

**Problem 2-3**

(a) Determine the factored axial load or the required axial strength,  $P_u$  of a column in an office building with a regular roof configuration. The service axial loads on the column are as follows

$$\begin{aligned} P_D &= 200 \text{ kips (dead load)} \\ P_L &= 300 \text{ kips (floor live load)} \\ P_S &= 150 \text{ kips (snow load)} \\ P_W &= \pm 60 \text{ kips (wind load)} \\ P_E &= \pm 40 \text{ kips (seismic load)} \end{aligned}$$

(b) Calculate the required nominal axial compression strength,  $P_n$  of the column.

$$\begin{aligned} 1: \quad P_u &= 1.4 P_D = 1.4 (200\text{k}) = 280 \text{ kips} \\ 2: \quad P_u &= 1.2 P_D + 1.6 P_L + 0.5 P_S \\ &= 1.2 (200) + 1.6 (300) + 0.5 (150) = \mathbf{795 \text{ kips}} \text{ (governs)} \\ 3 \text{ (a):} \quad P_u &= 1.2 P_D + 1.6 P_S + 0.5 P_L \\ &= 1.2 (200) + 1.6 (150) + 0.5 (300) = 630 \text{ kips} \\ 3 \text{ (b):} \quad P_u &= 1.2 P_D + 1.6 P_S + 0.5 P_W \\ &= 1.2 (200) + 1.6 (150) + 0.5 (60) = 510 \text{ kips} \\ 4: \quad P_u &= 1.2 P_D + 1.0 P_W + 0.5 P_L + 0.5 P_S \\ &= 1.2 (200) + 1.0 (60) + 0.5 (300) + 0.5 (150) = 525 \text{ kips} \\ 5: \quad P_u &= 1.2 P_D + 1.0 P_E + 0.5 P_L + 0.2 P_S \\ &= 1.2 (200) + 1.0 (40) + 0.5 (300) + 0.2 (150) = 460 \text{ kips} \end{aligned}$$

Note that  $P_D$  must always oppose  $P_W$  and  $P_E$  in load combination 6

$$\begin{aligned} 6: \quad P_u &= 0.9 P_D + 1.0 P_W \\ &= 0.9 (200) + 1.0 (-60) = 120 \text{ kips (no net uplift)} \\ 7: \quad P_u &= 0.9 P_D + 1.0 P_E \\ &= 0.9 (200) + 1.0 (-40) = 140 \text{ kips (no net uplift)} \end{aligned}$$

$$\phi P_n > P_u$$

$$\phi_c = 0.9$$

$$(0.9)(P_n) = (795 \text{ kips})$$

$$\mathbf{P_n = 884 \text{ kips}}$$

**Problem 2-4**

(a) Determine the ultimate or factored load for a roof beam subjected to the following service loads:

$$\begin{aligned}
 \text{Dead Load} &= 29 \text{ psf (dead load)} \\
 \text{Snow Load} &= 35 \text{ psf (snow load)} \\
 \text{Roof live load} &= 20 \text{ psf} \\
 \text{Wind Load} &= 25 \text{ psf } \mathbf{\text{upwards}} / 15 \text{ psf } \mathbf{\text{downwards}}
 \end{aligned}$$

(b) Assuming the roof beam span is 30 ft and tributary width of 6 ft, determine the factored moment and shear.

Since,  $S = 35 \text{ psf} > L_r = 20 \text{ psf}$ , use  $S$  in equations and ignore  $L_r$ .

$$\begin{aligned}
 1: \quad p_u &= 1.4D = 1.4 (29) = 40.6 \text{ psf} \\
 2: \quad p_u &= 1.2 D + 1.6 L + 0.5 S \\
 &= 1.2 (29) + 1.6 (0) + 0.5 (35) = 52.3 \text{ psf} \\
 3 \text{ (a):} \quad p_u &= 1.2D + 1.6S + 0.5W \\
 &= 1.2 (29) + 1.6 (35) + 0.5 (15) = \mathbf{98.3 \text{ psf}} \text{ (governs)} \\
 3 \text{ (b):} \quad p_u &= 1.2D + 1.6S + 0.5L \\
 &= 1.2 (29) + 1.6 (35) + (0) = 90.8 \text{ psf} \\
 4: \quad p_u &= 1.2 D + 1.0 W + L + 0.5S \\
 &= 1.2 (29) + 1.0 (15) + (0) + 0.5 (35) = 67.3 \text{ psf} \\
 5: \quad p_u &= 1.2 D + 1.0 E + 0.5L + 0.2S \\
 &= 1.2 (29) + 1.0 (0) + 0.5(0) + 0.2 (35) = 41.8 \text{ psf} \\
 6: \quad p_u &= 0.9D + 1.0W \text{ (D must always oppose **W** in load combinations 6 and 7)} \\
 &= 0.9 (29) + 1.0(-25) \text{ (*upward wind load is taken as negative*)} \\
 &= 1.1 \text{ psf (no net uplift)} \\
 7: \quad p_u &= 0.9D + 1.0E \text{ (D must always oppose **E** in load combinations 6 and 7)} \\
 &= 0.9 (29) + 1.6(0) \text{ (*upward wind load is taken as negative*)} \\
 &= 26.1 \text{ psf (no net uplift)}
 \end{aligned}$$

$$w_u = (98.3 \text{ psf})(6 \text{ ft}) = \mathbf{590 \text{ plf (downward)}}$$

downward	No net uplift
$V_u = \frac{w_u L}{2} = \frac{(590)(30)}{2} = 8850 \text{ lb.}$	.
$M_u = \frac{w_u L^2}{8} = \frac{(590)(30)^2}{8} = 66375 \text{ ft-lb}$ = 66.4 ft-kips	

### Problem 2-5

Occupancy	Uniform Load (psf)	Concentrated Load (lb)*
Library stack rooms	150	1000
Classrooms	40	1000
Heavy storage	250	-
Light Manufacturing	125	2000
Offices	50	2000

**\*Note:** Generally, the uniform live loads (in psf) are usually more critical for design than the concentrated loads

### Problem 2-6

*Determine the tributary widths and tributary areas of the joists, beams, girders and columns in the roof framing plan shown below.*

*Assuming a roof dead load of 30 psf and an essentially flat roof with a roof slope of  $\frac{1}{4}$ " per foot for drainage, determine the following loads using the ASCE 7 load combinations. Neglect the rain load,  $R$  and assume the snow load,  $S$  is zero:*

- The uniform dead and roof live load on the typical roof beam in lb/ft
- The concentrated dead and roof live loads on the typical roof girder in lb/ft
- The total factored axial load on the typical interior column, in lb.
- The total factored axial load on the typical corner column, in lb

Member	Tributary width (TW)	Tributary area ( $A_T$ )
Interior Beam	24 ft/4 spaces = 6 ft	6 ft x 32 ft = 192 ft <sup>2</sup>
Spandrel Beam	(24 ft/4 spaces)/2 + 0.75' = 3.75 ft	3.75 ft x 32 ft = 120 ft <sup>2</sup>
Interior Girder	32 ft/ 2 + 32 ft/2 = 32 ft	32 ft x 24 ft = 768 ft <sup>2</sup>
Spandrel Girder	32 ft/2 + 0.75 ft = 16.75 ft	16.75 ft x 24 ft = 402 ft <sup>2</sup>
Interior Column	-	32 ft x 24 ft = 768 ft <sup>2</sup>
Corner Column	-	(32 ft/2 + 0.75)(24 ft/2 + 0.75) ft = 214 ft <sup>2</sup>

$R_2 = 1.0$  (flat roof)

Member	$R_1$	$L_r$
Interior Beam	1.0	20psf
Spandrel Beam	1.0	20psf
Interior Girder	0.6	(0.6)(20) = 12psf
Spandrel Girder	1.2-0.001(402) = 0.798	(0.798)(20) = 15.96psf
Interior Column	0.6	(0.6)(20) = 12psf
Corner Column	1.2-0.001(214) = 0.986	(0.798)(20) = 19.72psf

Member	$p_u = 1.2D + 1.6L_r$	$w_u$ (plf)	$P_u$ (kips)
Interior Beam	$(1.2)(30) + (1.6)(20) = 68\text{psf}$	$(68\text{psf})(6\text{ft}) = 408\text{plf}$	-
Spandrel Beam	$(1.2)(30) + (1.6)(20) = 68\text{psf}$	$(68\text{psf})(3.75\text{ft}) = 255\text{plf}$	-
Interior Girder	$(1.2)(30) + (1.6)(12) = 55.2\text{psf}$	-	$(55.2\text{psf})(6\text{ft})(32\text{ft}) = 10.6\text{ kips}$
Spandrel Girder	$(1.2)(30) + (1.6)(15.96) = 61.5\text{psf}$	-	$(61.5\text{psf})(6\text{ft})(32/2\text{ft}) = 5.9\text{ kips}$
Interior Column	$(1.2)(30) + (1.6)(12) = 55.2\text{psf}$	-	$(55.2\text{psf})(768\text{ft}^2) = 42.4\text{ kips}$
Corner Column	$(1.2)(30) + (1.6)(19.72) = 67.6\text{psf}$	-	$(67.6\text{psf})(214\text{ft}^2) = 14.5\text{ kips}$

**Problem 2-7**

A 3-story building has columns spaced at 18 ft in both orthogonal directions, and is subjected to the roof and floor loads shown below. Using a column load summation table, calculate the cumulative axial loads on a typical interior column with and without live load reduction. Assume a roof slope of  $\frac{1}{4}$ " per foot for drainage.

Roof Loads:Dead Load,  $D_{\text{roof}} = 20$  psfSnow Load,  $S = 40$  psf2<sup>nd</sup> and 3<sup>rd</sup> Floor Loads:Dead Load,  $D_{\text{floor}} = 40$  psfFloor Live Load,  $L = 50$  psf

Member	$A_T$ (ft. <sup>2</sup> )	$K_{LL}$	$L_o$ (psf)	Live Load Red. Factor $0.25 + 15/\sqrt{(K_{LL} A_T)}$	Design live load, $L$ or $S$
<b>3<sup>rd</sup> floor</b>	N/A	-	-	-	<b>40 psf</b> (Snow load)
<b>2<sup>nd</sup> floor</b>	$(18)(18) = 324$ ft <sup>2</sup>	4	40 psf	$\left[ 0.25 + \frac{15}{\sqrt{(4)(324)}} \right] = 0.667$	$(0.667)(50) = \mathbf{34 \text{ psf}}$ $\geq 0.50 L_o = 25$ psf
<b>Ground Flr.</b>	2 floors x $(18)(18) = 648$ ft <sup>2</sup>	4	40 psf	$\left[ 0.25 + \frac{15}{\sqrt{(4)(648)}} \right] = 0.545$	$(0.545)(50) = \mathbf{28 \text{ psf}}$ $\geq 0.40 L_o = 20$ psf

Level	TA (ft <sup>2</sup> )	D (psf)	Live Load L <sub>o</sub> (S or L <sub>r</sub> or R) psf	LLredF	Design Live (psf) Floor: L Roof: S or L <sub>r</sub> or R	<b>W<sub>u1</sub></b> <b>(LC 2)</b> Roof: 1.2D + 0.5S (psf) Floor: 1.2D + 1.6L (psf)	<b>W<sub>u2</sub></b> <b>(LC 3)</b> Roof: 1.2D + 1.6S (psf) Floor: 1.2D + 0.5L (psf)	$P_u = (TA)(w_{u1})$ or $(TA)(w_{u2})$ (kips)	$\Sigma P$ <b>LC 2</b> (kips)	$\Sigma P$ <b>LC 3</b> (kips)	Maximum $\Sigma P$ (kips)
<b>With Floor Live Load Reduction</b>											
Roof	324	20	40	1	40	44	88	14.3 or 28.5	14.3	<b>28.5</b>	<b>28.5</b>
3 <sup>rd</sup> Flr	324	40	50	0.666	33.3	101	65	32.8 or 21	47.1	<b>49.5</b>	<b>49.5</b>
2 <sup>nd</sup> Flr	324	40	50	0.544	27.2	92	62	29.7 or 20	<b>74</b>	68	<b>74</b>
<b>Without Floor Live Load Reduction</b>											
Roof	324	20	40	1	40	44	88	14.3 or 28.5	14.3	<b>28.5</b>	<b>28.5</b>
3 <sup>rd</sup> Flr	324	40	50	1	50	128	73	41.5 or 23.7	55.7	<b>52.2</b>	<b>55.7</b>
2 <sup>nd</sup> Flr	324	40	50	1	50	128	73	41.5 or 23.7	<b>97.2</b>	75.9	<b>97</b>

**Problem 2-8**

(a) Determine the **dead load** (*with and without partitions*) in **psf** of floor area for a steel building floor system with W24x55 beams (weighs 55 lb/ft) spaced at 6'-0" o.c. and W30x116 girders (weighs 116 lb/ft) spaced at 35' o.c. The floor deck is 3.5" normal weight concrete on 1.5" x 20 gage composite steel deck.

