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 $= 1.2\ P_D + 1.6\ P_L + 0.5\ P_S$
$= 1.2\ (200) + 1.6\ (300) + 0.5\ (150) = 795$ kips (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 $= 1.2\ P_D + 1.6\ P_S + 0.8\ P_W$
$= 1.2\ (200) + 1.6\ (150) + 0.8\ (60) = 528$ kips

4: P_u $= 1.2\ P_D + 1.6\ P_W + 0.5\ P_L + 0.5\ P_S$
$= 1.2\ (200) + 1.6\ (60) + 0.5(300) + 0.5\ (150) = 561$ 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 $= 0.9\ P_D + 1.6\ P_W$
$= 0.9\ (200) + 1.6\ (-60) = 84$ 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)*

$\varphi P_n > P_u$

$\varphi_c = 0.9$

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

$P_n = 884$ 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 + 0.8W
= 1.2 (29) + 1.6 (35) + 0.8 (15) = 102.8 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 + 1.6 W + L + 0.5S
= 1.2 (29) + 1.6 (15) + (0) + 0.5 (35) = 76.3 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: pu = 0.9D + 1.6W (D <u>must</u> always oppose W in load combinations 6 and 7)
= 0.9 (29) + 1.6(-25) (*upward wind load is taken as negative*)
= -13.9 psf *(net uplift)*

7: p_u = 0.9D + 1.0E (D <u>must</u> 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= (102.8psf)(6ft) = 616.8 plf *(downward)*
w_u= (-13.9psf)(6ft) = -83.4 plf *(upward)*

downward	uplift
$V_u = \frac{w_u L}{2} = \frac{(616.8)(30)}{2}$ = 9252 lb.	$V_u = \frac{w_u L}{2} = \frac{(-83.4)(30)}{2}$ = 1251 lb.
$M_u = \frac{w_u L^2}{} = \frac{(616.8)(30)^2}{}$ = 69.4 ft-kips	$M_u = \frac{w_u L^2}{} = \frac{(-83.4)(30)^2}{}$ = 9.4 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	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
3rd Flr	324	40	50	34	102.4	65	33.2 or 21.1	47.5	49.6	49.6
2nd 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
3rd Flr	324	40	50	50	128	73	41.5 or 23.7	55.8	52.2	55.8
2nd 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 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 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

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

Part (b):

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)

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

1.4D = (1.4)(96) = 134.4psf
1.2D + 1.6L = (1.2)(96) + (1.6)(250)= 515psf Å controls

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

Design Load on Girder:
(515psf)(35 ft) = 18032 plf = 18.0 kips/ft

Part (e):

Beam:

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

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

Girder:

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

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

(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:

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 ≈ 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 ≈ 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

$H_d = \mu\ (0.43\ [L]^{1/3}\ [P_g + 10]^{¼} - 1.5)$

$= \quad 0.75\ (0.43\ [80]^{1/3}\ [40 + 10]^{¼} - 1.5) \quad = 2.6$ ft (governs)

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

$H_d = 1.0\ (0.43\ [40]^{1/3}\ [40 + 10]^{¼} - 1.5) \quad = 2.4$ ft

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

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

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:

$H_d = 2.8$ ft $\leq H_c = 13.54$ ft, therefore
$w = 4\ H_d = 4 \times 2.6$ ft $= 10.4$ ft (governs) $\leq 8\ H_c = 8 \times 13.54 = 108$ 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:

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

$SD_{average} = 31$ 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 i 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

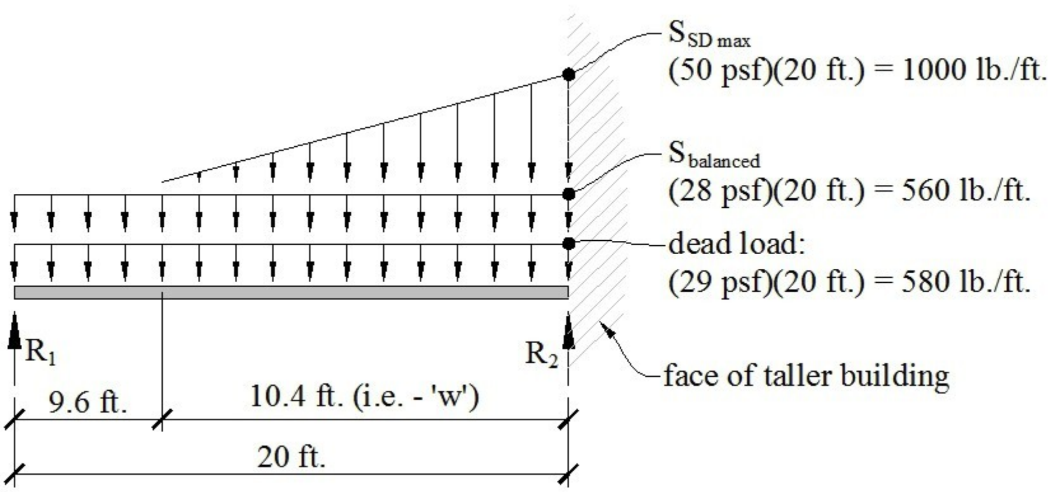
M_u = $w_u L^2/8$ = (517 lb/ft) x (20 ft)^2/8 = 25.9 ft-kips

