FUNDAMENTALS OF STRUCTURAL ANALYSIS

5th Edition

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SOLUTIONS MANUAL

CHAPTER 2: DESIGN LOADS AND STRUCTURAL FRAMING
P2.1. Determine the deadweight of a 1-ft-long segment of the prestressed, reinforced concrete tee-beam whose cross section is shown in Figure P2.1. Beam is constructed with lightweight concrete which weighs 120 lbs/ft³.

Compute the weight/ft. of cross section @ 120 lb/ft³.

Compute cross sectional area:

\[
\text{Area} = (0.5' \times 6') + 2 \left( \frac{1}{2} \times 0.5' \times 2.67' \right) + (0.67' \times 2.5') + (1.5' \times 1')
\]

\[
= 7.5 \text{ ft}^2
\]

Weight of member per foot length:

\[
\text{wt/ft} = 7.5 \text{ ft}^2 \times 120 \text{ lb/ft}^3 = 900 \text{ lb/ft}
\]
P2.2. Determine the deadweight of a 1-ft-long segment of a typical 20-in-wide unit of a roof supported on a nominal 2 × 16 in. southern pine beam (the actual dimensions are \( \frac{1}{2} \) in. smaller). The \( \frac{3}{4} \)-in. plywood weighs 3 lb/ft². 

See Table 2.1 for weights

\[
\text{wt/20'' unit} \\
\text{Plywood: } 3 \text{ psf} \times \frac{20''}{12} \times 1' = 5 \text{ lb} \\
\text{Insulation: } 3 \text{ psf} \times \frac{20''}{12} \times 1' = 5 \text{ lb} \\
\text{Roof'g Tar & G: } 5.5 \text{ psf} \times \frac{20''}{12} \times 1' = 9.17 \text{ lb} \\
\text{Wood Joist } = \frac{37 \text{ lb}}{\text{ft}^2} \left( \frac{1.5'' \times 15.5''}{14.4 \text{ in}^2/\text{ft}^2} \times 1' \right) = 5.97 \text{ lb} \\
\text{Total wt of 20'' unit } = 19.17 + 5.97 \\
= 25.14 \text{ lb. Ans.}
\]
Uniform Dead Load $W_{dl}$ Acting on the Wide Flange Beam:

Wall Load:

$$9.5'(0.09 \text{ ksf}) = 0.855 \text{ klf}$$

Floor Slab:

$$10'(0.05 \text{ ksf}) = 0.50 \text{ klf}$$

Steel Frm, Fireproof’g, Arch’l Features, Floor Finishes, & Ceiling:

$$10'(0.024 \text{ ksf}) = 0.24 \text{ klf}$$

Mech’l, Piping & Electrical Systems:

$$10'(0.006 \text{ ksf}) = 0.06 \text{ klf}$$

Total $W_{dl} = 1.66 \text{ klf}$
P2.4. Consider the floor plan shown in Figure P2.4. Compute the tributary areas for (a) floor beam B1, (b) floor beam B2, (c) girder G1, (d) girder G2, (e) corner column C1, and (f) interior column C.

(a) Method 1: \( A_r = \left( \frac{8 + 8}{2} \right) (40) = 320 \text{ ft}^2 \)
Method 2: \( A_r = 320 - 4 \left( \frac{1}{2} \right) (4(4)) = 288 \text{ ft}^2 \)

(b) Method 1: \( A_r = \left( \frac{6.67}{2} \right) (20) = 66.7 \text{ ft}^2 \)
Method 2: \( A_r = 66.7 - 2 \left( \frac{1}{2} \right) (3.33(3.33)) = 55.6 \text{ ft}^2 \)

(c) Method 1: \( A_r = \left( \frac{6.67}{2} \right) (20) + 10(10) \)
\( A_r = 166.7 \text{ ft}^2 \)
Method 2: \( A_r = 166.7 - 2 \left( \frac{1}{2} \right) (3.33(3.33)) + 2 \left( \frac{1}{2} \right) (5(5)) \)
\( A_r = 180.6 \text{ ft}^2 \)

(d) Method 1: \( A_r = \left( \frac{40}{2} + \frac{20}{2} \right) (36) \)
\( A_r = 1080 \text{ ft}^2 \)
Method 2: \( A_r = 1080 + 2 \left( \frac{1}{2} \right) (4(4)) \)
\( A_r = 1096 \text{ ft}^2 \)

(e) \( A_r = \frac{20}{2} (\frac{20}{2}); \quad A_r = 200 \text{ ft}^2 \)

(f) \( A_r = \left( \frac{40}{2} + \frac{20}{2} \right) \left( \frac{40}{2} + \frac{20}{2} \right); \quad A_r = 900 \text{ ft}^2 \)
P2.5. Refer to Figure P2.4 for the floor plan. Calculate the tributary areas for (a) floor beam B3, (b) floor beam B4, (c) girder G3, (d) girder G4, (e) edge column C3, and (f) corner column C4.

(a) Method 1: $A_y = (10)(20)$

\[ A_y = 200 \text{ ft}^2 \]

Method 2: $A_y = 200 - 4 \left( \frac{1}{2} \times 5 \right)$

\[ A_y = 150 \text{ ft}^2 \]

(b) Method 1: $A_y = (6.67)(20) = A_y = 133.4 \text{ ft}^2$

Method 2: $A_y = 133.4 - 4 \left( \frac{1}{2} \times 3.33 \right)$

\[ A_y = 111.2 \text{ ft}^2 \]

(c) Method 1: $A_y = (36)(20) = A_y = 720 \text{ ft}^2$

Method 2: $A_y = 720 + 2 \left( \frac{1}{2} \times 4 \right) = A_y = 736 \text{ ft}^2$

(d) Method 1: $A_y = (4)(40) + 33.33(10)$

\[ A_y = 493.4 \text{ ft}^2 \]

Method 2: $A_y = 493.4 - 2 \left( \frac{1}{2} \times 4 \right) + 2 \left( \frac{1}{2} \times 3.33 \right)$

\[ A_y = 488.5 \text{ ft}^2 \]

(e) $A_y = (30)(20); \quad A_y = 600 \text{ ft}^2$

(f) $A_y = (10)(10); \quad A_y = 100 \text{ ft}^2$
The uniformly distributed live load on the floor plan in Figure P2.4 is 60 lb/ft². Establish the loading for members (a) floor beam B1, (b) floor beam B2, (c) girder G1, and (d) girder G2. Consider the live load reduction if permitted by the ASCE standard.

