

Instructor's Manual

for

**Structural Steel Design:
A Practice - Oriented Approach**

Second Edition

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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
$\Sigma =$	63 in.²		850.5			I = 7844 in.⁴

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

$$\text{Self weight} = (63/144)(490 \text{ lb/ft}^3) = 214 \text{ lb/ft.}$$

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
$\Sigma =$	15.55 in.²		143.4			I = 939 in.⁴

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

$$\text{Self weight} = (15.55/144)(490 \text{ lb/ft}^3) = 52.9 \text{ lb/ft.}$$

c) From AISCM Table 1-20, $I_x = 314 \text{ in.}^4$
 Area = 13.8 in²
 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
5	5	24	5'-10"	130psf

← select

*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 = $350\text{psf} - 50\text{psf} - 2\text{psf} = 298\text{psf}$)

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

N.G.

N.G.

← select

$t = 3''$ (superimposed load = $350\text{psf} - 57\text{psf} - 2\text{psf} = 291\text{psf}$)

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

N.G.

← 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.

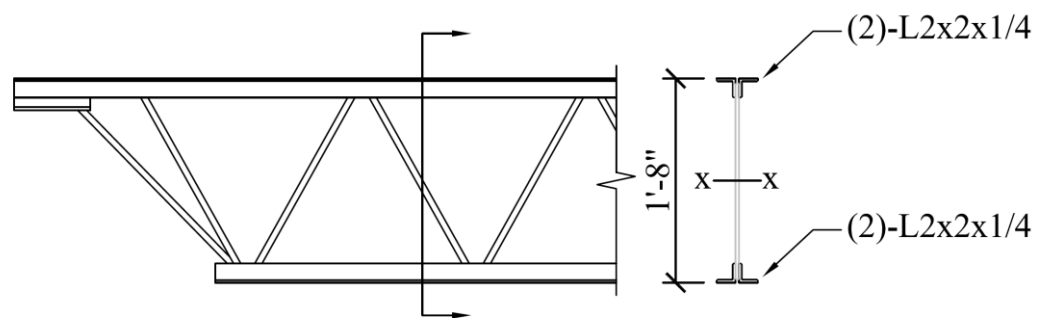
Problem 1-12

$$\text{Anticipated expansion or contraction} = (6.5 \times 10^{-6} \text{ in./in.})(300 \text{ ft.})(12 \text{ in./ft.})(70 \text{ }^{\circ}\text{F}) = 1.64 \text{ in.}$$

$$\text{Expansion joint width} = (2)(1.64 \text{ in.}) = 3.28 \text{ in.}$$

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 $(200/300)(3.28 \text{ in.}) = 2.2 \text{ in.}$ (i.e. 2¼ in. expansion joint)

Problems 1-17**B1-1a****Problem B1-1a**

Angle Properties - L2x2x1/4:

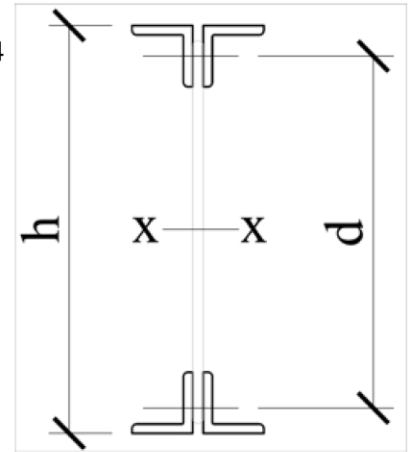
$$A_a := 0.944 \text{ in}^2 \quad w_{t_a} := 3.19 \text{ plf} \quad x_{\text{bar}} := 0.609 \text{ in} \quad I_a := 0.346 \text{ in}^4$$

