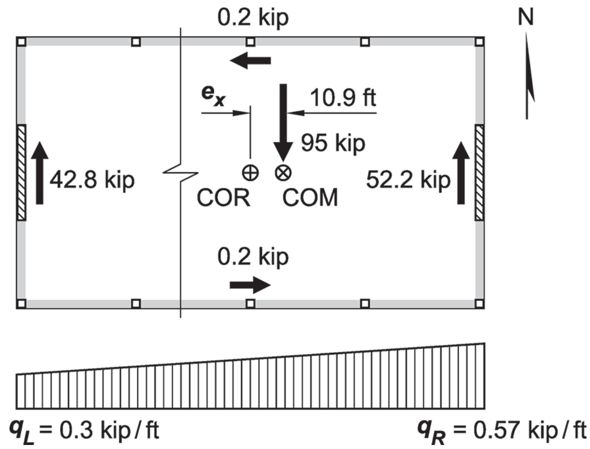
Step 8: Diaphragm lateral force distribution N-S		
	Lateral force is distributed to the walls as follows, refer to Fig. E1.4:	 *Fig. E1.4— Seismic forces in the lateral-force-resisting systems in the N-S direction.*
12.4.2.4(a) 12.5.1.3(a)	Diaphragm is idealized as rigid. Design moments, shear, and axial forces are calculated assuming the diaphragm is a beam supported by idealized rigid supports with a depth equal to full diaphragm depth. The wall and frame forces and the assumed direction of torque due to the eccentricity are shown in Fig. E1.4.	
12.4.2.4	The diaphragm force distribution is calculated by using q_L and q_R as the left and right diaphragm reactions per unit length (Fig. E1.4): Design force: 95 kip	
	Force equilibrium $q_L\left(\frac{L}{2}\right)+q_R\left(\frac{L}{2}\right)=F_{px,des(NS)}$	$q_L\left(\frac{218\text{ ft}}{2}\right)+q_R\left(\frac{218\text{ ft}}{2}\right)=95\text{ kip}\quad\text{(I)}$
	Moment equilibrium $q_L\left(\frac{L}{2}\right)\left(\frac{L}{3}\right)+q_R\left(\frac{L}{2}\right)\left(\frac{2L}{3}\right)=F_{px,des(NS)}\left(\frac{L}{2}+0.05L\right)$ From statics solve equations (I) and (II) for q_L and q_R:	$q_L\frac{(218\text{ ft})^2}{6}+q_R\frac{2(218\text{ ft})^2}{6}=(95\text{ kip})\left(\frac{218\text{ ft}}{2}+10.9\text{ ft}\right)\text{(II)}$ $q_L + q_R = 0.87$ kip/ft (I) $q_L + 2q_R = 1.44$ kip/ft (II) q_L =0.3 kip/ft and q_L = 0.57 kip/ft

