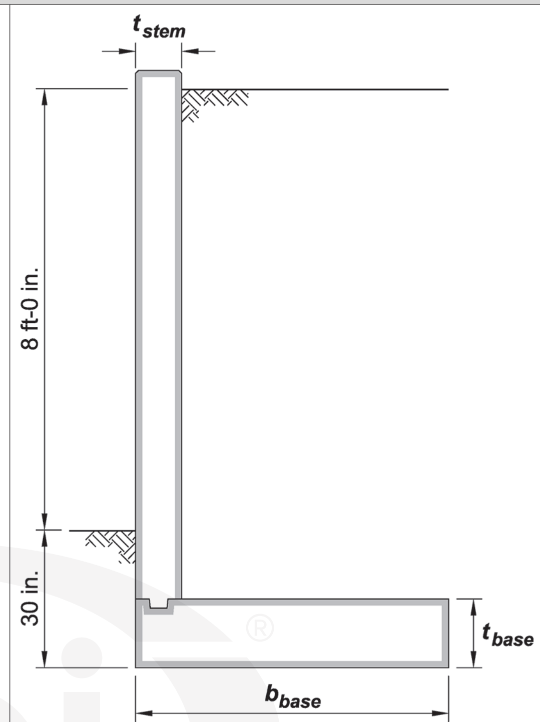
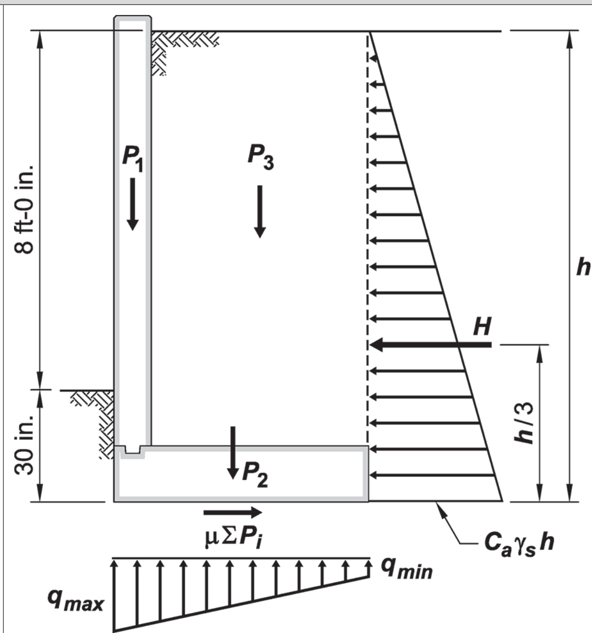
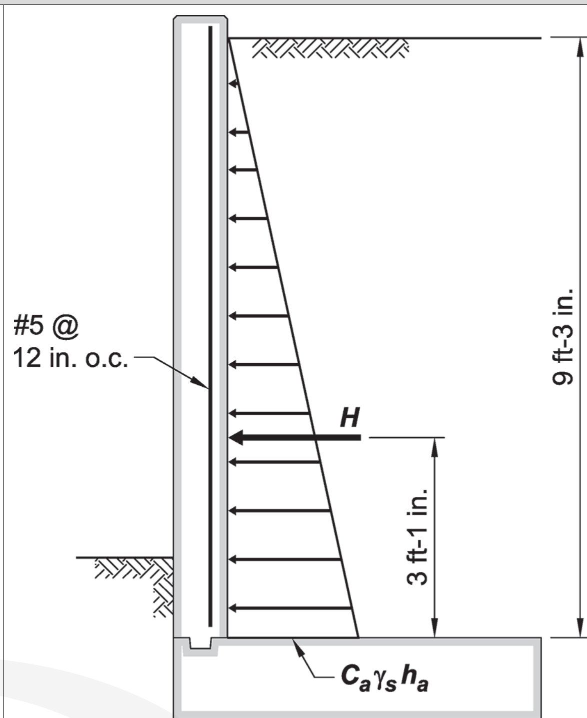
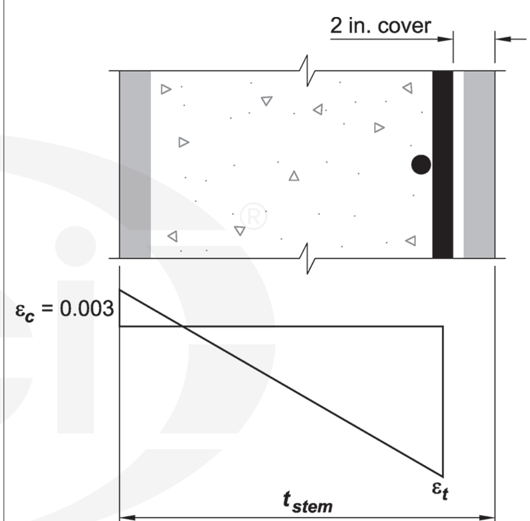
Step 2: Equivalent lateral pressure		
	The geotechnical report provides the equivalent lateral pressure the wall is required to resist.	p = (0.271)(120 pcf) = 32.5 pcf
Step 3: Preliminary cantilever wall data		
	General criteria The preliminary retaining wall dimensions (refer to Fig. E6.2) are determined from the author's experience and are presented in the text.	 *Fig. E6.2—Retaining L-Wall required dimensions.*
	Overall height of wall is:	$h = h_1 + h_2$ = 8 ft + 2.5 ft = 10.5 ft
	Estimating stem thickness: (0.07 to 0.12)h Estimating base thickness: (0.07 to 0.1)h, but at least 12 in. Engineers commonly specify the base be slightly thicker than the stem.	t_{stem} ~ 0.08(10.5 ft) = 0.83 ft, say, 10 in. t_{base} ~ 0.1(10.5 ft) = 1.05 ft, say, 1 ft 3 in.
20.6.1.3.1 13.3.1.2	Assuming 3 in. cover, the effective depth is:	(1 ft 3 in.) – (3 in.) = 1 ft > 6 in. **OK**
	Length of base (heel): (0.5 to 0.7)h Heel length: $b_{heel} = b_{base} - b_{toe} - t_{stem}$	b_{base} ~ 0.55 (10.5 ft) = 5.775 ft, say, 5 ft 8in. b_{heel} = (5 ft 8 in.) – (10 in.) = 4 ft 10 in.

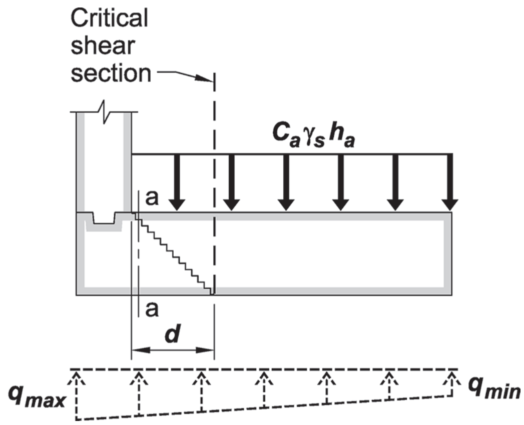
Step 4: Applied forces		
	For out-of-plane moment and shear, the cantilevered retaining wall is assumed to be continuous, and a representative 1 ft strip is analyzed for the maximum load effects (refer to Fig. E6.3).	
	Vertical loads The vertical weight is the self-weight of the retaining wall (stem and base) and the weight of the soil above the heel.	*Fig. E6.3—Applied forces on retaining wall.*
	Stem wall: $P_1 = \gamma_{conc}(t_{stem})(h - t_{base})$ Base: $P_2 = \gamma_{conc}(t_{base})(b)$ Soil: $P_3 = \gamma_s(h - t_{base})(b - t_{stem})$	P_1 = (150 pcf)(0.83 ft)(10.5 ft – 1.25 ft) = 1152 lb P_2 = (150 pcf)(1.25 ft)(5.67 ft) = 1063 lb P_3 = (120 pcf)(10.5 ft – 1.25 ft)(5.67 ft – 0.83 ft) = 5372 lb
