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 = (6.5×10^{-6} in./in.)(300 ft.)(12 in./ft.)(70 ^{o}F) = 1.64 in.

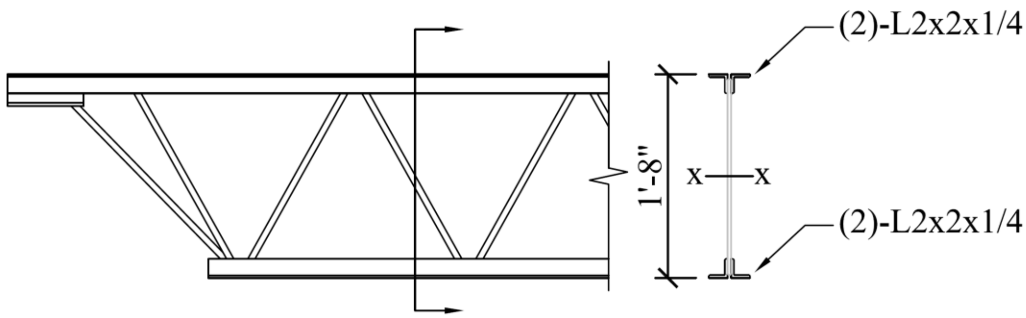
Expansion joint width = (2)(1.64 in.) = 3.28 in.

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

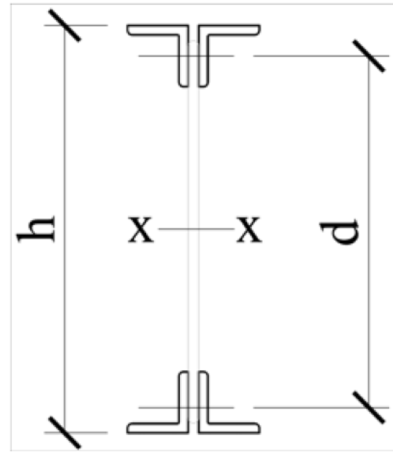
B1-1a



Problem B1-1a

Angle Properties - L2x2x1/4:

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



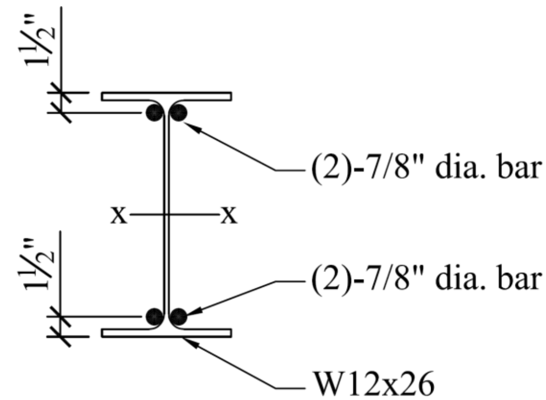
$h := 20in$

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

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

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

B1-1b



Problem B1-1b

beam := "W12X26"

Beam Properties

$A = 7.65 \cdot in^2$

$Ix = 204 \cdot in^4$

$d = 12.2 \cdot in$

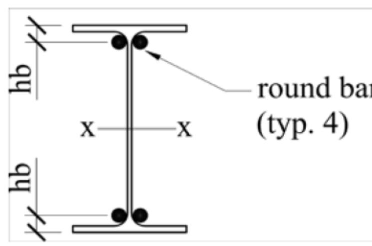
Round Bars

$d_b := 0.875in$

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

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

$h_b := 1.5in$

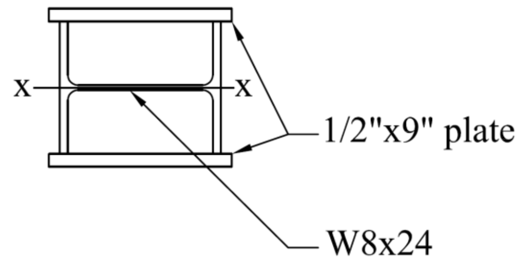


$$I_{comp} := Ix + 4 \cdot \left[A_b \cdot \left(\frac{d}{2} - h_b \right)^2 \right] = 254.9 \cdot in^4$$

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

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

B1-1c



Problem B1-1c

column := "W8X24"

Column Properties

$A = 7.08 \cdot in^2$

$Iy = 18.3 \cdot in^4$

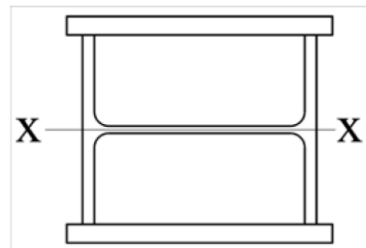
$bf = 6.5 \cdot in$

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

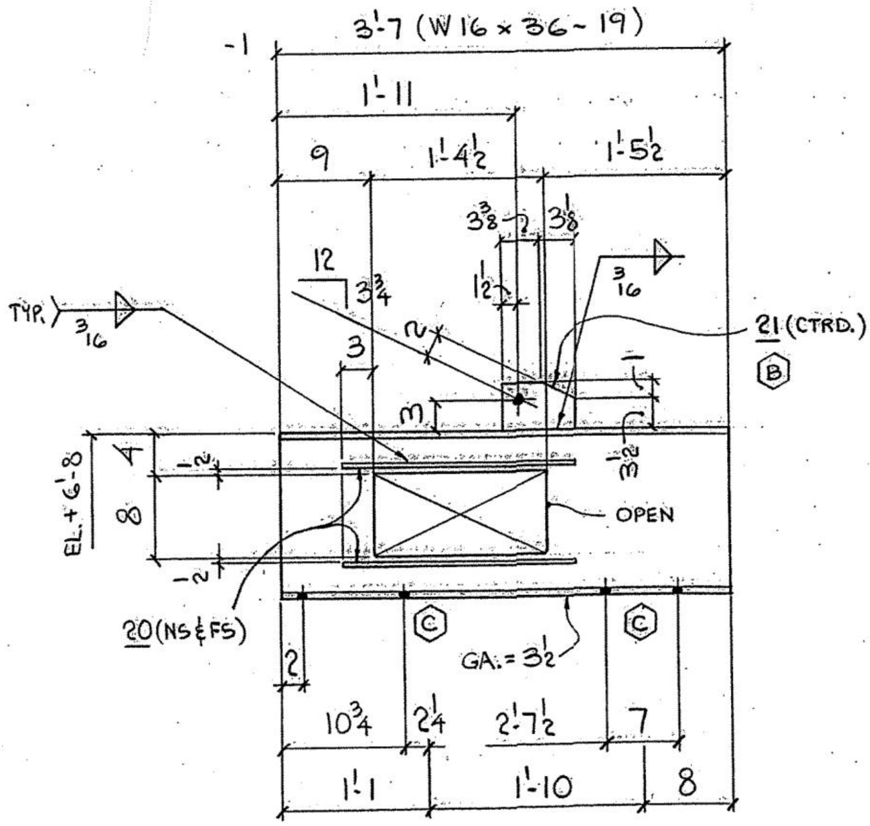
$$y_{bar} := \frac{bf}{2} + t_p = 3.75 \cdot in \qquad A_{comp} := A + (2) \cdot A_p = 16.08 \cdot in^2 \qquad wt_{comp} := A_{comp} \cdot 490pcf = 54.7 \cdot plf$$

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

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

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 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)*

$\phi P_n > P_u$

$\phi_c = 0.9$

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

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	