Instructor's Manual

for

Structural Steel Design: A Practice - Oriented Approach

Second Edition

Abi O. Aghayere
Jason Vigil

PEARSON

Boston Columbus Indianapolis New York San Francisco Hoboken

Amsterdam Cape Town Dubai London Madrid Milan Munich Paris Montreal Toronto

Delhi Mexico City Sao Paulo Sydney Hong Kong Seoul Singapore Taipei Tokyo

This work is protected by United States copyright laws and is provided solely for the use of instructors in teaching their courses and assessing student learning. Dissemination or sale of any part of this work (including on the World Wide Web) will destroy the integrity of the work and is not permitted. The work and materials from it should never be made available to students except by instructors using the accompanying text in their classes. All recipients of this work are expected to abide by these restrictions and to honor the intended pedagogical purposes and the needs of other instructors who rely on these materials.

Copyright © 2015 Pearson Education, Inc., publishing as Prentice Hall, Hoboken, New Jersey and Columbus, Ohio. All rights reserved. Manufactured in the United States of America. This publication is protected by Copyright, and permission should be obtained from the publisher prior to any prohibited reproduction, storage in a retrieval system, or transmission in any form or by any means, electronic, mechanical, photocopying, recording, or likewise. To obtain permission(s) to use material from this work, please submit a written request to Pearson Education, Inc., Permissions Department, 221 River Street, Hoboken, New Jersey.

Many of the designations by manufacturers and seller to distinguish their products are claimed as trademarks. Where those designations appear in this book, and the publisher was aware of a trademark claim, the designations have been printed in initial caps or all caps.

10 9 8 7 6 5 4 3 2 1

PEARSON

ISBN-13: 978-0-13-341883-5
ISBN-10: 0-13-341883-9

TABLE OF CONTENTS

Chapter 1	1
Chapter 2	9
Chapter 3	45
Chapter 4	72
Chapter 5	94
Chapter 6	108
Chapter 7	135
Chapter 8	151
Chapter 9	176
Chapter 10	194
Chapter 11	209
Chapter 12	229
Appendix B	248

Problem 1-4

The size and cross-sectional areas are obtained from Part 1 of the AISCM as follows:

Size	**Self-weight (lb/ft.)**	**Cross-sectional area (in^2)**
W14x22	22	6.49
W21x44	44	13.0
HSS 6x6x½	35.11	9.74
L6x4x½	16.2	4.75
C12x30	30	8.81
WT18x128	128	37.7

Problem 1-5

a)

Element	**A**	**y**	**Ay**	**I**	**d = y- $\bar{y}$**	**I + Ad^2**
top flange	21	26.25	551.25	3.94	-12.75	3418
web	21	13.5	283.5	1008	0	1008
bot flange	21	0.75	15.75	3.94	12.75	3418
Σ =	**63 in.^{2}**		**850.5**			**I = 7844 in.^{4}**

$$\bar{y} = \frac{\Sigma Ay}{\Sigma A} = \frac{850.5}{63} = 13.5 \text{ in.}$$

Self weight = (63/144)(490 lb/ft^3) = 214 lb/ft.

b)

Element	**A**	**y**	**Ay**	**I**	**d = y- $\bar{y}$**	**I + Ad^2**
top plate	2.63	18.26	47.93	0.03	-9.04	214.3
beam	10.3	9.23	95.02	510	0	510
bot plate	2.63	0.188	0.49	0.03	9.04	214.3
Σ =	**15.55 in.^{2}**		**143.4**			**I = 939 in.^{4}**

$$\bar{y} = \frac{\Sigma Ay}{\Sigma A} = \frac{143.4}{15.55} = 9.23 \text{ in.}$$

Self weight = (15.55/144)(490 lb/ft^3) = 52.9 lb/ft.

c) From AISCM Table 1-20, I_x = 314 in.^{4}
Area = 13.8 in^2
Self weight = 47.1 lb/ft.

Problem 1-7

Determine the most economical layout of the roof framing (joists and girders) and the gage (thickness) of the roof deck for a building with a 25 ft x 35 ft typical bay size. The total roof dead load is 25 psf and the snow load is 35 psf. Assume a 1½" deep galvanized wide rib deck and an estimated weight of roof framing of 6 psf.

