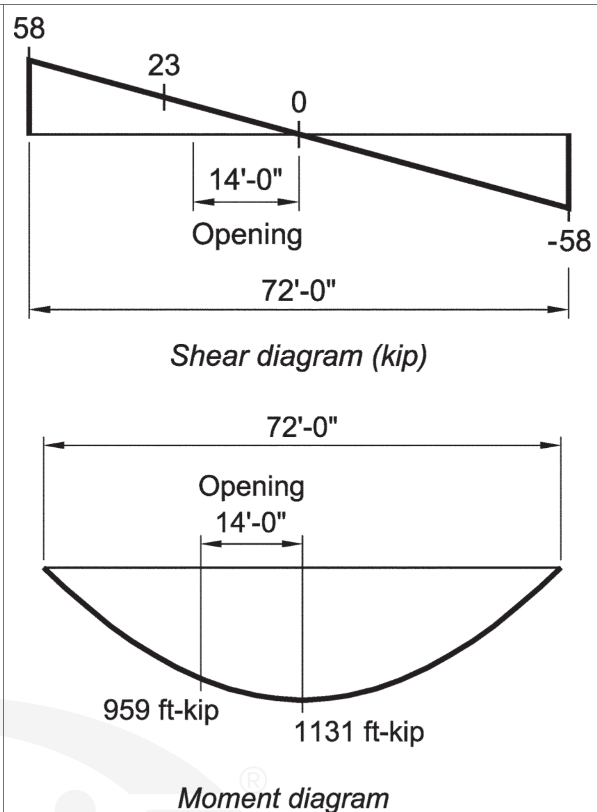
<table>
<tr><td></td><td>Maximum moment is taken at midspan.
Draw the shear and moment diagrams (Fig. E2.10).</td><td>

Fig. E2.10—Shear and moment diagrams.</td></tr>
<tr><td colspan="3">Step 12: Chord reinforcement E-W</td></tr>
<tr><td>12.5.2.3</td><td>Maximum moment is calculated above:

Chord reinforcement resisting tension must be located within $h/4$ of the tension edge of the diaphragm.

Assume tension reinforcement is placed in a 1 ft strip at both north and south sides of the slab edges at CLs 1 and 5.

<u>Chord force</u>
Maximum chord tension force that must be resisted by the chord at midspan:

$$T_u = \frac{M_u}{B - 1\text{ ft}}$$</td><td>M_u = 1131 ft-kip

$h/4$ = 218.0 ft/4 = 54.5 ft

1 ft < $h/4$ = 54.5 ft **OK**

$$T_{u,1} = \frac{1131\text{ ft-kip}}{(218\text{ ft} - 1\text{ ft})} = 5.2\text{ kip}$$</td></tr>
</table>

<u>Chord forces at opening</u>

The opening in the diaphragm results in local bending of the diaphragm segments on either side of the opening (Fig. E2.11).

1. The diaphragm sections to the east and west of the opening are idealized as fixed end beams.
2. The applied loading on the sections east and west the opening are based on the relative mass of each section (1:1).
3. The secondary chord forces are calculated based on the internal moment in the diaphragm sections adjacent to the opening.
4. The calculated tension and compression secondary chord forces are added to the primary tension and secondary chord forces.

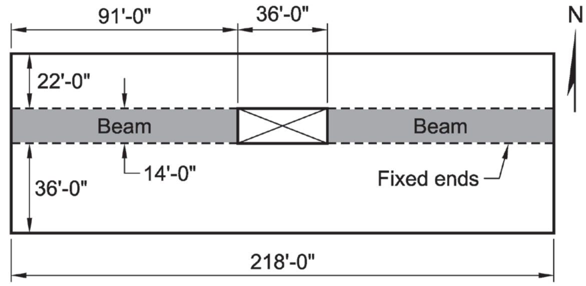


Fig. E2.11—Idealization of sections above and below opening.

