2-1. *Calculate the total uniformly distributed roof dead load in psf of horizontal plan area for a sloped roof with the design parameters given below.*

- *2x8 rafters at 24” on centers*
- *Asphalt shingles on ½” plywood sheathing*
- *6” insulation (fiberglass)*
- *Suspended Ceiling*
- *Roof slope: 6-in-12*
- *Mechanical & Electrical (i.e. ducts, plumbing etc) = 5 psf*

Solution:

2x8 rafters at 24” on-centers	= 1.2 psf
Asphalt shingles (**assume ¼” shingles**)	= 2.0 psf
½” plywood sheathing = (4 x 0.4 psf/1/8” plywood)	= 1.6 psf
6” insulation (fiberglass) = 6 x 1.1 psf/in.	= 6.6 psf
Suspended Ceiling	= 2.0 psf
Mechanical & Electrical (i.e. ducts, plumbing etc)	= 5.0 psf

Total roof dead load, D (psf of sloped roof area) = 18.4 psf

The total dead load in psf of horizontal plan area will be:

$$w_{DL} = D\left(\frac{\sqrt{6^2+12^2}}{12}\right) \text{, psf of horizontal plan area}$$

$$= 18.4\,\text{psf}\,(1.118) = 20.6 \text{ psf of horizontal plan area}$$

2-2. *Given the following design parameters for a sloped roof, calculate the uniform total load and the maximum shear and moment on the rafter. Calculate the horizontal thrust on the exterior wall if rafters are used.*

- *Roof dead load, D= 20 psf (of sloped roof area)*
- *Roof snow load, S = 40 psf (of horizontal plan area)*
- *Horizontal projected length of rafter,* L_2 *=14 ft*
- *Roof slope: 4-in-12*
- *Rafter or Truss spacing = 4’ 0*

Solutions:

Sloped length of rafter, $L_1 = \left(\frac{\sqrt{4^2+12^2}}{12}\right)(14\,ft) = 14.8\,ft$

Using the load combinations in section 2.1, the total load in psf of horizontal plan area will be:

$$w_{TL} = D\left(\frac{L_1}{L_2}\right) + (L_r \text{ or } S \text{ or } R), \text{ psf of horizontal plan area}$$

$$= 20\,\text{psf}\left(\frac{14.8'}{14'}\right) + 40 \text{ psf}$$

$= \mathbf{61.1\ psf}$ of horizontal plan area

The total load in pounds per horizontal linear foot (Ib/ft) is given as,
w_{TL} (Ib/ft) = w_{TL} (psf) x Tributary width (TW) or Spacing of rafters
= 61.1 psf (4 ft) = **244.4 lb/ft.**

h = (4/12) (14 ft) = 4.67 ft

The horizontal thrust H is,

$$H = \frac{w_{TL}(L_2)\left(\frac{L_2}{2}\right)}{h} = \frac{244.4\ \text{Ib/ft}(14')\left(\frac{14'}{2}\right)}{4.67'} = 5129 \text{ Ib.}$$

The collar or ceiling ties must be designed to resist this horizontal thrust.

L_2 = 14’

The maximum shear force in the rafter is,

$$V_{max} = w_{TL}\left(\frac{L_2}{2}\right) = 244.4\left(\frac{14'}{2}\right) = 1711 \text{ Ib}$$

The maximum moment in the rafter is,

$$M_{max} = \frac{w_{TL}(L_2)^2}{8} = \frac{244.4(14')^2}{8} = 5989 \text{ ft-Ib} = 5.9 \text{ ft-kip}$$

2-3. *Determine the tributary widths and tributary areas of the joists, beams, girders and columns in the panelized roof framing plan shown below. Assuming a roof dead load of 20 psf and an essentially flat roof with a roof slope of ¼” per foot for drainage, determine the following loads using the IBC load combinations. Neglect the rain load, R and assume the snow load, S is zero:*

a. The uniform total load on the typical roof joist in Ib/ft
b. The uniform total load on the typical roof girder in Ib/ft
c. The total axial load on the typical interior column, in Ib.
d. The total axial load on the typical perimeter column, in Ib

.