(a) \( A_r = 8(40) = 320 \text{ ft}^2 \), \( K_{L2} = 2 \), \( A_r K_{L2} = 640 > 400 \)

\[
L = 60 \left( 0.25 + \frac{15}{\sqrt{640}} \right) = 50.6 \text{ psf}, \text{ ok}
\]

\[ w = 8(50.6) = 404.8 \text{ lb/ft} = 0.40 \text{ kips/ft} \]

(b) \( A_r = \frac{6.67}{2} (20) = 66.7 \text{ ft}^2 \), \( K_{L2} = 2 \), \( A_r K_{L2} = 133.4 < 400 \), No Reduction

\[ w = \frac{6.67}{2} (60) = 200.1 \text{ lb/ft} = 0.20 \text{ kips/ft} \]

(c) \( A_r = \frac{6.67}{2} (20) + 10(10) = 166.7 \text{ ft}^2 \), \( K_{L2} = 2 \), \( A_r K_{L2} = 333.4 < 400 \), No Reduction

\[ w = \frac{6.67}{2} (60) = 200.1 \text{ lb/ft} = 0.20 \text{ kips/ft} \]

\[ P = q(W_{mb})(L_{beam}) = \frac{60(10)(20)}{2} = 6000 \text{ lbs} = 6 \text{ kips} \]

(d) \( A_r = \left( \frac{40}{2} + \frac{20}{2} \right) = 36 \text{ ft}^2 \), \( K_{L2} = 2 \), \( A_r K_{L2} = 2160 > 400 \)

\[
L = 60 \left( 0.25 + \frac{15}{\sqrt{2160}} \right) = 34.4 > \frac{60}{2}, \text{ ok}
\]

\[ L = 34.4 \text{ psf} \]

\[ P = 8(34.4) \left( \frac{40}{2} + \frac{20}{2} \right) = 8256 \text{ lbs} = 8.26 \text{ kips} \]
**P2.7.** The uniformly distributed live load on the floor plan in Figure P2.4 is 60 lb/ft². Establish the loading for members (a) floor beam B3, (b) floor beam B4, (c) girder G3, and girder G4. Consider the live load reduction if permitted by the ASCE standard.

(a) $A_y = 10(20) = 200 \text{ ft}^2$, $K_{ll} = 2$, $A_y K_{ll} = 400 > 400$

$L = 60 \left(0.25 + \frac{15}{400}\right) = 60 \text{ psf}$

$w = 10(60) = 600 \text{ lb/ft} = 0.60 \text{ kips/ft}$

(b) $A_y = 6.67(20) = 133.4 \text{ ft}^2$, $K_{ll} = 2$, $A_y K_{ll} = 266.8 < 400$, No Reduction

$w = 6.67(60) = 400.2 \text{ lb/ft} = 0.40 \text{ kips/ft}$

(c) $A_y = 36(20) = 720 \text{ ft}^2$, $K_{ll} = 2$, $A_y K_{ll} = 1440 > 400$

$L = 60 \left(0.25 + \frac{15}{1440}\right) = 38.7 \text{ psf} > \frac{60}{2} = 30 \text{ psf}$, ok

$P = \frac{q(W_{y2})(L_{max})}{2} = \frac{38.7(8)(40)}{2} = 6192 \text{ lbs} = 6.19 \text{ kips}$

(d) $A_y = \frac{8}{2} = 40 + 33.33(10) = 493.3 \text{ ft}^2$, $K_{ll} = 2$, $A_y K_{ll} = 986.6 > 400$

$L = 60 \left(0.25 + \frac{15}{986.6}\right) = 43.7 > \frac{60}{2} = 30 \text{ psf}$, ok

$w = 43.7(4) = 174.8 \text{ lb/ft} = 0.17 \text{ kips/ft}$

$P = \frac{43.7(6.67(20))}{2} = 2914.8 \text{ lbs} = 2.91 \text{ kips}$
P2.8. The building section associated with the floor plan in Figure P2.4 is shown in Figure P2.8. Assume a live load of 60 lb/ft² on all three floors. Calculate the axial forces produced by the live load in column C1 in the third and first stories. Consider any live load reduction if permitted by the ASCE standard.

(a) \[ A_y = \left( \frac{40}{2} + \frac{20}{2} \right) \left( \frac{40}{2} + \frac{20}{2} \right) = 900 \text{ ft}^2 \]

\[ K_y = 4, A_y K_y = 3600 > 400 \]

\[ L = 60 \left( 0.25 + \frac{15}{\sqrt{3600}} \right) = 30 \text{ psf} = \frac{60}{2}, \text{ ok (minimum permitted)} \]

\[ P_{36} = 900(30) = 27000 \text{ lbs} = 27 \text{ kips} \]

\[ P_{1m} = (3)900(30) = 27000 \text{ lbs} = 81 \text{ kips} \]
P2.9. The building section associated with the floor plan in Figure P2.4 is shown in Figure P2.8. Assume a live load of 60 lb/ft² on all three floors. Calculate the axial forces produced by the live load in column C3 in the third and first stories. Consider any live load reduction if permitted by the ASCE standard.

(a) $A_y = \left( \frac{40}{2} + \frac{20}{2} \right) = 600 \text{ ft}^2$, $K_{wL} = 4, A_y K_{wL} = 2400 > 400$

$L = 60 \left( 0.25 + \frac{15}{\sqrt{2400}} \right) = 33.4 \text{ psf} = \frac{60}{2}, \text{ ok}$

$P_{tot} = 600(33.4) = 20040 \text{ lbs} = 20.0 \text{ kips}$

$P_{tot} = (3)600(33.4) = 60120 \text{ lbs} = 60.1 \text{ kips}$
P2.10. A five-story building is shown in Figure P2.10. Following the ASCE standard, the wind pressure along the height on the windward side has been established as shown in Figure P2.10(c). (a) Considering the windward pressure in the east-west direction, use the tributary area concept to compute the resultant wind force at each floor level. (b) Compute the horizontal base shear and the overturning moment of the building.