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:

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

$R_{1\ L}$ = <u>560 lb/ft x (20') x (20'/2) + ½ x 1000 Ib/ft x 10.4' x (10.4'/3)</u>

20’

= 6501 lb = 6.5 kips

R_{2L} = 560 lb/ft x (20’) + ½ x 1000 Ib/ft x 10.4’ - R_{1LL}

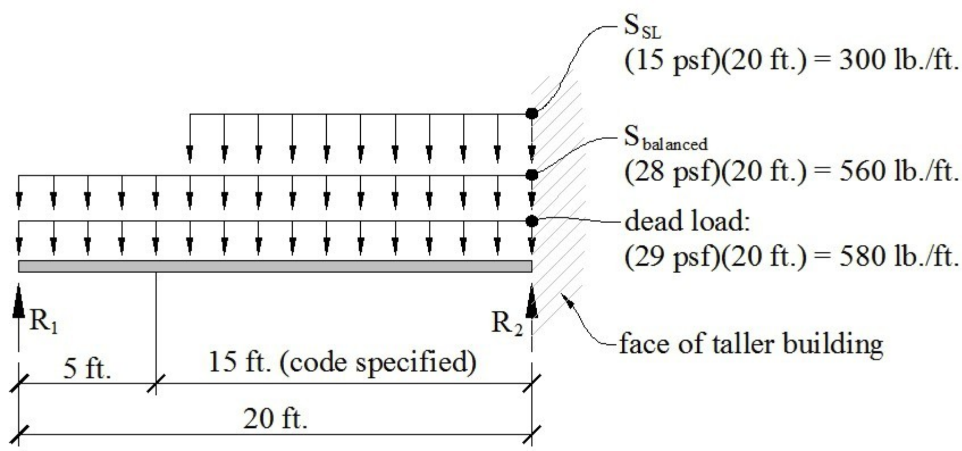
= 9899 Ib = 9.9 kips

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 x 5.8 kip + 1.6 x 6.5 kip = 17.4 kips

$R_{2u} = 1.2\,R_{2D} + 1.6\,R_{2L}$ = 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:

R_{1DL} = 580 Ib/ft x (20’/2) = 5800 Ib = 5.8 kips

R_{2DL} = 580 Ib/ft x (20’/2) = 5800 Ib = 5.8 kips

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

= 7288 Ib = 7.3 kips

R_{2LL} = 560 lb/ft x (20’) + 300 lb/ft x 15’ - R_{1LL}

= 8412 Ib = 8.4 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 x 5.8 kip + 1.6 x 7.3 kip = 18.6 kips

$R_{2u} = 1.2\,R_{2D} + 1.6\,R_{2L}$ = 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
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 8th floor)	1 floor + roof (i.e. supports the roof and the 8th floor)	1 floor x 900 ft^2 = 900 ft^2	4 K_{LL} A_T = 3600 > 400 ft^2 ⇒ Live Load reduction allowed	50 psf	0.5 x 50 = 25 psf ≥ 0.50 Lo = 25 psf

6^{th} 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 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
5^{th} 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 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
4^{th} 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 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
3^{rd} floor column (i.e. column below 4^{th} floor)	5 floors + roof (.e. supports the roof, 8^{th}, 7^{th}, 6^{th}, 5^{th} and 4^{th} 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
2^{nd} 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 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 1^{st} 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 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	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
8th Flr	900	120	50	25	184.0	156.5	165.6 or 140.9	270	284.9	284.9
7th Flr	900	120	50	22	179.2	155.0	161.3 or 139.5	431.3	424.4	431.3
6th Flr	900	120	50	20	176.0	154.0	158.4 or 138.6	589.7	563.0	589.7
5th Flr	900	120	50	20	176.0	154.0	158.4 or 138.6	748.1	701.6	748.1
4th Flr	900	120	50	20	176.0	154.0	158.4 or 138.6	906.5	840.2	906.5
3rd Flr	900	120	50	20	176.0	154.0	158.4 or 138.6	1064.9	978.8	1064.9
2nd 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
8th Flr	900	120	50	50	224	169	201.6 or 152.1	306.0	296.1	306.0
7th Flr	900	120	50	50	224	169	201.6 or 152.1	507.6	448.2	507.6
6th Flr	900	120	50	50	224	169	201.6 or 152.1	709.2	600.3	709.2
5th Flr	900	120	50	50	224	169	201.6 or 152.1	910.8	752.4	910.8
4th Flr	900	120	50	50	224	169	201.6 or 152.1	1112.4	904.5	1112.4
3rd Flr	900	120	50	50	224	169	201.6 or 152.1	1314.0	1056.6	1314.0
2nd Flr	900	120	50	50	224	169	201.6 or 152.1	1515.6	1208.7	1515.6