

Problem 24 **Manual-structural-steel-design-a-practice-oriented-approach-1e-aghayere**

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

1: $p_u = 1.4D = 1.4 (29) = 40.6 \text{ psf}$

2: $p_u = 1.2 D + 1.6 L + 0.5 S$
 $= 1.2 (29) + 1.6 (0) + 0.5 (35) = 52.3 \text{ psf}$

3 (a): $p_u = 1.2D + 1.6S + 0.8W$
 $= 1.2 (29) + 1.6 (35) + 0.8 (15) = \mathbf{102.8 \text{ psf}}$ (governs)

3 (b): $p_u = 1.2D + 1.6S + 0.5L$
 $= 1.2 (29) + 1.6 (35) + (0) = 90.8 \text{ psf}$

4: $p_u = 1.2 D + 1.6 W + L + 0.5S$
 $= 1.2 (29) + 1.6 (15) + (0) + 0.5 (35) = 76.3 \text{ psf}$

5: $p_u = 1.2 D + 1.0 E + 0.5L + 0.2S$
 $= 1.2 (29) + 1.0 (0) + 0.5(0) + 0.2 (35) = 41.8 \text{ psf}$

6: $p_u = 0.9D + 1.6W$ (**D must** always oppose **W** in load combinations 6 and 7)
 $= 0.9 (29) + 1.6(-25)$ (*upward wind load is taken as negative*)
 $= -13.9 \text{ psf}$ (*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*)
 $= 26.1 \text{ psf}$ (*no net uplift*)

$w_u = (102.8\text{psf})(6\text{ft}) = \mathbf{616.8 \text{ plf}}$ (*downward*)

$w_u = (-13.9\text{psf})(6\text{ft}) = \mathbf{-83.4 \text{ plf}}$ (*upward*)

downward	uplift
$V_u = \frac{w_u L}{2} = \frac{(616.8)(30)}{2} = 9252 \text{ lb.}$	$V_u = \frac{w_u L}{2} = \frac{(-83.4)(30)}{2} = 1251 \text{ lb.}$
$M_u = \frac{w_u L^2}{8} = \frac{(616.8)(30)^2}{8} = 69.4 \text{ ft-kips}$	$M_u = \frac{w_u L^2}{8} = \frac{(-83.4)(30)^2}{8} = 9.4 \text{ ft-kips}$

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 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	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, $D_{\text{roof}} = 20 \text{ psf}$

Snow Load, $S = 40 \text{ psf}$

2nd and 3rd Floor Loads:

Dead Load, $D_{\text{floor}} = 40 \text{ psf}$

Floor Live Load, $L = 50 \text{ psf}$

Member	$A_T \text{ (ft.}^2\text{)}$	K_{LL}	$L_o \text{ (psf)}$	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	$(18)(18) = 324 \text{ ft}^2$	4	40 psf	$\left[0.25 + \frac{15}{\sqrt{(4)(324)}} \right] = 0.667$	$(0.667)(50) = \mathbf{34 \text{ psf}}$ $\geq 0.50 L_o = 25 \text{ psf}$
Ground Flr.	$2 \text{ floors} \times (18)(18) = 648 \text{ ft}^2$	4	40 psf	$\left[0.25 + \frac{15}{\sqrt{(4)(648)}} \right] = 0.545$	$(0.545)(50) = \mathbf{28 \text{ psf}}$ $\geq 0.40 L_o = 20 \text{ psf}$

Level	TA (ft ²)	D (psf)	Live Load L _o (S or L _r or R) psf	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	40	44	88	14.3 or 28.5	14.3	28.5	28.5
3 rd Flr	324	40	50	34	102.4	65	33.2 or 21.1	47.5	49.6	49.6
2 nd Flr	324	40	50	28	92.8	62	30.1 or 20.1	77.5	69.7	77.5
Without Floor Live Load Reduction										
Roof	324	20	40	40	44	88	14.3 or 28.5	14.3	28.5	28.5
3 rd Flr	324	40	50	50	128	73	41.5 or 23.7	55.8	52.2	55.8
2 nd Flr	324	40	50	50	128	73	41.5 or 23.7	97.2	75.9	97.2

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

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

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

Part (b):

dead load on interior beam:

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

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

dead load on interior girder:

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

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

Part (d): 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:

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

Design Load on Girder:

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

Part (e):

Beam:
$$V_u = \frac{w_u L}{2} = \frac{(515)(35)}{2} = \mathbf{54\text{ kips}}$$

$$M_u = \frac{w_u L^2}{8} = \frac{(515)(35)^2}{8} = \mathbf{473\text{ ft-kips}}$$

