks in favor of 180-deg hooks or headed bars. In addition, for a conventional layout of reinforcement (Fig. E6.7(a)), experimental work has shown that opening moments applied to the corner joint can result in a flexural strength that is less than predicted by the sectional model for both the stem and heel reinforcement at the critical section (Nilsson and Losberg 1976). Campana et al. (2013) compiled results of tests on 90-deg. corner joints that used varying configurations of reinforcement detailing to control the joint opening stresses and to ensure that the joint remained stable up to and beyond the calculated moment strength of the joint. Some of the joint reinforcement detailing found to be relatively more effective resisting opening moments, however, would also be challenging to fabricate and place. Based on the previously cited references, one of the more effective corner joint details is shown in Fig. E6.7(b). The primary reinforcement for the stem and heel are bent to hairpin shape to ensure confinement of the joint concrete and full development of the reinforcement at the critical sections. To control reentrant corner cracking, a diagonal bar is also included. The amount of diagonal reinforcement required was experimentally determined to be about one-half of the primary flexural reinforcement.	