- Include the weights of 1" light-wt floor finish, suspended acoustical tile ceiling, Mechanical and Electrical (assume an industrial building), and partitions.
- Since the beam and girder sizes are known, you must calculate the **ACTUAL WEIGHT** in psf of the beam and girder by dividing their weights in lb/ft by their tributary widths)

(b) Determine the dead loads in **kips/ft** for a typical INTERIOR BEAM and a typical INTERIOR GIRDER. Assume the girder load is uniformly distributed because there are 4 or more beams framing into the girder.

(c) If the floor system in (a) is to be used as a **heavy manufacturing plant**, determine the controlling factored loads in **kips/ft** for the design of the typical beam and the typical girder.

- Use the Limit States (LSD) load combinations
- Note that *partition* loads need not be included in the dead load calculations when the floor live load is greater than 80 psf.

(d) Determine the factored,  $V_u$  and the factored moment,  $M_u$  for a typical beam and a typical girder.

- Assume the beams and girders are simply supported
- The span of the beam is 35 ft (i.e. the girder spacing)
- The span of the girder is 30 ft.

**Part (a): Dead Loads**

W24x55	55 plf / 6ft	=	9psf
W30x116	116 plf / 35 ft	=	3psf
Floor deck			
	(4.25"/12)(145pcf)	=	51psf
metal deck		=	3psf
light wt. floor finish		=	8psf
susp. ceiling		=	2psf
M/E (industrial)		=	20psf
Partitions		=	20psf

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$$\Sigma_{DL} = 116\text{psf (with partitions)}$$

$$\Sigma_{DL} = 96\text{psf (without partitions)}$$

**Part (b):**

*dead load on interior beam:*

$$(116 \text{ psf})(6') = 696 \text{ plf} = \mathbf{0.70 \text{ kips/ft. (with partitions)}}$$

$$(96 \text{ psf})(6') = 576 \text{ plf} = \mathbf{0.58 \text{ kips/ft. (without partitions)}}$$

*dead load on interior girder:*

$$(116 \text{ psf})(35') = 4060 \text{ plf} = \mathbf{4.1 \text{ kips/ft. (with partitions)}}$$

$$(96 \text{ psf})(35') = 3360 \text{ plf} = \mathbf{3.4 \text{ kips/ft. (without partitions)}}$$

**Part (c):** Heavy Mfr.: Live Load = 250psf

$$1.4D = (1.4)(96) = 134.4 \text{ psf}$$

$$1.2D + 1.6L = (1.2)(96) + (1.6)(250) = \mathbf{515 \text{ psf}} \leftarrow \text{controls}$$

*Design Load on Beam:*

$$(515 \text{ psf})(6 \text{ ft}) = 3091 \text{ plf} = \mathbf{3.1 \text{ kips/ft}}$$

Part (d)

*Design Load on Girder (assuming uniformly distributed load):*

$$(515 \text{ psf})(35 \text{ ft}) = 18032 \text{ plf} = \mathbf{18.0 \text{ kips/ft}}$$

Factored concentrated load from a beam on a typical interior girder:

$$(3.1 \text{ kips/ft})(35'/2 + 35'/2) = \mathbf{108.5 \text{ kips}}$$

**Part (d):**

Beam: 
$$V_u = \frac{w_u L}{2} = \frac{(3.1)(35)}{2} = \mathbf{54.3 \text{ kips}}$$

$$M_u = \frac{w_u L^2}{8} = \frac{(3.1)(35)^2}{8} = \mathbf{474.7 \text{ ft-kips}}$$

Girder: 
$$V_u = \frac{w_u L}{2} = \frac{(18.0)(30)}{2} = \mathbf{270 \text{ kips}}$$

$$M_u = \frac{w_u L^2}{8} = \frac{(18.0)(30)^2}{8} = \mathbf{2025 \text{ ft-kips}}$$

## Problem 2-9

The building with the **steel roof framing** shown in **Figure 2-16** is located in Rochester, New York. Assuming **terrain category C** and a **partially exposed roof**, determine the following:

- The **balanced** snow load on the lower roof,  $P_f$
- The **balanced** snow load on the upper roof,  $P_f$
- The design snow load on the upper roof,  $P_s$
- The snow load distribution on the lower roof **considering sliding snow from the upper pitched roof**
- The snow load distribution on the lower roof **considering drifting snow**
- The **factored** dead plus snow load in lb/ft for the **low roof Beam A** shown on plan. *Assume a steel framed roof and assuming a typical dead load of 29 psf for the steel roof*
- The factored moment,  $M_u$  and factored shear,  $V_u$  for Beam A  
*Note that the beam is simply supported*
- For the typical interior roof girder nearest the taller building (i.e. the interior girder supporting beam “A”, in addition to other beams), draw the dead load and snow load diagrams, showing all the numerical values of the loads in lb/ft for:
  - Dead load and **snow drift** loads
  - Dead load and **sliding snow** load
- For each of the two cases in part (h), determine the unfactored reactions at both supports of the simply supported interior girder due to dead load, snow load, and the factored reactions. Indicate which of the two snow loads (snow drift or sliding snow) will control the design of this girder.

**HINT:** Note that for the girder, the dead load is a uniform load, whereas the snow load may be uniformly distributed or trapezoidal in shape depending on whether sliding or drifting snow is being considered.

### Solution:

#### (a) Lower Roof: Balanced Snow Load, $P_f$

Ground snow load for Rochester, New York,  $P_g = 40$  psf (Building Code of New York State, Figure 1608.2)

Assume:

Category I building	$I_s = 1.0$
Terrain Category C & Partially exposed roof	$C_e = 1.0$ (ASCE 7 Table 7-2)
Slope factor ( $\theta \approx 0$ degrees for a flat roof)	$C_s = 1.0$ (ASCE 7 Fig 7-2)

Temperature factor,  $C_t = 1.0$  (ASCE 7 Table 7-3)

Flat roof snow load or Balanced Snow load on lower roof is,

$$P_{f \text{ lower}} = 0.7 C_e C_t I_s P_g = 0.7 \times 1.0 \times 1.0 \times 1.0 \times 40 \text{ psf} = \mathbf{28 \text{ psf}}$$

- (b) Design snow load for lower roof,  $P_{s \text{ lower}} = P_f C_s = 28 \text{ psf} \times 1.0 = \mathbf{28 \text{ psf}}$

### (c) Upper Roof: Balanced Snow Load, $P_f$

Ground snow load,  $P_g = 40$  psf

Assume:

Category I building	$I_s = 1.0$
Terrain Category C & Partially exposed roof	$C_e = 1.0$ (ASCE 7 Table 7-2)

Roof slope, $\theta = \arctan(6/12) = 27$ degrees	
Slope factor,	$C_s = 1.0$ (ASCE 7 Fig 7-2)

Temperature factor,	$C_t = 1.0$ (ASCE 7 Table 7-3)
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Flat roof snow load or Balanced Snow load on *upper* roof is,

$$P_{f \text{ upper}} = 0.7 C_e C_t I_s P_g = 0.7 \times 1.0 \times 1.0 \times 1.0 \times 40 \text{ psf} = \mathbf{28 \text{ psf}}$$

$$\text{Design snow load for } \textit{upper} \text{ roof, } P_{s \text{ upper}} = P_f C_s = 28 \text{ psf} \times 1.0 = \mathbf{28 \text{ psf}}$$

### (d) Sliding Snow Load on Lower Roof

$W$  = distance from ridge to eave of sloped roof = 20 ft

$$\begin{aligned} \text{Uniform sliding snow load, } P_{SL} &= 0.4 P_{f \text{ upper}} \times W / 15' \\ &= 0.4 \times 28 \text{ psf} \times 20' / 15' = \mathbf{15 \text{ psf}} \end{aligned}$$

- This sliding snow load is **uniformly distributed over a distance of 15 ft** (*Code specified*) **measured from the face of the taller building**. This load is added to the balanced snow load on the lower roof.
- Total maximum **total snow load,  $S$**  on the *lower* roof over the *Code specified* 15 ft distance = 28 psf + 15 psf  $\approx$  **43 psf**
- Beyond the distance of 15 ft from the face of taller building, the snow load on the lower roof is a uniform value of 28 psf.

$$\text{Average } \mathbf{\text{total snow load, } S} \text{ on beam A} = 28 \text{ psf (balanced snow)} + 15 \text{ psf} \approx \mathbf{43 \text{ psf}}$$

### (e) Drifting Snow Load on Lower Roof

$$\gamma = \text{density of snow} = 0.13 P_g + 14 = 0.13 \times 40 + 14 = 19.2 \text{ pcf}$$

$$H_b = P_f (\text{lower}) / \gamma = 28 \text{ psf} / 19.2 = 1.46 \text{ ft}$$

$H$  = height difference between low roof and eave of higher roof = 15 ft

$$H_c = H - H_b = 13.54 \text{ ft}$$

The maximum height of the drifting snow is obtained as follows:

**Windward Drift:** length of lower roof = 80 ft and  $\mu = 0.75$

$$H_d = \mu (0.43 [L]^{1/3} [P_g + 10]^{1/4} - 1.5)$$

$$= 0.75 (0.43 [80]^{1/3} [40 + 10]^{1/4} - 1.5) = \mathbf{2.6 \text{ ft}} \text{ (governs)}$$

**Leeward Drift:** length of upper roof = 40 ft and  $\mu = 1.0$

$$H_d = 1.0 (0.43 [40]^{1/3} [40 + 10]^{1/4} - 1.5) = 2.4 \text{ ft}$$

The maximum value of the *triangular* snow drift load,

$$P_{SD} = \gamma H_d = 19.2 \text{ pcf} \times 2.6 \text{ ft} = \mathbf{50 \text{ psf}}$$

**This load must be superimposed on the uniform balanced flat roof snow load,  $P_f$**

The length of the *triangular* portion of the snow drift load,  $w$ , is given as follows:

$$H_d = 2.8 \text{ ft} \leq H_c = 13.54 \text{ ft, therefore}$$

$$w = 4 H_d = 4 \times 2.6 \text{ ft} = \mathbf{10.4 \text{ ft}} \text{ (governs)} \leq 8 H_c = 8 \times 13.54 = 108 \text{ ft}$$

**This triangular snow drift load must be superimposed on the uniform balanced snow load on the lower roof.**

- Therefore, Maximum *total snow load* = 28 psf + 50 psf = 78 psf.
- The snow load varies from the maximum value of 78 psf to a value of 28 psf (i.e. balanced snow load) at a distance of 10.4 ft from the face of the taller building.
- Beyond the distance of 10.4 ft from the face of taller building, the snow load on the lower roof is a uniform value of 28 psf.

**(f) Factored Dead + Live Load on Low Roof Beam A**

From geometry, the *average* snow drift load on the **low roof beam A** is found using similar triangles:

$$(50 \text{ psf} / 10.4 \text{ ft}) = SD_{\text{average}} / (10.4 \text{ ft} - 4 \text{ ft})$$

$$SD_{\text{average}} = 31 \text{ psf} = \mathbf{\text{average "uniform" snow drift load on beam A}}$$

$$\text{Average total snow load, } S \text{ on beam A} = 28 \text{ psf (balanced snow)} + 31 \text{ psf} = \mathbf{59 \text{ psf}}$$

**NOTE:** This average total snow load is greater than the value of 43 psf for *sliding snow* obtained in part (d). Therefore, the **S** value for snow drift is more critical and therefore governs!