Solution:

The solution is presented in a tabular format as shown below:

Tributary Widths and Tributary Areas of Joists, Beams and Columns

Structural Member	Tributary Width (TW)	Tributary Area (TA)
Purlin	$\frac{10'}{2}+\frac{10'}{2}=10'$	10’ x 20’ = 200 ft^2
Glulam girder	$\frac{20'}{2}+\frac{20'}{2}=20'$	20’ x 60’ = 1200 ft^2
Typical interior Column		$\left(\frac{20'}{2}+\frac{20'}{2}\right)\left(\frac{60'}{2}+\frac{60'}{2}\right)=$ 1200 ft^2
Typical perimeter Column		$\left(\frac{20'}{2}+\frac{20'}{2}\right)\left(\frac{60'}{2}\right)=$ 600 ft^2

Since the snow and rain load are both zero, the roof live load, L_r will be critical.
With a roof slope of ¼” per foot, the number of inches of rise per foot, F = ¼ = 0.25

Purlin:

The tributary width TW = 10 ft and the tributary area, TA = 200 ft^2 < 200 ft^2
From section 2.5.1, we obtain:R_1 = 1.0 and R_2 = 1.0,

Using equation 2-4 gives the roof live load, L_r = 20 x 1 x 1 = 20 psf

The total loads are calculated as follows:
w_{TL} (psf) = (D + L_r) = 20 + 20 = 40 psf
w_{TL} (Ib/ft) = w_{TL} (psf) x tributary width (TW) = 40 psf x 10 ft = 400 Ib/ft

Glulam Girder:

The tributary width, TW = 20 ft and the tributary Area, TA = 1200 ft^2
Thus, TA > 600, and from section 2.4, we obtain:

R_1 = 0.6, and
R_2 = 1.0
Using equation 2-4 gives the roof live load, L_r = 20 x 0.6 x 1 = 12 psf

The total loads are calculated as follows:
w_{TL} (psf) = (D + L_r) = 20 + 12 = 32 psf
w_{TL} (Ib/ft) = w_{TL} (psf) x tributary width (TW) = 32 psf x 20 ft = 640 Ib/ft

Typical Interior Column:

The tributary tributary area of the typical interior column, TA = 1200 ft^2
Thus, TA > 600, and from section 2.4, we obtain:

R_1 = 0.6, and
R_2 = 1.0
Using equation 2-4 gives the roof live load, L_r = 20 x 0.6 x 1 = 12 psf

The total loads are calculated as follows:
w_{TL} (psf) = (D + L_r) = 20 + 12 = 32 psf
The Column Axial Load, P = 32 psf x 1200 ft^2 = 38, 400 Ib = 38.4 kips

Typical Perimeter Column:

The tributary tributary area of the typical perimeter column, TA = 600 ft^2
Thus, from section 2.5.1, we obtain:

R_1 = 0.6, and
R_2 = 1.0
Using equation 2-4 gives the roof live load, L_r = 20 x 0.6 x 1 = 12 psf

The total loads are calculated as follows:
w_{TL} (psf) = (D + L_r) = 20 + 12 = 32 psf
The Column Axial Load, P = 32 psf x 600 ft^2 = 19, 200 Ib = 19.2 kips

2-4. *A building has sloped roof rafters (5:12 slope) spaced at 2' 0" on centers and is located in Hartford, Connecticut. The roof dead load is 22 psf of sloped area. Assume a fully exposed roof with terrain category "C", and use the ground snow load from the IBC or ASCE 7 snow map*

(a) Calculate the total uniform load in lb/ft on a horizontal plane using the IBC.
(b) Calculate the maximum shear and moment in the roof rafter.