<u>Force east and west of opening</u>

$$q'_{u@bE} = q'_{u@bW} = \left(\frac{1.6\text{ kip/ft}}{2}\right) = 0.8\text{ kip/ft}$$

The opening is located at mid-length of the building floor plan in the E-W direction. The load on the north and south sections of the diaphragm bound by the opening are equal to one half of the overall applied trapezoidal load over this portion of the diaphragm (Fig. E2.12).

Because forces at both ends of openings are close, a uniform load is assumed.

Fixed end moment can be obtained from computer-aided aided design software programs or from Reinforced Concrete Design Handbook Design Aid – Analysis Tables, which can be downloaded at: https://www.concrete.org/MNL1721Download1

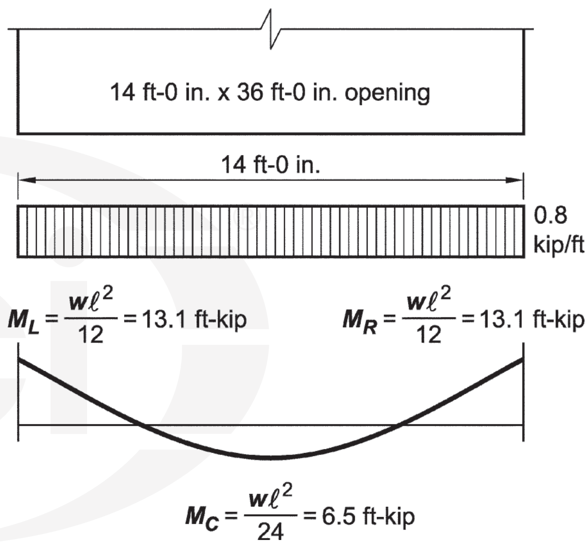


Fig. E2.12—Moment diagram of sections at opening.

The secondary chord forces are obtained from the moment diagram. Assuming a 2 ft strip (< B/4) at each end of the span between opening and diaphragm edge:

$T_{u,opening} = M_u/D$

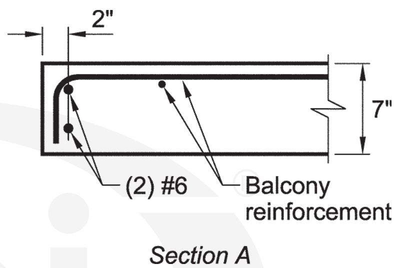
$$T^{+}_{u,op,S} = \frac{13.1\text{ ft-kip}}{89\text{ ft}} = 0.15\text{ kip}$$

Total moment to be resisted is the sum of the main chord force and the secondary chord force:

$T_{u,total} = T_{u1} + T_{u,O2}$

$T_{u,total,N} = C_{u,total,N} = 5.2\text{ kip} + 0.15\text{ kip} = 5.35\text{ kip}$
Use 5.4 kip

12.5.2.2	Chord reinforcement: Tension due to moment will be resisted by deformed bars conforming to Section 20.2.1 of ACI 318.	
12.5.1.5	Steel stress is the lesser of the specified yield strength and 60,000 psi.	
12.5.1.1 22.4.3.1	Required reinforcement $\phi T_n = \phi f_y A_s \geq T_u$	$A_s = \dfrac{5400\text{ lb}}{0.9(60{,}000\text{ psi})} = 0.1\text{ in.}^2$
	The chord forces north of and south of opening are equal.	One No.3 bar satisfies the requirement. The required collector reinforcement in the N-S direction, however, requires two No. 5 bars. Therefore, provided reinforcement is adequate and no additional reinforcement is required.
Step 13: Collector reinforcement E-W		
	Continuous reinforced concrete frame over the full length of the building will act 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.	
Step 14: Shrinkage and temperature reinforcement		
12.6.1 24.4.3.2	The minimum shrinkage and temperature Reinforcement, A_{S+T}: $A_{S+T} \geq 0.0018A_g$	$A_{S+T} = (0.0018)(7\text{ in.})(12\text{ in./ft}) = 0.15\text{ in.}^2$
24.4.3.3	Spacing of S+T reinforcement is the lesser of $5h$ and 18 in. $5h = 5(12\text{ in.}) = 60$ in. 18 in. **Controls**	Note: Shrinkage and temperature reinforcement may be part of the main 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 midspans and top reinforcing bars to resist negative moments at columns), continuity between top and bottom reinforcing bars may be achieved by providing adequate splice lengths between them.