Roof Dead Load = 29 psf (given)

Using the ASCE 7 strength load combinations, the factored load on the roof is:

$$w_{u \text{ roof}} = 1.2 \times 29 \text{ psf} + 1.6 \times 59 \text{ psf} = \mathbf{129.2 \text{ psf}}$$

Tributary width of beam A = 4 ft (see roof plan)

$$\begin{aligned} \text{Factored load on beam, } w_u &= w_{u \text{ roof}} \times \text{Beam Tributary width} \\ &= 129.2 \text{ psf} \times 4 \text{ ft} = \mathbf{517 \text{ lb/ft}} \end{aligned}$$

**(g) Factored Moment and Shear for the Low Roof Beam A**

Span of beam = 20 ft

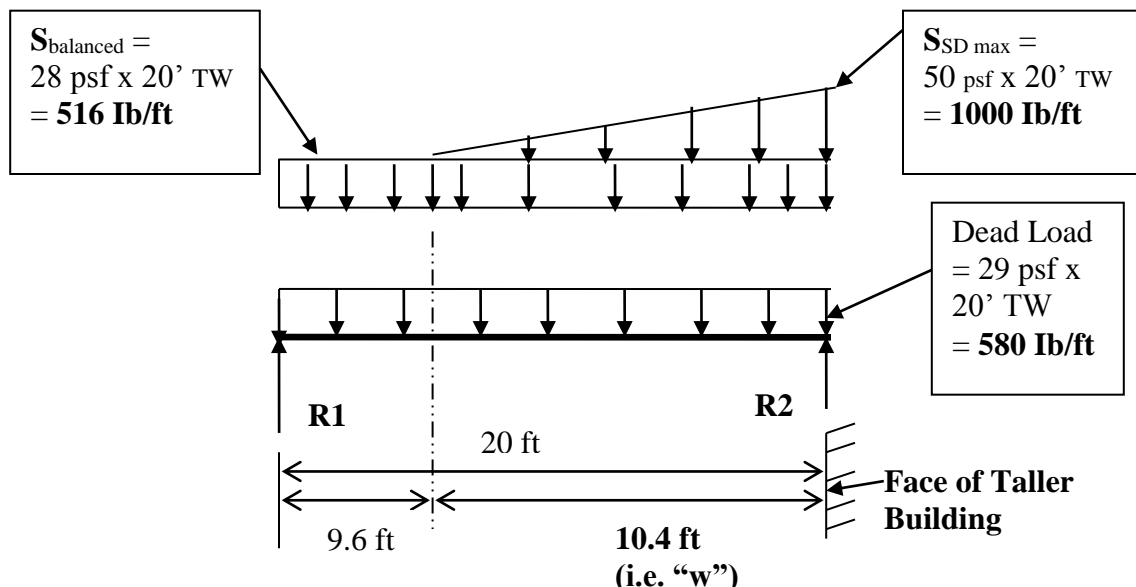
$$M_u = w_u L^2/8 = (517 \text{ lb/ft}) \times (20 \text{ ft})^2/8 = \mathbf{25.9 \text{ ft-kips}}$$

$$V_u = w_u L/2 = (517 \text{ lb/ft}) \times (20 \text{ ft})/2 = \mathbf{5.2 \text{ kips}}$$

**(h) Loading diagram for Typical Interior Low roof Girder that frames into the Taller Building column**

Consider both the snow drift and sliding snow loads and then determine which of these loads is more critical for this girder

**(1) Snow drift on *typical interior girder***



Using principles from statics, we can calculate the girder reactions as follows:

$$R_{1D} = 580 \text{ lb/ft} \times (20' / 2) = 5800 \text{ lb} = \mathbf{5.8 \text{ kips}}$$

$$R_{2D} = 580 \text{ lb/ft} \times (20'/2) = 5800 \text{ lb} = \mathbf{5.8 \text{ kips}}$$

$$R_{1L} = \frac{560 \text{ lb/ft} \times (20') \times (20'/2) + \frac{1}{2} \times 1000 \text{ lb/ft} \times 10.4' \times (10.4'/3)}{20'}$$

$$= 6501 \text{ lb} = \mathbf{6.5 \text{ kips}}$$

$$R_{2L} = 560 \text{ lb/ft} \times (20') + \frac{1}{2} \times 1000 \text{ lb/ft} \times 10.4' - R_{1LL}$$

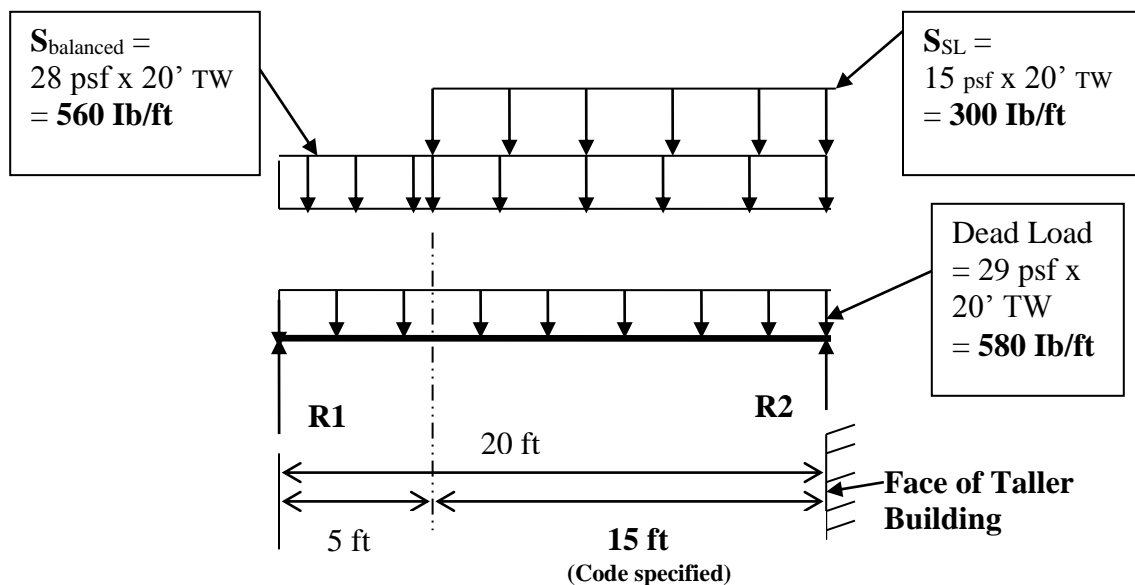
$$= 9899 \text{ lb} = \mathbf{9.9 \text{ kips}}$$

The factored reactions are calculated using the factored load combinations from the course text,

$$R_{1u} = 1.2 R_{1D} + 1.6 R_{1L} = 1.2 \times 5.8 \text{ kip} + 1.6 \times 6.5 \text{ kip} = \mathbf{17.4 \text{ kips}}$$

$$R_{2u} = 1.2 R_{2D} + 1.6 R_{2L} = 1.2 \times 5.8 \text{ kip} + 1.6 \times 9.9 \text{ kip} = \mathbf{22.8 \text{ kips}}$$

## (2) Sliding snow on typical interior girder



Using principles from statics, we can calculate the girder reactions as follows:

$$R_{1DL} = 580 \text{ lb/ft} \times (20'/2) = 5800 \text{ lb} = \mathbf{5.8 \text{ kips}}$$

$$R_{2DL} = 580 \text{ lb/ft} \times (20'/2) = 5800 \text{ lb} = \mathbf{5.8 \text{ kips}}$$

$$R_{1LL} = \frac{560 \text{ lb/ft} \times (20') \times (20'/2) + 300 \text{ lb/ft} \times 15' \times (15'/2)}{20'}$$

$$= 7288 \text{ lb} = \mathbf{7.3 \text{ kips}}$$

$$R_{2LL} = 560 \text{ lb/ft} \times (20') + 300 \text{ lb/ft} \times 15' - R_{1LL}$$

$$= 8412 \text{ lb} = \mathbf{8.4 \text{ kips}}$$

The factored reactions are calculated using the factored load combinations from the course text,

$$R_{1u} = 1.2 R_{1D} + 1.6 R_{1L} = 1.2 \times 5.8 \text{ kip} + 1.6 \times 7.3 \text{ kip} = \mathbf{18.6 \text{ kips}}$$

$$R_{2u} = 1.2 R_{2D} + 1.6 R_{2L} = 1.2 \times 5.8 \text{ kip} + 1.6 \times 8.4 \text{ kip} = \mathbf{20.4 \text{ kips}}$$

## Problem 2-10

An **eight-story** office building consists of columns located 30 ft apart in both orthogonal directions. The roof and typical floor gravity loads are given below:

### Roof loads:

Dead Load (RDL) = 80 psf;  
Snow Load (SL) = 40 psf

### Floor Loads:

Floor Dead Load (FDL) = 120 psf  
Floor Live Load (FLL) = 50 psf

- (a) Using the column tributary area and a column load summation table, determine the total unfactored and factored vertical loads in a typical interior column in the first story neglecting live load reduction.
- (b) Using the column tributary area and a column load summation table, determine the total unfactored and factored vertical loads in a typical interior column in the first story considering live load reduction.
- (c) Develop a spread sheet to solve parts (a) and (b) and verify your results.

## Solution:

### Column load summation table using tributary area

**GIVEN: 8-story building;** Typical Interior Column Tributary Area **per floor** = 30 ft x 30 ft = 900 ft<sup>2</sup>

Roof Loads:            D = 80 psf    S = 40 psf  
Typical floor loads:   D = 120 psf    L = 50 psf

**Floor Live Load Calculation Table**

Member	Levels supported	A <sub>T</sub> (summation of floor TA)	K <sub>LL</sub>	Unreduced Floor live load, L <sub>o</sub> (psf)	Design live load*, L
<b>8<sup>th</sup> floor Column</b> (i.e. column below roof)	Roof only	Floor live load reduction NOT applicable to roofs!!!	-	40 psf (snow)	40 psf (snow)
<b>7<sup>th</sup> floor column</b> (i.e. column below 8 <sup>th</sup> floor)	1 floor + roof (i.e. supports the roof and the 8 <sup>th</sup> floor)	1 floor x 900 ft <sup>2</sup> = <b>900 ft<sup>2</sup></b>	4  K <sub>LL</sub> A <sub>T</sub> = 3600 > 400 ft <sup>2</sup> ⇒ <b>Live Load reduction</b>	50 psf	0.5 x 50 = <b>25 psf</b> ≥ 0.50 L <sub>o</sub> = 25 psf

			<b>allowed</b>		
<b>6<sup>th</sup> floor column</b>  (i.e. column below 7 <sup>th</sup> floor)	2 floors + roof (i.e. supports the roof, 8 <sup>th</sup> and 7 <sup>th</sup> floors)	2 floors x 900 ft <sup>2</sup> = <b>1800 ft<sup>2</sup></b>	4  $K_{LL} A_T = 7200 > 400 \text{ ft}^2 \Rightarrow$ <b>Live Load reduction allowed</b>	50 psf	$0.43 \times 50 =$ <b>22 psf</b> $\geq 0.40 \text{ Lo} =$ 20 psf
<b>5<sup>th</sup> floor column</b>  (i.e. column below 6 <sup>th</sup> floor)	3 floors + roof (i.e. supports the roof, 8 <sup>th</sup> , 7 <sup>th</sup> and 6 <sup>th</sup> floors)	3 floors x 900 ft <sup>2</sup> = <b>2700 ft<sup>2</sup></b>	4  $K_{LL} A_T = 10800 > 400 \text{ ft}^2 \Rightarrow$ <b>Live Load reduction allowed</b>	50 psf	$0.394 \times 50 =$ <b>20 psf</b> $\geq 0.40 \text{ Lo} =$ 20 psf
<b>4<sup>th</sup> floor column</b>  (i.e. column below 5 <sup>th</sup> floor)	4 floors + roof (i.e. supports the roof, 8 <sup>th</sup> , 7 <sup>th</sup> , 6 <sup>th</sup> and 5 <sup>th</sup> floors)	4 floor x 900 ft <sup>2</sup> = <b>3600 ft<sup>2</sup></b>	4  $K_{LL} A_T = 14400 > 400 \text{ ft}^2 \Rightarrow$ <b>Live Load reduction allowed</b>	50 psf	$0.375 \times 50 = 19$ psf $\geq 0.40 \text{ Lo} =$ <b>20 psf</b>
<b>3<sup>rd</sup> floor column</b>  (i.e. column below 4 <sup>th</sup> floor)	5 floors + roof (i.e. supports the roof, 8 <sup>th</sup> , 7 <sup>th</sup> , 6 <sup>th</sup> , 5 <sup>th</sup> and 4 <sup>th</sup> floors)	5 floor x 900 ft <sup>2</sup> = <b>4500 ft<sup>2</sup></b>	4  $K_{LL} A_T = 18000 > 400 \text{ ft}^2 \Rightarrow$ <b>Live Load reduction allowed</b>	50 psf	$0.362 \times 50 = 18$ psf $\geq 0.40 \text{ Lo} =$ <b>20 psf</b>
<b>2<sup>nd</sup> floor column</b>  (i.e. column below 3 <sup>rd</sup> floor)	6 floors + roof (i.e. supports the roof, 8 <sup>th</sup> , 7 <sup>th</sup> , 6 <sup>th</sup> , 5 <sup>th</sup> , 4 <sup>th</sup> and 3 <sup>rd</sup> floors)	6 floor x 900 ft <sup>2</sup> = <b>5400 ft<sup>2</sup></b>	4  $K_{LL} A_T = 21600 > 400 \text{ ft}^2 \Rightarrow$ <b>Live Load reduction allowed</b>	50 psf	$0.352 \times 50 = 18$ psf $\geq 0.40 \text{ Lo} =$ <b>20 psf</b>
<b>Ground or 1<sup>st</sup> floor column</b>  (i.e. column below 2 <sup>nd</sup> floor)	7 floors + roof (i.e. supports the roof, 8 <sup>th</sup> , 7 <sup>th</sup> , 6 <sup>th</sup> , 5 <sup>th</sup> , 4 <sup>th</sup> , 3 <sup>rd</sup> and 2 <sup>nd</sup> floors)	7 floors x 900 ft <sup>2</sup> = <b>6300 ft<sup>2</sup></b>	4  $K_{LL} A_T = 25200 > 400 \text{ ft}^2 \Rightarrow$ <b>Live Load reduction allowed</b>	50 psf	$0.344 \times 50 = 17.3$ psf $\geq 0.40 \text{ Lo} =$ <b>20 psf</b>

$$*L = L_o [ 0.25 + \{ 15 / [K_{LL} A_T]^{0.5} \} ]$$

$\geq 0.50 L_o$  for members supporting **one floor** (e.g. slabs, beams, girders or columns)

$\geq 0.40 L_o$  for members supporting **two or more floors** (e.g. columns)

$L_o$  = unreduced design live load from the Code (ASCE 7-02 Table 4-1)

$K_{LL}$  = live load factor (ASCE 7-02 Table 4-2)

$A_T$  = summation of the floor tributary area in ft<sup>2</sup> supported by the member, excluding the roof area and floor areas with NON-REDUCIBLE live loads.