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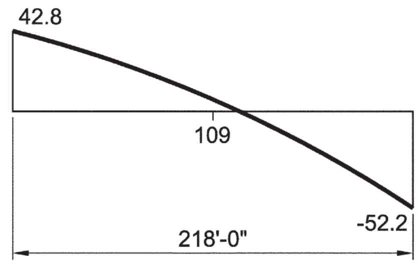
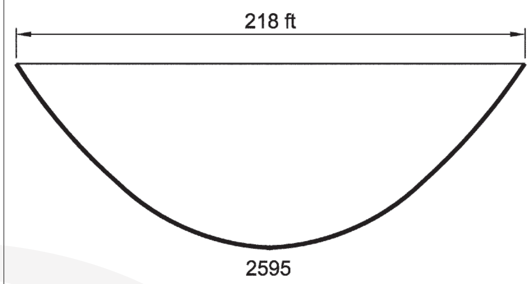
	Find the maximum moment by taking the first derivative of the moment equation expressed as a function of x (unknown distance) $dM/dx = 0$	$x = 114.3$ ft $M_{max} = 2595$ ft-kip
	Draw the shear and moment diagrams (Fig. E1.5). Note: In an Aug. 2010 National Institute and Standards Technology (NIST) report, GCR 10-917-4, "Seismic Design of Cast-in-Place Concrete Diaphragms, Chords, and Collectors," by Moehle et al. states that, "This approach leaves any moment due to the frame forces along column lines A and F unresolved. Sometimes this is ignored or, alternatively, it too can be incorporated in the trapezoidal loading." In this example the small moment due to the frame forces (0.2 kip) is ignored.	 *Diaphragm shear diagram (kip)*  *Diaphragm moment diagram (ft-kip)* *Fig. E1.5—Shear and moment diagrams.*
	Note: Experienced engineers sometimes simplify the calculations by distributing the load uniformly:	$q = \dfrac{95 \text{ kip}}{218 \text{ ft}} = 0.44$ kip/ft
	Resulting in a maximum moment of:	$M_{max} = \dfrac{(0.44 \text{ kip/ft})(218 \text{ ft})^2}{8} = 2614$ ft-kip Note: Both approaches, in this example, result in close maximum moments (less than 1 percent), but at different locations (114.3 ft versus 109 ft). Shear diagram for the second approach is a straight line with equal shear force at both ends. In this example, the detailed approach is presented.
Step 9: Chord reinforcement N-S		
	Maximum moment is calculated above:	$M_u = 2595$ ft-kip
12.5.2.3	Chord reinforcement resisting tension must be located within $h/4$ of the tension edge of the diaphragm.	$h/4 = 72.0$ ft/4 = 18 ft
	Note: Chord reinforcement can be placed either in the exterior edge of the balcony or it can be placed along the exterior frame of CL A. Placing chord reinforcement along the exterior frame is a simpler and cleaner load path for the forces in the diaphragm. Crack control reinforcement should be added in the balcony slab for crack control.	

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	Assume tension reinforcement is placed in a 3 ft strip at both north and south sides of the slab edges at CLs A and F.	3 ft < h/4 = 18 ft **OK**
	Chord force The overhang is placed monolithic with the rest of the slab. Chord forces are usually highest furthest from the geometric centroid, in this case, edge of the overhang. To prevent cracking, place chord reinforcement at the outside edge of the overhang. The maximum chord tension force is calculated at 114.3 ft east of CL 1:	
	$T_u = \dfrac{M_u}{B - 3\text{ ft}}$	$T_u = \dfrac{2595\text{ ft-kip}}{72\text{ ft} - 3\text{ ft}} = 37.6\text{ kip}$
12.5.2.2	Tension due to moment is resisted by deformed bars conforming to Section 20.2.1 of ACI 318-14.	
12.5.1.5	Steel stress is the lesser of the specified yield strength and 60,000 psi.	f_y = 60,000 psi
12.5.1.1	Required reinforcement $\phi T_n = \phi f_y A_s \geq T_u$	$A_{s,req'd} = \dfrac{37{,}600\text{ lb}}{(0.9)(60{,}000\text{ psi})} = 0.70\text{ in.}^2$
22.4.3.1	The building is assigned to SDC B. Therefore, Chapter 18 requirements for chord spacing and transverse reinforcement of Section 18.12.7.6 of ACI 318-14 do not apply.	
18.12.7.5	Overstrength factor Ω_o for chord design is not required. Therefore, use the compression stress limit in provision 18.12.7.5 of $0.2f_c'$.	
	Required chord width: $w_{chord} > \dfrac{C_{Chord}}{0.2f_c'h_{diaph}}$	$w_{chord} > \dfrac{37{,}600\text{ lb}}{(0.2)(5000\text{ psi})(7\text{ in.})} = 5.4\text{ in.}$ Less than 3 ft assumed. Therefore, **OK**
	Choose reinforcement:	Try two No. 6 chord bars. $A_{s,prov.} = 2(0.44\text{ in.}^2) = 0.88\text{ in.}^2$
	Check if provided reinforcement area is greater than required reinforcement area:	$A_{s,prov.} = 0.88\text{ in.}^2 > A_{s,req'd} = 0.70\text{ in.}^2$

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The engineer has two options for providing chord reinforcement along the exterior frames:
1. Excess amount of reinforcement used in the beams to resist gravity loads could be used to resist part of the tensile force of the chord. Additional reinforcement is provided to resist the difference.
2. The chord force is resisted with additional reinforcement.

