Problem 9.4 Solution

Given: The single-story wood-frame warehouse in Fig. 9.B. Roof sheathing of 3/8-in thick 24/0 span-rated STR I sheathing is adequate for vertical "sheathing" loads. Strength level seismic forces to the roof diaphragm are:

$$w_u = w_T = 330 \text{ lb/ft}$$
$$w_u = w_L = 570 \text{ lb/ft}$$

Calculation of unit shears requires consideration of the IBC load combinations. The applicable load combinations from IBC Sec. 1605.3.1 are:

D + H + F + (W or 0.7E)
D + H + F + 0.75(W or 0.7E) + 0.75 L + 0.75 (L_r or S or R)
0.6D + (W or 0.7E) + H

Because the diaphragm is assumed to only resist horizontal forces, the first and third equations can be simplified to just "0.7E." The redundancy / reliability factor will be assumed to be 1.0.

a. Design the roof diaphragm, considering lateral forces in both directions. Show all criteria including design shears, load cases, allowable shears, and nailing requirements. If blocking is required, determine at what points it can be omitted.

Transverse Direction Unit Shears

w_T = 330 plf
R = V = 330 plf (120 ft /2) = 19,800 lb
b = 70 ft
v = V/b= 19,800 lb / 70 ft = 283 plf strength
= 198 plf ASD

Longitudinal Direction Unit Shears

w_L = 570 plf
R = V = 570 plf (70 ft /2) = 19950 lb
b = 120 ft
v = V/b= 19950 lb / 120 ft = 166 plf strength
= 116 plf ASD

Diaphragm Aspect Ratio

Per IBC Table 2305.2.3, for unblocked wood structural panel diaphragms the maximum allowable span to width ratio is 3:1, while for blocked diaphragms it is

4:1. The actual maximum span-to-width ratio for this diaphragm is 120 ft/ 70 ft = 1.7, which is less than either maximum allowable, so OK.

Actual versus Allowable Shears

Transverse Loading. From IBC Table 2306.3.1, the panel layout and load direction for transverse loading correspond to Case 1. For 3/8 inch Structural I sheathing, 2x framing members, and an unblocked diaphragm, the allowable Case 1 shear using 8d common nails is 240 plf, which is greater than the 198 plf ASD demand. Transverse loading does not require blocking.

Longitudinal Loading. From IBC Table 2306.3.1, the panel layout and load direction for transverse loading correspond to Case 3. For 3/8 inch Structural I sheathing, 2x framing members, and an unblocked diaphragm, the allowable Case 3 shear is 180 plf, which is greater than the 116 plf ASD demand. Blocking is not required for longitudinal loading.

Diaphragm nailing requirements

8 common nails – unblocked diaphragm
8d @ 6" oc supported edges
8d @ 12" oc other supports (field nailing)

b. Calculate the maximum chord forces for both lateral force directions. Also, calculate the chord forces at the ends of all headers in the exterior walls.

Transverse Loading

Mid-Span

$$M_u = \frac{w_T L^2}{8} = \frac{330(120)^2}{8} = 594{,}000 lb - ft = 594 kip - ft$$

$$T_u = \frac{M_u}{b} = \frac{594 kip - ft}{70 ft} = 8.49 kips$$

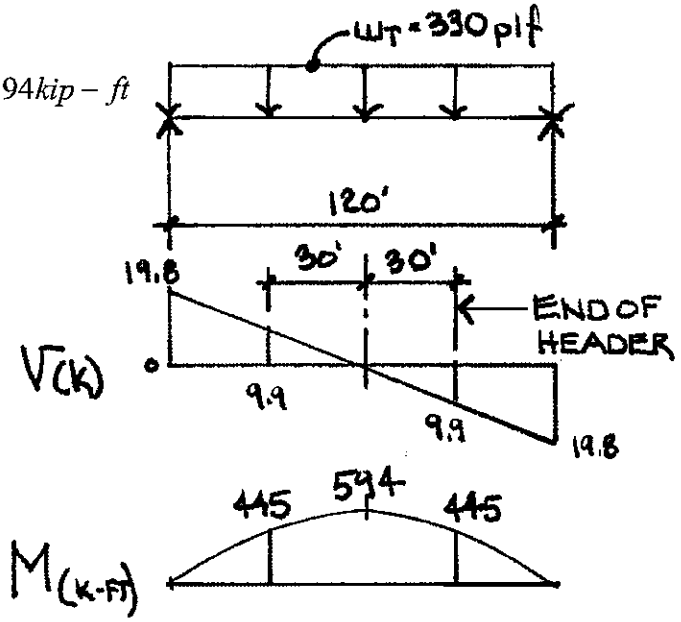
$$T = 0.7(T_u) = 0.7(8.49) = 5.94 kips$$