Step 15: Reinforcement detailing		
12.7.2.1	<u>Reinforcement spacing</u> Chord and collector reinforcement minimum and maximum spacing must satisfy 12.7.2.1 and 12.7.2.2.	
25.2.1	Section 25.2 requires minimum spacing of (a) 1 in. (b) 4/3$d_{agg.}$ (c) d_b No. 5	 Minimum spacing 1.0 in. **Controls** 4/3(3/4 in.) aggregate = 1.0 in. 0.625 in.
18.12.7.7a	Collector reinforcement spacing at a splice must be at least the larger of: (a) At least three longitudinal d_b (b) 1.5 in. (c) $c_c \geq$ max [2.5d_b, 2 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**
	<u>Edge reinforcement</u> The opening has four beams around its perimeter. Therefore, the beams reinforcement is adequate to resist the tension forces due to inertial forces and additional reinforcement is not required. Note: If beams are not constructed around the opening perimeter a minimum of two No. 5 is recommended around the opening as shown in the Fig. E2.13 and extended a minimum of its development length. See detailing in Fig. E2.14 and Fig. E2.15.	 *Fig. E2.13—Two No. 5 reinforcement around opening.*

Step 16: Details

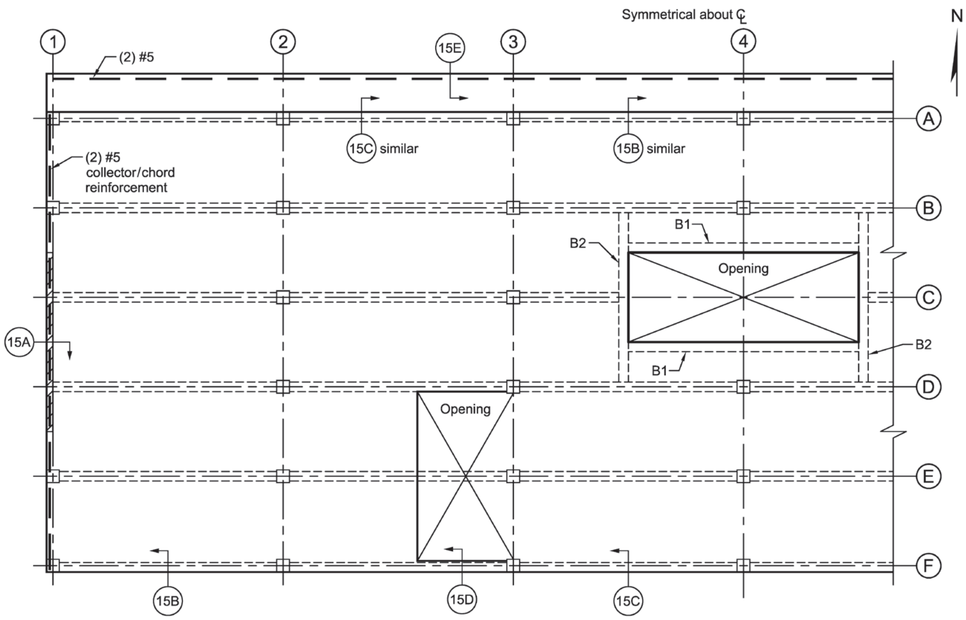


Fig. E2.14—Typical diaphragm to wall section. Note: Slab reinforcement not shown for clarity.