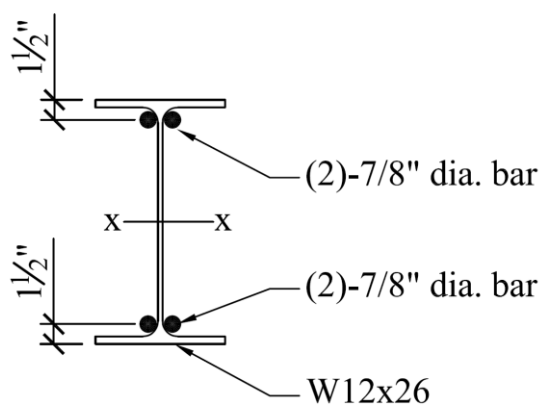
$$h := 20 \text{ in}$$

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

$$w_{t_{\text{comp}}} := 4 \cdot A_a \cdot 490 \text{ pcf} = 12.8 \cdot \text{plf}$$

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



B1-1b**Problem B1-1b**

beam := "W12X26"

Beam Properties

$$A = 7.65 \cdot \text{in}^2$$

$$I_x = 204 \cdot \text{in}^4$$

$$d = 12.2 \cdot \text{in}$$

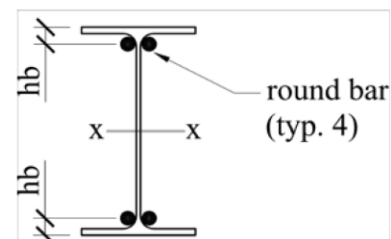
Round Bars

$$d_b := 0.875 \text{ in}$$

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

$$h_b := 1.5 \text{ in}$$

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



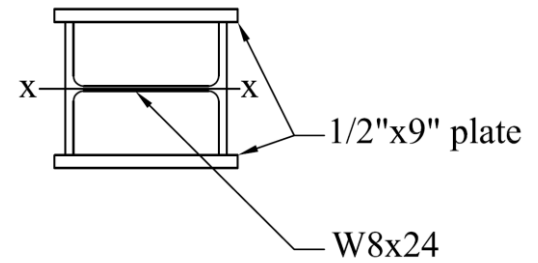
$$I_{\text{comp}} := I_x + 4 \cdot \left[A_b \cdot \left(\frac{d}{2} - h_b \right)^2 \right] = 254.9 \cdot \text{in}^4$$

$$y_{\text{bar}} := \frac{d}{2} = 6.1 \cdot \text{in}$$

$$A_{\text{comp}} := A + (4 \cdot A_b) = 10.1 \cdot \text{in}^2$$

$$wt_{\text{comp}} := A_{\text{comp}} \cdot 490 \text{ pcf} = 34.2 \cdot \text{plf}$$

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

B1-1c**Problem B1-1c**

column := "W8X24"

Column Properties

$$A = 7.08 \cdot \text{in}^2$$

$$I_y = 18.3 \cdot \text{in}^4$$

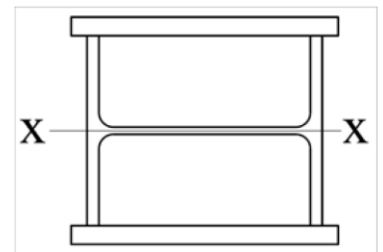
$$bf = 6.5 \cdot \text{in}$$

Cover Plates

$$t_p := 0.5 \text{ in} \quad b_p := 9 \text{ in}$$

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

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

**Composite Section Properties**

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

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

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

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.

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

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

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

$$\phi P_n > P_u$$

$$\phi_c = 0.9$$

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

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

Problem 2-4

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

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

(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 .

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

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

downward	No net uplift
$V_u = \frac{w_u L}{2} = \frac{(590)(30)}{2} = 8850 \text{ lb.}$	.
$M_u = \frac{w_u L^2}{8} = \frac{(590)(30)^2}{8} = 66375 \text{ ft-lb}$ = 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 $\frac{1}{4}$ " 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	$(1.2)(30) + (1.6)(19.72) = 67.6\text{psf}$	-	$(67.6\text{psf})(214\text{ft}^2) = 14.5\text{ kips}$

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