Concrete Reinforcing
Steel Institute

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Design Guide on the ACI 318 Building Code Requirements for Structural Concrete

1st Edition, 1st Printing, 2020

The following pages supersede the versions shown in the CRSI's ***Design Guide on the ACI 318 Building Code Requirements for Structural Concrete, 318-19 Edition***, and should be referenced as such. This errata applies to the 1st Edition, specifically noted as the "First Edition 2020, First Printing."

Updates and/or corrections have been made to the following:

Chapter 6; page 6-25

Chapter 7; page 7-100

Chapter 14; pages 14-133, 14-134, 14-137, 14-138, and 14-139

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$$M_{n1} = A_{sf} f_y \left(d - \frac{h}{2} \right) \tag{6.51}$$

The moment strength of the beam web, M_{u2}, is determined by subtracting the design strength ϕM_{n1} from M_u at the section:

$$M_{u2} = M_u - \phi M_{n1} \tag{6.52}$$

The area of flexural reinforcement, A_{sw}, required to resist M_{u2} is determined by Equations (6.34) and (6.33) where $M_u = M_{u2}$ and $b = b_w$.

The total required area of flexural reinforcement is the sum of the areas corresponding to the contributions from the overhanging flange(s) and the web:

$$A_s = A_{sf} + A_{sw} \tag{6.53}$$

Once A_s is determined by Equation (6.53), the assumption that the section is tension-controlled needs to be checked, that is, it must be verified that $A_{sw} \le A_{s,t}$ where $A_{s,t}$ is determined by Equation (6.37) or (6.38) with $b = b_w$. If the section is not tension-controlled, compression reinforcement may be added to the section to make it tension-controlled or the dimensions of the beam must be modified.

The minimum reinforcement requirements in ACI 9.6.1.2 are also applicable [see Equation (6.35)].

6.5.2 Required Shear Reinforcement

The required area A_v and spacing s of shear reinforcement perpendicular to the axis of a member is determined by Equation (6.8), which can be rewritten as follows:

$$\frac{A_v}{s} = \frac{V_u - \phi V_c}{\phi f_{yt} d} \tag{6.54}$$

The nominal shear strength provided by the concrete, V_c, is obtained from Table 6.4 and the nominal shear strength provided by the shear reinforcement (stirrups), V_s, is calculated by Equation (6.29).

A summary of the shear requirements in ACI 9.6.3, 9.7.6, and 22.5 is given in Table 6.5 assuming the effects from any torsional moments, if present, need not be considered. Note that these requirements are not applicable to the beam types in ACI Table 9.6.3.1, which are cases where the minimum shear reinforcement requirements of ACI 9.6.3.4 need not be satisfied if $V_u \le \phi V_c$. Also, "Along the length" in Table 6.5 refers to the maximum spacing of legs of shear reinforcement along the length of the member and "Across the width" refers to the maximum spacing of legs of shear reinforcement across the width of the member (see Figure 6.19).

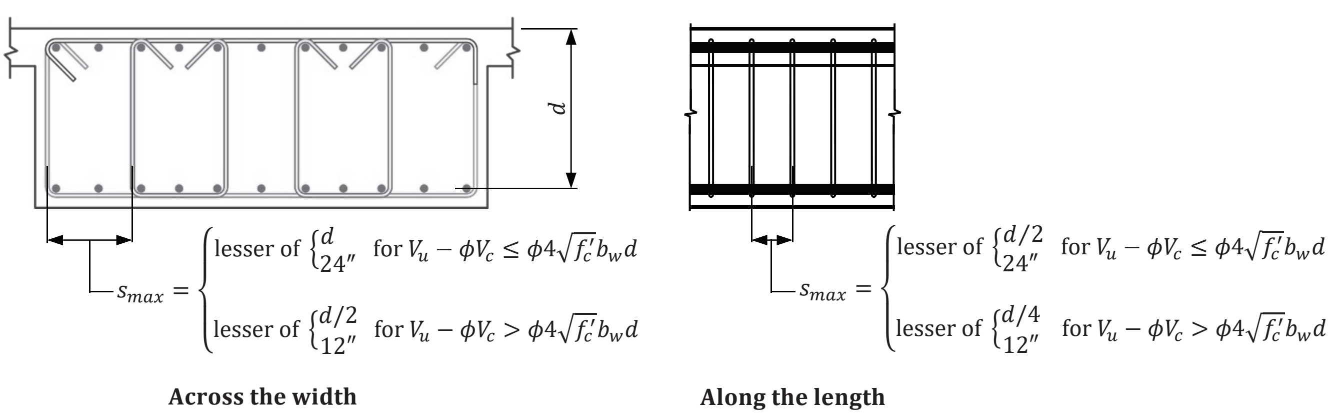

Figure 6.19 Maximum spacing requirements for legs of shear reinforcement.