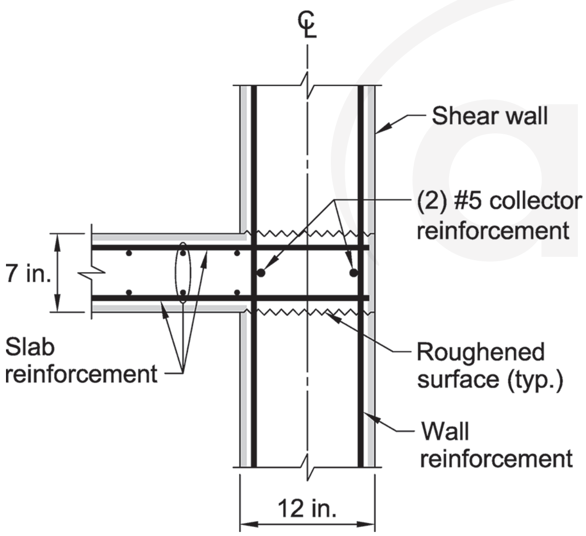


Fig. E2.15a—Collector reinforcement in shear walls along CL 1 and 7.

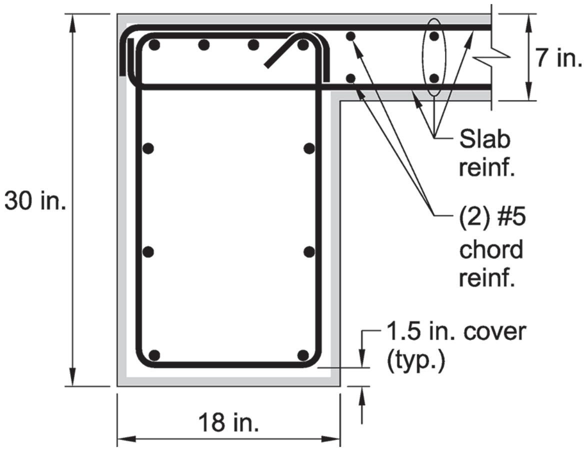


Fig. E2.15b—Chord reinforcement at midspan.

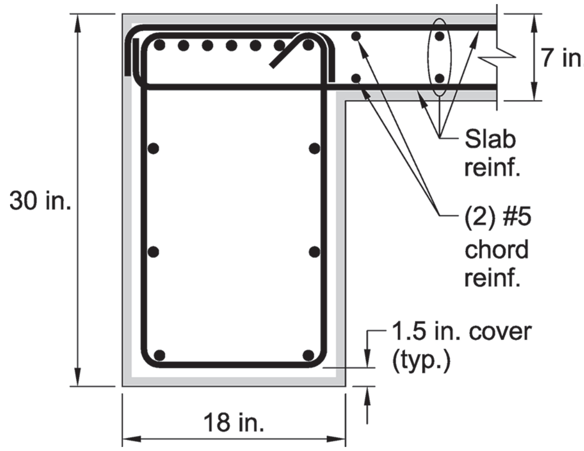


Fig. 2.15c—Chord reinforcement at supports.

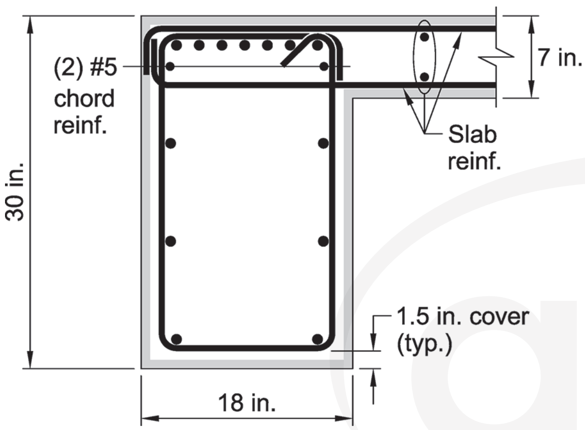


Fig. E2.15d—Chord reinforcement at opening.

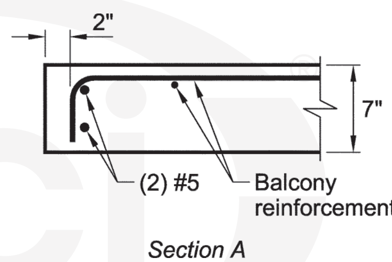


Fig. E2.15e—Crack control reinforcement at balcony edge.