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

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

(i) **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

$$C_e = 1.0 \text{ (ASCE 7 Table 7-2)}$$

Roof slope, $\theta = \arctan(6/12) = 27$ degrees

Slope factor,

$$C_s = 1.0 \text{ (ASCE 7 Fig 7-2)}$$

Temperature factor,

$$C_t = 1.0 \text{ (ASCE 7 Table 7-3)}$$

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

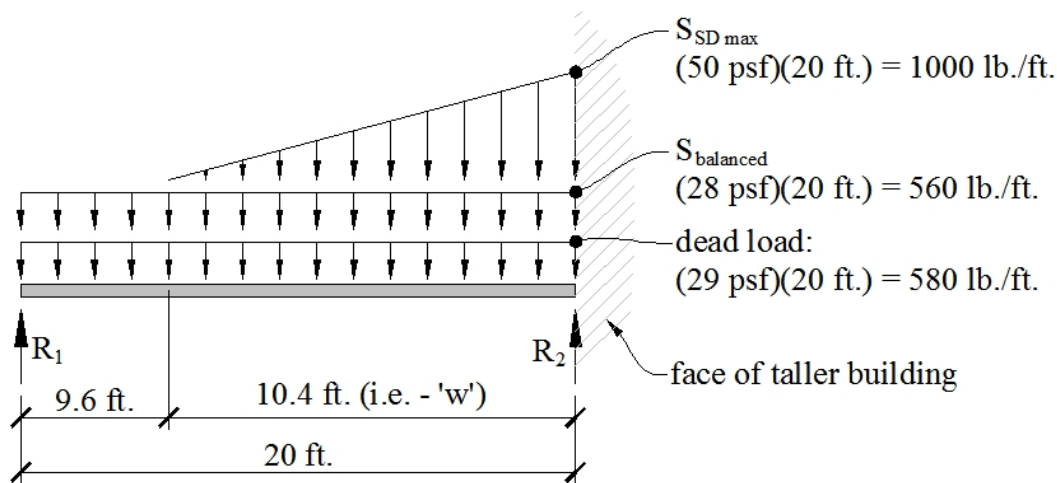
$$M_u = w_u L^2/8 = (517 \text{ lb/ft}) \times (20 \text{ ft})^2/8 = \mathbf{25.9 \text{ ft-kips}}$$

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

(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'

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

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

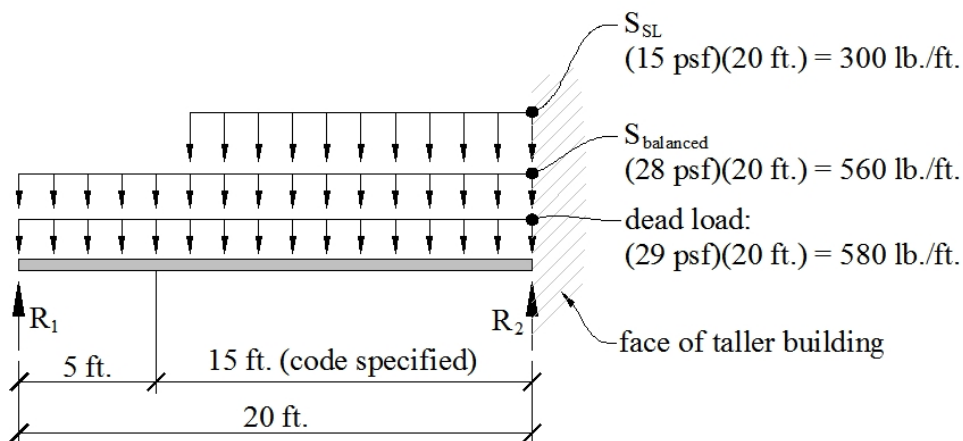
$$= 9899 \text{ lb} = \mathbf{9.9 \text{ kips}}$$

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

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

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

(2) Sliding snow on typical interior girder



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

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

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

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

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

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

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

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

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

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

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 ft²

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, L _o (psf)	Design live load*, L
8th floor Column (i.e. column below roof)	Roof only	Floor live load reduction NOT applicable to roofs!!!	-	40 psf (snow)	40 psf (snow)
7th 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 ft²	4 K _{LL} A _T = 3600 > 400 ft ² ⇒ Live Load reduction allowed	50 psf	0.5 x 50 = 25 psf ≥ 0.50 L _o = 25 psf