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**a) Resulant Wind Forces**

- Roof: 20 psf \((6 \times 90)\) = 10,800 lb
- 5th floor: 20 psf \((12 \times 90)\) = 21,600 lb
- 4th floor: 20 psf \((2 \times 90) + 15 (10 \times 90)\) = 17,100 lb
- 3rd floor: 15 psf \((10 \times 90) + 13 (2 \times 96)\) = 15,800 lb
- 2nd floor: 13 psf \((12 \times 90)\) = 14,040 lb

**b) Horizontal Base Shear**

\[ V_{\text{BASE}} = \Sigma \text{Forces at Each Level} = 10.8^t + 21.6^t + 17.1^t + 15.8^t + 14.04^t = \]

\[ V_{\text{BASE}} = 79.34^t \]

**Overturning Moment** of the Building =

\[ \Sigma (\text{Force} @ \text{Ea. Level} \times \text{Height above Base}) = 10.8^t (60^t) + 21.6 (48^t) + 17.1 (36^t) + 15.8^t (24^t) + 14.04^t (12^t) = \]

\[ M \text{ overturning} = 2,848^\text{in} \]
P2.11. A mechanical support framing system is shown in Figure P2.11. The framing consists of steel floor grating over steel beams and entirely supported by four tension hangers that are connected to floor framing above it. It supports light machinery with an operating weight of 4000 lbs, centrally located. (a) Determine the impact factor $I$ from the Live Load Impact Factor, Table 2.3. (b) Calculate the total live load acting on one hanger due to the machinery and uniform live load of 40 psf around the machine. (c) Calculate the total dead load acting on one hanger. The floor framing dead load is 25 psf. Ignore the weight of the hangers. Lateral bracing is located on all four edges of the mechanical floor framing for stability and transfer of lateral loads.

a) **Live Load Impact Factor** = 20%

b) **Total LL**

   Machinery = 1.20 (4 kips) = 4.8 kips

   Uniform LL = $((10' \times 16') - (5' \times 10')) (0.04 \text{ ksf}) = 4.4$ kips

   $\therefore \text{Total LL} = 9.2$ kips

   $\therefore \text{Total LL Acting on One Hanger} = 9.2 / 4 \text{ Hangers} = 2.3 \text{ kips}$

c) **Total DL**

   Floor Framing = $10' \times 16' (0.025 \text{ ksf}) = 4$ kips

   $\therefore \text{Total DL Acting on one Hanger} = 4 / 4 \text{ Hangers} = 1 \text{ kip}$

   $\therefore \text{Total DL + LL on One Hanger} = 2.3 + 1 = 3.3 \text{ kips}$
**P2.12.** The dimensions of a 9-m-high warehouse are shown in Figure P2.12. The windward and leeward wind pressure profiles in the long direction of the warehouse are also shown. Establish the wind forces based on the following information: basic wind speed = 40 m/s, wind exposure category = C, \( K_d = 0.85 \), \( K_z = 1.0 \), \( G = 0.85 \), and \( C_p = 0.8 \) for windward wall and –0.2 for leeward wall. Use the \( K_z \) values listed in Table 2.4. What is the total wind force acting in the long direction of the warehouse?

Use \( I = 1 \)

\[
q_z = 0.613 V^2 \quad \text{(Eq. 2.4b)}
\]

\[
q_z = 0.613(40)^2 = 980.8 \text{ N/m}^2
\]

\[
q_z = q_z I K_d K_z
\]

\[
q_z = 980.8(1)(0.85) = 833.7 \text{ N/m}^2
\]

\[
0 - 4.6 \text{ m: } q_z = 833.7(0.85) = 708.6 \text{ N/m}^2
\]

\[
4.6 - 6.1 \text{ m: } q_z = 833.7(0.90) = 750.3 \text{ N/m}^2
\]

\[
6.1 - 7.6 \text{ m: } q_z = 833.7(0.94) = 783.7 \text{ N/m}^2
\]

\[
7.6 - 9 \text{ m: } q_z = 833.7(0.98) = 817.1 \text{ N/m}^2
\]

For the Windward Wall

\[
p = q_z (0.85)(-0.2)
\]

\[
p = q_z (0.85)(-0.2) = 138.9 \text{ N/m}^2
\]

For the Windward Wall

\[
F_w = 481.8[4.6 \times 20] + 510.2[1.5 \times 20]
\]

\[
+ 532.9[1.5 \times 20] + 555.6[1.4 \times 20]
\]

\[
F_w = 91,180 \text{ N}
\]

For Leeward Wall

\[
p = q_z (0.85)(-0.2)
\]

\[
q_z = q_z \text{ at } 9 \text{ m} = 817.1 \text{ N/m}^2 \quad \text{(above)}
\]

\[
p = 817.1(0.85)(-0.2) = -138.9 \text{ N/m}^2
\]

Total Windforce, \( F_L \), on Leeward Wall

\[
F_L = (20 \times 9)(-138.9) = -25,003 \text{ N}
\]

Total Force

\[
F = F_w + F_L
\]

\[
= 91,180 \text{ N} + 25,003
\]

\[
= 116,183.3 \text{ N}
\]

*Both \( F \) and \( F_w \) Act in Same Direction.*
**Table P2.13 Roof Pressure Coefficient $C_p^*$**

<table>
<thead>
<tr>
<th>Angle $\theta$</th>
<th>Windward</th>
<th>Leeward</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$10$</td>
<td>$15$</td>
</tr>
<tr>
<td>$C_p$</td>
<td>$-0.9$</td>
<td>$-0.7$</td>
</tr>
<tr>
<td></td>
<td>$0.0$</td>
<td>$0.2$</td>
</tr>
</tbody>
</table>