Although the first option is more economical, the second option is detailed in this example. Here again the engineer has several options:
1. Place chord reinforcement outside the web width
2. Place chord reinforcement within the web width

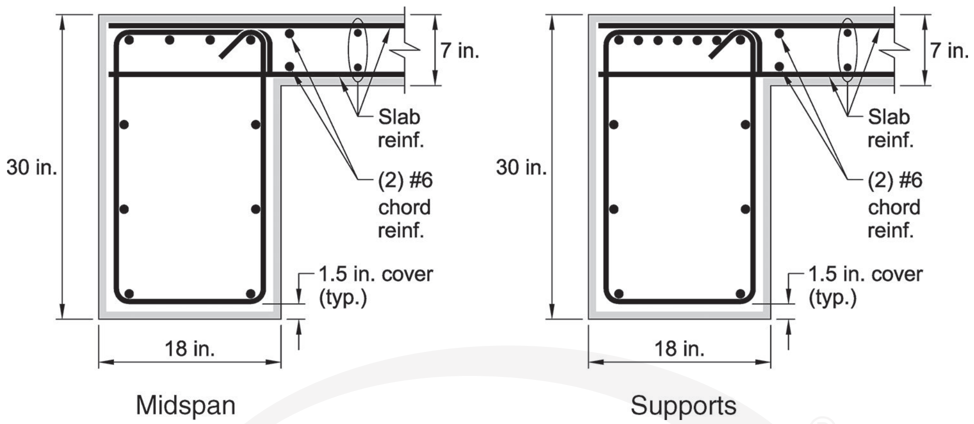

Fig. E1.6—Chord reinforcement along CLs A and F.

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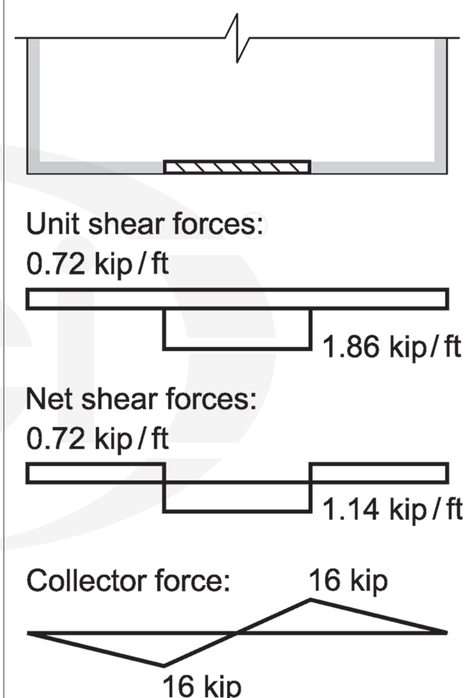
Step 10: Collector reinforcement N-S		
12.5.4.1	Collector elements transfer shear forces from the diaphragm to the vertical walls at both east and west ends along column lines 1 and 7 (Fig. E1.2). Collector elements extend over the full width of the diaphragm. Unit shear force:	
	$v_{u@F} = \frac{F_{u@F}}{B}$	From Step 6: F_u = 52.2 kip
	In diaphragm: $v_{u@F} = \frac{F_{px}}{L_{diaph}}$	$v_{u@F} = \frac{52.2\text{ kip}}{72\text{ ft}} = 0.72\text{ kip/ft}$
	In wall: $v_{u@F} = \frac{F_{px}}{L_{wall}}$	$v_{u@F} = \frac{52.2\text{ kip}}{28\text{ ft}} = 1.86\text{ kip/ft}$
	<u>Force at diaphragm to wall connection</u> East wall south end: $F_{7/D.5}$ = –(0.72 kip/ft)(22 ft) = –15.8 kip East wall north end: $F_{7/B.5}$ = –15.8 kip + (1.14 kip/ft)(28 ft) = 16.0 kip At diaphragm end: $F_{7/A}$ = +16 kip – (0.72 kip/ft)(22 ft) = 0.2 kip ≈ 0 kip due to number rounding. Per collector force diagram, the maximum axial force on the collector is $T_u = C_u$ = 16 kip. This force must be transferred from the diaphragm to the collector to the shear wall (Fig. E1.7). The building is assigned to SDC B. Therefore, the collector force and its connections to the shear wall will not be multiplied by the system overstrength factor, Ω_o = 2.5 (ASCE/SEI 7-10, Table 12.2-1).	
12.5.4.2	Collectors are designed as tension members, compression members, or both.	*Fig. E1.7—Collector force diagram.*