6th floor column (i.e. column below 7 th floor)	2 floors + roof (i.e. supports the roof, 8 th and 7 th floors)	2 floors x 900 ft ² = 1800 ft²	4 $K_{LL} A_T = 7200 > 400 \text{ ft}^2 \Rightarrow$ Live Load reduction allowed	50 psf	$0.43 \times 50 =$ 22 psf $\geq 0.40 \text{ Lo} =$ 20 psf
5th floor column (i.e. column below 6 th floor)	3 floors + roof (i.e. supports the roof, 8 th , 7 th and 6 th floors)	3 floors x 900 ft ² = 2700 ft²	4 $K_{LL} A_T = 10800 > 400 \text{ ft}^2 \Rightarrow$ Live Load reduction allowed	50 psf	$0.394 \times 50 =$ 20 psf $\geq 0.40 \text{ Lo} =$ 20 psf
4th floor column (i.e. column below 5 th floor)	4 floors + roof (i.e. supports the roof, 8 th , 7 th , 6 th and 5 th floors)	4 floor x 900 ft ² = 3600 ft²	4 $K_{LL} A_T = 14400 > 400 \text{ ft}^2 \Rightarrow$ Live Load reduction allowed	50 psf	$0.375 \times 50 = 19$ psf $\geq 0.40 \text{ Lo} =$ 20 psf
3rd floor column (i.e. column below 4 th floor)	5 floors + roof (i.e. supports the roof, 8 th , 7 th , 6 th , 5 th and 4 th floors)	5 floor x 900 ft ² = 4500 ft²	4 $K_{LL} A_T = 18000 > 400 \text{ ft}^2 \Rightarrow$ Live Load reduction allowed	50 psf	$0.362 \times 50 = 18$ psf $\geq 0.40 \text{ Lo} =$ 20 psf
2nd floor column (i.e. column below 3 rd floor)	6 floors + roof (i.e. supports the roof, 8 th , 7 th , 6 th , 5 th , 4 th and 3 rd floors)	6 floor x 900 ft ² = 5400 ft²	4 $K_{LL} A_T = 21600 > 400 \text{ ft}^2 \Rightarrow$ Live Load reduction allowed	50 psf	$0.352 \times 50 = 18$ psf $\geq 0.40 \text{ Lo} =$ 20 psf
Ground or 1st floor column (i.e. column below 2 nd floor)	7 floors + roof (i.e. supports the roof, 8 th , 7 th , 6 th , 5 th , 4 th , 3 rd and 2 nd floors)	7 floors x 900 ft ² = 6300 ft²	4 $K_{LL} A_T = 25200 > 400 \text{ ft}^2 \Rightarrow$ Live Load reduction allowed	50 psf	$0.344 \times 50 = 17.3$ psf $\geq 0.40 \text{ Lo} =$ 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 ft² 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	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	40	116.0	160.0	104.4 or 144.0	104.4	144.0	144.0
8 th Flr	900	120	50	25	184.0	156.5	165.6 or 140.9	270	284.9	284.9
7 th Flr	900	120	50	22	179.2	155.0	161.3 or 139.5	431.3	424.4	431.3
6 th Flr	900	120	50	20	176.0	154.0	158.4 or 138.6	589.7	563.0	589.7
5 th Flr	900	120	50	20	176.0	154.0	158.4 or 138.6	748.1	701.6	748.1
4 th Flr	900	120	50	20	176.0	154.0	158.4 or 138.6	906.5	840.2	906.5
3 rd Flr	900	120	50	20	176.0	154.0	158.4 or 138.6	1064.9	978.8	1064.9
2 nd Flr	900	120	50	20	176.0	154.0	158.4 or 138.6	1223.3	1117.4	1223.3
(a) Without Floor Live Load Reduction										
Roof	900	80	40	40	116	160	104.4 or 144.0	104.4	144.0	144.0
8 th Flr	900	120	50	50	224	169	201.6 or 152.1	306.0	296.1	306.0
7 th Flr	900	120	50	50	224	169	201.6 or 152.1	507.6	448.2	507.6
6 th Flr	900	120	50	50	224	169	201.6 or 152.1	709.2	600.3	709.2
5 th Flr	900	120	50	50	224	169	201.6 or 152.1	910.8	752.4	910.8
4 th Flr	900	120	50	50	224	169	201.6 or 152.1	1112.4	904.5	1112.4
3 rd Flr	900	120	50	50	224	169	201.6 or 152.1	1314.0	1056.6	1314.0
2 nd Flr	900	120	50	50	224	169	201.6 or 152.1	1515.6	1208.7	1515.6