Consider Positive Windward Pressure on Roof, i.e. left side. Interpolate in Table P2.10

$$C_p = 0.2 + \left( \frac{33.69 - 30}{35 - 30} \right) \times 0.1$$

$$C_p = 0.2738 \text{(Roof only)}$$

From Table 2.4 (see p48 of text)

$$K_z = 0.57, 0 - 15'$$
$$= 0.62, 15' - 20'$$
$$= 0.66, 20' - 25'$$
$$= 0.70, 25' - 30'$$
$$= 0.76, 30' - 32'$$

Mean Roof Height, $h = 24$ ft

$$\theta = \tan^{-1} \left( \frac{16'}{24'} \right) = 33.69^\circ \text{ (for Table 2.10)}$$
P2.13. Continued

\[ K_o = 1.0, \quad K_d = 0.85, \quad I = 1 \]
\[ q_e = 0.00256 V^2 \quad \text{(Eq. 2.4a)} \]
\[ q_e = 0.00256(100)^2 = 25.6 \text{ lb/ft}^2 \]

0–15': \[ q_e = 25.6 \times (1)(0.57)(1)(0.85) = 12.40 \text{ lb/ft}^2 \]
15–16': \[ q_e = 13.49 \text{ lb/ft}^2 \]
\[ h = 24; \quad q_e = 14.36 \text{ lb/ft}^2 \]

Wind Pressure on Windward Wall & Roof

\[ P = q_e \cdot G \cdot C_r \]

Wall 0–15' \[ P = 12.40 \times 0.85 \times 0.80 \]
\[ P = 8.43 \text{ psf} \]

Wall, 15'–16' \[ P = 13.49 \times 0.85 \times 0.8 = 9.17 \text{ psf} \]

Roof, \[ P = 14.36 \times 0.85 \times 0.2738 \]
\[ P = 3.34 \text{ psf} \]

Wind Pressure on Leeward Side

For Wall \[ P = q_h \cdot G \cdot C_r \]

For \[ h = 24' \]: \[ q_h = q_e = 14.36 \text{ lb/ft}^2 \]

\[ C_r = -0.2 \text{ for wall, } 0.6 \text{ for roof} \]

For Wall \[ P = 14.36 \times (0.85)(0.2) \]
\[ P = 2.44 \text{ lb/ft}^2 \]

For Roof \[ P = 14.36 \times (0.85)(-0.6) \]
\[ P = -7.32 \text{ lb/ft}^2 \text{ (uplift)} \]
P2.14. Establish the wind pressures on the building in Problem P2.13 when the windward roof is subjected to an uplift wind force.

### TABLE P2.13 Roof Pressure Coefficient $C_p$

<table>
<thead>
<tr>
<th>Angle $\theta$</th>
<th>10</th>
<th>15</th>
<th>20</th>
<th>25</th>
<th>30</th>
<th>35</th>
<th>45</th>
<th>$\geq 60$</th>
<th>Windward</th>
<th>Leeward</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_p$</td>
<td>-0.9</td>
<td>-0.7</td>
<td>-0.4</td>
<td>-0.3</td>
<td>-0.2</td>
<td>-0.2</td>
<td>0.0</td>
<td>0.01$\theta^\circ$</td>
<td>10</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>0.0</td>
<td>0.2</td>
<td>0.2</td>
<td>0.3</td>
<td>0.4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

See P2.13 Solution

Windward Roof (Negative Pressure)

$\theta = 33.7^\circ$

Interpolate between 30° and 35° for negative $C_p$ value in Table P2.12

$C_p = -0.274$

$p = q_h GC_p = 21.76(0.66) 0.85(-0.274)$

$= -3.34$ lb/ft$^2$ (Suction)

Note: Wind pressures on other 3 surfaces are the same as in P2.13.
(a) Compute Variation of Wind Pressure on Windward Face

\[ q_z = q_s I K_s K_d \] \hspace{1cm} Eq 2.8
\[ q_s = 0.00256V^2 \] \hspace{1cm} Eq 2.6a
\[ = 0.00256(140)^2 \]
\[ q_s = 50.176 \text{ psf}; \text{ Round to 50.18 psf} \]
\[ I = 1.15 \text{ for hospitals} \]
\[ K_s = 1; K_d = 0.85 \]
\[ K_z \text{ Read in Table 2.4} \]

\[
\begin{array}{c|cccc}
\text{Elev. (ft)} & 0 & 35' & 70' & 105' & 140' \\
\hline
K_z & 1.03 & 1.19 & 1.34 & 1.44 & 1.52 \\
\end{array}
\]

\[ q_z = 50.18 (1.15)(K_z) (1)(0.85) \]
\[ q_z = 49.05 K_z \]

Compute Wind Pressure “p” on Windward Face

\[ p = q_s GC_p = 49.05 K_z GC_p \]
where \( G = 0.85 \) for natural period less than 1 sec.
\[ C_p = -0.5 \]
\[ q_z = 49.05(1.52) = \frac{74.556}{0.85} \]
\[ p = -31.68 \text{ psf} \hspace{1cm} \text{ANS.} \]

Compute Wind Pressure on Leeward Wall

\[ p = q_s GC_p \]
\[ = 49.05 (1.52)(0.85)(-0.7) \]
\[ p = -44.36 \text{ psf} \]

(b) Variation of Wind Pressure on Windward and Leeward Sides

Compute “p” for Various Elevations

\[
\begin{array}{c|cccc}
\text{Elev. (ft)} & 0 & 35' & 70' & 105' & 140' \\
\hline
p (psf) & 34.36 & 39.69 & 44.69 & 48.03 & 50.70 \\
\end{array}
\]
P2.15. Continued

Compute Total Wind Force (kips)

\[ F_1 = \frac{50.7 + 48.02}{2} \left( \frac{35 \times 160}{1000} \right) = 276.42 \text{ kips} \]

\[ F_2 = \frac{48.03 + 44.69}{2} \left( \frac{35 \times 160}{1000} \right) = 259.62 \text{ k} \]

\[ F_3 = \frac{44.69 + 39.69}{2} \left( \frac{35 \times 160}{1000} \right) = 236.26 \text{ k} \]

\[ F_4 = \frac{39.69 + 34.36}{2} \left( \frac{35 \times 160}{1000} \right) = 207.39 \text{ k} \]

\[ F_5 = \frac{31.68 \times (140 \times 160)}{1000} = 709.63 \text{ k} \]

Total Wind Force = \( \Sigma F_1 + F_2 + F_3 + F_4 + F_5 \)

\[ = 1689.27 \text{ kips} \]
**P2.16.** Consider the five-story building shown in Figure P2.10. The average weights of the floor and roof are 90 lb/ft² and 70 lb/ft², respectively. The values of $S_{ds}$ and $S_{d}$ are equal to 0.9g and 0.4g, respectively. Since steel moment frames are used in the north-south direction to resist the seismic forces, the value of $R$ equals 8. Compute the seismic base shear $V$. Then distribute the base shear along the height of the building.