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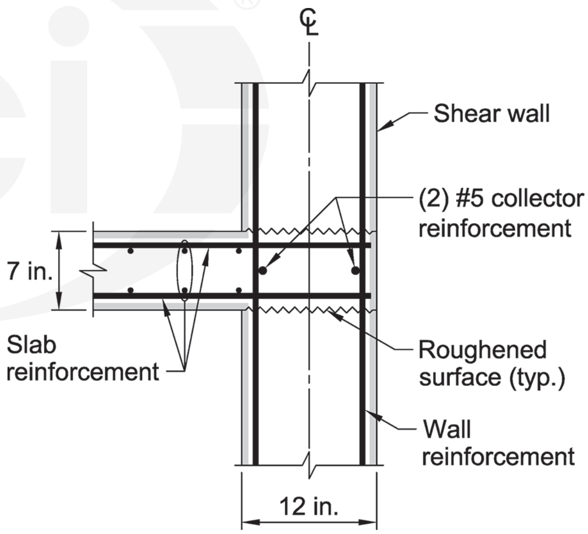
12.5.1.1 22.4.3.1	Tension is resisted by reinforcement as calculated above. Required reinforcement: $\phi T_n = \phi f_y A_s \geq T_u$	$A_{s,req'd} = \dfrac{T_u}{0.9 f_y} = \dfrac{16{,}000 \text{ lb}}{(0.9)(60{,}000 \text{ psi})} = 0.3 \text{ in.}^2$ Although one No. 5 bar suffices, two No. 5 bars are provided to maintain symmetry of load being transferred from the slab into the wall.
18.12.7.5	Check if collector compressive force exceeds $0.2f_c'$. Calculate minimum required collector width using $0.2f_c'$ $w_{coll.} > \dfrac{C_{Coll.}}{0.2 f_c' t_{diaph}}$	$w_{coll} > \dfrac{16{,}000 \text{ lb}}{(0.2)(5000 \text{ psi})(7 \text{ in.})} = 2.3 \text{ in.}$
	This results in compressive stress on the concrete diaphragm collector being relatively low. The section is adequate to transfer shear stress without additional reinforcement. The collector compression and tension forces are transferred to the lateral force-resisting system within its width (shear wall). Therefore, no eccentricity is present and no-in-plane bending occurs.	Use 12 in. wide collector (same width as shear wall). Provide two No. 5 bars at mid-depth of slab to prevent additional out-of-plane bending stresses in the slab. Space the two No. 5 bars at 8 in. on center starting at 2 in. from the edge of the diaphragm within the 12 in. wide collector/shear wall (Fig. E1.8).
12.5.4.1	ACI 318-14 permits to discontinue the collector along the length of the shear wall where transfer of design collector is not required.	 *Fig. E1.8—Collector reinforcement.*
	<u>Check slab shear strength along shear walls</u>	
12.4.2.4	Slab shear strength along walls: L = 28 ft and slab thickness t = 7 in. From	
12.5.3.3	$\phi V_c = \phi A_{cv} 2\lambda\sqrt{f_c'}$	$\phi V_c = (0.75)(2)(1.0)\left(\sqrt{5000 \text{ psi}}\right)(28 \text{ ft})(12)(7 \text{ in.})$
21.2.4.2	$\phi = 0.75$	ϕV_c = 249,467 lb ~249 kip
12.5.1.1	Is the provided shear strength adequate?	ϕV_c = 249 kip > V_u = 52 kip (from Step 7) **OK**
12.5.3.4	By inspection, the diaphragm shear design force satisfies the requirement of Section 12.5.3.4 of ACI 318-14. $\phi V_c = \phi A_{cv} 8\lambda\sqrt{f_c'}$	

