

/solution-manual-structural-steel-design-a-practice-oriented-approach-2e-aghayere-310

Problem 2-3

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

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

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

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

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

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

$$\square P_n > P_u$$

$$\square \phi_c = 0.9$$

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

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

Problem 2-4

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

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

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

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

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

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

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

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 6" 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
3 rd floor	N/A	-	-	-	40 psf (Snow load)
2 nd floor	$(18)(18) = 324 \text{ ft}^2$	4	40 psf	$0.25 + \frac{15}{\sqrt{(4)(324)}} = 0.667$	$(0.667)(50) = 34 \text{ psf}$ $\geq 0.50 L_o = 25 \text{ psf}$
Ground Flr.	$2 \text{ floors} \times (18)(18) = 648 \text{ ft}^2$	4	40 psf	$0.25 + \frac{15}{\sqrt{(4)(648)}} = 0.545$	$(0.545)(50) = 28 \text{ psf}$ $\geq 0.40 L_o = 20 \text{ psf}$

Level	TA (ft ²)	D (psf)	Live Load L _o (S or L _r or R) psf	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
3 rd Flr	324	40	50	0.666	33.3	101	65	32.8 or 21	47.1	49.5	49.5
2 nd 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
3 rd Flr	324	40	50	1	50	128	73	41.5 or 23.7	55.7	52.2	55.7
2 nd 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 (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

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

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

Part (b):

dead load on interior beam:

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

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

dead load on interior girder:

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

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

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

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

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

Design Load on Beam:

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

Part (d)

Design Load on Girder (assuming uniformly distributed load):

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

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

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

Part (d):

$$\begin{aligned} \text{Beam:} \quad V_u &= \frac{w_u L}{2} = \frac{(3.1)(35)}{2} = 54.3 \text{ kips} \\ M_u &= \frac{w_u L^2}{8} = \frac{(3.1)(35)^2}{8} = 474.7 \text{ ft-kips} \end{aligned}$$

$$\begin{aligned} \text{Girder:} \quad V_u &= \frac{w_u L}{2} = \frac{(18.0)(30)}{2} = 270 \text{ kips} \\ M_u &= \frac{w_u L^2}{8} = \frac{(18.0)(30)^2}{8} = 2025 \text{ ft-kips} \end{aligned}$$