Rigid Diaphragm Example 3: *Lateral force distribution of a rigid diaphragm to shear walls*—A three-story wood apartment building is built on a normalweight reinforced concrete one-story slab. The slab is 200 ft x 90 ft with f_c' = 4000 psi and f_y = 60,000 psi. Assume that the structure is located in an active earthquake region Seismic Design Category (SDC) D and that the seismic analysis of the structural analysis based on ASCE/SEI 7, resulting in a base shear coefficient of 0.316. The slab supporting the wood structure is 10 in. thick and the wall lengths, height, and thicknesses are shown as follows. Assume the weight of the wood frame building imparts an equivalent uniform dead load of 135 psf to the slab. In addition, add a 10 psf miscellaneous dead load to the slab. Refer to Fig. E3.1 for geometric information.

This example will determine the seismic forces that are resisted by the shear walls, design the diaphragm, chords, and collectors to resist these forces and transmit them to the walls, and then detail the flatwork accordingly.

Given:

Project data—
Diaphragm size 200 ft 0 in. x 90 ft 0 in.
Wall 1: 90 ft 0 in. x 8 in.
Wall 2: 30 ft 0 in. x 10 in.
Wall 3: 30 ft 0 in. x 10 in.
Wall 4: 28 ft 0 in. x 10 in.
Wall 5: 40 ft 0 in. x 10 in.
Slab thickness: t = 10 in.
Parking structure (top of slab) height is 12 ft above the foundation

Concrete—
f_c' = 4000 psi
f_y = 60,000 psi

Seismic criteria—
SDC D
C_S = 0.316

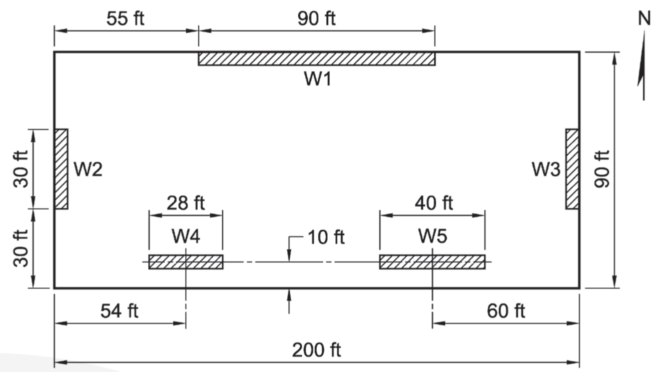


Fig. E3.1—Slab that supports a four-story wood building.

Note: Nonparticipating columns in the lateral-force-resisting system are not shown for clarity.

ACI 318	Discussion	Calculation
Step 1: Material requirements		
7.2.2.1	The mixture proportion must satisfy the durability requirements of Chapter 19 and structural strength requirements (ACI 318). The designer determines the durability classes. Please refer to Chapter 4 of this Manual for an in-depth discussion of the categories and classes. ACI 301 is a reference specification that is coordinated with ACI 318. ACI encourages referencing ACI 301 into job specifications. There are several mixture options within ACI 301, such as admixtures and pozzolans, which the designer can require, permit, or review if suggested by the contractor.	By specifying that the concrete mixture shall be in accordance with ACI 301 and providing the exposure classes, Chapter 19 requirements are satisfied. Based on durability and strength requirements, and experience with local mixtures, the compressive strength of concrete is specified at 28 days to be at least 4000 psi.
Step 2: Slab geometry		
12.3.1.1 18.13.7.1	Assume that diaphragm thickness satisfies the requirements for stability, strength, and stiffness under factored load combinations. Diaphragm thickness satisfies Chapter 18 minimum thickness requirements.	Given: h = 10 in.