Solution:

The roof slope, θ for this building is 22.6°,

Roof Live Load, L_r:

From Section 2.4, the roof slope factor is obtained as,
F = 5 $\therefore$ R_2 = 1.2 – 0.05 (5) = 0.95

Assume the tributary area (TA) of the rafter < 200 ft^2, $\therefore$ R_1 = 1.0
The roof live load will be,

$L_r = 20R_1R_2$ = 20(1.0)(0.95) = 19 psf

Snow Load:

Using IBC Figure 1608.2 or ASCE 7 Figure 7-1, the ground snow load, P_g for Hartford, Connecticut is 30 psf.

Assuming a building with a warm roof and fully exposed, and a building site with terrain category "C", we obtain the coefficients as follows:

Exposure coefficient, $C_e = 0.9$ (ASCE 7 Table 7-2)
The thermal factor, $C_t = 1.0$ (ASCE 7 Table 7-3)
The Importance Factor, I = 1.0 ASCE Table 7-4
The slope factor, $C_s = 1.0$ (ASCE Figure 7-2 with roof slope, $\theta = 22.6°$ and a warm roof)

The flat roof snow load, $P_f = 0.7\, C_e\, C_t\, I\, P_g = 0.7 \times 0.9 \times 1.0 \times 1.0 \times 30 = 18.9$ psf

Minimum flat roof snow load, $Pm = 20I_s = 20\,(1.0) = 20$ psf (governs)

Thus, the design roof snow load, $P_s = C_s\, P_f = 1.0 \times 20 = 20$ psf

Therefore, the snow load, S = 20 psf

The total load in psf of horizontal plan area is given as,

$$w_{TL} = D\left(\frac{L_1}{L_2}\right) + (L_r \text{ or } S \text{ or } R)\text{, psf of horizontal plan area}$$

Since the roof live load, L_r (18 psf) is smaller that the snow load, S (20 psf), the snow load is more critical and will be used in calculating the total roof load.

$$w_{TL} = 22\,\text{psf}\left(\frac{\sqrt{5^2+12^2}}{12}\right) + 20\text{ psf}$$

$= \;$ **43.83 psf** of horizontal plan area

The total load in pounds per horizontal linear foot (Ib/ft) is given as,
w_{TL} (Ib/ft) = w_{TL} (psf) x Tributary width (TW) or Spacing of rafters
= 43.83 psf (2 ft) = **87.7 lb/ft.**

Assume $L_2 = 14'$

The maximum moment in the rafter is,

$$M_{max} = \frac{w_{TL}(L_2)^2}{8} = \frac{87.7(14')^2}{8} = 2149\text{ ft-Ib} = 2.15\text{ ft-kip}$$

2-5. 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:
Dead Load, D_{roof} = 20 psf
Snow Load, S = 40 psf

2^{nd} and 3^{rd} Floor Loads:
Dead Load, D_{floor} = 40 psf
Floor Live Load, L = 50 psf

Solution:

At each level, the tributary area (TA) supported by a typical interior column is

18' x 18' = 324 ft^2

Roof Live Load, L_r:

From section 2.4, the roof slope factor is obtained as,
F = ¼ = 0.25 $\therefore$ $R_2 = 1.0$
Since the tributary area (TA) of the column = 324 ft^2, $\therefore$ $R_1 = 1.2 - 0.001\,(324) = 0.88$
The roof live load will be,

$L_r = 20R_1R_2 = 20(0.88)(1.0) = 17.6$ psf < Snow load, S = 40 psf

The governing load combination from Section 2.1.1 for calculating the column axial loads is D + L + (L_r or S or R). Since the snow load is greater than the roof live load, the critical load combination reduces to D + L + S.