*Assume beams (or joists) span the 35' direction
* Assume 3-span condition
*Total roof load = (25psf + 35psf) – 6psf = **54psf**

# of beam spaces	beam spacing (ft.)	Selected deck gage	max. constr. span	Deck Load capacity*	
2	12.5	none	-	-	
3	8.33	16	10'-3"	85psf	
4	**6.25**	**22**	**6'-11"**	**76psf**	← *select*
5	5	24	5'-10"	130psf	

*Vulcraft deck assumed

1-10 *Determine the most economical layout of the floor framing (beams and girders), the total depth of the floor slab, and the gage (thickness) of the floor deck for a building with a 30 ft x 47 ft typical bay size. The total floor dead load is 110 psf and the floor live load is 250 psf. Assume normal weight concrete, a 3" deep galvanized composite wide rib.*

*Assume beams span the 47' direction
* Assume 3-span condition
* Assume weight of the framing = 10psf
*Total floor load = (110psf +250psf) – 10psf = 350psf

t =2.5" *(superimposed load = 350psf – 50psf – 2psf) =* ***298psf****)*

# of beam spaces	beam spacing (ft.)	Selected deck gage	max. constr. span	Deck Load capacity*	
2	15	16	15'-5"	none	N.G.
3	10	16	15'-5"	218psf	N.G.
4	7.5	18	13'-11"	298psf	← *select*

t =3" *(superimposed load = 350psf – 57psf – 2psf) =* ***291psf****)*

# of beam spaces	beam spacing (ft.)	Selected deck gage	max. constr. span	Deck Load capacity*	
2	15	none	-	-	
3	10	16	14'-11"	245psf	N.G.
4	7.5	18	13'-4"	334psf	← *select*

*Vulcraft deck assumed

Problem 1-11

From Equation 1-1, the carbon content is

CE = 0.16 + (0.20 + 0.25)/15 + (0.10 + 0.15 + 0.06)/5 + (0.80 + 0.20)/6 = 0.419 < 0.5

Therefore, the steel member is weldable.

Problem 1-12

Anticipated expansion or contraction = (6.5 x 10^{-6} in./in.)(300 ft.)(12 in./ft.)(70 ^{o}F) = 1.64 in.

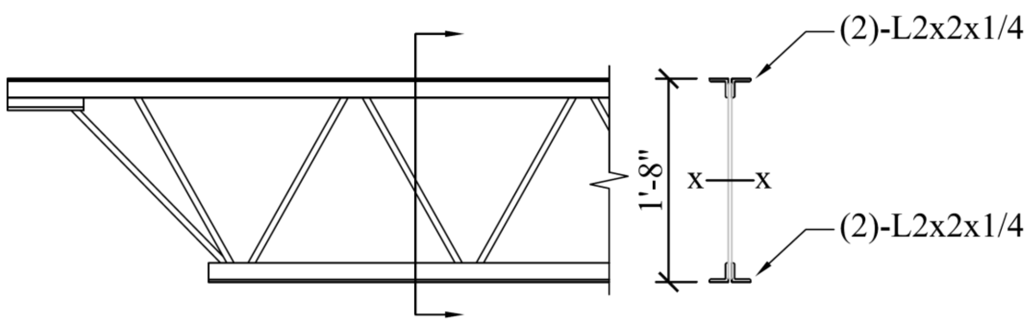
Expansion joint width = (2)(1.64 in.) = 3.28 in.

Therefore, use a 3¼ in. wide expansion joint.

The width of the required expansion joint appears large, and one way to reduce this width is to reduce the length between expansion joints from 300 ft to say 200 ft. That will bring the required expansion joint width down to (200/300)(3.28 in.) = 2.2 in. (i.e. 2¼ in. expansion joint)

Problems 1-17

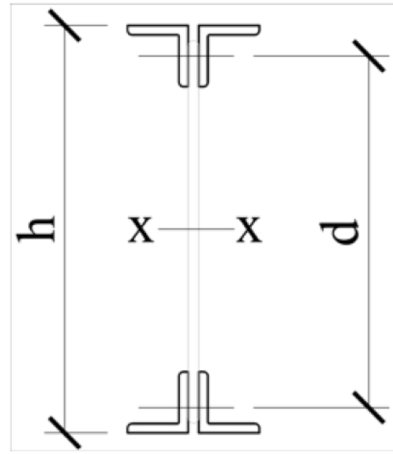
<u>B1-1a</u>



<u>Problem B1-1a</u>

Angle Properties - L2x2x1/4:

$A_a := 0.944in^2$ $\quad wt_a := 3.19plf$ $\quad x_{bar} := 0.609in$ $\quad I_a := 0.346in^4$