	Total vertical load:	$\sum P$ = 1152 lb + 1063 lb + 5372 lb = 7587 lb
	The self-weight of the retaining wall and the soil above the heel tend to counteract the overturning moment. Moments taken about the front edge of base (stem):	
	Stem wall: $M_1 = P_1(t_{stem}/2)$ Base: $M_2 = P_2(b_{base}/2)$ Soil: $M_3 = P_3(b - b_{heel}/2)$	M_1 = (1152 lb)(0.83 ft/2) = 478 ft-lb M_2 = (1063 lb)(5.67 ft/2) = 3014 ft-lb M_3 = (5372 lb)(5.67 ft – 4.83 ft/2) = 17,486 ft-lb
	Restoring moment:	$\sum M_R$ = 478 ft-lb + 3014 ft-lb + 17,486 ft-lb = 20,978 ft-lb
	The retained soil behind the wall exerts lateral pressure H on the wall: $H_1 = (C_a)(\gamma_s)(h^2/2)$	H_1 = (0.271)(120 pcf)(10.5 ft)^2/2 = 1793 lb/ft
	Therefore, this lateral force tends to overturn the retaining wall about the front edge of the stem:	
	$M_{OTM} = H(h/3)$	M_{OTM} = (1793 ft)(10.5 ft/3) = 6276 ft-lb
	Summation of moments: $\Delta M = \sum M_R - M_{OTM}$	ΔM = (20,978 ft-lb) – (6276 ft-lb) = 14,702 ft-lb

Step 5: Soil pressure		
13.3.1.1	The aforementioned determined cantilever wall base is checked using unfactored forces and allowable soil bearing pressure. To calculate soil pressure, the location of the vertical resultant force must be determined. The distance of the resultant to the front face of stem:	
	$a = \dfrac{\Delta M}{\sum P}$	$a = \dfrac{14{,}702\text{ ft-lb}}{7587\text{ lb}} = 1.94\text{ ft}$
	Eccentricity is the difference between the resultant location and the base mid-length: $e = b_{base}/2 - a$	$e = 5.67\text{ ft}/2 - 1.94\text{ ft} = 0.90\text{ ft}$
	Check if resultant falls within the middle third of the base.	$\dfrac{b_{base}}{6} = \dfrac{5.83\text{ ft}}{6} = 0.97\text{ ft} > e = 0.9\text{ ft}$