Step 3 – Check the adequacy of the tie bar size and spacing for shear ACI 22.5

Maximum factored shear force, V_u, is obtained from ACI Eq. (5.3.1d) based on the factored bending moments at the top and bottom of the column (see Table 7.41):

- North-south wind and SSL:

$$(V_u)_{NS} = \frac{27.2 + 65.7}{14.0 - \left(\frac{24.0}{2 \times 12}\right)} = 7.2 \text{ kips}$$

- East-west wind and SSR:

$$(V_u)_{EW} = \frac{34.8 + 48.7}{14.0 - \left(\frac{24.0}{2 \times 12}\right)} = 6.4 \text{ kips}$$

Check if the design shear strength of the concrete, ϕV_c, alone is adequate, which is determined based on whether $A_{v,min}$ is provided or not (see Table 7.12).

$$A_{v,min} = \text{greater of} \begin{cases} \dfrac{0.75\sqrt{f'_c}b_w s}{f_{yt}} = \dfrac{0.75 \times \sqrt{4{,}000} \times 20.0 \times 18.0}{60{,}000} = 0.29 \text{ in.}^2 \\[2ex] \dfrac{50 b_w s}{f_{yt}} = \dfrac{50 \times 20.0 \times 18.0}{60{,}000} = 0.30 \text{ in.}^2 \end{cases}$$ Eq. (7.26)

The minimum area of shear reinforcement is greater than $A_v = 0.15$ in.^{2} provided by the #3 ties spaced 18 in. on center. Because V_u is relatively small in both directions, it is not necessary to increase the tie bar size and/or decrease the spacing, so V_c is determined based on $A_v < A_{v,min}$:

$$V_c = \left[8\lambda_s \lambda (\rho_w)^{1/3} \sqrt{f'_c} + \frac{N_u}{6A_g}\right] b_w d$$ Table 7.12

$$\lambda_s = \sqrt{\frac{2}{1 + (d / 10)}} = \sqrt{\frac{2}{1 + (17.56 / 10)}} = 0.85$$ Eq. (7.23)

where $d = 20.0 - 1.5 - 0.375 - (1.128 / 2) = 17.56$ in.

$\lambda = 1.0$ for normalweight concrete Table 7.13

$\rho_w = \dfrac{A_s}{b_w d} = \dfrac{2 \times 1.00}{20.0 \times 17.56} = 0.0057$ (there are 2-#9 bars located more than two-thirds of the overall member depth from the extreme compression fiber).

- North-south wind:

  Use axial force $N_u = 224.5$ kips, which corresponds to ACI Eq. (5.3.1d) and sidesway to the left.

$$N_u / 6A_g = 224{,}500 / (6 \times 20.0^2) = 93.5 \text{ psi} < 0.05 f'_c = 200.0 \text{ psi}$$ ACI 22.5.5.1.2

The unit shear forces in the walls along column lines C and E and in the wall along column line D are equal to the following:

- Along column lines C and E: $(v_u)_C = (v_u)_E = 124.0 / 30.0 = 4.13$ kips/ft
- Along column line D: $(v_u)_D = 135.5 / 30.0 = 4.52$ kips/ft

