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 $= 1.2\ P_D + 1.6\ P_S + 0.5\ P_W$
$= 1.2\ (200) + 1.6\ (150) + 0.5\ (60) = 510$ 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)

□$P_n > P_u$

□$_c = 0.9$

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

$P_n = 884$ 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 <u>must</u> 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 <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 = (98.3psf)(6ft) = 590 plf (downward)

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

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 ó" 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)	□P LC 2 (kips)	□P LC 3 (kips)	Maximum □P (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

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

$\square_{DL}$ = 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)= 515psf $\Rightarrow$ 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