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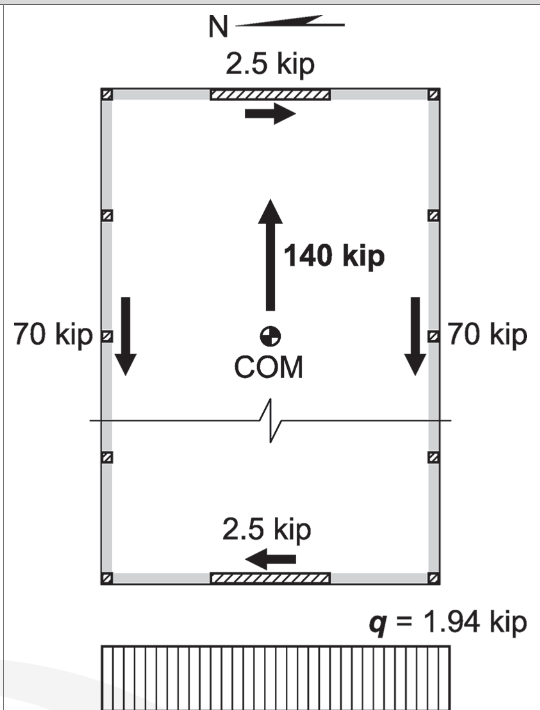
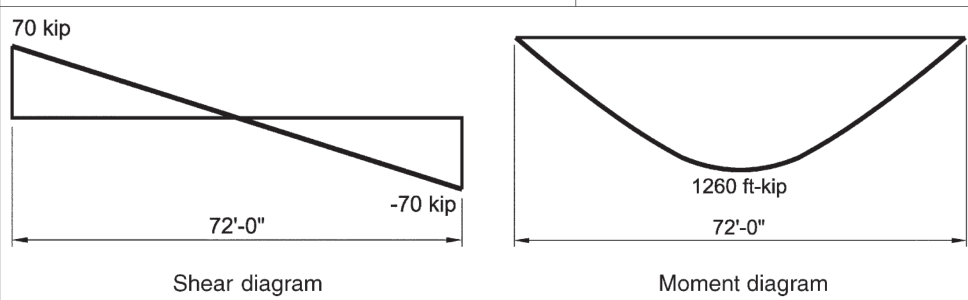
Step 11: Lateral force distribution in diaphragm E-W		
	Design force: 140 kip	
12.4.2.4(a) 12.5.1.3(a)	Design moments, shear, and axial forces are calculated assuming a simply supported beam with depth equal to full diaphragm length (refer to Fig. E1.9).	*Fig. E1.9—Seismic forces in the lateral force resisting systems in the E-W direction.*
	Because of the negligible effect of accidental torsion, the inertial force is uniformly distributed across the diaphragm width.	$q_L = \left(\dfrac{140\text{ kip}}{72\text{ ft}}\right) = 1.94\text{ kip/ft}$
		Shear force: V = (1.94 kip/ft)(36 ft) = 70 kip
	Maximum moment is located at midlength.	x = 36 ft
	Draw the shear and moment diagrams (Fig. E1.10).	$M_{max} = \dfrac{w\ell^2}{8} = \dfrac{(1.94\text{ kip/ft})(72\text{ ft})^2}{8} = 1260\text{ ft-kip}$
	 *Fig. E1.10—Shear and moment diagrams.*	