![Diagram of a five-story building](image)

**Fundamental Period**

$T = C_i \cdot h^{1/4}$  

$T = 0.035(60)^{3/4}$  

$T = 0.75 \text{ sec.}$  

$W = 4(100 \times 90) \text{ lb/ft}^2 + (100 \times 90) \text{ lb/ft}^2$  

$W = 3,870,000 \text{ lbs} = 3,870 \text{ kips}$  

$V = \frac{S_{ds} W}{T(R/I)}$  

$I = 1$ for office bldgs.  

$V = \frac{0.4(3870)}{0.75(8/1)} = 258 \text{ kips}$  

$V_{max} = \frac{S_{d} W}{R/I}$  

$V_{max} = \frac{0.9(3870)}{8/1}$  

$V_{max} = 435 \text{ kips}$

$V_{max} = 0.0441 I S_{ds} W$  

$V_{max} = 0.0441 (1)(0.9)(3870)$  

$V_{max} = 153.6 \text{ kips}$

Therefore, Use $V = 258 \text{ kips}$

$k = 1 + \frac{1 - 0.5}{2} = 1.125$

$F_i = \frac{W_i h_i^2}{\sum W_i h_i^2} V$
### P2.16. Continued

**Forces at Each Floor Level**

<table>
<thead>
<tr>
<th>Floor</th>
<th>Weight $W_i$ (kips)</th>
<th>Floor Height $h_i$ (ft)</th>
<th>$W_i h_i^2$</th>
<th>$\frac{W_i h_i^2}{\Sigma W_i h_i^2}$</th>
<th>$F_i$ (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>630</td>
<td>60</td>
<td>63,061</td>
<td>0.295</td>
<td>76.1</td>
</tr>
<tr>
<td>5th</td>
<td>810</td>
<td>48</td>
<td>63,079</td>
<td>0.295</td>
<td>76.1</td>
</tr>
<tr>
<td>4th</td>
<td>810</td>
<td>36</td>
<td>45,638</td>
<td>0.213</td>
<td>56.0</td>
</tr>
<tr>
<td>3rd</td>
<td>810</td>
<td>24</td>
<td>28,922</td>
<td>0.135</td>
<td>34.8</td>
</tr>
<tr>
<td>2nd</td>
<td>810</td>
<td>12</td>
<td>13,261</td>
<td>0.062</td>
<td>16.0</td>
</tr>
<tr>
<td>$\Sigma = 3,870$</td>
<td>$\Sigma = 213,961$</td>
<td>$\Sigma = 258$</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
P2.17. When a moment frame does not exceed 12 stories in height and the story height is at least 10 ft, the ASCE standard provides a simpler expression to compute the approximate fundamental period:

\[ T = 0.1N \]

where \( N \) = number of stories. Recompute \( T \) with the above expression and compare it with that obtained from Problem P2.16. Which method produces a larger seismic base shear?

ASCE Approximate Fundamental Period:

\[ T = 0.1N \]

\( N = 5 \quad \therefore T = 0.5 \text{ seconds} \)

\[ V = \frac{0.3 \times 6750}{0.5(5/1)} = 810 \text{ kips} \]

The simpler approximate method produces a larger value of base shear.
(a) Wind Loads Using Simplified Procedure:
Design Wind Pressure \( P_s = \lambda K_z IP_{50} \)
\( \lambda = 1.66 \) Table 2.8, Mean Roof Height = 30’

<table>
<thead>
<tr>
<th>Zones</th>
<th>( P_s )</th>
<th>( P_s = 1.66(1.15)P_{50} = 1.909P_{50} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>12.8 psf</td>
<td>24.44 psf</td>
</tr>
<tr>
<td>C</td>
<td>8.5 psf</td>
<td>16.22 psf</td>
</tr>
</tbody>
</table>

Resultant Force at Each Level: Where Distance \( a = 0.1(100’) = 10’; 0.4(30’) = 12’; 3’ \)
\( a = 10’ \) Controls & 2a = 20’ Region (A)

\[
F_{\text{1st}}: \text{Zone (A): } \frac{15’}{2}(24.44 \text{ psf}) \frac{20’}{1000} = 3.67^4 \\
\text{Zone (C): } \frac{15’}{2}(16.3 \text{ psf}) \frac{80’}{1000} = 9.78^4 \\
F_{\text{1st, Resultant}} = 13.45^4
\]

\[
F_{\text{2nd}}: \text{Zone (A): } 15’(24.44 \text{ psf}) \frac{20’}{1000} = 7.33^4 \\
\text{Zone (C): } 15’(16.3 \text{ psf}) \frac{80’}{1000} = 19.56^4 \\
F_{\text{2nd, Resultant}} = 26.89^4
\]

Base Shear \( V_{\text{base}} = F_{\text{1st}} + F_{\text{2nd}} = 40.34^7 \)

Overturning Moment \( M_{O,T} = \Sigma F_i h_i \)
\( M_{O,T} = 13.45^4(30’) + 26.89^4(15’) = 806.9^9^4 \)
P2.18. Continued

(b) Seismic Loads by Equivalent Lateral Force Procedure

Given:

\( W = 90 \text{ psf Floor & Roof} \)

