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

P_D = 200 kips (dead load)
P_L = 300 kips (floor live load)
P_S = 150 kips (snow load)
P_W = ±60 kips (wind load)
P_E = ±40 kips (seismic load)

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

1: P_u = 1.4 P_D = 1.4 (200k) = 280 kips

2: P_u = 1.2 P_D + 1.6 P_L + 0.5 P_S
= 1.2 (200) + 1.6 (300) + 0.5 (150) = **795 kip**s (governs)

3 (a): P_u = 1.2 P_D + 1.6 P_S + 0.5P_L
= 1.2 (200) + 1.6 (150) + 0.5(300) = 630 kips

3 (b): P_u = 1.2 P_D + 1.6 P_S + 0.5 P_W
= 1.2 (200) + 1.6 (150) + 0.5 (60) = 510 kips

4: 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 kips

5: 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 kips

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

6: P_u = 0.9 P_D + 1.0 P_W
= 0.9 (200) +1.0 (-60) = 120 kips *(no net uplift)*

7: P_u = 0.9 P_D + 1.0 P_E
= 0.9 (200) + 1.0 (-40) = 140 kips *(no net uplift)*

$\phi P_n > P_u$

$\phi_c = 0.9$

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

P_n = 884 kips

Problem 2-4

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

Dead Load = 29 psf (dead load)
Snow Load = 35 psf (snow load)
Roof live load = 20 psf
Wind Load = 25 psf **upwards** / 15 psf **downwards**

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

Since, S = 35psf > L_r = 20psf, use S in equations and ignore L_r.

1: p_u = 1.4D = 1.4 (29) = 40.6 psf

2: p_u = 1.2 D + 1.6 L + 0.5 S
= 1.2 (29) + 1.6 (0) + 0.5 (35) = 52.3 psf

3 (a): p_u = 1.2D + 1.6S + 0.5W
= 1.2 (29) + 1.6 (35) + 0.5 (15) = **98.3 psf** (governs)

3 (b): p_u = 1.2D + 1.6S + 0.5L
= 1.2 (29) + 1.6 (35) + (0) = 90.8 psf

4: p_u = 1.2 D + 1.0 W + L + 0.5S
= 1.2 (29) + 1.0 (15) + (0) + 0.5 (35) = 67.3 psf

5: 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 psf

6: p_u = 0.9D + 1.0W (**D** must always oppose **W** in load combinations 6 and 7)
= 0.9 (29) + 1.0(**-**25) (*upward wind load is taken as negative*)
= 1.1 psf *(no net uplift)*

7: p_u = 0.9D + 1.0E (**D** must always oppose **E** in load combinations 6 and 7)
= 0.9 (29) + 1.6(0) (*upward wind load is taken as negative*)
= 26.1 psf *(no net uplift)*`

w_u = (98.3psf)(6ft) = **590 plf** *(downward)*

downward	**No net uplift**
$V_u = \frac{w_u L}{2} = \frac{(590)(30)}{2}$ = 8850 lb.	.
$M_u = \frac{w_u L^2}{8} = \frac{(590)(30)^2}{8}$ = 66375 ft-Ib = 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 ¼" 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:

a. The uniform dead and roof live load on the typical roof beam in Ib/ft
b. The concentrated dead and roof live loads on the typical roof girder in Ib/ft
c. The total factored axial load on the typical interior column, in Ib.
d. The total factored axial load on the typical corner column, in Ib

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

R_2 = 1.0 (flat roof)

Member	R_1	Lr
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) = **68psf**	(68psf)(6ft) = **408plf**	-
Spandrel Beam	(1.2)(30)+(1.6)(20) = **68psf**	(68psf)(3.75ft) = **255plf**	-
Interior Girder	(1.2)(30)+(1.6)(12) = **55.2psf**	-	(55.2psf)(6ft)(32ft) = **10.6 kips**
Spandrel Girder	(1.2)(30)+(1.6)(15.96) = **61.5psf**	-	(61.5psf)(6ft)(32/2ft) = **5.9 kips**
Interior Column	(1.2)(30)+(1.6)(12) = **55.2psf**	-	(55.2psf)(768ft^2) = **42.4 kips**
Corner Column	(1.2)(30)+(1.6)(19.72) = **67.6psf**	-	(67.6psf)(214ft^2) = **14.5 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 ¼” per foot for drainage.

Roof Loads:		2nd and 3rd Floor Loads:
Dead Load, D_{roof}	= 20 psf	Dead Load, D_{floor} = 40 psf
Snow Load, S	= 40 psf	Floor Live Load, L = 50 psf

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

Level	TA (ft^2)	D **(psf)**	Live Load L_o (S or L_r or R) **psf**	LLredF	Design Live **(psf)** Floor: L Roof: S or L_r or R	**W_{u1} (LC 2)** Roof: 1.2D +0.5S **(psf)** Floor: 1.2D + 1.6L**(psf)**	**W_{u2} (LC 3)** Roof: 1.2D + 1.6S **(psf)** Floor: 1.2D + 0.5L **(psf)**	P_u = (TA)(w_{u1}) or (TA)(w_{u2}) **(kips)**	ΣP **LC 2** (kips)	ΣP **LC 3** (kips)	Maximum ΣP (kips)
	With Floor Live Load Reduction										
Roof	324	20	40	1	40	44	88	14.3 or 28.5	14.3	**28.5**	**28.5**
3rd Flr	324	40	50	0.666	33.3	101	65	32.8 or 21	47.1	**495**	**49.5**
2nd Flr	324	40	50	0.544	27.2	92	62	29.7 or 20	**74**	68	**74**
	Without Floor Live Load Reduction										
Roof	324	20	40	1	40	44	88	14.3 or 28.5	14.3	**28.5**	**28.5**
3rd Flr	324	40	50	1	50	128	73	41.5 or 23.7	55.7	**52.2**	**55.7**
2nd Flr	324	40	50	1	50	128	73	41.5 or 23.7	**97.2**	75.9	**97**

Problem 2-8

(a) Determine the **dead load** (<u>*with and without partitions*</u>) **in psf** of floor area for a steel building floor system with W24x55 beams (weighs 55 Ib/ft) spaced at 6'-0" o.c. and W30x116 girders (weighs 116 Ib/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 Ib/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 <u>typical beam</u> and the <u>typical girder</u>.

- 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.