The reduced or design floor live load for the 2nd and 3rd floors are calculated using the table below:

Reduced or Design Floor Live Load Calculation Table

Member	**Levels supported**	**A_T (summation of floor tributary area)**	**K_{LL}**	**Unreduced Floor live load, Lo (psf)**	**Live Load Reduction Factor $0.25 + 15/\sqrt{(K_{LL} A_T)}$**	**Design floor live load, L**

3^{rd} floor Column (i.e. column below roof)	Roof only	Floor live load reduction NOT applicable to roofs!!!	-	-	-	40 psf (Snow load)
2^{nd} floor column (i.e. column below 3^{rd} floor)	1 floor + roof	1 floor x 324 ft^2 = **324 ft^2**	4 K_{LL} A_T = 1296 > 400 ft^2 $\therefore$ **Live Load reduction allowed**	50 psf	0.25 + 15/√(4 x 324) = 0.67	0.67 x 50 = **33.5 psf** ≥ 0.50 Lo = 25 psf
Ground or 1^{st} floor column (i.e. column below 2^{nd} floor)	2 floors + roof	2 floors x 324 ft^2 = **648 ft^2**	4 K_{LL} A_T = 2592 > 400 ft^2 $\therefore$ **Live Load reduction allowed**	50 psf	0.25 + 15/√(4 x 648) = 0.54	0.54 x 50 = **27 psf** ≥ 0.40 Lo = 20 psf

The column axial loads with and without floor live load reduction are calculated using the column load summation tables below:

Column Load Summation Table

Level	Tributary area, (TA) (ft^2)	Dead Load, D (psf)	Live Load, L_o (S or L_r or R on the roof) (psf)	Design Live Load Roof: S or L_r or R Floor: L (psf)	Unfactored total load at each level, w_{s1} Roof: D Floor: $D + L$ (psf)	Unfactored total load at each level, w_{s2} Roof: D+0.75S Floor: D+0.75L (psf)	Unfactored Column Axial Load at each level, P = (TA)(w_{s1}) or (TA)(w_{s2}) (kips)	Cumulative Unfactored Axial Load, ΣP_{D+L} (kips)	Cumulative Unfactored Axial Load, $\Sigma P_{D+0.75L+0.75S}$ (kips)	Maximum Cumulative Unfactored Axial Load, ΣP (kips)
With Floor Live Load Reduction										
Roof	324	20	40	40	20	50	6.5 or 16.2	6.5	16.2	**16.2**
3rd Flr	324	40	50	33.5	73.5	65.1	23.8 or 21.1	30.3	37.3	**37.3**
2nd Flr	324	40	50	27	67	60.3	21.7 or 19.5	52	56.8	**56.8**
Without Floor Live Load Reduction										
Roof	324	20	40	40	20	50	6.5 or 16.2	6.5	16.2	**16.2**
Third floor	324	40	50	50	90	77.5	29.2 or 25.1	35.7	41.3	**41.3**
Second floor	324	40	50	50	90	77.5	29.2 or 25.1	64.9	66.4	**66.4**

2-6. *A 2-story wood framed structure 36 ft x 75 ft in plan is shown below with the following given information. The floor to floor height is 10 ft and the truss bearing (or roof datum) elevation is at 20 ft and the truss ridge is 28 ft 4" above the ground floor level. The building is "enclosed" and located in Rochester, New York on a site with a category "C" exposure. Assuming the following additional design parameters, calculate:*

Floor Dead Load =	*30 psf*
Roof Dead Load =	*20 psf*
Exterior Walls =	*10 psf*
Snow Load (P_f) =	*40 psf*

Site Class =	*D*
Importance (I_e)=	*1.0*
S_S =	*0.25%*
S_1 =	*0.07%*
R =	*6.5*

(a) The total horizontal wind force on the main wind force resisting system (MWFRS) in both the transverse and longitudinal directions.
(b) The gross vertical wind uplift pressures and the net vertical wind uplift pressures on the roof (MWFRS) in both the transverse and longitudinal directions.
(c) The seismic base shear, V, in kips
(d) The lateral seismic load at each level in kips

Solution:

(a) Lateral Wind

Roof Slope: Run = 18', Rise = 8'-4", θ = 25°

Assuming a Category II building

V = 115mph (ASCE 7 Table 26.5-1A)

Wind Pressures (from ASCE 7, Figure 28.6-1):

Transverse (θ = 25°):

Horizontal	*Vertical*
Zone A: 26.3 psf	Zone E: -11.7 psf
Zone B: 4.2 psf	Zone F: -15.9 psf
Zone C: 19.1 psf	Zone G: -8.5 psf
Zone D: 4.3 psf	Zone H: -12.8 psf

Longitudinal: ($\theta = 0°$):

Horizontal	*Vertical*
Zone A: 21 psf	Zone E: -25.2 psf
Zone B: N/A	Zone F: -14.3 psf
Zone C: 13.9 psf	Zone G: -17.5 psf
Zone D: N/A	Zone H: -11.1 psf

End Zone width:

a: ≤ 0.1 x least horizontal dimension of building
≤ 0.4 x mean roof height of the building and
≥ 0.04 x least horizontal dimension of building
≥ 3 feet

a ≤ 0.1 (36') = 3.6' (governs)

$$\le (0.4)\left(\frac{20' + 28.33'}{2}\right) = 9.67'$$

≥ 0.04 (36') = 1.44'
≥ 3 feet

Therefore the Edge Zone = 2a = 2 (3.6') = 7.2'

Average horizontal pressures:

Transverse:

$$q_{\text{avg}} = \left(\frac{(\text{end zone})(\text{end zone pressure}) + (\text{bldg width} - \text{end zone})(\text{interior zone pressure})}{(\text{bldg width})}\right)$$

$$q_{avg}(wall) = \left(\frac{(7.2')(26.3\,psf) + (75' - 7.2')(19.1\,psf)}{(75')}\right) = 19.8 \text{ psf (Zones A, C)}$$

$$q_{avg}(roof) = \left(\frac{(7.2')(4.2\,psf) + (75' - 7.2')(4.3\,psf)}{(75')}\right) = 4.3 \text{ psf (Zones B, D)}$$

Longitudinal:

$$q_{avg}(wall) = \left(\frac{(7.2')(21psf) + (36' - 7.2')(13.9psf)}{(36')}\right) = 15.32 \text{ psf (Zones A, C)}$$

Design wind pressures:

Height and exposure coefficient:

Mean roof height = $\left(\frac{20'+28.33'}{2}\right)$ = 24.2'

λ = 1.35 (ASCE 7 Figure 28.6-1, Exposure = C, h ≈ 25')

Transverse wind:

$P = q_{avg}\lambda$

P_{wall} = (19.8 psf)(1.35) = 26.73 psf

P_{roof} = (4.3 psf)(1.35) = 5.81 psf

Longitudinal wind:

P_{wall} = (15.32 psf)(1.35) = 20.7 psf

Total Wind Force:

Transverse wind:

$P_T = \left[(26.73psf)(10'+10') + (5.81psf)(8.33')\right](75')$ = **43.7 kips** *(base shear, transverse)*

Longitudinal wind:

$P_T = \left(20' + \frac{8.33'}{2}\right)(20.7psf)(36')$ = **18.0 kips** *(base shear, longitudinal)*

(b) Wind Uplift

Average vertical pressures:

$P = q_{avg}\lambda$

From Part (a), base uplift pressures:

Transverse:	*Longitudinal:*
Zone E: -11.7 psf	Zone E: -25.2 psf
Zone F: -15.9 psf	Zone F: -14.3 psf
Zone G: -8.5 psf	Zone G: -17.5 psf
Zone H: -12.8 psf	Zone H: -11.1 psf

Transverse:

$$P_{u,avg} = \left[(-11.7\,psf - 15.9\,psf)(7.2')\left(\frac{36'}{2}\right) + (-8.5\,psf - 12.8\,psf)(75' - 7.2')\left(\frac{36'}{2}\right) \right](1.35)$$