The **COLUMN LOAD SUMMATION TABLES** are shown on the following pages for the two cases:

1. Live load reduction ignored
2. Live load reduction considered

Level	TA (ft <sup>2</sup> )	D (psf)	Live Load L <sub>o</sub> (S or L <sub>r</sub> or R) psf	LLredF	Design Live (psf) Floor: L Roof: S or L <sub>r</sub> or R	W <sub>u1</sub> (LC 2)  Roof: 1.2D + 0.5S (psf) Floor: 1.2D + 1.6L (psf)	W <sub>u2</sub> (LC 3)  Roof: 1.2D + 1.6S (psf) Floor: 1.2D + 0.5L (psf)	P <sub>u</sub> = (TA)(w <sub>u1</sub> ) or (TA)(w <sub>u2</sub> ) (kips)	ΣP LC 2 (kips)	ΣP LC 3 (kips)	Maximum ΣP (kips)
<b>(b) With Floor Live Load Reduction</b>											
Roof	900	80	40	1	40	116.0	160.0	104.4 or 144.0	104.4	<b>144.0</b>	<b>144.0</b>
8 <sup>th</sup> Flr	900	120	50	0.5	25	184.0	157	165.6 or 140.9	270	<b>284.9</b>	<b>284.9</b>
7 <sup>th</sup> Flr	900	120	50	0.43	21.3	178	155.0	160.3 or 139.2	<b>430.3</b>	424.1	<b>430.3</b>
6 <sup>th</sup> Flr	900	120	50	0.4	20	176.0	154.0	158.4 or 138.6	<b>588.7</b>	562.7	<b>588.7</b>
5 <sup>th</sup> Flr	900	120	50	0.4	20	176.0	154.0	158.4 or 138.6	<b>747.1</b>	701.3	<b>747.1</b>
4 <sup>th</sup> Flr	900	120	50	0.4	20	176.0	154.0	158.4 or 138.6	<b>905.5</b>	839.9	<b>905.5</b>
3 <sup>rd</sup> Flr	900	120	50	0.4	20	176.0	154.0	158.4 or 138.6	<b>1063.9</b>	978.5	<b>1063.9</b>
2 <sup>nd</sup> Flr	900	120	50	0.4	20	176.0	154.0	158.4 or 138.6	<b>1222.3</b>	1117.1	<b>1222.3</b>
<b>(a)</b>	<b>(b) Without Floor Live Load Reduction</b>										
Roof	900	80	40	1	40	116	160	104.4 or 144.0	104.4	<b>144.0</b>	<b>144.0</b>
8 <sup>th</sup> Flr	900	120	50	1	50	224	169	201.6 or 152.1	<b>306.0</b>	296.1	<b>306.0</b>

7 <sup>th</sup> Flr	900	120	50	1	50	224	169	201.6 or 152.1	<b>507.6</b>	448.2	<b>507.6</b>
6 <sup>th</sup> Flr	900	120	50	1	50	224	169	201.6 or 152.1	<b>709.2</b>	600.3	<b>709.2</b>
5 <sup>th</sup> Flr	900	120	50	1	50	224	169	201.6 or 152.1	<b>910.8</b>	752.4	<b>910.8</b>
4 <sup>th</sup> Flr	900	120	50	1	50	224	169	201.6 or 152.1	<b>1112.4</b>	904.5	<b>1112.4</b>
3 <sup>rd</sup> Flr	900	120	50	1	50	224	169	201.6 or 152.1	<b>1314.0</b>	1056.6	<b>1314.0</b>
2 <sup>nd</sup> Flr	900	120	50	1	50	224	169	201.6 or 152.1	<b>1515.6</b>	1208.7	<b>1515.6</b>

**Problem 2-11** (see framing plan and floor section)

Framing Members:

Interior Beam: W16x31

Spandrel beam: W21x50

Interior Girder: W24x68

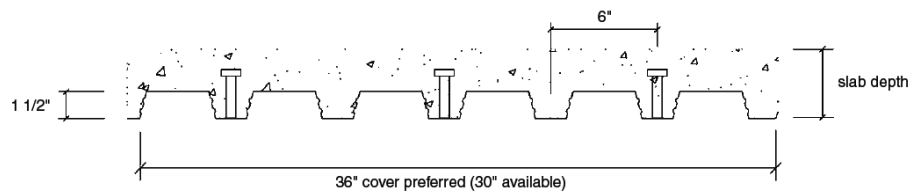
Floor Deck: see below

Assume Office occupancy, LL=50psf

- Determine the floor dead load in PSF to the interior beam
- Determine the weight of the perimeter wall (brick & stud wall) in PLF
- Determine the service dead and live loads to the spandrel and **interior** beams in PLF
- Determine the factored loads to the spandrel and **interior** beams in PLF
- Determine the factored maximum moment and shear in the to the spandrel and **interior** beams
- Determine the factored loads to the interior girder
- Determine the factored maximum moment and shear in the interior girder

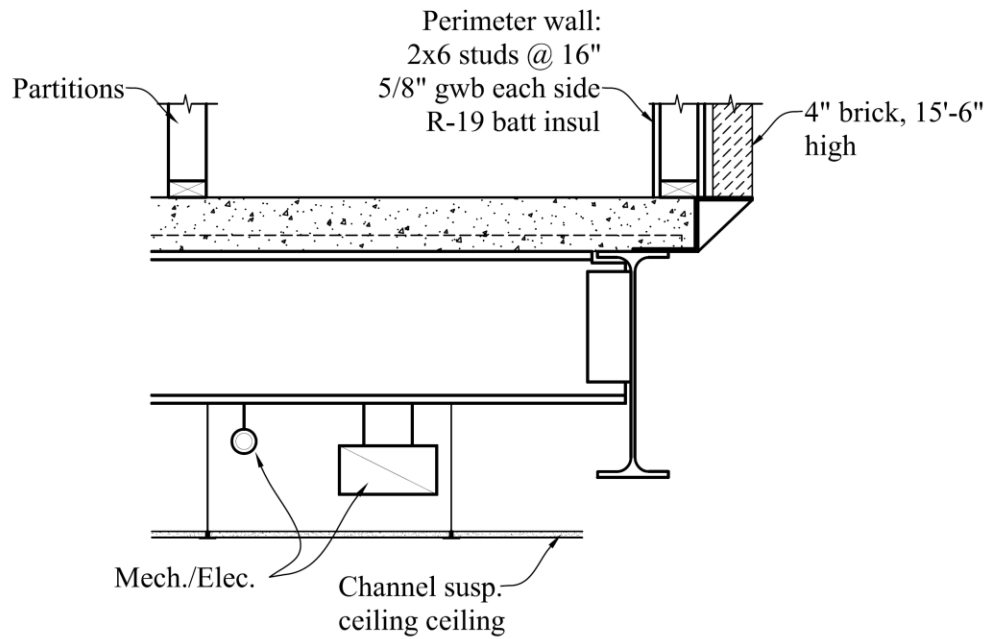
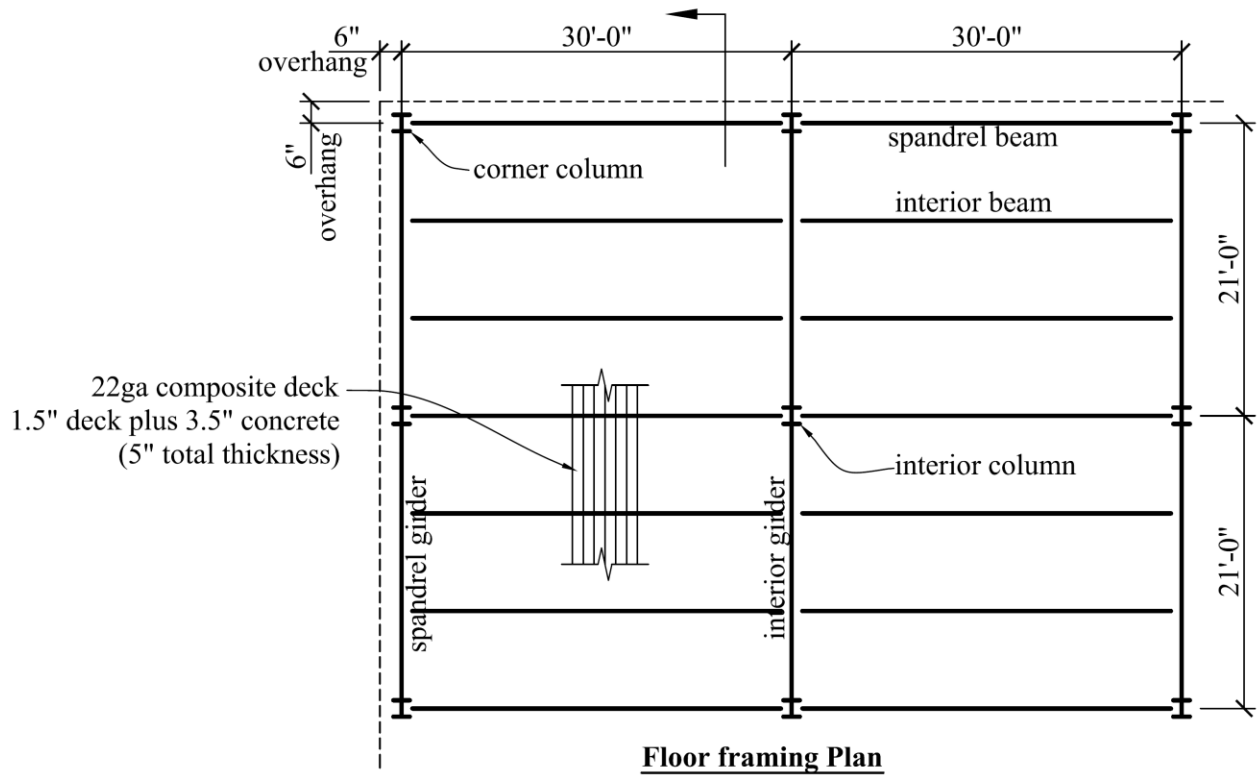
**B-LOK**1.5" x 6" deck  $F_y = 40 \text{ ksi}$   $f'_c = 3 \text{ ksi}$  145 pcf concrete

Studs are not required for composite slab action. Studs on the cross-section indicate that it is possible to install studs at the beams.