\( S_{d0} = 0.27g, \ S_{d1} = 0.06g, \ R = 8, \ I = 1.5 \)

Base Shear \( V_{\text{base}} = \frac{S_{d0} W}{T(R/I)} \)

Where \( W \) Total Building Dead Load =

\[
W_{\text{roof}} = 90 \text{ psf (100')^2} = 900^i \\
W_{2\text{nd}} = 90 \text{ psf (100')^2} = 900^i \\
W_{\text{total}} = 1800^i
\]

And \( T = C_f h_i^k = 0.342 \text{ sec.} \)

\( C_f = 0.016 \) Reinf. Concrete Frame

\( X = 0.9 \) Reinf. Concrete Frame

\( h = 30' \) Building Height

\[
V_{\text{base}} = \frac{0.06(1800^i)}{(0.342 \text{ sec})(8/1.5)} = 0.033W = \frac{59.2^i}{\text{Controls}}
\]

\[
V_{\text{max}} = \frac{S_{d0} W}{R/I} = \frac{0.27(1800^i)}{(8/1.5)} = 0.051W = 91.1^i
\]

\[
V_{\text{min}} = 0.044 S_{d0} IW = 0.044(0.27)(1.5)(1800^i) = 0.0178W = 32.1^i
\]

Force @ Each Level \( F_x = \frac{W_i h_i^k}{\Sigma W_i h_i^k} V_{\text{base}} \), Where \( V_{\text{base}} = 59.2^i \)

\( T < 0.5 \text{ Sec.} \) Thus \( K = 1.0 \)

<table>
<thead>
<tr>
<th>Level</th>
<th>( W_i )</th>
<th>( H_i )</th>
<th>( W_i h_i^k )</th>
<th>( W_i h_i^k / W_i h_i^k )</th>
<th>Force @ Ea. Level:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>900^i</td>
<td>30'</td>
<td>27000</td>
<td>0.667</td>
<td>( F_{\text{roof}} = 39.5^i )</td>
</tr>
<tr>
<td>2nd</td>
<td>900^i</td>
<td>15'</td>
<td>13500</td>
<td>0.333</td>
<td>( F_{2\text{nd}} = 19.76^i )</td>
</tr>
</tbody>
</table>

\[
\Sigma W_i h_i^k = 40500 \quad \Sigma F_x = V_{\text{base}} = 59.2^i
\]

Overturning Moment \( M_{o,T} = \Sigma F_x h_i \)

\[
M_{o,T} = 39.5^i(30') + 19.76^i(15') = 1481.4 \text{ ft} \cdot \text{k}
\]

(c) Seismic Forces Govern the Lateral Strength Design.
P2.19. In the gabled roof structure shown in Figure P2.13, determine the sloped roof snow load \( P_s \). The building is heated and is located in a windy area in Boston. Its roof consists of asphalt shingles. The building is used for a manufacturing facility, placing it in a type II occupancy category. Determine the roof slope factor, \( C_s \) using the ASCE graph shown in Figure P2.19. If roof trusses are spaced at 16 ft on center, what is the uniform snow load along a truss?

Sloped Roof Snow Load \( P_s = C_s \cdot pf \)

Where \( pf \) Flat Roof Snow Load

\( pf = 0.7 \cdot C_e \cdot C_t \cdot I \cdot pg \)

\( C_e = 0.7 \) Windy Area

\( C_t = 1.0 \) Heated Building

\( I = 1.0 \) Type II Occupancy

\( pg = 40 \) psf for Boston

\( C_s = \) Based on Roof Slope \( \theta = \tan^{-1}\left(\frac{16}{24}\right) = 33.7^\circ \)

From Fig. P2.17 \( C_s \) is Approximately 0.9 (Non-Slippery Surface)

\( P_f = 0.7 \cdot (0.7) \cdot (1.0) \cdot (40 \text{ psf}) = 19.6 \text{ psf} \)

\( P_s = C_s \cdot P_f = 0.9 \cdot 19.6 \text{ psf} = 17.64 \text{ psf} \)

Uniform Load Acting on Trusses Spaced @ 16' o.c.

\( W_{\text{snow}} = 17.64 \text{ psf} \cdot (16') = 282.2 \text{ plf} \)
P2.20. A beam that is part of a rigid frame has end moments and mid-span moments for dead, live, and earthquake loads shown below. Determine the governing load combination for both negative and positive moments at the ends and mid-span of the beam. Earthquake load can act in either direction, generating both negative and positive moments in the beam.

<table>
<thead>
<tr>
<th>End Moments (ft-kip)</th>
<th>Mid-Span Moments (ft-kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead Load</td>
<td>−180</td>
</tr>
<tr>
<td>Live Load</td>
<td>−150</td>
</tr>
<tr>
<td>Earthquake</td>
<td>±80</td>
</tr>
</tbody>
</table>

End Moments

\[
1.4 DL = 1.4 \left(-180 \times \frac{\text{ft}}{\text{k}}\right) = -252 \text{ ft} \cdot \text{k} \\
1.2 DL + 1.6 LL + 0.5 \left(\frac{L}{S}\right) = 1.2 \left(-180\right) + 1.6 \left(-150\right) = -456 \text{ ft} \cdot \text{k} \\
1.2 DL \pm 1.0 E + LL + 0.2 \left(\frac{S}{L}\right) = 1.2 \left(-180\right) + \left(-80\right) + \left(-150\right) = -446 \text{ ft} \cdot \text{k}
\]

Mid-Span Moments

\[
1.4 DL = 1.4 \left(+90 \times \frac{\text{ft}}{\text{k}}\right) = +126 \text{ ft} \cdot \text{k} \\
1.2 DL + 1.6 LL + 0.5 \left(\frac{L}{S}\right) = 1.2 \left(+90\right) + 1.6 \left(+150\right) = +348 \text{ ft} \cdot \text{k} \\
1.2 DL \pm 1.0 E + LL + 0.2 \left(\frac{S}{L}\right) = 1.2 \left(90\right) + 0 + \left(150\right) = +258 \text{ ft} \cdot \text{k}
\]