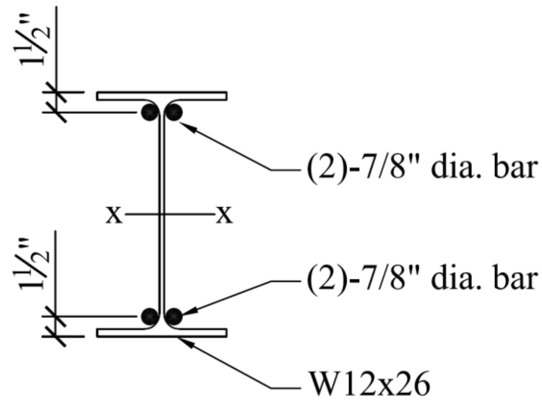
$h := 20in$

$d := h - (2)\cdot(x_{bar}) = 18.782\,in$

$wt_{comp} := 4\cdot A_a\cdot 490pcf = 12.8\cdot plf$

$I_{comp} := (4)(I_a) + \left[4\cdot A_a\cdot\left[\left(\frac{d}{2}\right)^2\right]\right] = 334.4\,in^4$

B1-1b



Problem B1-1b

beam := "W12X26"

Beam Properties

$A = 7.65 \cdot in^2$

$Ix = 204 \cdot in^4$

$d = 12.2 \cdot in$

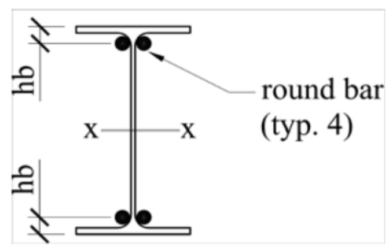
Round Bars

$d_b := 0.875in$

$I_b := \frac{\pi \cdot d_b^4}{64} = 0.029 \cdot in^4$

$A_b := \frac{\pi \cdot d_b^2}{4} = 0.601 \cdot in^2$

$h_b := 1.5in$

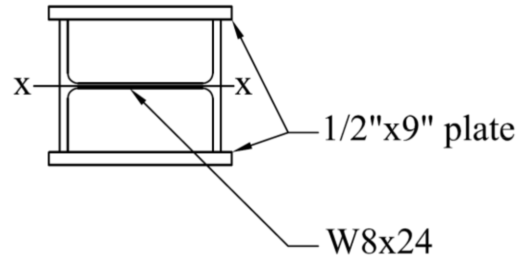


$$I_{comp} := Ix + 4 \cdot \left[A_b \cdot \left(\frac{d}{2} - h_b \right)^2 \right] = 254.9 \cdot in^4$$

$y_{bar} := \frac{d}{2} = 6.1 \cdot in$ $\quad A_{comp} := A + (4 \cdot A_b) = 10.1 \cdot in^2$ $\quad wt_{comp} := A_{comp} \cdot 490pcf = 34.2 \cdot plf$

$$S_{comp} := \frac{I_{comp}}{y_{bar}} = 41.8 \cdot in^3$$

B1-1c



Problem B1-1c

column := "W8X24"