End of Header

$$M_u = \frac{(19.8 + 9.9k)(30 ft)}{2} = 445 kip - ft$$

$$T_u = \frac{M_u}{b} = \frac{445 kip - ft}{70 ft} = 6.36 kips$$

$$T = 0.7(T_u) = 0.7(6.36) = 4.45 kips$$



Longitudinal Loading

West Wall
Mid-Span

$$M_u = \frac{w_L L^2}{8} = \frac{570(70)^2}{8} = 349{,}000 lb - ft = 349 kip - ft$$

$$T_u = \frac{M_u}{b} = \frac{349 kip - ft}{120 ft} = 2.91 kips$$

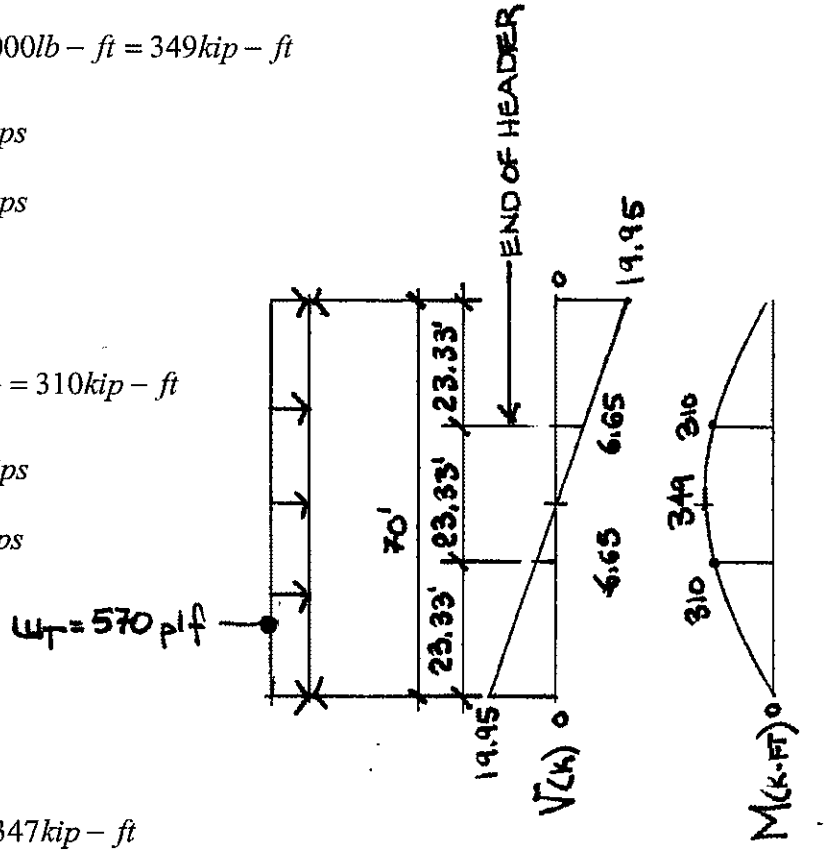
$$T = 0.7(T_u) = 0.7(2.91) = 2.04 kips$$

End of Header

$$M_u = \frac{(19.95 + 6.65k)(23.33 ft)}{2} = 310 kip - ft$$

$$T_u = \frac{M_u}{b} = \frac{310 kip - ft}{120 ft} = 2.58 kips$$

$$T = 0.7(T_u) = 0.7(2.58) = 1.81 kips$$



East Wall
End of Header 1

$$M_u = \frac{(19.95 + 1.14k)(33 ft)}{2} = 347 kip - ft$$

$$T_u = \frac{M_u}{b} = \frac{347 kip - ft}{120 ft} = 2.89 kips$$

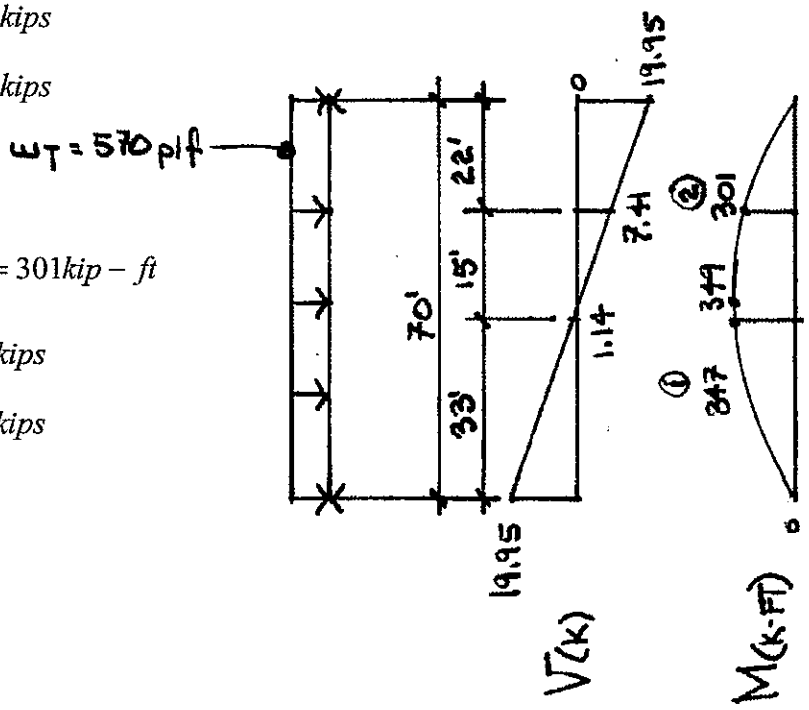
$$T = 0.7(T_u) = 0.7(2.89) = 2.02 kips$$