Beam Needs to be Designed for Max. End Moment = −456 ft·k
Max. Mid-Span Moment = +348 ft·k
P2.21. Calculate the vertical hydrostatic load on the 5100-lb empty shipping container in Figure P2.19 subjected to a tsunami inundation height of 3’. Assuming the container is water-tight, will the tsunami wave be capable of carrying away the container as debris?

\[ F_v = \gamma V_w = 70.4(3)(8)(20) \]
\[ F_v = 33792 \text{ lbs} \]
\[ F_v = 33.8 \text{ kips} \]

33.8 kips > \( W_{\text{container}} = 5.1 \) kips

Yes, the container will be carried away.
P2.22. Consider the building in Figure P2.22, which has a width into the page of 35 ft. Maximum inundation height, $h_{\text{max}}$, and flow velocity, $u_{\text{max}}$, have been determined as 33 ft and 20 ft/sec, respectively. Calculate the hydrodynamic and hydrostatic resultant load and location on the walls $ABC$ and $IJKL$ for Load Cases 2 and 3, due to both inflow and outflow directions. If windows are inundated, calculate the expected hydrostatic loading on the adjacent outside walls due to water retained by the floor, or floors. Finally, calculate the debris impact load to be applied to the free-standing column CD. Assume $I_{\text{tsu}} = 1.0$ and $C_{\text{d}} = 1.25$.

Load Case 2:

$$\frac{2}{3} h_{\text{max}} = h_{\text{des}} = 22 \text{ ft}$$

$$\frac{1}{3} u_{\text{max}} = u_{\text{des}} = 20 \text{ ft/sec}$$

Hydrodynamic, Load Case 2

$$h_{\text{des},x} = \text{Trib height} = 8 + 6 = 14 \text{ ft}$$

$$F_{\text{d}x} = \frac{1}{2} \gamma_{\text{f}} \left( I_{\text{max}x} \right) \left( C_{\text{d}} \right) \left( B \right) \left( h_{\text{des},x} \right)(20^2)$$

$$F_{\text{d}x} = \frac{1}{2} 70.4(1.0)(1.25)(1.0)(35)(14)(20^2)$$

$$F_{\text{d}x} = 8624 \text{ kips}$$

Hydrostatic on interior walls

$$F_{\text{s}} = \frac{1}{2} \gamma_{\text{f}} b h_{\text{des}}^2 = \frac{1}{2} 70.4(35)3^2$$

$$F_{\text{s}} = 11.1 \text{ kips}$$

Debris Impact on CD

$$F_{\text{i}} = 330 C_{\text{d}} I_{\text{max}} = 330(0.65)(1.0)$$

$$F_{\text{i}} = 214.5 \text{ kips}$$

Load Case 3:

$$h_{\text{max}} = h_{\text{des}} = 33 \text{ ft}$$

$$\frac{1}{3} u_{\text{max}} = u_{\text{des}} = 6.67 \text{ ft/sec}$$

Hydrodynamic, Load Case 3

$$h_{\text{des},x} = \text{Trib height} = 1 + 8 = 9 \text{ ft}$$

$$F_{\text{d}x} = \frac{1}{2} \gamma_{\text{f}} \left( I_{\text{max}x} \right) \left( C_{\text{d}} \right) \left( B \right) \left( h_{\text{des},x} \right)(20^2)$$

$$F_{\text{d}x} = \frac{1}{2} 70.4(1.0)(1.25)(1.0)(35)(9)(6.67^2)$$

$$F_{\text{d}x} = 616 \text{ kips}$$

$$F_{\text{d}x} = \frac{1}{2} 70.4(1.0)(1.25)(1.0)(35)(16)(6.67^2)$$

$$F_{\text{d}x} = 1096.2 \text{ kips}$$

Hydrostatic on inside walls

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Figure 2.1
three-ply felt with gravel topping

2” rigid insulation

T-beam

36”

10” average

14”

4”
Figure 2.3

(a) Plan view of the building layout with panels A, B, C, and D.

(b) Section 1-1 showing the 5" slab and suspended ceiling.

(c) Section 2-2 showing Beam B1.

(d) Calculations for beams:
- Panel B1:
  - Vertical load: \( w = q \times 1 \text{ ft} \)
  - Moments:
    - Left support: \( wL/2 \)
    - Right support: \( wL/2 \)

- Beam B2:
  - Right support: \( wL/2 \)

(e) Beam B1:
- Load: \( 8.875 \text{ kips} \)
- Moment at 25'

(f) Beam B2:
- Loads: \( 8.875 \text{ kips} \)
- Moment at 24'

Duct placement and other details are also shown in the figure.
Figure 2.4
Figure 2.5

(a) Plan

(b) Elevation

\[ R = 32.3 \text{ kips} \]
(c) Tributary area to column C shown shaded

\[ A_T = 480 \text{ ft}^2 \]

\[ w_L = 0.4 \text{ kip/ft} \]

(d) Beam A

\[ L = 20' \]

\[ R = 4 \text{ kips} \]

6.736 kips

8'

(e) Beam B

\[ L = 24' \]

\[ R = 6.736 \text{ kips} \]

\[ R = 6.736 \text{ kips} \]
Figure 2.6
Figure 2.7

(a) W = Combined weight on the first two axles, which is the same as for the corresponding Design Truck
V = Variable spacing—14 ft to 30 ft inclusive. Spacing to be used is that which produces maximum stresses.

(b)

(c) uniform load
640 lb per linear foot of lane load

Traffic direction

Transverse direction

curb

dimensional limits

10'-0" clearance and load lane width

2'-0"
6'-0"

V

0.25 W
0.25 W

0.25 W
0.25 W

8000 lb
32,000 lb
32,000 lb

14'-0"
0.1 W
0.4 W
0.1 W
0.4 W

0.8 W
0.4 W
0.8 W
6'-0"
Figure 2.8

Axle spacing:
- 8'-5'-5'-5'-9'-5'-6'-5'-8'-5'-5'-5'-9'-5'-6'-5'-5'

E80 loads:

| 40 | 80 | 80 | 80 | 52 | 52 | 52 | 40 | 80 | 80 | 80 | 52 | 52 | 52 | 52 | 8 kips/ft |

First locomotive
Second locomotive
Figure 2.9

(a) Shear wall with forces $F_1, F_2, F_3, F_4, F_5$.