Step 3: Lateral forces		
	The lateral force is obtained by multiplying the self-weight of the reinforced concrete slab, wood frame building dead load, miscellaneous dead load, and the contribution of the shear walls, by the base shear coefficient.	
	Gravity loads The reinforced concrete slab self-weight: $W_{slab} = (L)(B)(h)(\gamma_c)$	W_{slab}= (200 ft)(90 ft)(10 in./12 in/ft)(150 lb/ft^3) = 2,250,000 lb = 2250 kip
	Weight of wood frame building dead load and miscellaneous dead load:	W_D =(135 psf + 10 psf)(200 ft)(90 ft) =2610 kip
	Total gravity dead load:	W = 2250 kip + 2610 kip = 4860 kip
	Shear wall self-weight contribution to diaphragm lateral force calculation is half the wall height.	
	N-S direction $W_i = (L)(H/2)(t_w)(\gamma_c)$	W_1 =(90 ft)(12 ft/2)(8 in.)/(12 in./ft)(150 lb/ft^3) W_1 = 54,000 lb = 54 kip W_4 =(28 ft)(12 ft/2)(10in.)/(12in./ft)(150 lb/ft^3) W_4 = 21,000 lb = 21 kip W_5 =(40 ft)(12 ft/2)(10in.)/(12in./ft)(150 lb/ft^3) W_5 = 30,000 lb = 30 kip
	Total gravity dead load in the N-S direction:	$\sum W$ = 4860 kip + 54 kip + 21 kip + 30 kip = 4965 kip
	E-W direction $W_i = (L)(H/2)(t_w)(\gamma_c)$	W_2 =(30 ft)(12 ft/2)(10in.)/(12in./ft)(150 lb/ft^3) W_2 = 22,500 lb = 22.5 kip W_3 =(30 ft)(12 ft/2)(10in.)/(12in./ft)(150 lb/ft^3) W_3 = 22,500 lb = 22.5 kip
	Total gravity dead load in the E-W direction:	$\sum W$ = 4860 kip + 22.5 kip + 22.5 kip = 4905 kip
	Lateral loads Base shear is obtained from ASCE/SEI 7 Section 12.8.1: $V = C_sW$ C_s is calculated using ASCE/SEI 7 Section 12.8.1.1; not shown here for brevity:	C_s = 0.316 given
	The equivalent lateral force distribution over the building height is per ASCE/SEI 7 Eq. (12.8-11).	
	The diaphragm design forces F_{px} are calculated per ASCE/SEI 7 Eq. (12.10-1).	
	F_{px} and F_{py} must be in accordance with ASCE/SEI 7 Eq. (12.10-2) and (12.10-3). Calculations not shown here as it is outside the scope of this Manual. Equivalent lateral force at the concrete level is:	F_x = 363.1 kip

	Diaphragm design forces: N-S: E-W:	 F_{py} = 745 kip F_{px} = 726 kip

Note

Conservatively, the weight of all walls–parallel and perpendicular–to the direction of the analysis can be included. In this example, the contribution of wall weights parallel to the applied seismic force is considered in the calculation of diaphragm shears. Walls perpendicular to the applied seismic force are included in determining the lateral force of concrete diaphragms.

Step 4: Center of mass (COM)

Determine center of mass

Assume that the diaphragm is rigid.

Assume the (0,0) coordinate is located at the bottom left corner of the diaphragm. Center of mass of walls is shown in Table E.1:

Table E.1—Determining shear walls center of gravity

Wall no.	Weight, psf	Length, ft	Area, ft^2	Weight, kip	Direction	x_{cg}, ft	Wx_{cg}, ft-kip	y_{cg}, ft	Wy_{cg}, ft-kip
1 (8 in.)	100	90	540	54	x	100	5400	89.67	4842
2 (10 in.)	125	30	180	22.5	y	0.417	9.38	45	1012.5
3 (10 in.)	125	30	180	22.5	y	199.583	4490.6	45	1012.5
4 (10 in.)	125	28	168	21	x	54.0	1134	10	210
5 (10 in.)	125	40	240	30	x	140.0	4200	10	300
$\sum$				150			15,234		7377.2

The values of x_{cg} and y_{cg} are the center of mass of each wall. For example:

Wall 1 has the following coordinates: x_{cg} = 55 ft + 90 ft/2 = 100 ft and y = 90 ft – (8 in./12)/2 = 89.67 ft

Wall 2 has the following coordinates: x_{cg} = 0 ft + (10 in./12)/2 = 0.417 ft and y = 30 ft + 30 ft/2 = 45 ft