<u>**Part (a):** Dead Loads</u>

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

Σ_{DL} = 116psf (with partitions)
Σ_{DL} = 96psf (without partitions)

<u>**Part (b):**</u>

dead load on interior beam:
(116 psf)(6’) = 696 plf = **0.70 kips/ft.** (with partitions)
(96 psf)(6’) = 576 plf = **0.58 kips/ft.** (without partitions)

dead load on interior girder:
(116 psf)(35’) = 4060 plf = **4.1 kips/ft.** (with partitions)
(96 psf)(35’) = 3360 plf = **3.4 kips/ft.** (without partitions)

<u>**Part (c):**</u> Heavy Mfr.: Live Load = 250psf

1.4D = (1.4)(96) = 134.4psf
1.2D + 1.6L = (1.2)(96) + (1.6)(250)= **515ps**f ← controls

Design Load on Beam:
(515psf)(6 ft) = 3091 plf = **3.1 kips/ft**

Part (d)

Design Load on Girder (assuming uniformly distributed load):

(515psf)(35 ft) = 18032 plf = **18.0 kips/ft**

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

(3.1 kips/ft)(35’/2 + 35’/2) = **108.5 kips**

Part (d):

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

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

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

$M_u = \frac{w_u L^2}{8} = \frac{(18.0)(30)^2}{8}$ = **2025 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:

a) The **balanced** snow load on the lower roof, P_f
b) The **balanced** snow load on the upper roof, P_f
c) The design snow load on the upper roof, P_s
d) The snow load distribution on the lower roof **considering sliding snow from the upper pitched roof**
e) The snow load distribution on the lower roof **considering drifting snow**
f) The **factored** dead plus snow load in Ib/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*
g) The factored moment, M_u and factored shear, V_u for Beam A
Note that the beam is ***simply supported***
h) 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 Ib/ft for:

   a. Dead load and **snow drift** loads
   b. Dead load and **sliding snow** load

i) 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 lower = $0.7\, C_e C_t I_s P_g$ = 0.7 x 1.0 x 1.0 x 1.0 x 40 psf = **28 psf**

(b) Design snow load for lower roof, P_s lower = $P_f\, C_s$ = 28 psf x 1.0 = **28 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, θ = arc tan (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)

Flat roof snow load or Balanced Snow load on *upper* roof is,

P_f upper = 0.7 C_e C_t I_s P_g = 0.7 x 1.0 x 1.0 x 1.0 x 40 psf = **28 psf**

Design snow load for *upper* roof, P_s upper = P_f C_s = 28 psf x 1.0 = **28 psf**

(d) Sliding Snow Load on Lower Roof

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

Uniform sliding snow load, P_{SL} = 0.4 $P_{f\ upper}$ x W / 15'
= 0.4 x 28 psf x 20'/15' = **15 psf**

- 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.

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

(e) Drifting Snow Load on Lower Roof

γ = density of snow = 0.13 P_g + 14 = 0.13 x 40 + 14 = 19.2 pcf
H_b = P_f (lower)/ γ = 28 psf / 19.2 = 1.46 ft
H = height difference between low roof and eave of higher roof = 15 ft
H_c = H – H_b = 13.54 ft

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

Windward Drift: length of lower roof = 80 ft and μ = 0.75

H_d = μ (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) = **2.6 ft** (governs)

Leeward Drift: length of upper roof = 40 ft and μ = 1.0

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

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

P_{SD} = γ H_d = 19.2 pcf x 2.6 ft = **50 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 ft ≤ H_c= 13.54 ft, therefore
w = 4 H_d = 4 x 2.6 ft = **10.4 ft** (governs) ≤ 8 H_c = 8 x 13.54 = 108 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 psf / 10.4 ft) = $SD_{average}$ /(10.4 ft – 4 ft)

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

Average **total snow load, S** on beam A = 28 psf (balanced snow) + 31 psf = **59 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:

$w_{u\ roof}$ = 1.2 x 29 psf + 1.6 x 59 psf = **129.2 psf**

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

Factored load on beam, w_u = $w_{u\ roof}$ x Beam Tributary width
= 129.2 psf x 4 ft = **517 lb/ft**

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

Span of beam = 20 ft

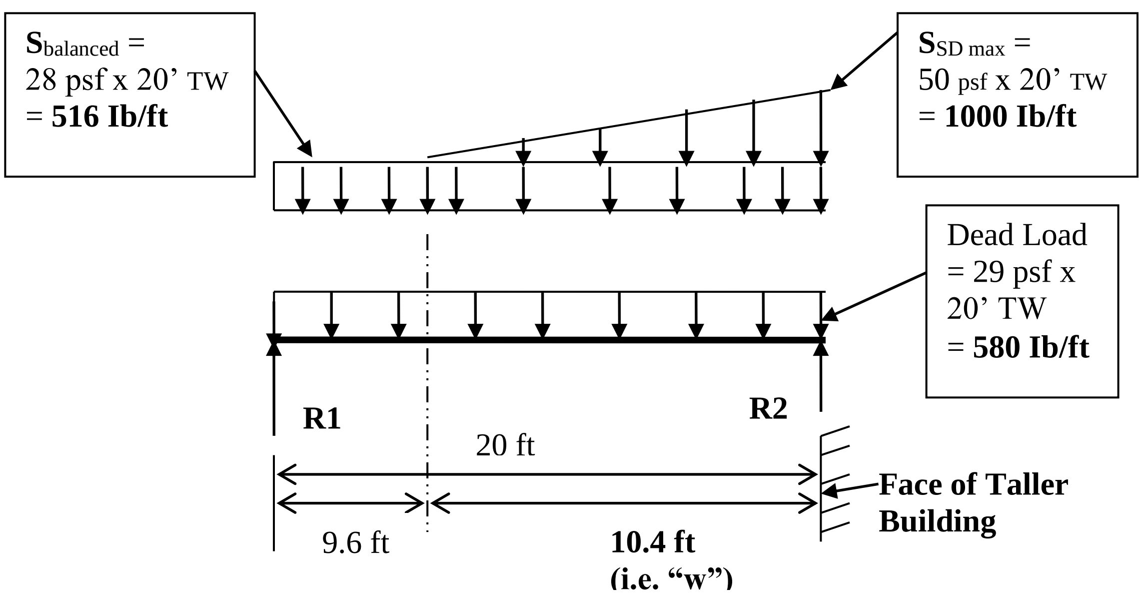
M_u = $w_u L^2/8$ = (517 lb/ft) x (20 ft)^2/8 = **25.9 ft-kips**

V_u = $w_u L/2$ = (517 lb/ft) x (20 ft)/2 = **5.2 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:

$\mathbf{R_{1\ D}}$ = 580 Ib/ft x (20'/2) = 5800 Ib = **5.8 kips**

$\mathbf{R_{2\,D}}$ = 580 Ib/ft x (20'/2) = 5800 Ib = **5.8 kips**

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

= 6501 lb = **6.5 kips**

$\mathbf{R_{2\,L}}$ = 560 lb/ft x (20') + ½ x 1000 Ib/ft x 10.4' – $R_{1\,LL}$

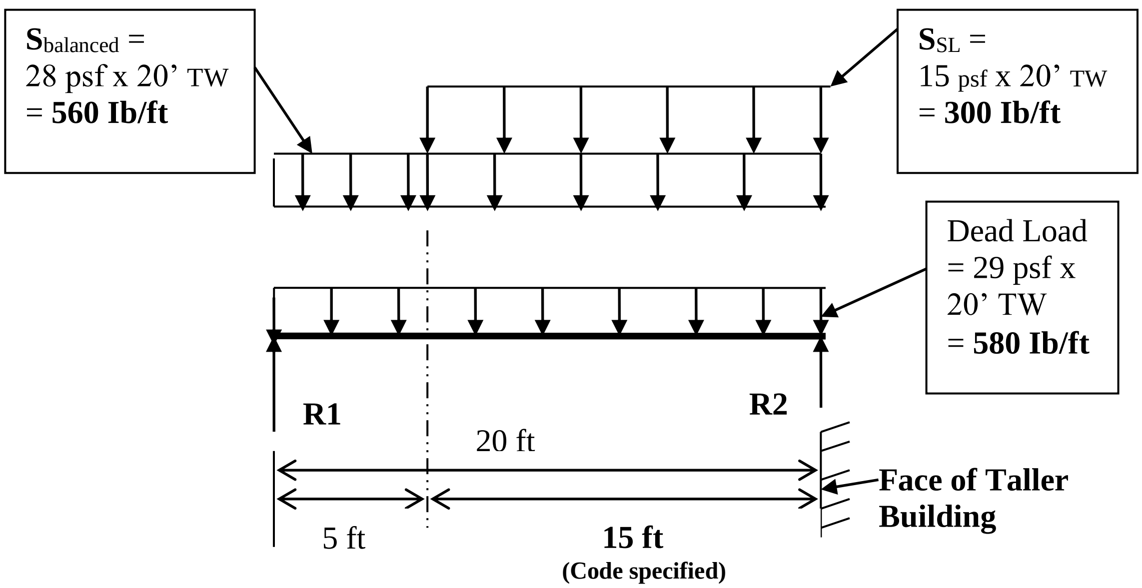
= 9899 Ib = **9.9 kips**

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

$\mathbf{R_{1\,u}}$ = 1.2 $R_{1\,D}$ + 1.6 $R_{1\,L}$ = 1.2 x 5.8 kip + 1.6 x 6.5 kip = **17.4 kips**

$\mathbf{R_{2\,u}}$ = 1.2 $R_{2\,D}$ + 1.6 $R_{2\,L}$ = 1.2 x 5.8 kip + 1.6 x 9.9 kip = **22.8 kips**

(2) Sliding snow on typical interior girder



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

$\mathbf{R_{1\,DL}}$ = 580 Ib/ft x (20'/2) = 5800 Ib = **5.8 kips**

$\mathbf{R_{2\,DL}}$ = 580 Ib/ft x (20'/2) = 5800 Ib = **5.8 kips**

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

= 7288 Ib = **7.3 kips**

$\mathbf{R_{2\,LL}}$ = 560 lb/ft x (20') + 300 lb/ft x 15' – $R_{1\,LL}$

= 8412 Ib = **8.4 kips**

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

$\mathbf{R_{1\,u}}$ = 1.2 $R_{1\,D}$ + 1.6 $R_{1\,L}$ = 1.2 x 5.8 kip + 1.6 x 7.3 kip = **18.6 kips**

$\mathbf{R_{2\,u}}$ = 1.2 $R_{2\,D}$ + 1.6 $R_{2\,L}$ = 1.2 x 5.8 kip + 1.6 x 8.4 kip = **20.4 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^2

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_T (summation of floor TA)	K_{LL}	Unreduced Floor live load, Lo (psf)	Design live load*, L
8^{th} floor Column (i.e. column below roof)	Roof only	Floor live load reduction NOT applicable to roofs!!!	-	40 psf (snow)	40 psf (snow)
7^{th} floor column (i.e. column below 8^{th} floor)	1 floor + roof (i.e. supports the roof and the 8^{th} floor)	1 floor x 900 ft^2 = **900 ft^2**	4 K_{LL} A_T = 3600 > 400 ft^2 ⇒ **Live Load reduction**	50 psf	0.5 x 50 = **25 psf** ≥ 0.50 Lo = 25 psf

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

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

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

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

Framing Members:
Interior Beam: W16x31
Spandrel beam: W21x50
Interior Girder:W24x68

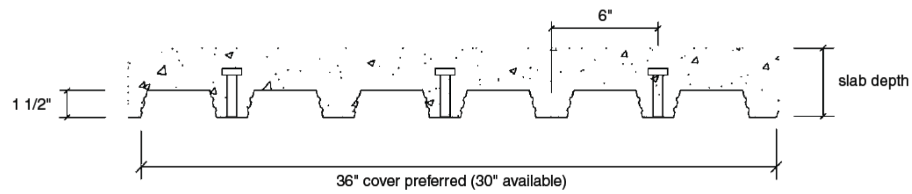
Floor Deck: see below

Assume Office occupancy, LL=50psf

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

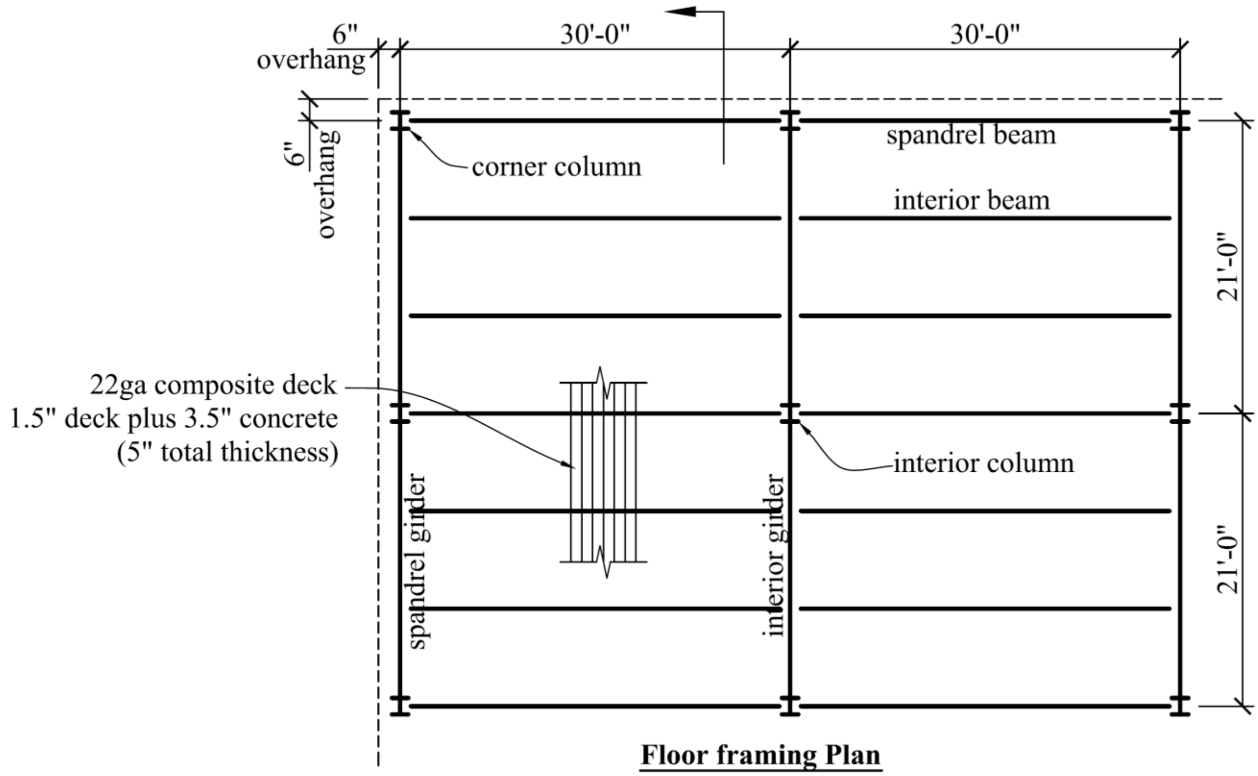
B-LOK 1.5" x 6" deck F_y = 40 ksi f'_c = 3 ksi 145 pcf concrete