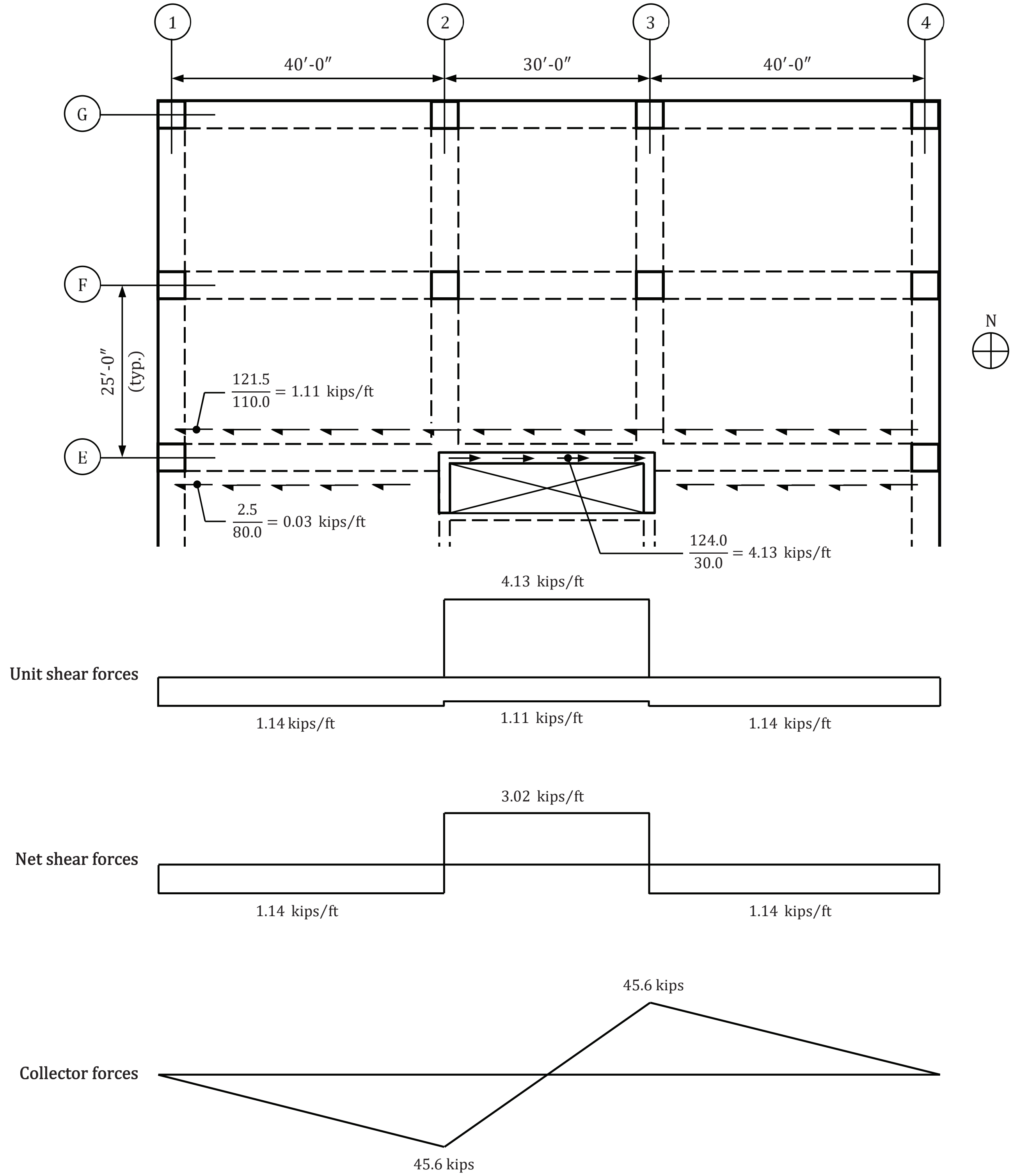

Figure 14.81 Unit shear forces, net shear forces, and collector forces along column line E for the diaphragm in Example 14.13.

The unit shear forces, net shear forces, and collector forces along column E are given in Figure 14.81 (the forces are the same along column line C). Forces along column line D can be obtained in a similar fashion.

Collectors are not required along column lines A and G because the special moment frames extend the entire depth of the diaphragm in the east-west direction.

Collectors are required along column lines C, D, and E. It is assumed the collector beams along these column lines transfer the entire axial force directly into the web of the walls. The beams must be designed for combined flexure due to gravity loads and axial tension and compression forces due to in-plane seismic forces (see Step 14 below).

Step 10 – Determine combined load effects

Combined load effects are determined using the applicable strength design load combinations in ACI 5.3 (see Table 14.1 of this publication). The governing in-plane load effects are due to design seismic forces. In ACI Eq. (5.3.1e), the effects of gravity and seismic ground motion are additive, so $E = \rho Q_E + 0.2S_{DS}D = 1.3Q_E + (0.2 \times 0.517)D = 1.3Q_E + 0.10D$ where $\rho = 1.3$ in accordance with ASCE/SEI 12.3.4.2. Therefore, this load combination becomes the following:

$$1.2D + 0.5L + 1.0E = 1.2D + 0.5L + (1.3Q_E + 0.10D) = 1.3D + 0.5L + 1.3Q_E$$

In buildings assigned to SDC D, collectors and their connections must be designed for the maximum of the three forces given in ASCE/SEI 12.10.2.1:

1. Forces calculated using the seismic load effects including overstrength of ASCE/SEI 12.4.3 with the seismic forces determined by the ELF Procedure of ASCE/SEI 12.8 or the modal response spectrum analysis procedure of ASCE/SEI 12.9.1.