End of Header 2

$$M_u = \frac{(19.95 + 7.41k)(22 ft)}{2} = 301 kip - ft$$

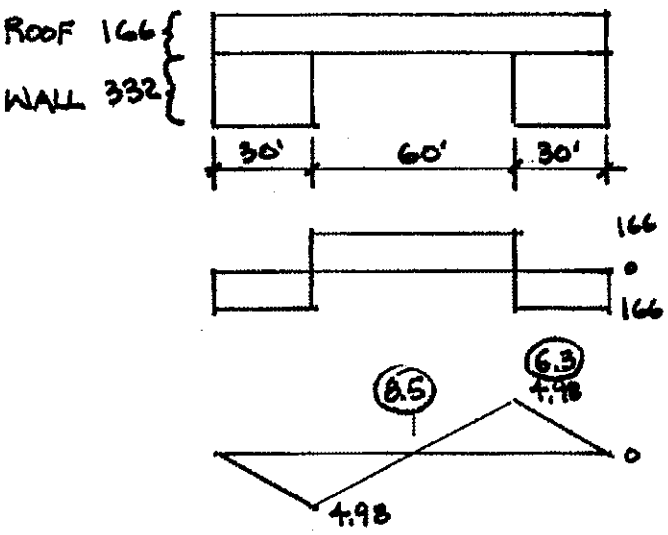
$$T_u = \frac{M_u}{b} = \frac{301 kip - ft}{120 ft} = 2.51 kips$$

$$T = 0.7(T_u) = 0.7(2.51) = 1.76 kips$$



c. Plot the distribution of the collector (strut) force for each of the walls with openings. Compare the collector (strut) forces and chord forces to determine the critical loading.

SOUTH WALL



WALL UNIT SHEAR = $\frac{19950\#}{60'} = 332$ plf

ROOF UNIT SHEAR = $\frac{19950\#}{120'} = 166$ plf

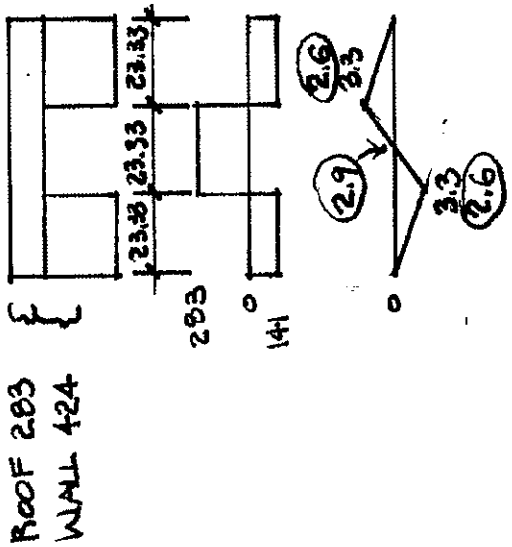
UNIT SHEARS (PLF)

NET UNIT SHEARS (PLF)

STRENGTH LEVEL COLLECTOR FORCE (k)
(CHORD FORCES SHOWN IN CIRCLES FOR COMPARISON)