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Step 12: Chord reinforcement E-W		
	Calculate chord reinforcement Maximum moment is calculated above (Fig. E1.8).	M_u = 1260 ft-kip
12.5.2.3	Chord reinforcement must be located within h/4 of the tension edge of the diaphragm.	$h/4 = 218.0\text{ ft}/4 = 54.5\text{ ft}$
	Assume tension reinforcement is placed within a 1 ft strip of the slab edge at both east and west sides of the slab.	$1\text{ ft} < h/4 = 54.5\text{ ft}$ **OK**
	Chord force The maximum chord tension force is at midspan: $T_u = \dfrac{M_u}{L - 1\text{ ft}}$	$T_u = \dfrac{1260\text{ ft-kip}}{218\text{ ft} - 1\text{ ft}} = 5.8\text{ kip}$
12.5.2.2	Chord reinforcement Tension due to moment is resisted by deformed bars confirming to Section 20.2.1 of ACI 318-14.	
12.5.1.5	Steel stress is the lesser of the specified yield strength and 60,000 psi.	f_y = 60,000 psi
12.5.1.1 22.4.3.1	Required reinforcement $\phi T_n = \phi f_y A_s \geq T_u$	$A_{s,req'd} = \dfrac{T_u}{0.9 f_y} = \dfrac{5800\text{ lb}}{(0.9)(60{,}000\text{ psi})} = 0.1\text{ in.}^2$
	Along column lines 1 and 7, two No. 5 bars collector reinforcement are provided to resist inertia force in the N-S direction. These bars can be used for chord reinforcement in the E-W direction (refer to Fig. E1.8).	$A_{s,prov.} = (2)(0.31\text{ in.}^2) = 0.62\text{ in.}^2 > 0.1\text{ in.}^2$ **OK**
	Maximum shear in the E-W direction occurs at CLs 1 and 7: Unit shear force in frame: $v_{u@1,7} = \dfrac{F_{u@1,7}}{L}$	$v_{u@1,7} = \dfrac{70\text{ kip}}{218\text{ ft}} = 0.32\text{ kip/ft}$
Step 13: Collector reinforcement		
	Collector along CLs A and F: The continuous reinforced concrete frame over the full length of the building acts as a collector. Note: Provide continuous reinforcement with tension splices (Step 15).	
12.5.3.7	In cast-in-place diaphragms, where shear is transferred from the diaphragm to a collector, or from the diaphragm or collector to a shear wall, temperature and shrinkage reinforcement is usually adequate to transfer that force.	

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Step 14: Shrinkage and temperature reinforcement		
12.6.1 24.4.3.2	The minimum area of shrinkage and temperature reinforcement, A_{S+T}: $A_{S+T} \geq 0.0018A_g$	 $A_{S+T} = (0.0018)(7\text{ in.})(12\text{ in./ft}) = 0.15\text{ in.}^2/\text{ft}$
24.4.3.3	Spacing of S+T reinforcement is the lesser of 5*h* and 18 in.: (a) $5h = 5(12\text{ in.}) = 60$ in. (b) 18 in. **Controls**	Note: Shrinkage and temperature reinforcement may be part of the reinforcing bars resisting diaphragm in-plane forces and gravity loads. If provided reinforcement is not continuous (placing bottom reinforcing bars to resist positive moments at mid-spans and top reinforcing bars to resist negative moments at columns), continuity between top and bottom reinforcing bars can sometimes be achieved by providing adequate splice lengths between them.
Step 15: Reinforcement detailing		
 12.7.2.1	<u>Reinforcement spacing</u> Minimum and maximum spacing of chord and collector reinforcement must satisfy 12.7.2.1 and 12.7.2.2.	
25.2.1	Section 25.2 limits minimum spacing of (a) 1 in. (b) $4/3d_{agg.}$ (d_{agg} = 3/4 in.) (c) d_b (No. 5)	 Minimum spacing 1.0 in. **Controls** (4/3)(3/4 in.) = 1.0 in. 0.625 in.
18.12.7.6a	The minimum collector reinforcement spacing at a splice must be at least the largest of: (a) Three longitudinal d_b (b) 1.5 in. (c) $c_c \geq \max[2.5d_b, 2\text{ in.}]$	 3(0.625 in.) = 1.875 in. 1.5 in. 2 in. **Controls**
12.7.2.2	Maximum spacing is the smaller of 5h or 18 in.	18 in. **Controls**