   For dual systems with special reinforced concrete shear walls and special reinforced concrete moment frames capable of resisting at least 25 percent of the prescribed seismic forces, the overstrength factor, Ω_o, is equal to 2.5 from ASCE/SEI Table 12.2-1. The required in-plane diaphragm force at the second-floor level based on this requirement is equal to $2.5 \times 1.0 = 2.5$ kips (see Table 3.33 of this publication).
2. Forces calculated using the seismic load effects including overstrength of ASCE/SEI 12.4.3 with the seismic forces determined by ASCE/SEI Eq. (12.10-1).

   Using the information in Table 3.33, the diaphragm force, F_{px}, at the second-floor level determined by ASCE/SEI Eq. (12.10-1) is equal to the following:

   $$F_{p2} = \left(\sum_{i=2}^{R} F_i \Big/ \sum_{i=2}^{R} w_i \right) w_{p2} = (2{,}528.9 / 109{,}954) \times 4{,}092 = 0.0230 \times 4{,}092 = 94.1 \text{ kips}$$

   The required diaphragm in-plane force including overstrength is equal to $2.5 \times 94.1 = 235.3$ kips based on this requirement.
3. Forces calculated using the load combinations of ASCE/SEI 2.3.6 with the seismic forces determined by ASCE/SEI Eq. (12.10-2).

   The diaphragm force, F_{px}, at the second-floor level determined by ASCE/SEI Eq. (12.10-2) is equal to 421.5 kips (see Table 3.33). The required diaphragm in-plane force based on this requirement is equal to ρ times this value: $1.3 \times 421.5 = 548.0$ kips.

Therefore, the collectors and their connections to the vertical elements of the SFRS must be designed for the effects due to the 548.0-kip in-plane diaphragm force stipulated in the third requirement. Thus, the axial tension and compression forces determined in Step 9 above for the 421.5-kip diaphragm force must be increased by 1.3 (see Figure 14.81): $1.3 \times 45.6 = 59.3$ kips. The collector beams must be designed for an 59.3-kip axial compression and

tension forces in combination with the effects due to the tributary factored gravity loads along the length of the member (see Step 14 below).

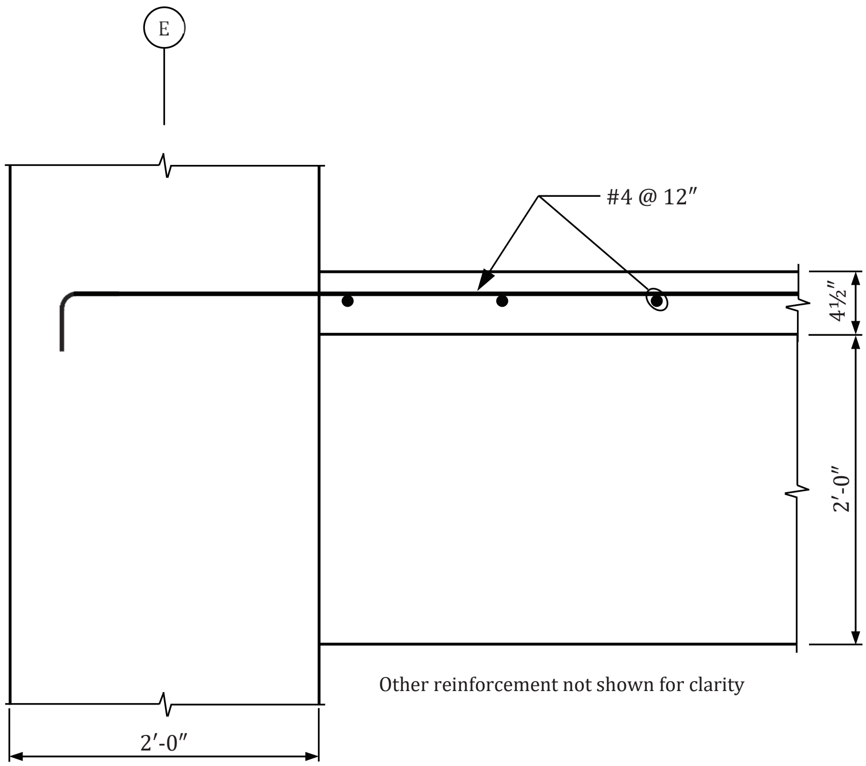

Figure 14.83 Shear transfer reinforcement between the diaphragm and the special structural wall along column line E in Example 14.13 for seismic forces in the east-west direction.