	Maximum and minimum soil pressure: $q_{1,2} = \dfrac{\sum P}{A} \pm \dfrac{\sum Pe}{S}$	Therefore, there is no uplift. $q_{1,2} = \dfrac{7587\text{ lb}}{(5.67\text{ ft})(1\text{ ft})} \pm \dfrac{(7587\text{ lb})(0.90\text{ ft})}{(5.67\text{ ft})^2/6}$ $q_{1,2} = 1338\text{ psf} \pm 1274\text{ psf}$ $q_1 = q_{max} = 2612\text{ psf} < q_{all} = 3000\text{ psf}$ $q_2 = q_{min} = 64\text{ psf} > 0\text{ psf}$, no tension Soil bearing pressure is acceptable.
Step 6: Stability requirements		
	Calculate factor of safety against overturning: $FS = \dfrac{\sum M_R}{M_{OTM}} \geq 2.0$	$FS = \dfrac{20{,}978\text{ ft-lb}}{6276\text{ ft-lb}} = 3.3$ $FS = 3.5 > 2.0$ **OK**
	Calculate the factor of safety against sliding: $FS = \dfrac{\sum \mu P}{\sum H} \geq 1.5$	$FS = \dfrac{(0.4)(7587\text{ lb})}{(32.5\text{ pcf})(10.5\text{ ft})(10.5\text{ ft}/2)} = 1.69$ $FS = 1.69 > 1.5$ **OK**
	This calculation neglects the passive pressure against the toe (conservative).	
Conclusion: The retaining wall preliminary dimensions are adequate to resist overturning, sliding, preventing uplift, and limiting pressure on the soil to less than the allowable provided soil pressure in the geotechnical report. In the following steps, the retaining wall is designed for strength. If any of the aforementioned determined dimensions are not satisfactory, then all the previous steps must be revised. Note: Unfactored loads were used to determine the stability of the retaining wall and to calculate the soil pressure.		

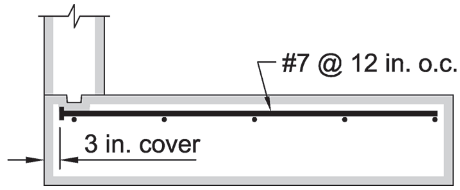
Step 7: Stem design		
	The cantilevered concrete stem is a determinate member and is modeled as a 1 ft wide cantilever beam (refer to Fig. E6.4).	 *Fig. E6.4—Soil lateral force on stem.*
13.2.7.1	<u>Flexure</u> The maximum design moment in the stem is calculated at the face of the base foundation. Vertical reinforcement in the stem resists the lateral earth pressure and is placed near the face of the stem wall that is against the retained soil. Adequate concrete cover protects reinforcement against moisture changes in soil. Cover is measured from the concrete surface to the outermost surface of the reinforcing bar.	