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

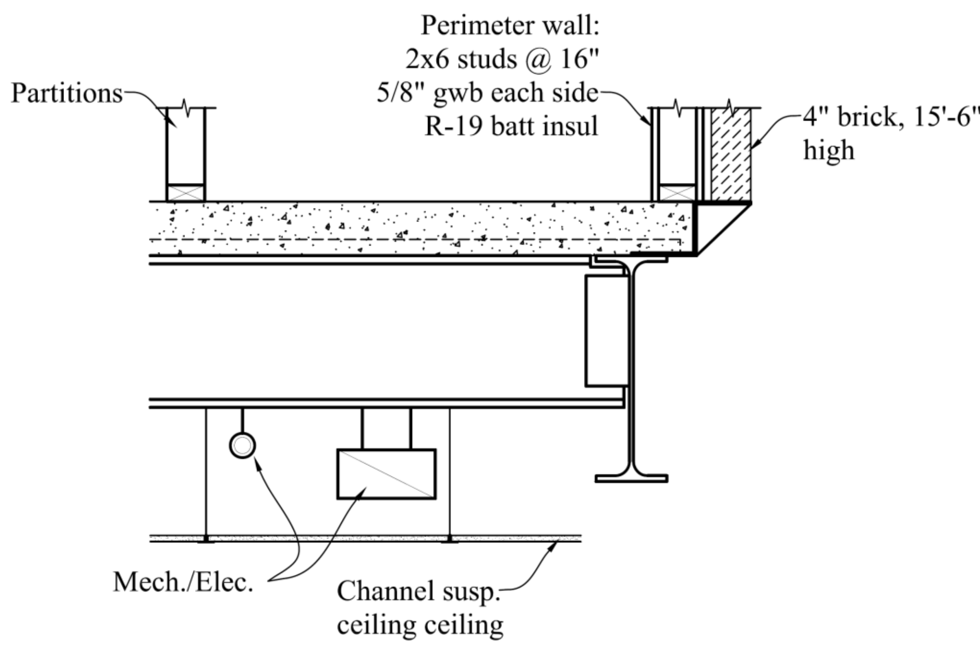


DECK PROPERTIES										
Gage	t	w	A_s	I_p	S_p	S_n	ϕR_{be}	ϕR_{bi}	ϕV_n	studs
22	0.0295	1.6	0.470	0.158	0.189	0.191	1290	1690	2830	0.52
20	0.0358	1.9	0.570	0.205	0.233	0.241	1830	2440	3420	0.63
19	0.0418	2.3	0.670	0.251	0.276	0.283	2420	3270	3980	0.74
18	0.0474	2.6	0.760	0.294	0.317	0.322	3040	4140	4500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

	COMPOSITE PROPERTIES											
	Slab Depth	ϕM_{nf} in.k	A_c in²	Vol. ft³/ft²	W psf	S_c in³	I_{av} in⁴	ϕM_{no} in.k	ϕV_{nt} lbs.	Max Unshored Span, ft. 1 span	2 span	3 span	A_{wwf} in²/ft
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

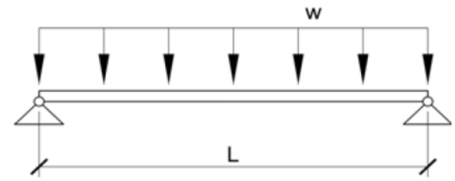


Floor framing Plan



Typical Floor Section

Dead Loads



$w_{deck} := 1.6psf$ $\quad H_{wall} := 15.5ft$

$w_{conc} := 49psf$ $\quad w_{studs} := 1.4psf$

$w_{part} := 15psf$ $\quad w_{ins} := 3psf$

$w_{ME} := 5psf$ $\quad w_{gwb} := 2 \cdot (5psf) \cdot 0.625 = 6.25\,psf$

$w_{clg} := 2psf$ $\quad w_{brick} := 40psf$

$LL := 50psf$

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

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

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

Loads to Interior Beam:

$L_{IB} := 30ft$

$TW_{IB} := 7ft$

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

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

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

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

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

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

Loads to Spandrel Beam:

$L_{SB} := 30ft$

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

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

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

Part (c)

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

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

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

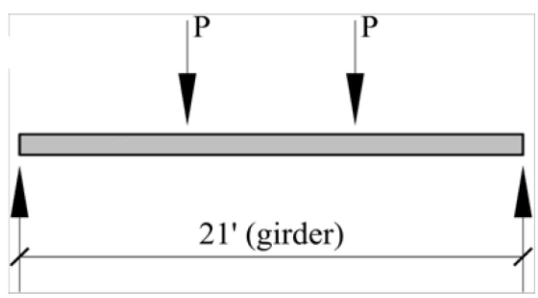
Part (e)

$V_{uSB} := \frac{w_{uSB} \cdot L_{SB}}{2} = 25.1 \cdot kips$

Load to Interior Girder:

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

$L_G := 21ft$



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

Part (g)

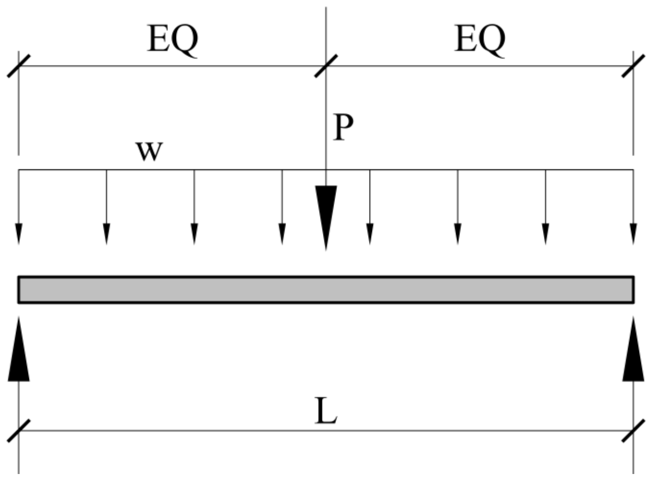
$M_{uG} := \frac{P_{uG} \cdot L_G}{3} = 253.5\,ft \cdot kips$

Problem 2-12

Given Loads:

<u>Uniform load, w</u>
D = 500plf
L = 800plf
S = 600plf
Beam length = 25 ft.