(b) Shear diagram and moment diagram.

(c) Plan view with weights $W_1, W_2$.

(d) Structural frame with forces $F_1, F_2, F_3, F_4, F_5$. 

Shear Diagram: 

Moment Diagram:
Figure 2.10

(a) Relationship between elevation above ground and increasing wind velocity.

(b) Relationship between elevation above ground and wind pressure.
Figure 2.11

(a) Path of air particle

(b)
Figure 2.15

(a) 

(b)
Figure 2.16
Figure 2.17

(a) Figure showing a building with windward and leeward faces. Wind speed = 130 mph, dimensions: $B = 60'$, $L = 60'$, height = 100'.

(b) Diagram illustrating forces applied to the building. Forces per square foot: $11.66 \text{ lb/ft}^2$, $8.33 \text{ lb/ft}^2$, $13.3 \text{ lb/ft}^2$, and $11.3 \text{ lb/ft}^2$.
Figure 2.18

Case 2

Case 1

Cases 1 and 2
Figure 2.19

(a) Schematic view of a shear wall system with forces:
- \( P_s = 13 \text{ lb/ft}^2 \)
- \( 2a = 6' \)
- \( 34' \)
- \( 15' \)
- Shear wall

(b) Detailed view of the forces:
- \( w = 0.0975 \text{ kip/ft} \)
- \( 18.8' \)
- \( R = 4.2 \text{ kips} \)
- \( w = 0.195 \text{ kip/ft} \)
- \( 18.8' \)
- \( R = 8.4 \text{ kips} \)
- \( w = 0.147 \text{ kip/ft} \)
- \( 18.8' \)
- \( w = 0.294 \text{ kip/ft} \)
- \( 18.8' \)
- \( w = 0.294 \text{ kip/ft} \)
- Roof
- 3rd floor
- 2nd floor
- 1st floor

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Figure 2.19 (Continued)

R = 21 kips

V\(_1\) = 11.13 kips

V\(_2\) = 9.87 kips

M\(_1\) = 300.6 kip \(\cdot\) ft

(c) 2.23 kips

4.45 kips

4.45 kips

(d)
Figure 2.20

(a) 

(b) $V = \sum F_i$
Figure 2.21

The graph shows the relationship between $k$ and $T$ (seconds) given by the equation:

$$k = 1 + \frac{T - 0.5}{2}$$
Figure 2.22

(a) A schematic of a building structure showing floors labeled from 2nd to 6th, with a roof at the top. Dimensions include:
- 5 @ 12" = 60'
- 15' height

(b) A graph showing force (kips) versus height (ft), with forces at:
- 70.8 kips at 0 ft
- 57.4 kips at 20 ft
- 44.6 kips at 40 ft
- 32.3 kips at 60 ft
- 20.8 kips at 80 ft
- 10.1 kips at 100 ft
Figure 2.23

Offshore Tsunami Height, $H_T$

Depth of 33 ft (100 m)

Sea-Level

Structure being analyzed

Inundation Depth, $h_{max}$

Inundation Limit Distance, $X_r$
Figure 2.24

Load Case 3

Load Case 2

Load Case 1

h_{des} / h_{max}

0

0.1

0.2

0.3

0.4

0.5

1

0.9

0.8

0.7

0.6

0.5

0.4

0.3

0.2

0.1

0

1/3

2/3

0

0.1

0.2

0.3

0.4

0.5

time/period of wave

u_{des} / u_{max}

1

0.9

0.8

0.7

0.6

0.5

0.4

0.3

0.2

0.1

0

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Figure 2.25

(a) Height of water, \( h_{\text{max}} \)

- \( h_{\text{des}} \)

- \( \gamma_s h_{\text{max}} \)

(b) Resulting horizontal force on surface of width \( b \)

- \( F_h = \frac{1}{2} \gamma_s h_{\text{des}}^2 \)

- \( \frac{1}{3} h_{\text{des}} \)
Figure 2.26

(a) at maximum inundation

(b) after wave recession
Figure 2.27

\[ F_v = \gamma_s V_{tank} \]
Figure 2.28

(a) Front view

(b) Side view

30 ft

4 ft

4 ft

3 ft

10 ft

2 ft

open

2

Load Case 1

h_{des} = 15 \text{ ft}

Direction of Flow
\[ F_{d2} = 1000.8 \text{ kips} \]

\[ F_{d1} = 769.8 \text{ kips} \]

422.4 kips
Figure P2.3

- 8” concrete masonry partition
- Concrete floor slab
- Mechanical duct
- Piping
- Wide flange steel beam with fireproofing
- Ceiling tile and suspension hangers
- Section
Figure P2.8

Building Section

3 @ 12' = 36'

40' - 20'
Figure P2.10

Plan
(a)

Building Section

wind pressures in lb/ft²
(b) (c)
Figure P2.11

Mechanical Floor Plan (beams not shown)

Section

floor framing above supports
vertical lateral bracing beyond
Figure P2.12

The diagram represents a building with the following dimensions:
- Height: 9 m
- Width: 40 m
- Depth: 20 m

The diagram includes arrows indicating forces or stresses, labeled as $q_z G C_p$ and $q_h G C_p$.

The text within the diagram notes that the dimensions are not to scale.
Figure P2.19

Roof slope factor $C_s$ with warm roofs and $C_t \leq 1.0$

- Unobstructed slippery surfaces with thermal resistance, $R \geq 30^\circ F \cdot h \cdot ft^2/\text{Btu} (5.3^\circ C \cdot m^2/W)$ for unventilated roofs
- $R \geq 20^\circ F \cdot h \cdot ft^2/\text{Btu} (3.5^\circ C \cdot m^2/W)$ for ventilated roofs

Roofs with obstructions or non-slippery surfaces

Roof Slope

$C_s$

0
0.2
0.4
0.6
0.8
1.0

0°
30°
60°
90°