- Diaphragms and the collectors

  According to ASCE/SEI 12.10.2.1, the shear transfer reinforcement providing the connection between the diaphragm and the collectors must be based on the largest of the three forces determined in accordance with that section. From Step 10, the required diaphragm force is equal to 548.0 kips.

  The unit shear force between the diaphragm and the collector beams at column lines C and E is equal to 1.54 kips/ft based on a diaphragm force of 421.5 kips (see Figure 14.81). Therefore, the required shear-friction reinforcement at these locations is equal to the following:

$$A_{vf} = \frac{(v_u)_C}{\phi \mu f_y} = \frac{1.14 \times (548.0\ /\ 421.5)}{0.75 \times 1.4 \times 1.0 \times 60} = 0.02 \text{ in.}^2/\text{ft}$$

  The #4 bars spaced at 12 in. on center in the slab oriented in the north-south direction are adequate for combined flexure and shear transfer $A_{s,provided} = 0.20 \text{ in.}^2/\text{ft} > 0.10 + 0.02 = 0.12 \text{ in.}^2/\text{ft}$.

  Because the unit shear force along column line D is less than that along column lines C and E, #4 bars spaced at 12 in. are also adequate for combined flexure and shear transfer at this location.

Step 14 – Determine the required collector reinforcement

Reinforcement is determined for the 36.0 in. by 28.5 in. collector beams along column lines C and E (determination of the reinforcement for the collector beam along column line D is similar). These beams must be designed for the combined effects from gravity loads (bending moments and shear forces) and seismic forces (axial compression and tension). A summary of the axial forces, bending moments, and shear forces is given in Table 14.21. The 59.3-kip axial compression and tension force in the collectors is determined in Step 10.

Table 14.21 Summary of Design Axial Forces, Bending Moments, and Shear Forces for the Collector Beams Along Column Lines C and E in Example 14.13

Load Case		Axial Force (kips)	Bending Moment (ft-kips)		Shear Force (kips)
			Negative	Positive	
Dead (D)		0	−121.8	57.4	16.5
Live (L)		0	−36.5	18.3	4.1
Seismic (Q_E)		±59.3	0	0	0
Load Combination					
ACI Eq. (5.3.1a)	$1.4D$	0	−170.5	80.4	23.1
ACI Eq. (5.3.1b)	$1.2D + 1.6L$	0	−204.6	98.2	26.4
ACI Eq. (5.3.1e)	$1.3D + 0.5L + Q_E$	±59.3	−176.6	83.8	23.5
ACI Eq. (5.3.1g)	$0.8D + Q_E$	±59.3	−97.4	45.9	13.2

The design strength interaction diagram for the collector beam with 5-#8 top bars, 5-#8 bottom bars, and 1-#5 bar on each side face is given in Figure 14.84. It is evident from the figure that the longitudinal reinforcement is adequate for the load combinations in Table 14.21.