<u>Concentrated Load, P</u>
D = 11k
S = 15k
W = +12k or -12k
E = +8k or - 8k



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

Uniform Loads **Conccentrated Loads** $L_B := 25ft$

$w_D := 500plf$ $P_D := 11kips$

$w_L := 800plf$ $P_S := 15kips$

$w_S := 600plf$ $P_W := 12kips$ $P_{Wup} := -12kips$

$P_E := 8kips$ $P_{Eup} := -8kips$

$$M_D := \frac{w_D \cdot L_B^2}{8} + \frac{P_D \cdot L_B}{4} = 108\,ft \cdot kips \qquad M_W := \frac{P_W \cdot L_B}{4} = 75\,ft \cdot kips \qquad M_{Wup} := \frac{P_{Wup} \cdot L_B}{4} = -75\,ft \cdot kips$$

$$M_L := \frac{w_L \cdot L_B^2}{8} = 62\,ft \cdot kips \qquad M_E := \frac{P_E \cdot L_B}{4} = 50\,ft \cdot kips \qquad M_{Eup} := \frac{P_{Eup} \cdot L_B}{4} = -50\,ft \cdot kips$$

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

$$LC1 := (1.4 \cdot M_D) = 151\,ft \cdot kips$$

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

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

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

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

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

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

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

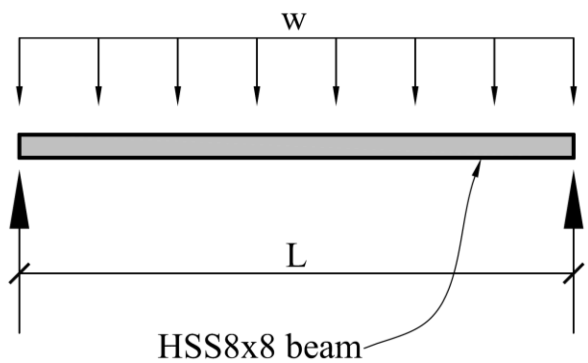
$$M_{max} := \max(LC1, LC2, LC3a, LC3b, LC4, LC5) = 414\,ft \cdot kips$$

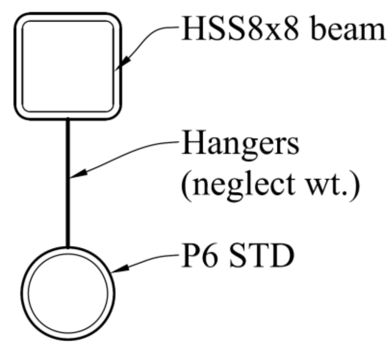
$$M_{maxUp} := \min(LC6, LC7) = -23\,ft \cdot kips$$

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





Dead Loads

$w_{8x8} := 37.61plf$

$w_{P6} := 19plf$

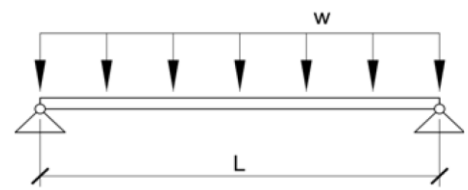
$O_{dia} := 6.63in$

$I_{dia} := 6.07in$

$\gamma_{ice} := 56pcf$

$\gamma_{water} := 62.4pcf$

$t_{ice} := 0.625in$



$L_B := 28ft$

$A_{ice8x8} := t_{ice} \cdot (4)(8in + 0.625in) = 21.563 \cdot in^2$

$$A_{iceP6} := \pi \cdot \frac{\left[\left[O_{dia} + (2 \cdot t_{ice})\right]^2 - O_{dia}^2\right]}{4} = 14.245 \cdot in^2$$

$w_{ice8x8} := \gamma_{ice} \cdot A_{ice8x8} = 8.385 \cdot plf$

$w_{iceP6} := \gamma_{ice} \cdot A_{iceP6} = 5.54 \cdot plf$

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

$w_{total} := w_{ice8x8} + w_{iceP6} + w_{water} + w_{8x8} + w_{P6} = 83.075 \cdot plf$

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

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

Uniform Loads **Conccentrated Loads** $L_B := 25ft$

$w_D := 500plf$ $P_D := 11kips$

$w_L := 800plf$ $P_S := 15kips$

$w_S := 600plf$ $P_W := 12kips$ $P_{Wup} := -12kips$

$P_E := 8kips$ $P_{Eup} := -8kips$

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

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

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

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

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

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

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

$$LC1 := \left(1.4 \cdot M_D\right) = 151\,ft \cdot kips$$

$$LC2 := \left(1.2 \cdot M_D\right) + \left(1.6 \cdot M_L\right) + \left(0.5 \cdot M_S\right) = 300\,ft \cdot kips$$

$$LC3a := \left(1.2 \cdot M_D\right) + \left(0.8 \cdot M_L\right) + \left(1.6 \cdot M_S\right) = 404\,ft \cdot kips$$

$$LC3b := \left(1.2 \cdot M_D\right) + \left(0.8 \cdot M_W\right) + \left(1.6 \cdot M_S\right) = 414\,ft \cdot kips$$

$$LC4 := \left(1.2 \cdot M_D\right) + \left(1.6 \cdot M_W\right) + \left(M_L\right) + \left(0.5 \cdot M_S\right) = 382\,ft \cdot kips$$

$$LC5 := \left(1.2 \cdot M_D\right) + \left(M_E\right) + \left(M_L\right) + \left(0.2 \cdot M_S\right) = 270\,ft \cdot kips$$

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

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

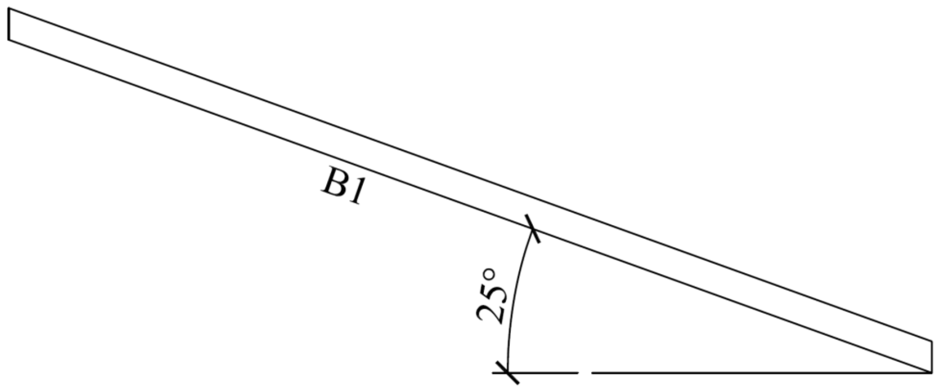
$$M_{max} := \max(LC1, LC2, LC3a, LC3b, LC4, LC5) = 414\,ft \cdot kips$$