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Step 16: Details

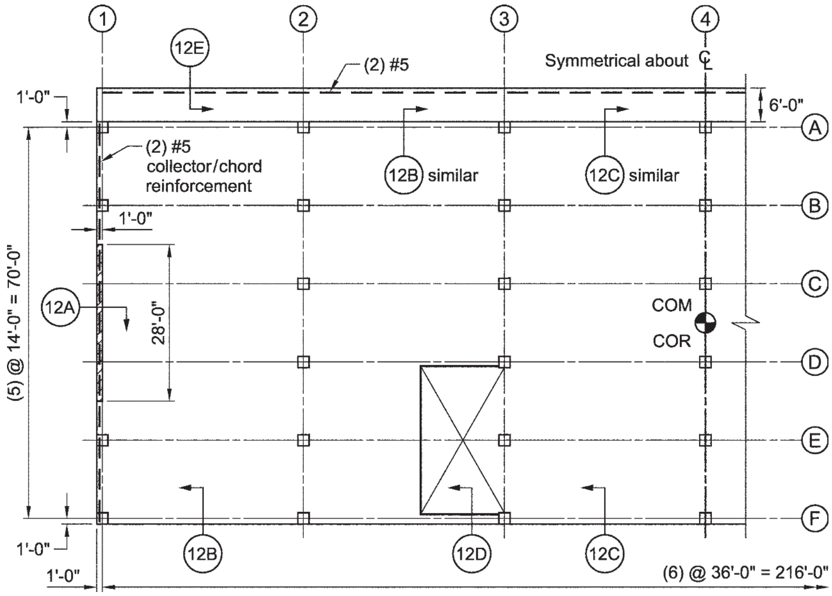
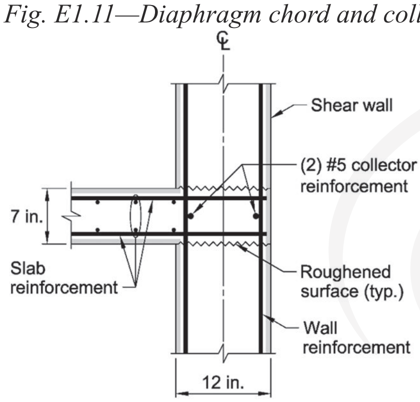
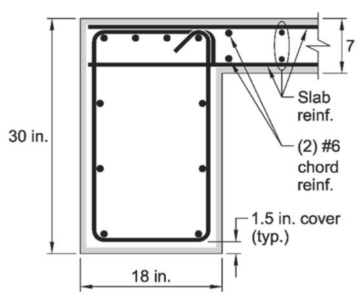


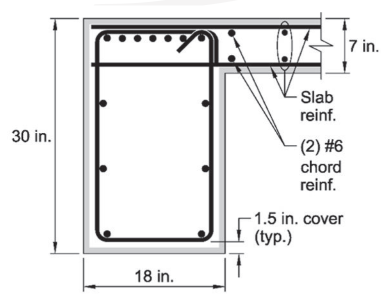
Fig. E1.11—Diaphragm chord and collector reinforcement.



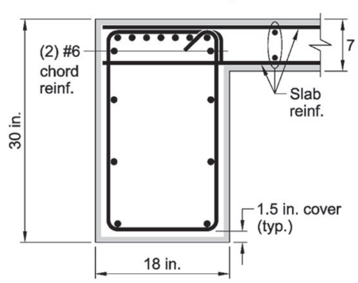
Section 12A—Chord and collector reinforcement at east and west ends of the diaphragm.



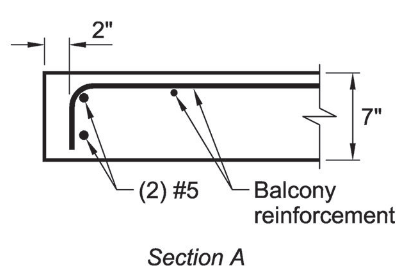
Section 12B—Chord reinforcement at midspan.



Section 12C—Chord reinforcement at support.



Section 12D—Chord reinforcement within the beam at midspan and support at opening location.



Section 12E—Chord reinforcement at overhang.

Note: Shrinkage and temperature reinforcement not shown for clarity.

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