	Stem height:	h_{stem} = (8 ft) + (2 ft 6 in.) – (1 ft 3 in.) = 9 ft 3 in.
20.6.1.3.1	From Table 20.6.1.3.1 use, 2 in. cover	
21.2.2	Assume that the member is tension controlled; steel strain $\varepsilon_t = 0.005$ and $\phi = 0.9$	
7.4.1.1 5.3.8(a)	Load factor: $U = H_u = 1.6H$; when lateral pressure acts alone.	H_u = 1.6(32.5 pcf)(9.25 ft)(9.25 ft)/2 = 2225 lb
	The moment is taken at the bottom of the stem and above the base; h_1 = 9.25 ft	M_u = (2225 lb)(9.25/3) = 6860 ft-lb ≅ 82,300 in.-lb
7.5.1.1	Satisfy: $\phi M_n \geq M_u$	
22.2.2.1	The concrete compressive strain at which ultimate moments are developed is equal to $\varepsilon_c = 0.003$.	
22.2.2.2	The tensile strength of concrete in flexure is a variable property and its value is approximately 10 to 15 percent of the concrete compressive strength. For calculating nominal strength ACI 318 neglects the concrete tensile strength.	

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	$\beta_1 = 0.85 - \frac{0.05(4500\text{ psi} - 4000\text{ psi})}{1000\text{ psi}} = 0.825$
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 $T = A_sf_y$	$0.85(4500\text{ psi})(12\text{ in.})(a) = A_s(60{,}000\text{ psi})$ $a = \frac{A_s(60{,}000\text{ psi})}{0.85(4500\text{ psi})(12\text{ in.})} = 1.31A_s$
	Calculate required reinforcement area:	Equating design moment strength and required moment strength, A_s is:
7.5.2.1 22.3	$M_n = A_sf_y\left(d - \frac{a}{2}\right)$	$M_n = (60{,}000\text{ psi})A_s\left(7.68\text{ in.} - \frac{1.31A_s}{2}\right)$
21.2.1	Use flexure strength reduction factor:	$\phi = 0.9$
20.6.1.3.1	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: $\phi M_n \geq M_u$	$(0.9)(60\text{ ksi})A_s\left(7.68\text{ in.} - \frac{1.31A_s}{2}\right) = 82.3\text{ in.-kip}$
	$M_u = 82.3$ 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.
9.6.1.2	The cantilevered retaining wall calculated required tensile reinforcement is usually very small compared to the member concrete section. The stem reinforcement is checked against the beam minimum required flexural reinforcement area rather than the one-way slab minimum reinforcement area because of the lack of redundancy. The Code requires that the beam reinforcement area at least the greater of:	$A_{s,prov'd} = 0.31\text{ in.}^2/\text{ft} > A_{s,req'd} = 0.2\text{ in.}^2/\text{ft}$
	(a) $A_{s,min} \geq \frac{3\sqrt{f_c'}}{f_y}b_wd$	$A_{s,\min} = \frac{3\sqrt{4500\text{ psi}}}{60{,}000\text{ psi}}(12\text{ in.})(7.68\text{ in.}) = 0.31\text{ in.}^2$

	Equation (9.6.1.2(a)) controls, because concrete compressive strength, f_c' = 4500 psi.	Use No. 5 at 12 in. on center. $A_{s,prov'd}$ = 0.31 in.^{2}/ft $A_{s,prov'd}$ = 0.31 in.^{2}/ft = $A_{s,min}$ = 0.31 in.^{2}/ft > $A_{s,req'd}$ = 0.2 in.^{2}/ft OK
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	$\varepsilon_t = \dfrac{\varepsilon_c}{c}(d-c)$	$\varepsilon_t = \dfrac{0.003}{0.49\text{ in.}}(7.68\text{ in.} - 0.49\text{ in.}) = 0.044$ $\varepsilon_t = 0.044 > 0.005$
	where: $c = \dfrac{a}{\beta_1}$ and $a = 1.31A_s$ derived previously.	Section is tension controlled and $\phi = 0.9$.  *Fig. E6.5—Strain distribution across stem.*