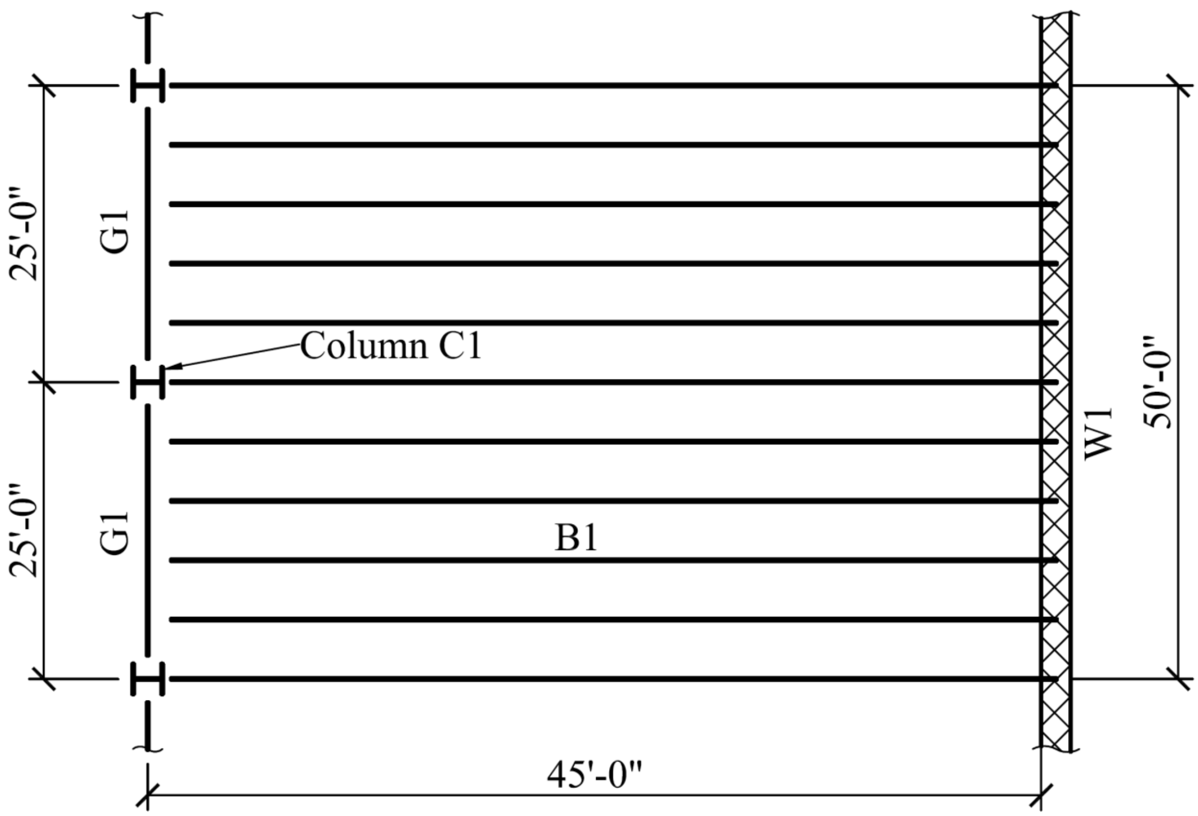
$$M_{maxUp} := \min(LC6, LC7) = -23\,ft \cdot kips$$

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.)





Roof Framing Plan

$Slope := 25$ $F := 12 \cdot \tan\left(Slope \cdot \frac{\pi}{180}\right) = 5.596$ $L_B := 45ft$ $TW_B := 5ft$ $D := 25psf$

$L_G := 25ft$ $TW_W := 2 \cdot L_G = 50\,ft$

$R_2 := 1.2 - (0.05 \cdot F) = 0.92$

Part (a):

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

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

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

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

Part (b):

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

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

Part (c):

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

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

Part (d):

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

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

Part (e):

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

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

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

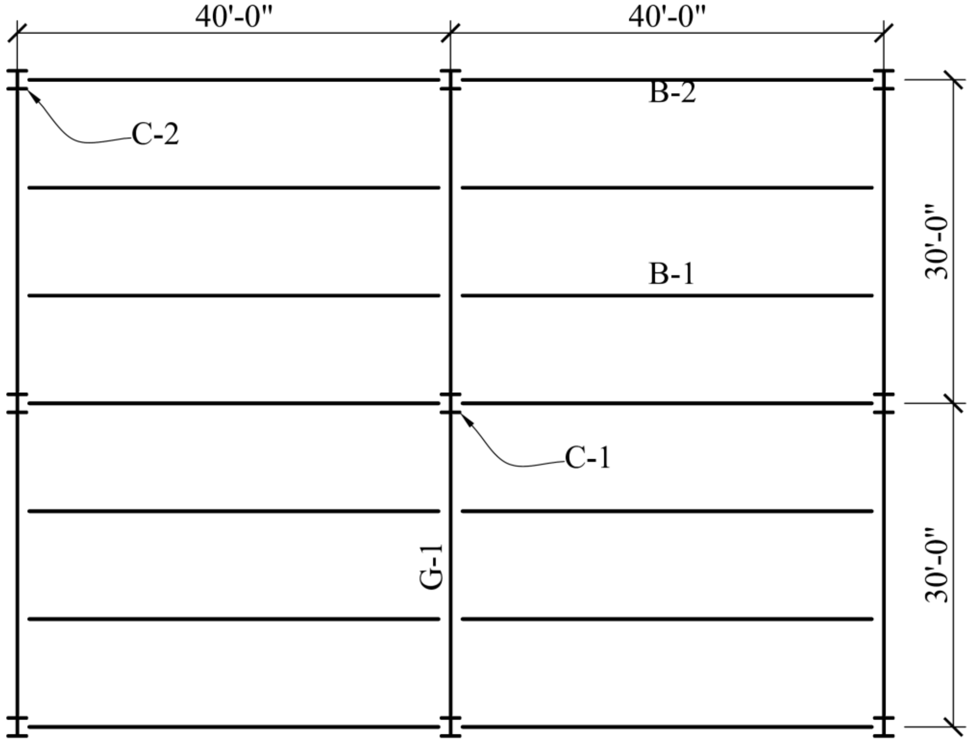
L1 = 25 ft
L2 = 25 ft

Level	TA ($ft.^2$)	D (psf)	S (psf)	L (psf)	wu1 (psf)	wu2 (psf)	Pu1 (kips)	Pu2 (kips)	Cumulative Pu1 (kips)	Cumulative Pu2 (kips)	Max. Load (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 = 60psf. 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