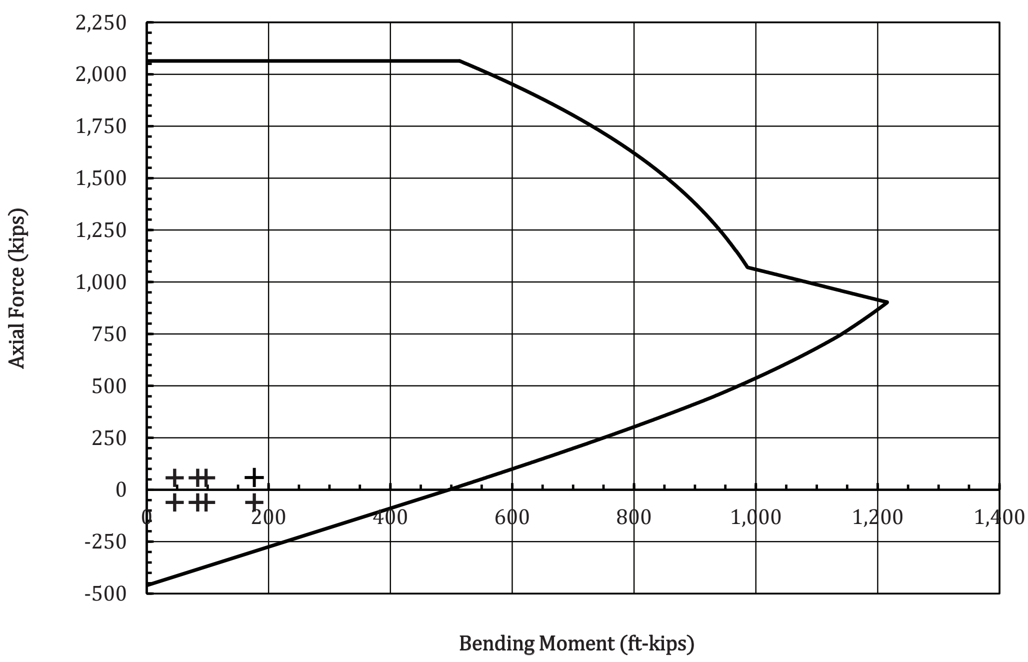

Figure 14.84 Design strength interaction diagram for the collector beams along column lines C and E in Example 14.13.

Calculate the average tensile stress in the reinforcing bars over the length of the collector and check the requirement in ACI 18.12.7.5 (note: this calculation is not necessary if the collector is designed using $f_y = 60{,}000$ psi; it is provided here for illustration purposes only):

$$\text{Average tensile stress} = \frac{59{,}300}{(10 \times 0.79) + (2 \times 0.31)} = 6{,}960 \text{ psi} < \phi f_y = 0.9 \times 60{,}000 = 54{,}000 \text{ psi}$$

Check if confinement reinforcement in accordance with ACI 18.12.7.6 must be provided.

$$\text{Maximum compressive stress} = \frac{59{,}300}{36.0 \times 28.5} = 57.8 \text{ psi} < 0.2 f_c' = 800 \text{ psi}$$

Thus, transverse reinforcement satisfying ACI 18.12.7.6 need not be provided. The limit of $0.2f_c'$ is used because the maximum axial force in the collector is based on ASCE/SEI Eq. (12.10-2) with the load combinations determined in accordance with ASCE/SEI 2.3.6 (see Step 10) and not on the overstrength factor, Ω_o.

The maximum shear force in the collector is equal to 23.5 kips when the beam is subjected to combined gravity and seismic effects (see Table 14.21). The design shear strength of the concrete, ϕV_c, is determined from ACI Table 22.5.5.1 assuming at least minimum shear reinforcement is provided in the section:

$$\phi V_c = \phi \left(2\lambda \sqrt{f_c'} + \frac{N_u}{6A_g} \right) b_w d$$

$$\frac{N_u}{6A_g} = \frac{59{,}300}{6 \times 36.0 \times 28.5} = 9.6 \text{ psi} < 0.05 f_c' = 200.0 \text{ psi}$$

Thus,

$$\phi V_c = 0.75 \times \left[\left(2 \times 1.0 \sqrt{4{,}000} \right) - 9.6 \right] \times 36.0 \times 26.0 / 1{,}000 = 82.1 \text{ kips}$$

The required spacing of #3 ties with one crosstie is equal to the following (the #3 crosstie is required to satisfy the maximum spacing of legs of shear reinforcement across the width of the member; see ACI Table 9.7.6.2.2):

$$s = \frac{\phi A_v f_{yt} d}{V_u - \phi V_c} = \frac{0.75 \times (3 \times 0.11) \times 60 \times 26.0}{23.5 - 82.1} < 0$$

$$\leq \frac{d}{2} = \frac{26.0}{2} = 13.0 \text{ in.}$$

$$\leq \frac{A_v f_{yt}}{0.75 \sqrt{f_c'} b_w} = 11.6 \text{ in.}$$

$$\leq \frac{A_v f_{yt}}{50 b_w} = 11.0 \text{ in.}$$

Provide #3 ties and crossties spaced at 11.0 in. on center over the entire length of the collector beams. Note that this transverse reinforcement satisfies the detailing requirement of ACI 18.12.7.7(b) at splices and anchorage zones:

$$\text{Provided } A_v = 0.33 \text{ in.}^2 \geq \begin{cases} 0.75 \sqrt{f_c'} b_w s / f_{yt} = 0.31 \text{ in.}^2 \\ \\ 50 b_w s / f_{yt} = 0.33 \text{ in.}^2 \end{cases}$$

Reinforcement details for the collector beams are given in Figure 14.85.