DECK PROPERTIES										
Gage	t	w	$A_s$	$I_p$	$S_p$	$S_n$	$\phi R_{be}$	$\phi R_{bl}$	$\phi V_n$	studs
22	0.0295	1.6	0.470	0.158	0.189	0.191	1290	1690	2830	0.52
20	0.0358	1.9	0.570	0.205	0.233	0.241	1830	2440	3420	0.63
19	0.0418	2.3	0.670	0.251	0.276	0.283	2420	3270	3980	0.74
18	0.0474	2.6	0.760	0.294	0.317	0.322	3040	4140	4500	0.84
16	0.0598	3.3	0.960	0.380	0.406	0.408	4620	6390	5620	0.84

COMPOSITE PROPERTIES													
	Slab Depth	$\phi M_{nt}$ in.k	$A_c$ in <sup>2</sup>	Vol. ft <sup>3</sup> /ft <sup>2</sup>	W psf	$S_c$ in <sup>3</sup>	$I_{sv}$ in <sup>4</sup>	$\phi M_{no}$ in.k	$\phi V_{nt}$ lbs.	Max Unshored Span, ft.			$A_{wrt}$ in <sup>2</sup> /ft
										1 span	2 span	3 span	
22 gage	4.00	45.43	21.3	0.255	37	0.96	4.0	32.66	3970	5.31	7.10	7.19	0.023
	4.50	53.42	24.8	0.297	43	1.16	5.7	39.48	4610	5.04	6.76	6.84	0.027
	5.00	61.41	28.3	0.339	49	1.37	7.8	46.48	5280	4.81	6.47	6.54	0.032
	5.50	69.40	32.1	0.380	55	1.58	10.4	53.61	5820	4.61	6.21	6.28	0.036
	6.00	77.39	36.0	0.422	61	1.79	13.4	60.83	6180	4.45	5.99	6.06	0.041
	6.50	85.38	40.1	0.464	67	2.00	17.0	68.14	6560	4.34	5.79	5.85	0.045
	6.75	89.37	42.2	0.484	70	2.11	19.1	71.81	6760	4.29	5.69	5.76	0.047
	7.00	93.37	44.3	0.505	73	2.22	21.3	75.50	6960	4.24	5.61	5.67	0.050

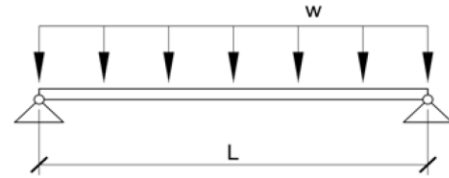


**Dead Loads**

$$\begin{aligned}
 w_{\text{deck}} &:= 1.6\text{psf} & H_{\text{wall}} &:= 15.5\text{ft} \\
 w_{\text{conc}} &:= 49\text{psf} & w_{\text{studs}} &:= 1.4\text{psf} \\
 w_{\text{part}} &:= 15\text{psf} & w_{\text{ins}} &:= 3\text{psf} \\
 w_{\text{ME}} &:= 5\text{psf} & w_{\text{gwb}} &:= 2 \cdot (5\text{psf}) \cdot 0.625 = 6.25\text{psf} \\
 w_{\text{clg}} &:= 2\text{psf} & w_{\text{brick}} &:= 40\text{psf}
 \end{aligned}$$

$$DL_{\text{floor}} := w_{\text{deck}} + w_{\text{conc}} + w_{\text{part}} + w_{\text{ME}} + w_{\text{clg}} = 72.6\text{psf}$$

$$DL_{\text{wall}} := w_{\text{studs}} + w_{\text{ins}} + w_{\text{gwb}} + w_{\text{brick}} = 50.65\text{psf}$$



$$LL := 50\text{psf}$$

$$w_{\text{wall}} := H_{\text{wall}} \cdot DL_{\text{wall}} = 785.1\text{plf} \quad \text{Part (b)}$$

**Loads to Interior Beam:**

$$L_{\text{IB}} := 30\text{ft}$$

$$TW_{\text{IB}} := 7\text{ft}$$

$$DL_{\text{IB}} := DL_{\text{floor}} + \frac{31\text{plf}}{TW_{\text{IB}}} = 77\text{psf} \quad \text{Part (a)}$$

$$w_{\text{DIB}} := TW_{\text{IB}} \cdot DL_{\text{IB}} = 539.2\text{plf}$$

$$w_{\text{LIB}} := TW_{\text{IB}} \cdot LL = 350\text{plf}$$

$$w_{\text{uIB}} := (1.2)(w_{\text{DIB}}) + (1.6)(w_{\text{LIB}}) = 1207\text{plf}$$

$$M_{\text{uIB}} := \frac{w_{\text{uIB}} \cdot L_{\text{IB}}^2}{8} = 136\text{ft}\cdot\text{kip}$$

$$V_{\text{uIB}} := \frac{w_{\text{uIB}} \cdot L_{\text{IB}}}{2} = 18.1\text{kips}$$

**Load to Interior Girder:**

$$P_{\text{uG}} := 2 \cdot V_{\text{uIB}} = 36.211\text{kips} \quad \text{Part (f)}$$

$$V_{\text{uG}} := P_{\text{uG}} = 36.211\text{kips}$$

$$M_{\text{uG}} := \frac{P_{\text{uG}} \cdot L_{\text{G}}}{3} = 253.5\text{ft}\cdot\text{kips} \quad \text{Part (g)}$$

$$L_{\text{G}} := 21\text{ft}$$

**Loads to Spandrel Beam:**

$$L_{\text{SB}} := 30\text{ft}$$

$$TW_{\text{SB}} := 3.5\text{ft} + 6\text{in} = 4\text{ft}$$

$$DL_{\text{SB}} := DL_{\text{floor}} + \frac{50\text{plf}}{TW_{\text{SB}}} = 85.1\text{psf}$$

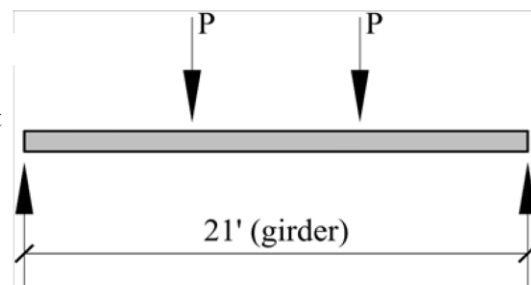
$$w_{\text{DSB}} := TW_{\text{SB}} \cdot DL_{\text{SB}} + w_{\text{wall}} = 1125.5\text{plf}$$

$$w_{\text{LSB}} := TW_{\text{SB}} \cdot LL = 200\text{plf}$$

$$w_{\text{uSB}} := (1.2)(w_{\text{DSB}}) + (1.6)(w_{\text{LSB}}) = 1671\text{plf} \quad \text{Part (d)}$$

$$M_{\text{uSB}} := \frac{w_{\text{uSB}} \cdot L_{\text{SB}}^2}{8} = 188\text{ft}\cdot\text{kip}$$

$$V_{\text{uSB}} := \frac{w_{\text{uSB}} \cdot L_{\text{SB}}}{2} = 25.1\text{kips} \quad \text{Part (e)}$$



**Problem 2-12**

Given Loads:

Uniform load,  $w$

$D = 500\text{plf}$

$L = 800\text{plf}$

$S = 600\text{plf}$

Beam length = 25 ft.

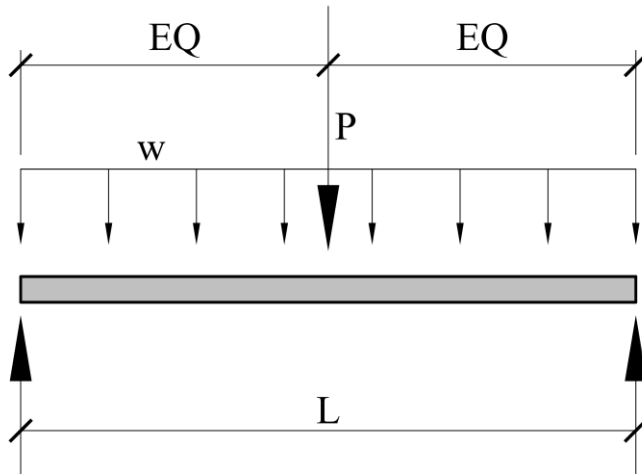
Concentrated Load,  $P$

$D = 11\text{k}$

$S = 15\text{k}$

$W = +12\text{k}$  or  $-12\text{k}$

$E = +8\text{k}$  or  $-8\text{k}$



Do the following:

- Describe a practical framing scenario where these loads could all occur as shown.
- Determine the maximum moment for each individual load effect ( $D$ ,  $L$ ,  $S$ ,  $W$ ,  $E$ )
- Develop a spreadsheet to determine the worst-case bending moments for the code-required load combinations.

**Uniform Loads****Concentrated Loads**

$$L_B := 25\text{ft}$$

$$w_D := 500\text{plf}$$

$$P_D := 11\text{kips}$$

$$w_L := 800\text{plf}$$

$$P_S := 15\text{kips}$$

$$w_S := 600\text{plf}$$

$$P_W := 12\text{kips}$$

$$P_{Wup} := -12\text{kips}$$

$$P_E := 8\text{kips}$$

$$P_{Eup} := -8\text{kips}$$

$$M_D := \frac{w_D \cdot L_B^2}{8} + \frac{P_D \cdot L_B}{4} = 108 \text{ ft} \cdot \text{kips}$$

$$M_W := \frac{P_W \cdot L_B}{4} = 75 \text{ ft} \cdot \text{kips}$$

$$M_{Wup} := \frac{P_{Wup} \cdot L_B}{4} = -75 \text{ ft} \cdot \text{kips}$$

$$M_L := \frac{w_L \cdot L_B^2}{8} = 62 \text{ ft} \cdot \text{kips}$$

$$M_E := \frac{P_E \cdot L_B}{4} = 50 \text{ ft} \cdot \text{kips}$$

$$M_{Eup} := \frac{P_{Eup} \cdot L_B}{4} = -50 \text{ ft} \cdot \text{kips}$$

$$M_S := \frac{w_S \cdot L_B^2}{8} + \frac{P_S \cdot L_B}{4} = 141 \text{ ft} \cdot \text{kips}$$

$$LC1 := (1.4 \cdot M_D) = 151 \text{ ft} \cdot \text{kips}$$

$$LC2 := (1.2 \cdot M_D) + (1.6 \cdot M_L) + (0.5 \cdot M_S) = 300 \text{ ft} \cdot \text{kips}$$

$$LC3a := (1.2 \cdot M_D) + (0.8 \cdot M_L) + (1.6 \cdot M_S) = 404 \text{ ft} \cdot \text{kips}$$

$$LC3b := (1.2 \cdot M_D) + (0.8 \cdot M_W) + (1.6 \cdot M_S) = 414 \text{ ft} \cdot \text{kips}$$

$$LC4 := (1.2 \cdot M_D) + (1.6 \cdot M_W) + (M_L) + (0.5 \cdot M_S) = 382 \text{ ft} \cdot \text{kips}$$

$$LC5 := (1.2 \cdot M_D) + (M_E) + (M_L) + (0.2 \cdot M_S) = 270 \text{ ft} \cdot \text{kips}$$

$$LC6 := (0.9 \cdot M_D) + (1.6 \cdot M_{Wup}) = -23 \text{ ft} \cdot \text{kips}$$

$$LC7 := (0.9 \cdot M_D) + (M_{Eup}) = 47 \text{ ft} \cdot \text{kips}$$

$$M_{\max} := \max(LC1, LC2, LC3a, LC3b, LC4, LC5) = 414 \text{ ft} \cdot \text{kips}$$

$$M_{\maxUp} := \min(LC6, LC7) = -23 \text{ ft} \cdot \text{kips}$$

**Problem 2-13**Given:

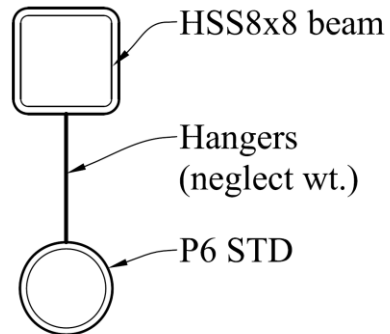
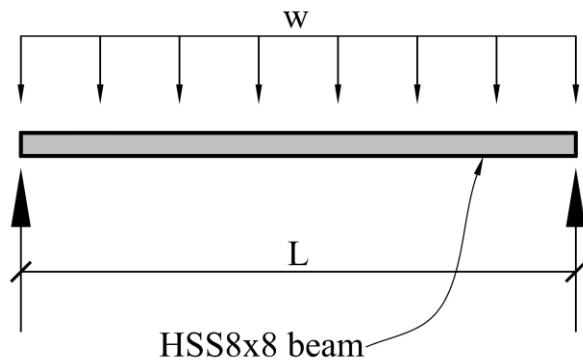
Beam is HSS8x8x3/8, Length = 28 ft.