$L_B := 40ft \quad L_G := 30ft \quad TW_B := 10ft \quad L_o := 60psf$

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

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

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

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

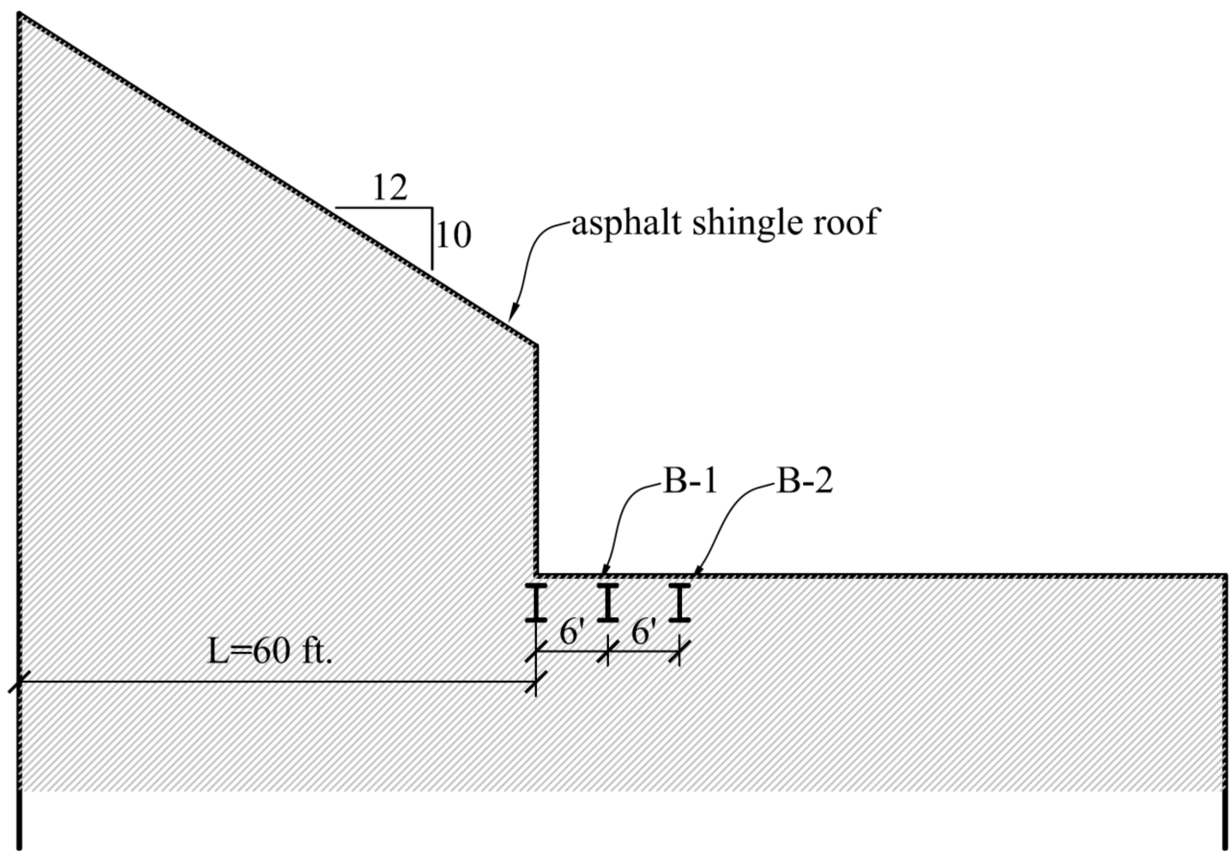
$TA_{C2} := \frac{L_G}{2} \cdot \frac{L_B}{2} = 300\,ft^2 \qquad K_{LLC2} := 2 \qquad L_{C2} := \max\left[(0.5 \cdot L_o), L_o \cdot \left(0.25 + \frac{15}{\sqrt{\frac{K_{LLC2} \cdot TA_{C2}}{1ft^2}}}\right)\right] = 51.7 \cdot psf$

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).

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



Problem 2-17

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

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

$P_f := 0.7p_g \cdot C_e \cdot C_t \cdot I_s = 42 \cdot psf \qquad P_s := P_f \cdot C_s = 32.848 \cdot psf$ ***part (a)***

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

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

$h_{bal} := \frac{P_f}{\gamma_{snow}} = 1.927\,ft \qquad h_{SL} := \frac{P_{SL}}{\gamma_{snow}} = 3.083\,ft$ ***part (c)***

$L_B := 30ft \qquad TW := 6ft \qquad D := 25psf$

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

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

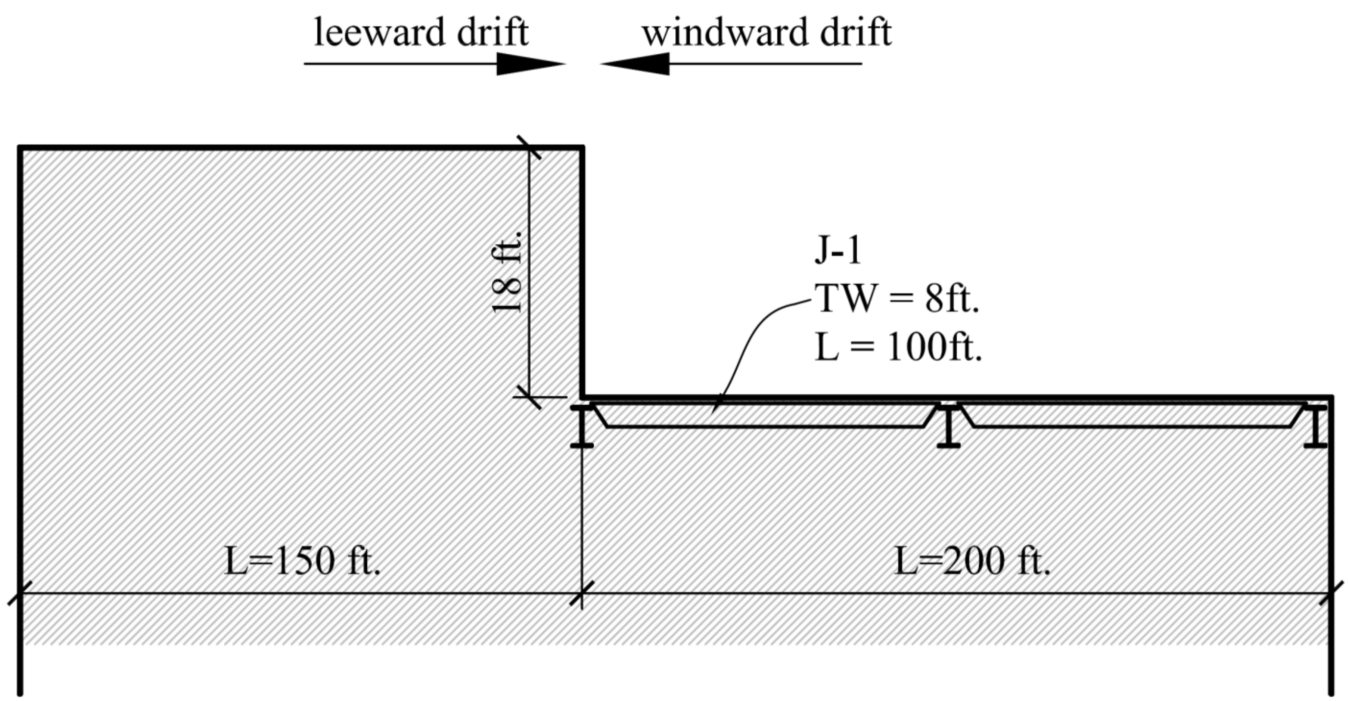
$M_u := \frac{w_u \cdot L_B^2}{8} = 138.2 \cdot ft \cdot kips \qquad V_u := \frac{w_u \cdot L_B}{2} = 18.4 \cdot kips$ ***part (e)***

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).

