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Problem 2-3

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

PD = 200 kips (dead load)

PL = 300 kips (floor live load)

PS = 150 kips (snow load)

PW = ±60 kips (wind load)

PE = ±40 kips (seismic load)

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

1: Pu = 1.4 PD = 1.4 (200k) = 280 kips

2: Pu = 1.2 PD + 1.6 PL + 0.5 PS

= 1.2 (200) + 1.6 (300) + 0.5 (150) = 795 kips (governs)

3 (a): Pu = 1.2 PD + 1.6 PS + 0.5PL

= 1.2 (200) + 1.6 (150) + 0.5(300) = 630 kips

3 (b): Pu = 1.2 PD + 1.6 PS + 0.5 PW

= 1.2 (200) + 1.6 (150) + 0.5 (60) = 510 kips

4: Pu = 1.2 PD + 1.0 PW + 0.5 PL + 0.5 PS

= 1.2 (200) + 1.0 (60) + 0.5(300) + 0.5 (150) = 525 kips

5: Pu = 1.2 PD + 1.0 PE + 0.5 PL + 0.2 PS

= 1.2 (200) + 1.0 (40) + 0.5 (300) + 0.2 (150) = 460 kips

Note that PD must always oppose PW and PE in load combination 6

6: Pu

7: Pu

Pn > Pu

c = 0.9

(0.9)(Pn) = (795 kips) Pn = 884 kips

= 0.9 PD + 1.0 PW

= 0.9 (200) +1.0 (-60) = 120 kips (no net uplift)

= 0.9 PD + 1.0 PE
= 0.9 (200) + 1.0 (-40) = 140 kips (no net uplift)

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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 > Lr = 20psf, use S in equations and ignore Lr.

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

2: pu = 1.2 D + 1.6 L + 0.5 S

= 1.2 (29) + 1.6 (0) + 0.5 (35) = 52.3 psf

3 (a): pu = 1.2D + 1.6S + 0.5W

= 1.2 (29) + 1.6 (35) + 0.5 (15) = 98.3 psf (governs)

3 (b): pu = 1.2D + 1.6S + 0.5L

= 1.2 (29) + 1.6 (35) + (0) = 90.8 psf

4: pu = 1.2 D + 1.0 W + L + 0.5S

= 1.2 (29) + 1.0 (15) + (0) + 0.5 (35) = 67.3 psf

5: pu = 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 + 1.0W (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: pu = 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)`

wu = (98.3psf)(6ft) = 590 plf (downward)

|  |  |
| --- | --- |
| downward | No net uplift |
| w L (590)(30)uV   = 8850 lb.u2 2 |  |
| 2 2w L (590)(30)uM   = 66375 ft-Ibu8 8= 66.4 ft-kips |  |

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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 (AT)

Interior Beam 24 ft/4 spaces = 6 ft 6 ft x 32 ft = 192 ft2

Spandrel Beam (24 ft/4 spaces)/2 + 0.75’ 3.75 ft x 32 ft = 120 ft2

= 3.75 ft

Interior Girder 32 ft/ 2 + 32 ft/2 = 32 ft 32 ft x 24 ft = 768 ft2

Spandrel Girder 32 ft/2 + 0.75 ft = 16.75 ft 16.75 ft x 24 ft = 402 ft2

Interior Column - 32 ft x 24 ft = 768 ft2

Corner Column - (32 ft/2 + 0.75)(24 ft/2 + 0.75) ft = 214 ft2

R2 = 1.0 (flat roof)

Member R1 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)(20) = 15.96psf

= 0.798

Interior Column 0.6 (0.6)(20) = 12psf

Corner Column 1.2-0.001(214) (0.798)(20) = 19.72psf

= 0.986

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Member pu = 1.2D+1.6Lr wu (plf) Pu (kips)

Interior Beam (1.2)(30)+(1.6)(20) = (68psf)(6ft) = 408plf -

68psf

Spandrel Beam (1.2)(30)+(1.6)(20) = (68psf)(3.75ft) = -

68psf 255plf

Interior Girder (1.2)(30)+(1.6)(12) = - (55.2psf)(6ft)(32ft) = 10.6

55.2psf kips

Spandrel Girder (1.2)(30)+(1.6)(15.96) - (61.5psf)(6ft)(32/2ft) = 5.9

= 61.5psf kips

Interior Column (1.2)(30)+(1.6)(12) = - (55.2psf)(768ft2) = 42.4 kips

55.2psf

Corner Column (1.2)(30)+(1.6)(19.72) - (67.6psf)(214ft2) = 14.5 kips

= 67.6psf

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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, Droof = 20 psf Dead Load, Dfloor = 40 psf

Snow Load, S = 40 psf Floor Live Load, L = 50 psf

|  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- |
| Member | AT (ft.2) | KLL | Lo (psf) | Live Load Red. Factor0.25 + 15/(KLL AT) | Design live load, Lor S |
| 3rd floor | N/A | - | - | - | 40 psf(Snow load) |
| 2nd floor | (18)(18) =324 ft2 | 4 | 40 psf |  150.25 = 0.667 (4)(324) | (0.667)(50)= 34 psf≥ 0.50 Lo = 25 psf |
| GroundFlr. | 2 floors x(18)(18) =648 ft2 | 4 | 40 psf |  150.25 = 0.545 (4)(648) | (0.545)(50)= 28 psf≥ 0.40 Lo = 20 psf |

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|  |  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
|  |  |  |  |  |  | wu1(LC 2) | wu2(LC 3) |  |  |  |  |
|  | 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 |

Level

TA (ft2 )

D (psf)

Live Load

Lo (S or Lr or R) psf

LLredF

Design Live (psf)

Floor: L

Roof: S or Lr or R

Roof: 1.2D +0.5S (psf)

Floor: 1.2D + 1.6L(psf)

Roof: 1.2D + 1.6S (psf)

Floor: 1.2D + 0.5L (psf)

Pu = (TA)(wu1) or

(TA)(wu2) (kips)

P

LC 2 (kips)

P

LC 3 (kips)

Maximum P (kips)

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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, Vu and the factored moment, Mu 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

\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_

DL = 116psf (with partitions)

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 (c): Heavy Mfr.: Live Load = 250psf

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1.4D = (1.4)(96) = 134.4psf

1.2D + 1.6L = (1.2)(96) + (1.6)(250)= 515psf  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):

w L (3.1)(35)

u

Beam: V

u



2

w

u

 = 54.3 kips

2

2 2

L (3.1)(35)

M

  = 474.7 ft-kips

u

8 8

w L (18.0)(30)

Girder:

V  u

u

 = 270 kips

2

w

2

2 2

L (18.0)(30)

M  u

u



= 2025 ft-kips

8 8