Pipe 6 STD is hung from the beam and is full of water (assume load is uniformly distributed)

5/8" thick ice is around the HSS8x8 and P6

Find:

- The uniform load in PLF for each load item (self wt., ice, water)
- The maximum bending moment in the beam



**Dead Loads**

$$w_{8 \times 8} := 37.61 \text{ plf}$$

$$\gamma_{\text{ice}} := 56 \text{ pcf}$$

$$w_{P6} := 19 \text{ plf}$$

$$\gamma_{\text{water}} := 62.4 \text{ pcf}$$

$$O_{\text{dia}} := 6.63 \text{ in}$$

$$t_{\text{ice}} := 0.625 \text{ in}$$

$$I_{\text{dia}} := 6.07 \text{ in}^4$$

$$A_{\text{ice}8 \times 8} := t_{\text{ice}}(4)(8 \text{ in} + 0.625 \text{ in}) = 21.563 \cdot \text{in}^2$$

$$A_{\text{ice}P6} := \pi \cdot \frac{[O_{\text{dia}} + (2 \cdot t_{\text{ice}})]^2 - O_{\text{dia}}^2}{4} = 14.245 \cdot \text{in}^2$$

$$w_{\text{ice}8 \times 8} := \gamma_{\text{ice}} \cdot A_{\text{ice}8 \times 8} = 8.385 \cdot \text{plf}$$

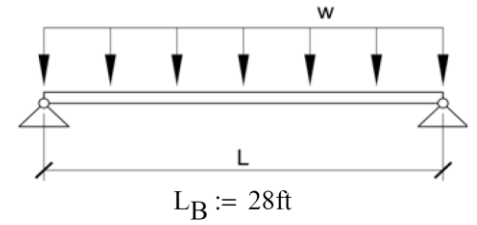
$$w_{\text{ice}P6} := \gamma_{\text{ice}} \cdot A_{\text{ice}P6} = 5.54 \cdot \text{plf}$$

$$w_{\text{water}} := \gamma_{\text{water}} \cdot \frac{\pi \cdot I_{\text{dia}}^2}{4} = 12.54 \cdot \text{plf}$$

$$w_{\text{total}} := w_{\text{ice}8 \times 8} + w_{\text{ice}P6} + w_{\text{water}} + w_{8 \times 8} + w_{P6} = 83.075 \cdot \text{plf}$$

$$M_B := \frac{w_{\text{total}} L_B^2}{8} = 8.141 \cdot \text{ft} \cdot \text{kip} \quad \text{part } b$$

$$V_B := \frac{w_{\text{total}} L_B}{2} = 1.16 \cdot \text{kips}$$



**Uniform Loads****Concentrated Loads**

$$L_B := 25\text{ft}$$

$$w_D := 500\text{plf}$$

$$P_D := 11\text{kips}$$

$$w_L := 800\text{plf}$$

$$P_S := 15\text{kips}$$

$$w_S := 600\text{plf}$$

$$P_W := 12\text{kips}$$

$$P_{Wup} := -12\text{kips}$$

$$P_E := 8\text{kips}$$

$$P_{Eup} := -8\text{kips}$$

$$M_D := \frac{w_D \cdot L_B^2}{8} + \frac{P_D \cdot L_B}{4} = 108 \text{ ft} \cdot \text{kips}$$

$$M_W := \frac{P_W \cdot L_B}{4} = 75 \text{ ft} \cdot \text{kips}$$

$$M_{Wup} := \frac{P_{Wup} \cdot L_B}{4} = -75 \text{ ft} \cdot \text{kips}$$

$$M_L := \frac{w_L \cdot L_B^2}{8} = 62 \text{ ft} \cdot \text{kips}$$

$$M_E := \frac{P_E \cdot L_B}{4} = 50 \text{ ft} \cdot \text{kips}$$

$$M_{Eup} := \frac{P_{Eup} \cdot L_B}{4} = -50 \text{ ft} \cdot \text{kips}$$

$$M_S := \frac{w_S \cdot L_B^2}{8} + \frac{P_S \cdot L_B}{4} = 141 \text{ ft} \cdot \text{kips}$$

$$LC1 := (1.4 \cdot M_D) = 151 \text{ ft} \cdot \text{kips}$$

$$LC2 := (1.2 \cdot M_D) + (1.6 \cdot M_L) + (0.5 \cdot M_S) = 300 \text{ ft} \cdot \text{kips}$$

$$LC3a := (1.2 \cdot M_D) + (0.8 \cdot M_L) + (1.6 \cdot M_S) = 404 \text{ ft} \cdot \text{kips}$$

$$LC3b := (1.2 \cdot M_D) + (0.8 \cdot M_W) + (1.6 \cdot M_S) = 414 \text{ ft} \cdot \text{kips}$$

$$LC4 := (1.2 \cdot M_D) + (1.6 \cdot M_W) + (M_L) + (0.5 \cdot M_S) = 382 \text{ ft} \cdot \text{kips}$$

$$LC5 := (1.2 \cdot M_D) + (M_E) + (M_L) + (0.2 \cdot M_S) = 270 \text{ ft} \cdot \text{kips}$$

$$LC6 := (0.9 \cdot M_D) + (1.6 \cdot M_{Wup}) = -23 \text{ ft} \cdot \text{kips}$$

$$LC7 := (0.9 \cdot M_D) + (M_{Eup}) = 47 \text{ ft} \cdot \text{kips}$$

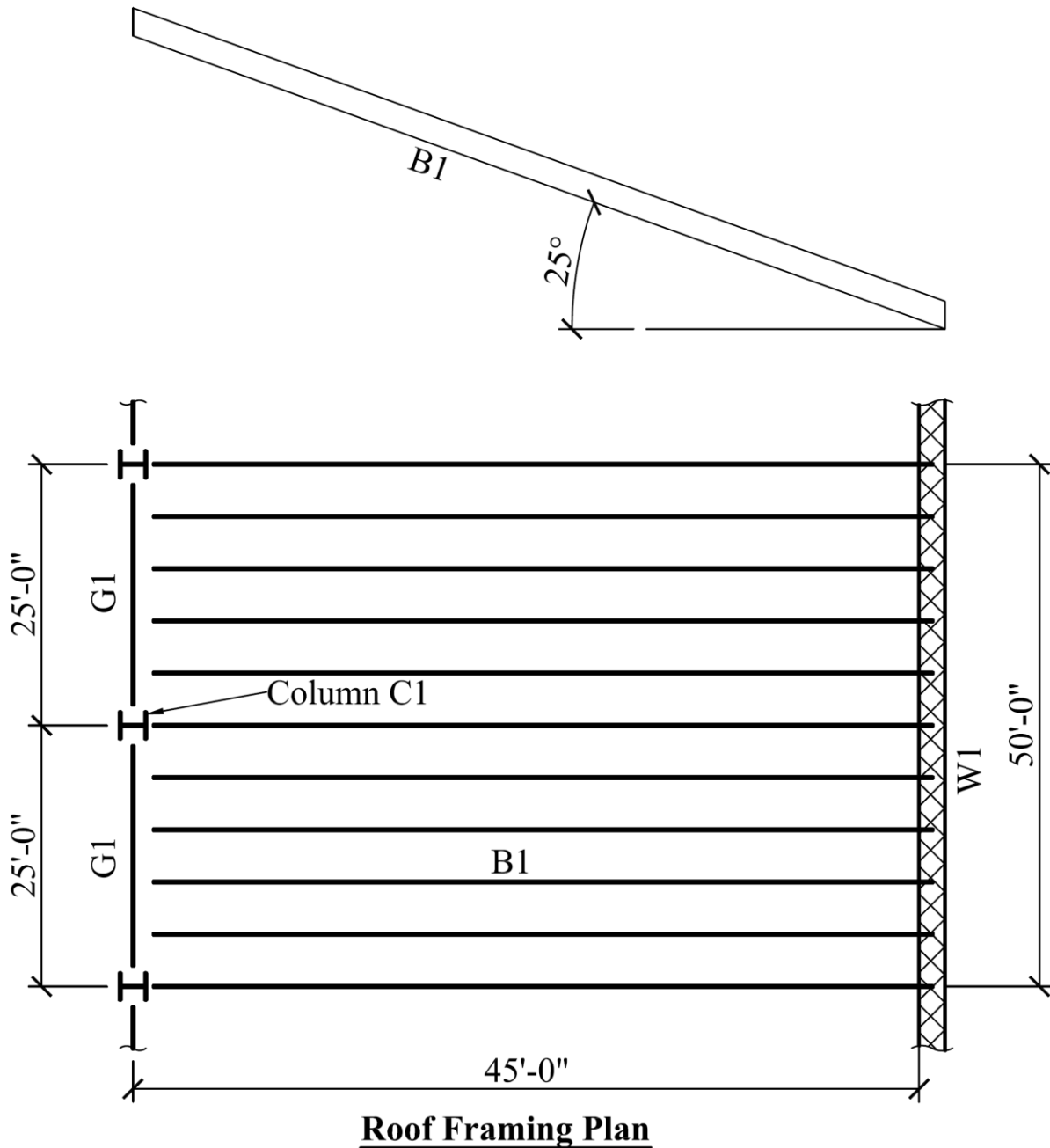
$$M_{\max} := \max(LC1, LC2, LC3a, LC3b, LC4, LC5) = 414 \text{ ft} \cdot \text{kips}$$

$$M_{\maxUp} := \min(LC6, LC7) = -23 \text{ ft} \cdot \text{kips}$$

**Problem 2-14** (see framing plan)

Assuming a roof dead load of 25 psf and a 25 degree roof slope, determine the following using the IBC factored load combinations. Neglect the rain load, R and assume the snow load, S is zero:

- d. Determine the tributary areas of B1, G1, C1, and W1
- e. The uniform dead and roof live load and the factored loads on B1 in PLF
- f. The uniform dead and roof live load on G1 and the factored loads in PLF (Assume G1 is uniformly loaded)
- g. The total factored axial load on column C1, in kips
- h. The total factored uniform load on W1 in PLF (assume trib. length of 50 ft.)



$$\begin{aligned} \text{Slope} &:= 25 & \underline{\underline{F}} &:= 12 \cdot \tan\left(\text{Slope} \cdot \frac{\pi}{180}\right) = 5.596 & L_B &:= 45\text{ft} & TW_B &:= 5\text{ft} & D &:= 25\text{psf} \\ & & & & L_G &:= 25\text{ft} & TW_W &:= 2 \cdot L_G = 50\text{ft} \\ R_2 &:= 1.2 - (0.05 \cdot F) = 0.92 \end{aligned}$$

**Part (a):**

$$TA_{B1} := L_B \cdot TW_B = 225\text{ft}^2 \quad R_{1B1} := 1.2 - \frac{0.001 \cdot TA_{B1}}{1\text{ft}^2} = 0.975$$

$$TA_{G1} := L_G \cdot \frac{L_B}{2} = 562.5\text{ft}^2 \quad R_{1G1} := 1.2 - \frac{0.001 \cdot TA_{G1}}{1\text{ft}^2} = 0.638$$

$$TA_{C1} := L_G \cdot \frac{L_B}{2} = 563\text{ft}^2 \quad R_{1C1} := 1.2 - \frac{0.001 \cdot TA_{C1}}{1\text{ft}^2} = 0.638$$

$$TW_{W1} := TW_W \cdot \frac{L_B}{2} = 1125\text{ft}^2 \quad R_{1W1} := 0.6$$

**Part (b):**

$$L_{rB1} := \max\left[0.6 \cdot 20\text{psf}, (R_{1B1} \cdot R_2 \cdot 20\text{psf})\right] = 17.9 \cdot \text{psf}$$

$$w_{DB1} := TW_B \cdot D = 125 \cdot \text{plf} \quad w_{LrB1} := TW_B \cdot L_{rB1} = 90 \cdot \text{plf} \quad w_{uB1} := (1.2 \cdot w_{DB1}) + (1.6 \cdot w_{LrB1}) = 294 \cdot \text{plf}$$

**Part (c):**

$$L_{rG1} := \max\left[0.6 \cdot 20\text{psf}, (R_{1G1} \cdot R_2 \cdot 20\text{psf})\right] = 12 \cdot \text{psf}$$

$$w_{DG1} := \frac{L_B}{2} \cdot D = 563 \cdot \text{plf} \quad w_{LrG1} := \frac{L_B}{2} \cdot L_{rG1} = 270 \cdot \text{plf} \quad w_{uG1} := (1.2 \cdot w_{DG1}) + (1.6 \cdot w_{LrG1}) = 1107 \cdot \text{plf}$$

**Part (d):**

$$L_{rC1} := \max\left[0.6 \cdot 20\text{psf}, (R_{1C1} \cdot R_2 \cdot 20\text{psf})\right] = 12 \cdot \text{psf}$$

$$P_{DC1} := TA_{C1} \cdot D = 14 \cdot \text{kips} \quad P_{LrC1} := TA_{C1} \cdot L_{rC1} = 7 \cdot \text{kips} \quad P_{uC1} := (1.2 \cdot P_{DC1}) + (1.6 \cdot P_{LrC1}) = 28 \cdot \text{kips}$$

**Part (e):**

$$L_{rW1} := \max\left[0.6 \cdot 20\text{psf}, (R_{1W1} \cdot R_2 \cdot 20\text{psf})\right] = 12 \cdot \text{psf}$$

$$w_{DW1} := \frac{L_B}{2} \cdot D = 563 \cdot \text{plf} \quad w_{LrW1} := \frac{L_B}{2} \cdot L_{rW1} = 270 \cdot \text{plf} \quad w_{uW1} := (1.2 \cdot w_{DW1}) + (1.6 \cdot w_{LrW1}) = 1107 \cdot \text{plf}$$

**Problem 2-15**

A 3-story building has columns spaced at 25 ft in both orthogonal directions, and is subjected to the roof and floor loads shown below. Using a column load summation table, calculate the cumulative axial loads on a typical interior column. Develop this table using a spreadsheet.