7.4.3.2	<u>Shear</u> The closest inclined crack to the support of the cantilevered wall will extend upward from the face of the base reaching the compression zone approximately d from the face of the base. The lateral load applied to the cantilever between the face of the base and point d away from the face are transferred directly to the base by compression in the cantilever above the crack. Accordingly, the Code permits design for a maximum factored shear force V_u at a distance d from the support for nonprestressed members. For simplicity, the critical section for design shear strength in this example is calculated at the bottom of the stem:	$h_{stem} = h - t_{base}$ = 10.5 ft – 1.25 ft = 9.25 ft V_u = 1.6(32.5 pcf)(9.25 ft)(9.25 ft/2) = 2225 lb
7.5.3.1 22.5.5.1	$V_n = (V_c + V_s) = 2\lambda\sqrt{f_c'}b_w d$ with $V_s = 0$	$V_n = 2\sqrt{4500\text{ psi}}(12\text{ in.})(7.68\text{ in.}) = 12{,}365\text{ lb}$
21.2.1	Shear strength reduction factor:	$\phi = 0.75$
	Therefore, $\phi V_n = \phi V_c$ is:	ϕV_n = (0.75)(12,365 lb) = 9273 lb
7.5.1.1	Is $\phi V_n \geq V_u$ satisfied?	ϕV_n = 9273 lb > V_u = 2225 lb **OK** Shear reinforcement is not required

Step 8: Heel design		
	Shear The base heel is designed for shear caused by the superimposed weight of soil, including self-weight of heel. The soil pressure counteracting the applied gravity loads is neglected as the soil pressure may not be linear as assumed (refer to Fig. E6.6).	 *Fig. E6.6—Force on heel.*
21.2.1	Shear strength reduction factor:	$\phi = 0.75$
7.4.3.2 7.5.3.1 22.5.5.1	The critical section for shear strength is taken at a distance d from the bottom of the stem: $V_n = (V_c + V_s) = 2\lambda\sqrt{f_c'}b_w d$ with $V_s = 0$	
20.6.1.3.1	$d = t_{base}$ – cover – $d_b/2$ From Table 20.6.1.3.1, use 3 in. cover to tension reinforcement. Assume No. 6 reinforcement Then, $\phi V_n = \phi V_c$ is: A load factor of 1.2 is used for the concrete self-weight and 1.6 for backfill self-weight.	d =15 in. – 3 in. – 0.375 in. = 11.6 in. ϕV_n=(0.75)2$\sqrt{4500\text{ psi}}$(12 in.)(11.6 in.) = 14,007 lb V_u = (1.2)(150 pcf)(4.84 ft – 11.3 in./12)(1.25 ft) + (1.6)(120 pcf)(9.25 ft)(4.84 ft – 11.6 in./ft) V_u = 7750 lb
7.5.1.1	Is $\phi V_n \geq V_u$ satisfied?	ϕV_n = 14,007 lb > V_u = 750 lb **OK** Shear reinforcement is not required

	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 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	$M_n = f_y A_s\left(d - \dfrac{a}{2}\right)$	$M_n = (60{,}000\text{ psi})A_s\left(11.6\text{ in.} - \dfrac{1.31A_s}{2}\right)$
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:	$0.9(60\text{ ksi})A_s\left(11.6\text{ in.} - \dfrac{1.31A_s}{2}\right) \geq 281\text{ in.-kip}$ A_s = 0.46 in.^{2}
9.6.1.2	The cantilevered retaining wall calculated tensile reinforcement is very small compared to the member concrete section. To prevent a sudden failure, the Code requires that the flexural reinforcement area is at least the greater of: (a) $A_{s,min} \geq \dfrac{3\sqrt{f_c'}}{f_y} b_w d$ Equation (9.6.1.2(a)) controls, because concrete compressive strength f_c' = 4500 psi.	$A_{s,min} = \dfrac{3\sqrt{4500\text{ psi}}}{60{,}000\text{ psi}}(12\text{ in.})(11.5\text{ in.}) = 0.46\text{ in.}^2$ Use No. 7 @ 12 in. on center (refer to Fig. E6.7). $A_{s,prov'd}$ = 0.6 in.^{2}/ft > $A_{s,req'd}$ = 0.46 in.^{2}/ft **OK**
21.2.2	Check if the tension controlled assumption and the use of ϕ = 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.	$c = \frac{(1.31)(0.60 \text{ in.}^2)}{0.825} = 0.95 \text{ in.}$ $\varepsilon_t = \frac{0.003}{0.95 \text{ in.}}(11.6 \text{ in.} - 0.95 \text{ in.}) = 0.0336$ $\varepsilon_t = 0.0336 > 0.005$ Section is tension controlled and $\phi = 0.9$.  *Fig. E6.7—Heel reinforcement.*
Step 9: Toe design		
	Because the cantilever wall is built at the property line, there is no toe at the base.	