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



Problem 2-18

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

$P_f := 0.7 p_g \cdot C_e \cdot C_t \cdot I_s = 56 \cdot psf$ ***part (a)***

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

part (b)

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

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

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

The Leeward drift will control the design

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

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

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

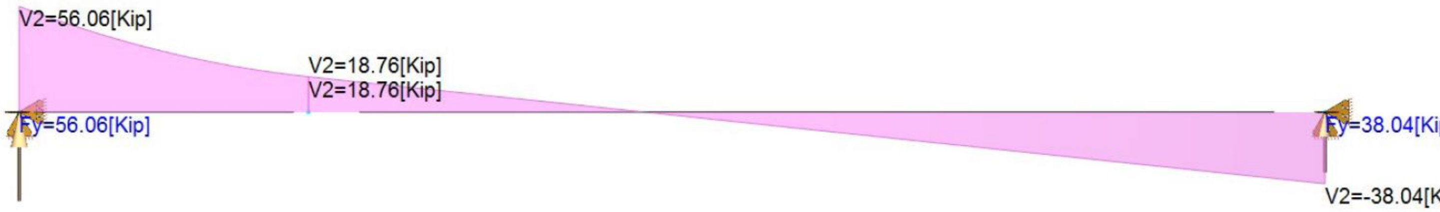
$w_u := (1.2 \cdot w_D) + [1.6 \cdot (w_S + w_{SD})] = 2638 \cdot plf$ ***part (d)***

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

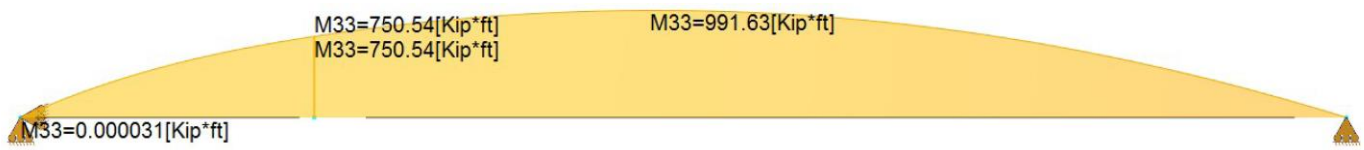
Factored - Dead Load; ws = 160plf

Factored - Snow Load; ws = 448plf, wSD = 1081plf

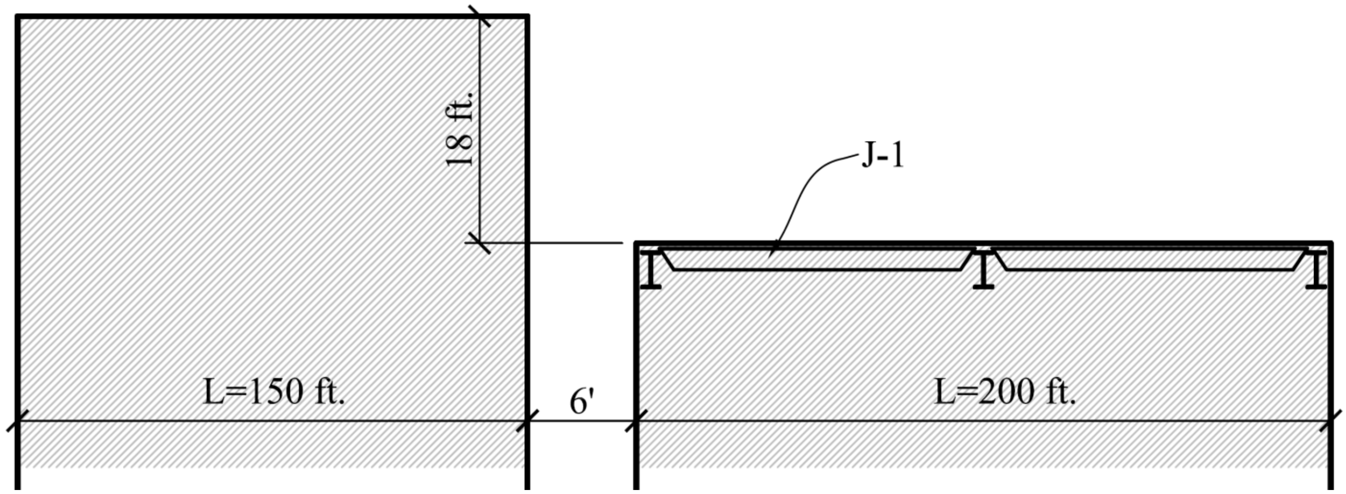
Factored - Shear



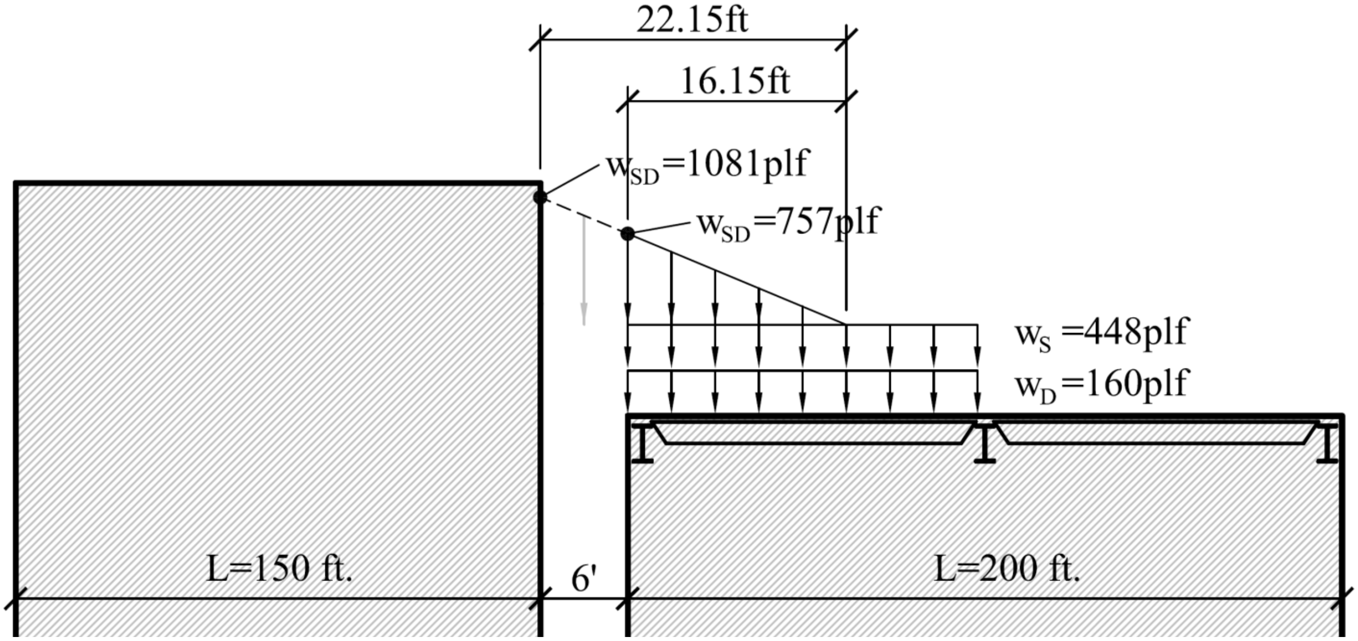
Factored - Moment



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.



$S := 6ft \qquad SD_6 := \dfrac{\left(20 - \dfrac{S}{1ft}\right)}{20} \cdot w_{SD} = 757\,plf$



At elevation 1500, pg = 70psf + (2)(1500-1000)/100 = 80psf (Pottersville)