Roof Loads:	2nd & 3rd floor loads
Dead, D = 20psf	Dead, D = 60psf
Snow, S = 45psf	Live, L = 100psf
All other loads are 0	

**Column Load Table**

L1 = 25 ft

L2 = 25 ft

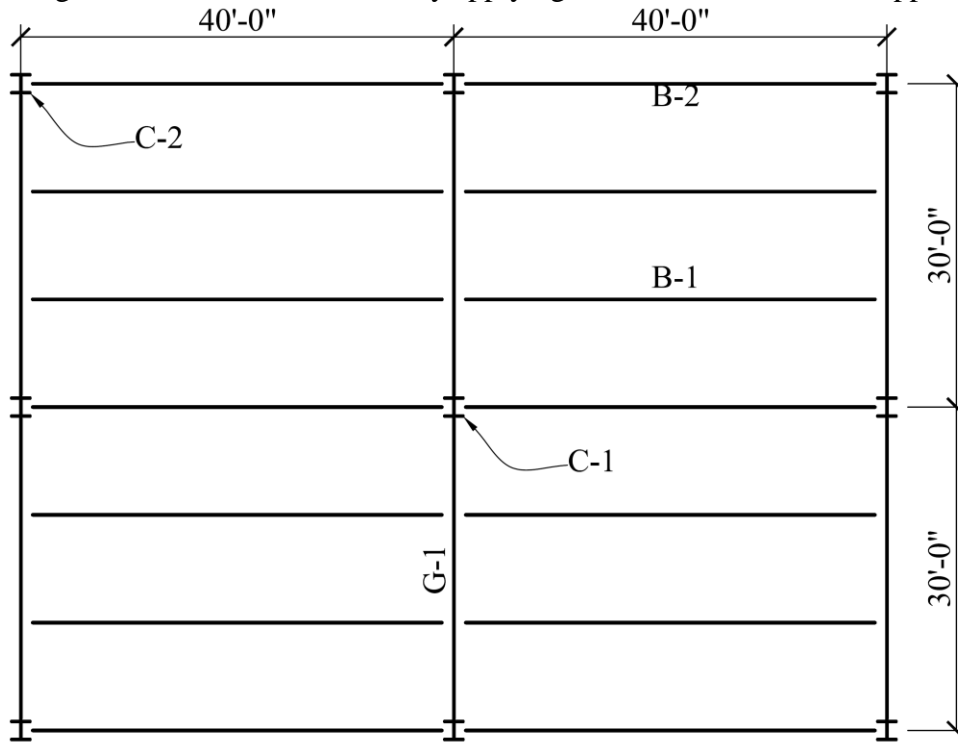
Level	Cumulative										Max. Load
	TA (ft. <sup>2</sup> )	D (psf)	S (psf)	L (psf)	wu1 (psf)	wu2 (psf)	Pu1 (kips)	Pu2 (kips)	Pu1 (kips)	Pu2 (kips)	
Roof	625	20	45	0	46.5	96	29.06	60.00	29.06	60.00	60.00
3rd	625	60	0	100	232	122	145.00	76.25	174.06	136.25	174.06
2nd	625	60	0	100	232	122	145.00	76.25	319.06	212.50	319.06

**Pu1, wu1 = 1.2D+1.6L+0.5S**

**Pu2, wu2 = 1.2D+0.5L+1.6S**

**Problem 2-16**

Using the floor plan below, assume a floor live load,  $L_O = 60\text{psf}$ . Determine the tributary areas and the design floor live load,  $L$  in PSF by applying a live load reduction, if applicable.



B-1, interior beam  
 B-2, Spandrel beam  
 G-1, interior girder  
 C-1, interior column  
 C-2, corner column

$$L_B := 40\text{ft} \quad L_G := 30\text{ft} \quad TW_B := 10\text{ft} \quad L_O := 60\text{psf}$$

$$TA_{B1} := L_B \cdot TW_B = 400\text{ft}^2 \quad K_{LLB1} := 2 \quad L_{B1} := \min \left[ L_O, L_O \cdot \left( 0.25 + \frac{15}{\sqrt{\frac{K_{LLB1} \cdot TA_{B1}}{1\text{ft}^2}}} \right) \right] = 46.82 \cdot \text{psf}$$

$$TA_{B2} := L_B \cdot \frac{TW_B}{2} = 200\text{ft}^2 \quad K_{LLB2} := 1 \quad L_{B2} := \min \left[ L_O, L_O \cdot \left( 0.25 + \frac{15}{\sqrt{\frac{K_{LLB2} \cdot TA_{B2}}{1\text{ft}^2}}} \right) \right] = 60 \cdot \text{psf}$$

$$TA_{G1} := 4 \cdot TW_B \cdot \frac{L_B}{2} = 800\text{ft}^2 \quad K_{LLG1} := 2 \quad L_{G1} := \min \left[ L_O, L_O \cdot \left( 0.25 + \frac{15}{\sqrt{\frac{K_{LLG1} \cdot TA_{G1}}{1\text{ft}^2}}} \right) \right] = 37.5 \cdot \text{psf}$$

$$TA_{C1} := L_G \cdot L_B = 1200\text{ft}^2 \quad K_{LLC1} := 4 \quad L_{C1} := \max \left[ (0.5 \cdot L_O), L_O \cdot \left( 0.25 + \frac{15}{\sqrt{\frac{K_{LLC1} \cdot TA_{C1}}{1\text{ft}^2}}} \right) \right] = 30 \cdot \text{psf}$$

$$TA_{C2} := \frac{L_G}{2} \cdot \frac{L_B}{2} = 300\text{ft}^2 \quad K_{LLC2} := 2 \quad L_{C2} := \max \left[ (0.5 \cdot L_O), L_O \cdot \left( 0.25 + \frac{15}{\sqrt{\frac{K_{LLC2} \cdot TA_{C2}}{1\text{ft}^2}}} \right) \right] = 51.7 \cdot \text{psf}$$

**Problem 2-17**

**NOTE:** Use the NYS snow map for this assignment (see [http://publicecodes.cyberregs.com/st/ny/st/b200v07/st\\_ny\\_st\\_b200v07\\_16\\_par085.htm](http://publicecodes.cyberregs.com/st/ny/st/b200v07/st_ny_st_b200v07_16_par085.htm)).

Given:

Location - Massena, NY; elevation is less than 1000 feet

Total roof DL = 25psf

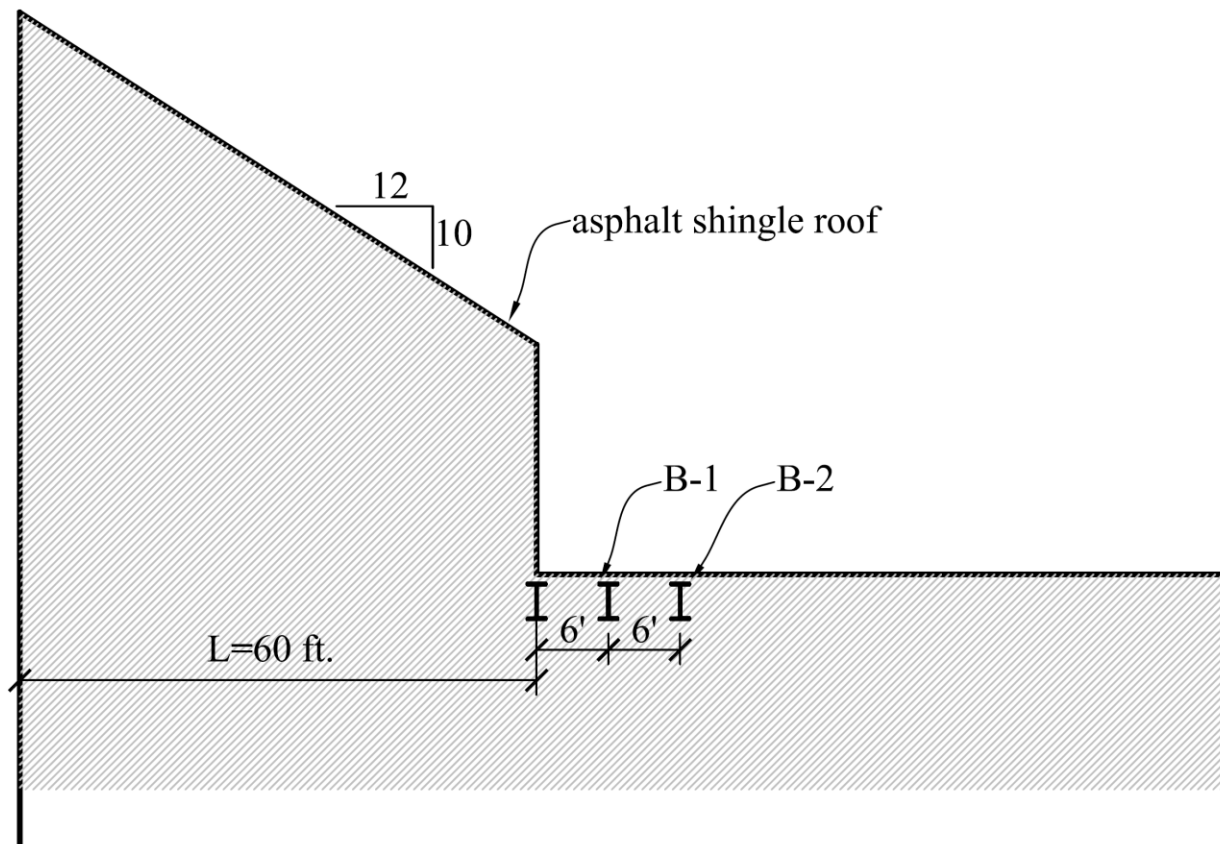
Ignore roof live load; consider load combination  $1.2D+1.6S$  only

Use normal occupancy, temperature, and exposure conditions

Length of B-1, B-2 is 30 ft.

Find:

- Flat roof snow load and sloped roof snow load in psf
- Sliding snow load in psf
- Determine the depth of the balanced snow load and the sliding snow load on B-1 and B-2 in feet.
- Draw a free-body diagram of B-1 showing the service dead and snow loads in plf
- Find the factored Moment and Shear in B-1.



**Problem 2-17**

$$p_g := 60 \text{ psf} \quad C_e := 1.0 \quad C_t := 1.0 \quad I_s := 1.0 \quad \theta := \text{atan}\left(\frac{10}{12}\right) \cdot \left(\frac{180}{\pi}\right) = 39.806$$

$$C_s := \frac{5}{3} - \frac{\theta}{45} = 0.782 \quad W_{SL} := 60 \text{ ft}$$

$$P_f := 0.7 p_g \cdot C_e \cdot C_t \cdot I_s = 42 \cdot \text{psf} \quad P_s := P_f \cdot C_s = 32.848 \cdot \text{psf} \quad \textit{part (a)}$$

$$P_{SL} := \frac{0.4 \cdot P_f \cdot W_{SL}}{15 \text{ ft}} = 67.2 \cdot \text{psf} \quad \textit{part (b)}$$

$$\gamma_{\text{snow}} := \frac{0.13}{1 \text{ ft}} \cdot p_g + 14 \text{ pcf} = 21.8 \cdot \text{pcf}$$

$$h_{\text{bal}} := \frac{P_f}{\gamma_{\text{snow}}} = 1.927 \text{ ft} \quad h_{SL} := \frac{P_{SL}}{\gamma_{\text{snow}}} = 3.083 \text{ ft} \quad \textit{part (c)}$$

$$L_B := 30 \text{ ft} \quad TW := 6 \text{ ft} \quad D := 25 \text{ psf}$$

$$w_D := TW \cdot D = 150 \cdot \text{plf} \quad w_S := TW \cdot P_f = 252 \cdot \text{plf} \quad w_{SL} := TW \cdot P_{SL} = 403.2 \cdot \text{plf} \quad \textit{part (d)}$$

$$w_u := (1.2 \cdot w_D) + [1.6 \cdot (w_S + w_{SL})] = 1228.3 \cdot \text{plf}$$

$$M_u := \frac{w_u \cdot L_B^2}{8} = 138.2 \cdot \text{ft} \cdot \text{kips} \quad V_u := \frac{w_u \cdot L_B}{2} = 18.4 \cdot \text{kips} \quad \textit{part (e)}$$

**Problem 2-18**

**NOTE:** Use the NYS snow map for this assignment (see [http://publicecodes.cyberregs.com/st/ny/st/b200v07/st\\_ny\\_st\\_b200v07\\_16\\_par085.htm](http://publicecodes.cyberregs.com/st/ny/st/b200v07/st_ny_st_b200v07_16_par085.htm)).