Center of mass of all walls:

$$x_1 = \frac{\sum W_i x_{cg,i}}{\sum W_i} = \frac{15{,}234\text{ ft-kip}}{150\text{ kip}} = 101.6\text{ ft}$$

$$y_1 = \frac{\sum W_i y_{cg,i}}{\sum W_i} = \frac{7377.2\text{ ft-kip}}{150\text{ kip}} = 49.2\text{ ft}$$

Center of mass of the slab is: x_2 = 200 ft/2 = 100 ft and y_2 = 90 ft/2 = 45 ft

Location of center of mass of the slab and walls combined:

$$x_m = \frac{\sum W_i x_i}{\sum W_i} = \frac{(4860\text{ kip})(100\text{ ft}) + (150\text{ kip})(101.6\text{ ft})}{4860\text{ kip} + 150\text{ kip}} = 100.05\text{ ft}$$

and

$$y_m = \frac{\sum W_i y_i}{\sum W_i} = \frac{(4860\text{ kip})(45\text{ ft}) + (150\text{ kip})(49.2\text{ ft})}{4860\text{ kip} + 150\text{ kip}} = 45.13\text{ ft}$$

where 4860 kip and 150 kip are the weight of the slab and walls, respectively.

Step 5: Center of rigidity (COR) and lateral system stiffness

Determine center of rigidity

From the lateral analysis, the diaphragm is assumed rigid and therefore, diaphragm flexibility is not considered. Therefore, lateral forces are distributed to shear walls in both directions in proportion to their relative stiffnesses. Lateral displacement is the sum of flexural and shear displacements.

Apply a lateral force of 1 kip is applied at the top of a cantilevered wall as shown in Fig. E3.2. The wall's lateral displacement under a unit load, which is related to its stiffness, is the sum of flexural and shear displacements:

$\Delta = \Delta_{Flexure} + \Delta_{Shear}$

$$\Delta = \frac{Ph^3}{3EI} + \frac{1.2Ph}{AG} \quad \text{where } G \cong 0.4E \text{ and } E = 3{,}605{,}000 \text{ psi}$$

$$\Delta_{Flexure} = \frac{Ph^3}{3EI} = \frac{Ph^3}{3E\dfrac{L^3t}{12}} = \frac{4P\left(\dfrac{h}{L}\right)^3}{Et}$$

$$\Delta_{Shear} = \frac{1.2Ph}{AG} = \frac{(1.2)Ph}{(Lt)0.4E} = \frac{3P\left(\dfrac{h}{L}\right)}{Et}$$

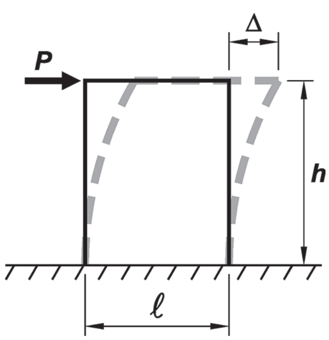


Fig. E3.2—Cantilever wall deflection.

Rigidity $k_i = 1/\Delta_i$ (refer to Table E.2)

Table E.2—Determining walls' relative stiffnesses

Wall no.	Height h, ft	Length L, ft	h/L	t, in.	$\Delta_i \times 10^{-4}$, in.	$k_i = 1/\Delta_i \times 10^4$, 1/in.
1	12	90	0.1333	8	0.14	7.043
2	12	30	0.4000	10	0.40	2.476
3	12	30	0.4000	10	0.40	2.476
4	12	28	0.4286	10	0.44	2.252
5	12	40	0.3000	10	0.28	3.576

Table E.3—Determining walls' rigidity

Wall no.	Direction	x, ft	y, ft	k_{ix}	k_{iy}	$(k_{iy})x$	$(k_{ix})y$
1	x	—	89.67	7.043	—	—	631.55
2	y	0.417	—	—	2.476	1.03	—
3	y	199.58	—	—	2.476	494.16	—
4	x	—	10.0	2.252	—	—	22.52
5	x	—	10.0	3.576	—	—	35.76
Σ				12.872	4.952	495.19	689.83