Column Properties

$A = 7.08 \cdot in^2$

$Iy = 18.3 \cdot in^4$

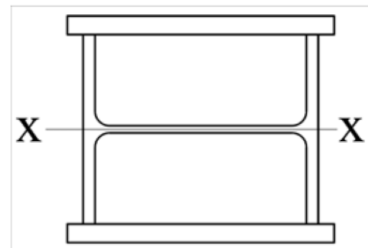
$bf = 6.5 \cdot in$

Cover Plates

$t_p := 0.5in \qquad b_p := 9in$

$A_p := t_p \cdot b_p = 4.5 \cdot in^2$

$I_{yp} := \frac{b_p \cdot t_p^3}{12} = 0.094 \cdot in^4$



Composite Section Properties

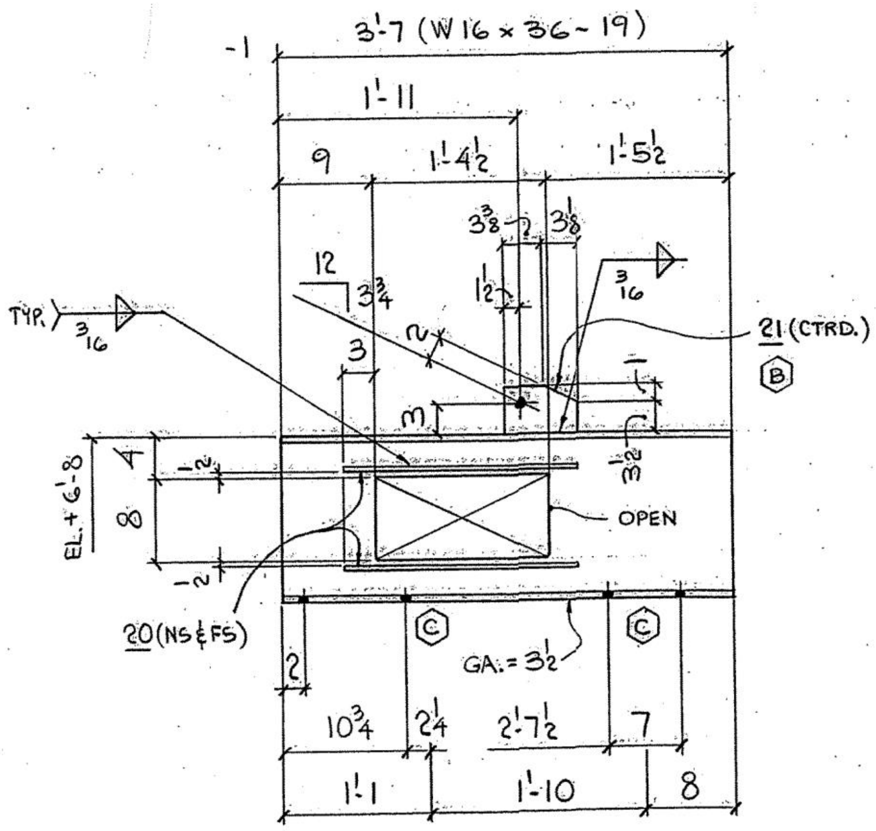
$y_{bar} := \frac{bf}{2} + t_p = 3.75 \cdot in \qquad A_{comp} := A + (2) \cdot A_p = 16.08 \cdot in^2 \qquad wt_{comp} := A_{comp} \cdot 490pcf = 54.7 \cdot plf$

$I_{comp} := Iy + \left(2 \cdot I_{yp}\right) + 2 \cdot \left[A_p \cdot \left[\left(\frac{t_p}{2} + \frac{bf}{2}\right)^2\right]\right] = 128.7 \cdot in^4$

$S_{comp} := \frac{I_{comp}}{y_{bar}} = 34.3 \cdot in^3$

Problem 1-18

S-1



ONE- BEAM ~ 2-8

50	2-8	ONE	BEAM			/170		
51	19	1	W16 × 36	3	7		129	∅
52	20	4	PL - ½ × 3	1	10½		38	
53	21	1	PL - ⅜ × 4½	0	6½		3	

Element	A	y	Ay	I	d = y- $\bar{y}$	Ad²
beam	10.6	7.93	84.06	448	-0.02	0
hole	-2.36	7.86	-18.55	-12.587	0.05	0
upper pls.	3	12.61	37.83	0.063	-4.7	66.21
lower pls.	3	3.11	9.33	0.063	4.8	69.18
Σ =	**14.24**		**112.67**	**435.54**		**135.4**

$$\bar{y} = \frac{\Sigma Ay}{\Sigma A} = \frac{112.67}{14.24} = 7.91 \text{ in.}$$

$\Sigma I + Ad^2 = 435.54+135.4 = 571 \text{ in.}^4$

Wt = (14.24)(490pcf)/144 = 48.5 plf

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 kip**s (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)*

$\phi P_n > P_u$

$\phi_c = 0.9$

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

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** 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: 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 psf *(no net uplift)*`

w_u = (98.3psf)(6ft) = **590 plf** *(downward)*

downward	**No net uplift**
$V_u = \frac{w_u L}{2} = \frac{(590)(30)}{2}$ = 8850 lb.	.
$M_u = \frac{w_u L^2}{8} = \frac{(590)(30)^2}{8}$ = 66375 ft-Ib = 66.4 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:		2^{nd} and 3^{rd} 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**
3^{rd} floor	N/A	-	-	-	**40 psf** (Snow load)
2^{nd} floor	(18)(18) = 324 ft^2	4	40 psf	$\left[0.25+\dfrac{15}{\sqrt{(4)(324)}}\right] = 0.667$	(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	$\left[0.25+\dfrac{15}{\sqrt{(4)(648)}}\right] = 0.545$	(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)**	ΣP **LC 2** (kips)	ΣP **LC 3** (kips)	Maximum ΣP (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

Σ_{DL} = 116psf (with partitions)
Σ_{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** ← 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: $$V_u = \frac{w_u L}{2} = \frac{(3.1)(35)}{2} = \mathbf{54.3\ kips}$$

$$M_u = \frac{w_u L^2}{8} = \frac{(3.1)(35)^2}{8} = \mathbf{474.7\ ft\text{-}kips}$$

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

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

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**