Given:

Location - Pottersville, NY; elevation is 1500 feet

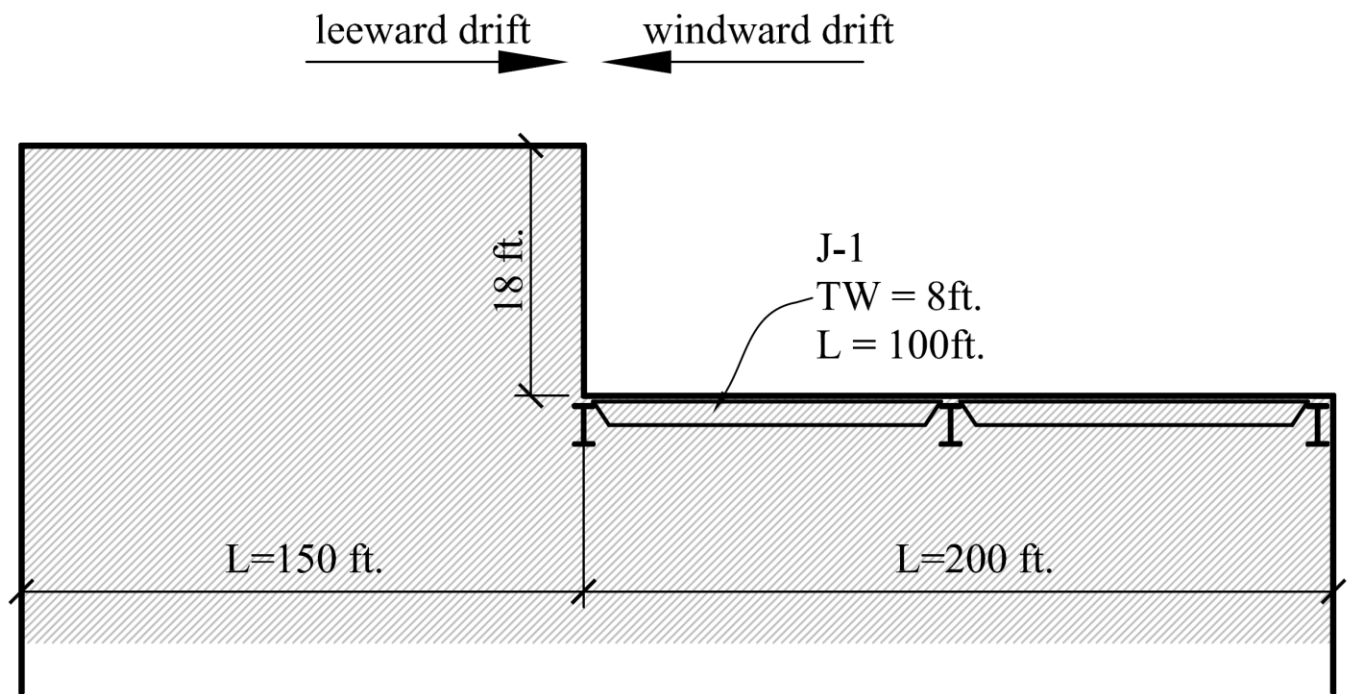
Total roof DL = 20psf

Ignore roof live load; consider load combination  $1.2D+1.6S$  only

Use normal occupancy, temperature, and exposure conditions

Find:

- Flat roof snow load
- Depth and width of the leeward drift and windward drifts; which one controls the design of J-1?
- Determine the depth of the balanced snow load and controlling drift snow load
- Draw a free-body diagram of J-1 showing the service dead and snow loads in PLF



**Problem 2-18**

$$p_g := 70\text{psf} + 10\text{psf} = 80\cdot\text{psf} \quad C_e := 1.0 \quad C_t := 1.0 \quad I_s := 1.0$$

$$P_f := 0.7p_g \cdot C_e \cdot C_t \cdot I_s = 56\cdot\text{psf} \quad \textit{part (a)}$$

$$L_{uW} := 200\text{ft} \quad h_{dW} := 0.75\text{ft} \cdot \left[ 0.43 \cdot \left( \frac{L_{uW}}{1\text{ft}} \right)^{\frac{1}{3}} \cdot \left[ \left( \frac{p_g + 10\text{psf}}{1\text{psf}} \right)^{\frac{1}{4}} \right] - 1.5 \right] = 4.684\text{ft}$$

$$L_{uL} := 150\text{ft} \quad h_{dL} := 1\text{ft} \cdot \left[ 0.43 \cdot \left( \frac{L_{uL}}{1\text{ft}} \right)^{\frac{1}{3}} \cdot \left[ \left( \frac{p_g + 10\text{psf}}{1\text{psf}} \right)^{\frac{1}{4}} \right] - 1.5 \right] = 5.537\text{ft} \quad \textit{part (b)}$$

$$\gamma_{\text{snow}} := \frac{0.13}{1\text{ft}} \cdot p_g + 14\text{pcf} = 24.4\cdot\text{pcf} \quad h_{\text{bal}} := \frac{P_f}{\gamma_{\text{snow}}} = 2.295\text{ft}$$

$$w_W := 4 \cdot h_{dW} = 18.736\text{ft} \quad w_L := 4 \cdot h_{dL} = 22.148\text{ft} \quad \textit{part (b)}$$

**The Leeward drift will control the design**

$$SD := \gamma_{\text{snow}} \cdot h_{dL} = 135.1\cdot\text{psf} \quad \textit{part (c)}$$

$$L_B := 100\text{ft} \quad TW := 8\text{ft} \quad D := 20\text{psf}$$

$$w_D := TW \cdot D = 160\cdot\text{plf} \quad w_S := TW \cdot P_f = 448\cdot\text{plf} \quad w_{SD} := TW \cdot SD = 1081\cdot\text{plf}$$

$$w_u := (1.2 \cdot w_D) + [1.6 \cdot (w_S + w_{SD})] = 2638\cdot\text{plf} \quad \textit{part (d)}$$

$$w_{uS} := 1.6 \cdot (w_S + w_{SD}) = 2446\cdot\text{plf}$$

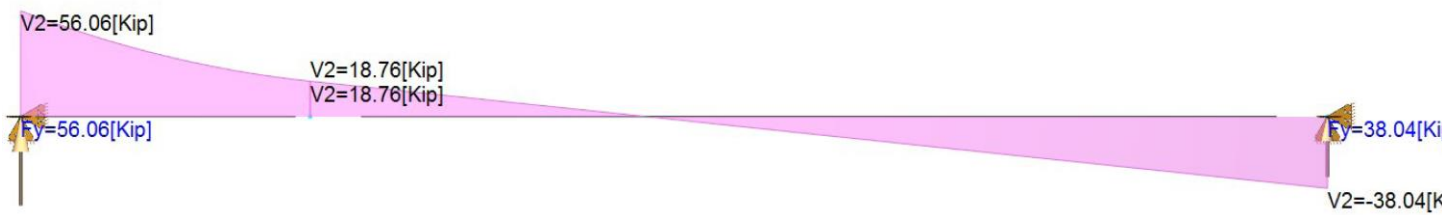
Factored - Dead Load;  $w_s = 160\text{plf}$



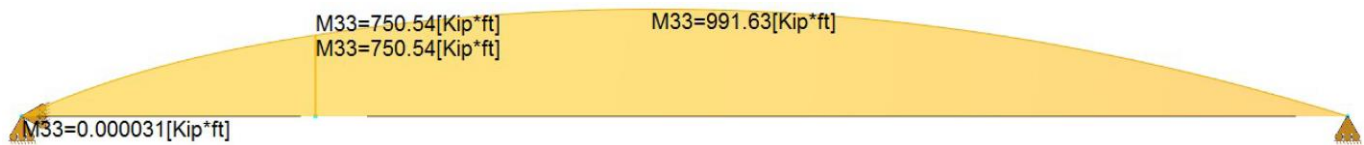
Factored - Snow Load;  $w_s = 448\text{plf}$ ,  $w_{SD} = 1081\text{plf}$



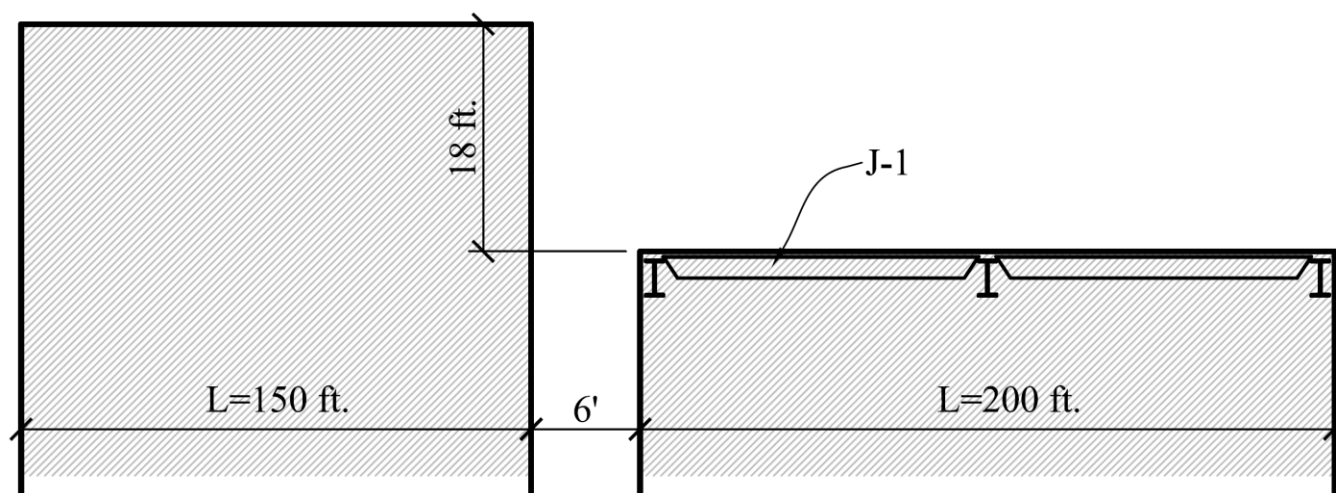
Factored - Shear



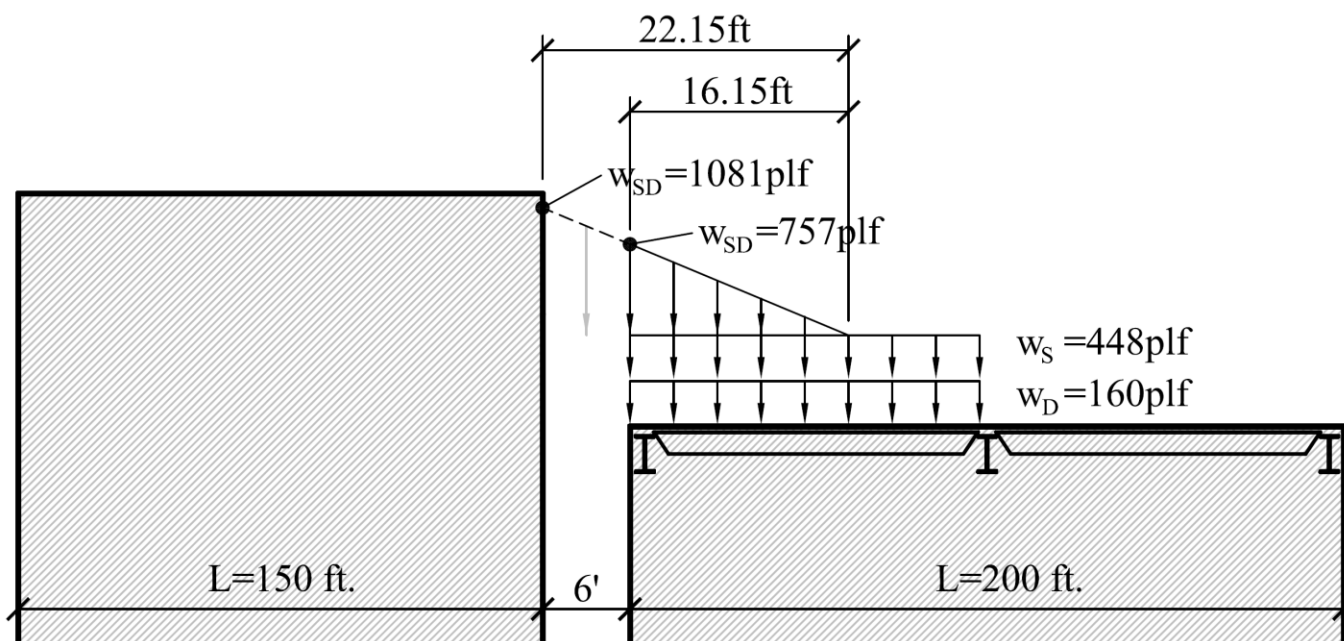
Factored - Moment



Using the values from the previous section, draw a free-body diagram of J-1 assuming the 150' and 200' buildings are separated by a distance of 6ft. Use the maximum drift load from the leeward side only for this part.



$$S_w := 6\text{ft} \quad SD_6 := \frac{\left(20 - \frac{S}{1\text{ft}}\right)}{20} \cdot w_{SD} = 757\text{plf}$$



At elevation 1500,  $p_g = 70\text{psf} + (2)(1500-1000)/100 = 80\text{psf}$  (Pottersville)