Step 10: Minimum transverse reinforcement		
11.6.1	<u>Stem</u> Assume that No. 5 bars or smaller will be used for temperature and shrinkage reinforcement. Per Table 11.6.1: $\rho_{min} = 0.002$ Distribute shrinkage and temperature reinforcement equally between the front and back face of the stem wall. Provide vertical reinforcement at front face to support the transverse wall reinforcement.	$A_{s,min} = (0.002)(12 \text{ in.})(10 \text{ in.}) = 0.24 \text{ in.}^2$ $(A_{s,min})_{front} = (A_{s,min})_{back} = 0.24 \text{ in.}^2/2 = 0.12 \text{ in.}^2$ Try No. 4 at 18 in. on center $A_{s,prov.} = (0.2 \text{ in.}^2)\left(\frac{12 \text{ in.}}{18 \text{ in.}}\right) = 0.133 \text{ in.}^2$ $> A_{s,req'd} = 0.12 \text{ in.}^2\text{/face}$ Use No. 4 spaced at 18 in. on center
24.4.3	<u>Base</u> Assume that No. 5 bars or smaller will be used for temperature and shrinkage reinforcement. The reinforcement can be located at the top, bottom, or allocated between the two faces.	$(0.0018)(1.25 \text{ ft})(12 \text{ in./ft})(5.67 \text{ ft})(12 \text{ in./ft}) = 1.84 \text{ in.}^2$ Use five No. 5 placed at the top and two No.5 at the bottom as continuous nose bars for the dowel reinforcement extended into the stem (refer to Fig. E6.8).

Step 11: Dowels		
7.7.1.2 25.4.2	The development length concept is based on the attainable average bond stress over the embedment length of reinforcement. Development lengths are required because of the tendency of highly stressed bars to split relatively thin sections of restraining concrete. In application, the development length concept requires minimum lengths or extensions of reinforcement beyond all points of peak stress in the reinforcement. Such peak stresses generally occur at the points of critical sections for development of reinforcement in flexural members and at points within a span where adjacent reinforcement terminates, or is bent. For the cantilevered wall, heel reinforcement must be properly developed. Heel reinforcement is developed beyond the stem critical section.	
25.4.2.1	Development length, ℓ_d, is the greater of Eq. (25.4.2.2) or (25.4.2.3) of ACI 318-14 and 12 in.:	
25.4.2.2	1. $\ell_d = \left(\dfrac{f_y \psi_t \psi_e}{200\lambda\sqrt{f'_c}}\right)d_b$	
25.4.2.3	2. $\ell_d = \left(\dfrac{3}{40}\dfrac{f_y}{\lambda\sqrt{f'_c}}\dfrac{\psi_t\psi_e\psi_s}{\dfrac{c_b+K_{tr}}{d_b}}\right)d_b$	
25.4.2.4	In this example Eq. (25.4.2.3a) will be used. ψ_t = bar location; not more than 12 in. of fresh concrete below horizontal reinforcement ψ_e = coating factor; uncoated ψ_s = bar size factor; No. 7 and larger c_b = spacing or cover dimension to center of bar, whichever is smaller K_{tr} = transverse reinforcement index It is permitted to use $K_{tr} = 0$.	ψ_t = 1.0, because not more than 12 in. of concrete is placed below bars. ψ_e = 1.0, because bars are uncoated ψ_s = 1.0, because bras are not larger than No. 7 $\dfrac{2.3125\text{ in.}+0}{0.625\text{ in.}} = 3.7 > 2.5$ $\dfrac{2.44\text{ in.}+0}{0.875\text{ in.}} = 2.79 > 2.5$
25.4.2.3	However, the expression: $\dfrac{c_b+K_{tr}}{d_b}$ must not be taken greater than 2.5. Note: the development length in the stem must be checked against the splice length of stem reinforcement and the larger length controls.	$\dfrac{2.25\text{ in.}+0}{0.5\text{ in.}} = 4.5 > 2.5$ Therefore, use 2.5 for all three bar sizes in Eq. (25.4.2.3a) $\ell_d = \left(\dfrac{3}{40}\dfrac{60{,}000\text{ psi}}{(1.0)\sqrt{4500\text{ psi}}}\dfrac{(1.0)(1.0)(1.0)}{2.5}\right)(d_b)$

No.	$\ell_{d,req'd}$, in.	$\ell_{d,prov.}$, in.
7	23.5	24
5	16.8	18
4	13.4	15