SOUTH WALL MAX FORCE = 8.5k CHORD NORTH WALL SAME

WEST WALL

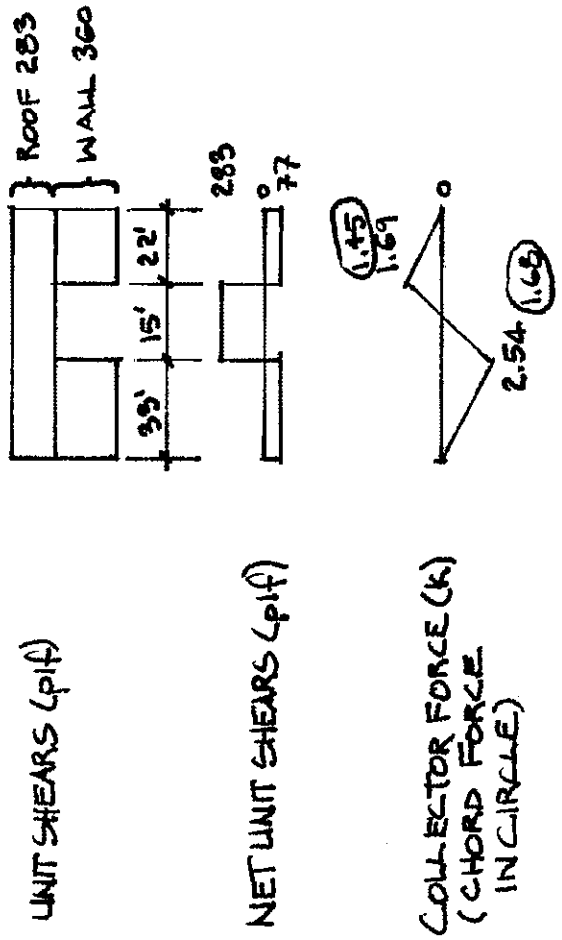


WALL UNIT SHEAR = $\frac{19800}{46.7} = 424$ plf

ROOF UNIT SHEAR = $\frac{19800}{70} = 283$ plf

WEST WALL MAX FORCE = 3.3k CHORD

EAST WALL



$$\text{WALL UNIT SHEAR} = \frac{19800}{55'} = 360 \text{ plf}$$

$$\text{ROOF UNIT SHEAR} = \frac{19800}{70'} = 283 \text{ plf}$$

EAST WALL MAX FORCE = 2.54 k STR CHORD

Problem 9.10 Solution

Given: The one-story building in Fig. 9.D. Roof sheathing is 15/32-in. 32/16 span-rated panels spanning between roof joists spaced 24 in oc. Nails are 10d common. For the transverse strength level seismic force, the panel layout is load case 1.

a. Design the roof diaphragm assuming that only the exterior walls are effective in resisting the lateral force. Show all design criteria including span-width ratio, nailing, and blocking requirements. Omit blocking where possible. Do not consider deflection of the diaphragm.

Calculation of critical lateral forces requires consideration of the IBC load combinations. The applicable load combinations from IBC Sec. 1605.3.1 are:

D + H + F + (W or 0.7E)
D + H + F + 0.75(W or 0.7E) + 0.75L + 0.75(L_r or S or R)
0.6D + (W or 0.7E) + H

Because the diaphragm is assumed to only resist horizontal forces, the first and third equations can be simplified to just "0.7E." The redundancy / reliability factor will be assumed to be 1.0.

$w_T = w_L$ = 335 plf Strength
= 0.7(335) = 235 plf ASD

Transverse Loading

w_T = 235 plf
R = V = 235 plf (110 ft /2) = 12,920 lb
b = 50 ft
v = V/b= 12,920 lb / 50 ft = 258 plf

Span to Width ratio = 110 ft / 50 ft = 2.2. This is less than the 3.0 permitted by IBC Table 2305.2.3.

From IBC Table 2306.3.1, for 15/32 inch rated sheathing, unblocked with 10d nails, the allowable Case 1 shear is 255 plf. Because this is approximately equal to the 258 plf demand, blocking will not be required.

b. Redesign the roof diaphragm, assuming that the interior and exterior walls are effective shearwalls. Show all criteria except deflection.

Transverse Loading

Use tributary area method, design diaphragm for longer 80 ft span.

w_T = 235 plf
R = V = 235 plf (80 ft /2) = 9400 lb
b = 50 ft
v = V/b= 9400 lb / 50 ft = 188 plf

Span to Width ratio = 80 ft / 50 ft = 1.6. This is less than the 3.0 permitted by IBC Table 2305.2.3.

From IBC Table 2306.3.1, for 15/32 inch rated sheathing, unblocked with 10d nails, the allowable Case 1 shear is 255 plf. Because this is less than the 188 plf demand, blocking will not be required.

c. Compare the maximum chord forces for a and b.

Chord - Transverse Loading, 110 ft span

$$M = \frac{w_T L^2}{8} = \frac{235(110)^2}{8} = 355{,}000 lb - ft = 355 kip - ft$$

$$T = \frac{M}{b} = \frac{355 kip - ft}{50 ft} = 7.1 kips \text{ ASD}$$

Chord - Transverse Loading, 80 ft span

$$M = \frac{w_T L^2}{8} = \frac{235(80)^2}{8} = 188{,}000 lb - ft = 188 kip - ft$$

$$T = \frac{M}{b} = \frac{188 kip - ft}{50 ft} = 3.8 kips \text{ ASD}$$

d. Plot the collector force diagram for the interior shearwall.

ROOF SHEAR $v_1 = \frac{235\ plf\ (30/2\ ft)}{50} = 71\ plf$

$v_2 = 188\ plf$

WALL SHEAR $v = \frac{235\ plf\ (110/2\ ft)}{30} = 431\ plf$