= - 39,922 lb.

$$q_{u,avg} = \frac{-39{,}922\,lb}{(75')(36')} = \mathbf{-\ 14.8\ psf}\ \textit{(gross uplift, transverse)}$$

Longitudinal:

$$P_{u,avg} = \left[(-25.2\,psf - 14.3\,psf)(7.2')\left(\frac{75'}{2}\right) + (-17.5\,psf - 11.1\,psf)(36' - 7.2')\left(\frac{75'}{2}\right) \right](1.35)$$

= - 56,097 lb.

$$q_{u,avg} = \frac{-56{,}097\ lb}{(75')(36')} = \mathbf{-\ 20.8\ psf}\ \textit{(gross uplift, longitudinal)}$$

Net factored uplift (Longitudinal controls):

q_{net} = 0.9D+W

=(0.9)(20psf) + (-20.8psf) = **-2.8psf** *(net uplift)*

(c) Seismic base shear

Calculate "W" for each level

Level	Area (ft.^{2})	Trib. Height (ft.)	Wt. Level (kip)*	Wt. Walls (kip)	W_{total} (kip)
Roof	75' x 36' = **2700 ft.^{2}**	(10'/2)= **5'**	(2700 ft^2) x [20psf + (0.2x40psf)] = **75.6k**	(5') x (10psf) x (2) x (75' + 36') = **11.1k**	75.6k + 11.1k = **86.7k**
2nd	75' x 36' = **2700 ft.^{2}**	(10'/2) + (10'/2) = **10'**	(2700 ft^2) x (30psf) = **81.0k**	(10') x (10psf) x (2) x (75' + 36') = **22.2k**	81.0k + 22.2k = **103.2k**
				∑W = 86.7k + 103.2k = **189.9k**	

** Note: Where the flat roof snow load, P_f, is greater than 30psf, then 20% of the flat roof snow load shall be included in "W" for the roof (ASCE 7 Section 12.14.8.1)*

<u>Seismic Variables:</u>

$F_a =$ 1.6 *(ASCE 7 Table 11.4-1)*
$F_v =$ 2.4 *(ASCE 7 Table 11.4-2))*

$S_{MS} = F_a S_S = (1.6)(0.25) = 0.40$
$S_{M1} = F_a S_1 = (2.4)(0.07) = 0.168$

$S_{DS} = (2/3) S_{MS} = (2/3)(0.40) = \mathbf{0.267}$
$S_{D1} = (2/3) S_{M1} = (2/3)(0.168) = \mathbf{0.112}$

<u>Base Shear:</u>

$$V = \frac{F S_{DS} W}{R}$$

$$V = \frac{(1.1)(0.267)(189.9)}{(6.5)} = \mathbf{8.58k}$$

(d) Seismic Forces at each level:

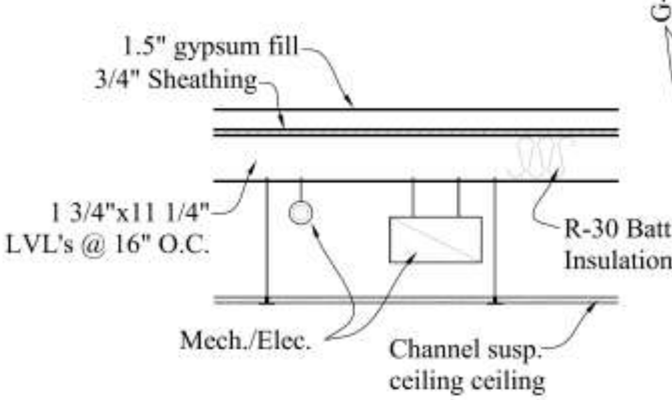
$$F_x = \frac{F S_{DS} W_x}{R}$$

$$F_R = \frac{(1.1)(0.267)(86.7)}{(6.5)} \mathbf{= 3.92k}$$

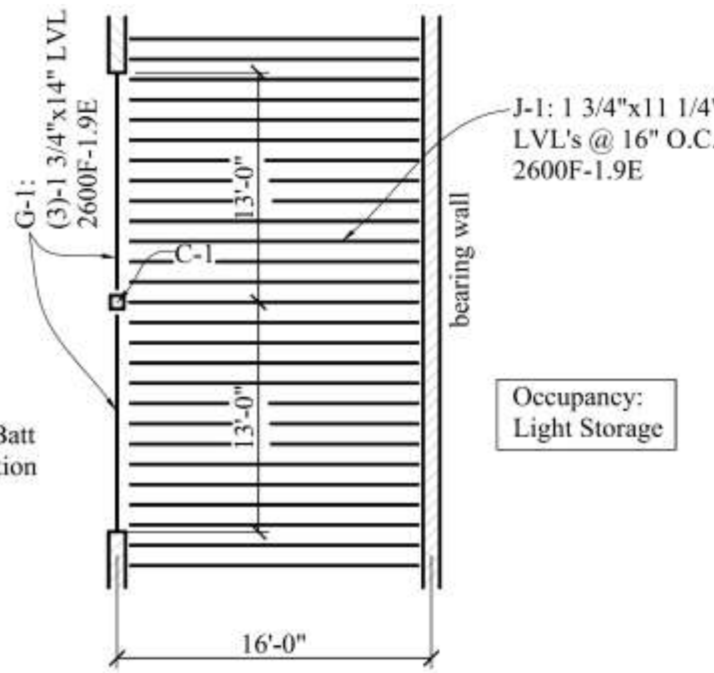
$$F_2 = \frac{(1.1)(0.267)(103.2)}{(6.5)} \mathbf{= 4.67k}$$

....

2-7 (see framing plan and floor section)
a) Determine the floor dead load in PSF
b) Determine the service dead and live loads to J-1 and G-1 in PLF
c) Determine the maximum factored loads in PLF to J-1 and G-1
d) Determine the factored maximum moment and shear in J-1 and G-1
e) Determine the maximum service and factored load in kips to C-1



Typical Floor Section

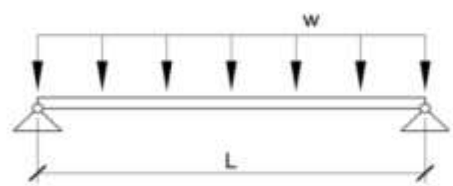


Floor framing plan

Dead Loads

$w_{gyp} := (1.5in) \cdot \left(6 \frac{lb}{ft^2 \cdot in}\right) = 9 \cdot psf$

$LL := 125psf$



$w_{sht} := (0.75in)(0.4psf)\left(\frac{1}{0.125in}\right) = 2.4 \cdot psf$

$w_{ME} := 5psf \quad w_{ins} := 4psf \quad w_{clg} := 2psf$

$w_{LVL} := \frac{(5.7plf)}{1.33ft} = 4.286 \cdot psf$

$DL := w_{gyp} + w_{sht} + w_{ME} + w_{ins} + w_{clg} + w_{LVL} = 26.7 \cdot psf$ *part a*

Loads to J-1:

$L_j := 16ft$

$TWj := 16in$

$w_{Dj} := TWj \cdot DL = 35.6 \cdot plf$

$w_{Lj} := TWj \cdot LL = 166.7 \cdot plf$

$w_{Sj} := w_{Dj} + w_{Lj} = 202.2 \cdot plf$ *part b*

$w_{uj} := (1.2)(w_{Dj}) + (1.6)(w_{Lj}) = 309 \cdot plf$ *part c*

$M_{Sj} := \frac{w_{Sj} \cdot L_j^2}{8} = 6472 \cdot ft \cdot lb$

$V_{Sj} := \frac{w_{Sj} \cdot L_j}{2} = 1618\,lbf$

$M_{uj} := \frac{w_{uj} \cdot L_j^2}{8} = 9900 \cdot ft \cdot lb$

part d

$V_{uj} := \frac{w_{uj} \cdot L_j}{2} = 2474.9\,lbf$

Loads to G-1:

$L_G := 13ft$

$TW_G := 8ft$

$w_{DG} := TW_G \cdot DL + (3) \cdot (7.1plf) = 234.8 \cdot plf$

$w_{LG} := TW_G \cdot LL = 1000 \cdot plf$

$w_{SG} := w_{DG} + w_{LG} = 1234.8 \cdot plf$ *part b*

$w_{uG} := (1.2)(w_{DG}) + (1.6)(w_{LG}) = 1882 \cdot plf$ *part c*

$M_{SG} := \frac{w_{SG} \cdot L_G^2}{8} = 26085 \cdot ft \cdot lb$

Service Loads

$V_{SG} := \frac{w_{SG} \cdot L_G}{2} = 8026\,lbf$

$M_{uG} := \frac{w_{uG} \cdot L_G^2}{8} = 39752 \cdot ft \cdot lb$

Factored Loads
part d

$V_{uG} := \frac{w_{uG} \cdot L_G}{2} = 12231\,lbf$

Load to C-1:

$P_s := 2 \cdot V_{SG} = 16052\,lbf$

$P_u := 2 \cdot V_{uG} = 24463\,lbf$ *part e*

Problem 2.8

$b := 1.875\text{in} \qquad d := 12.5\text{in} \qquad G := 0.5$

$A_x := b \cdot d = 23.438 \cdot \text{in}^2$

$\boxed{S_x := \frac{b \cdot d^2}{6} = 48.8 \cdot \text{in}^3} \qquad \boxed{I_x := \frac{b \cdot d^3}{12} = 305.2 \cdot \text{in}^4}$ **part a**

$\boxed{w_{self} := A_x \cdot G \cdot 62.4\text{pcf} = 5.1 \cdot \text{plf}}$ **part b**

$L_b := 17\text{ft} \qquad TW := 16\text{in}$

$DL := 15\text{psf} \qquad LL := 40\text{psf}$

$w_{DL} := TW \cdot DL + w_{self} = 25.1 \cdot \text{plf} \qquad w_{LL} := LL \cdot TW = 53.3 \cdot \text{plf}$

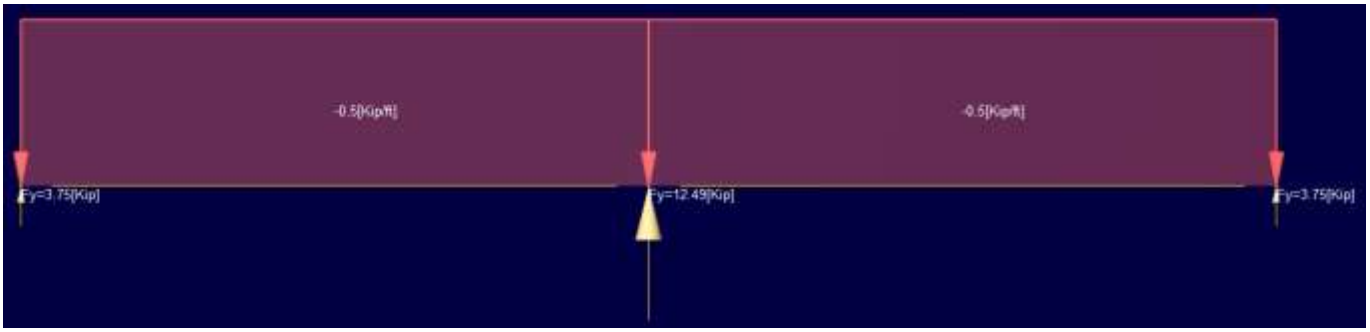
$w_{TL} := w_{DL} + w_{LL} = 78.4 \cdot \text{plf}$

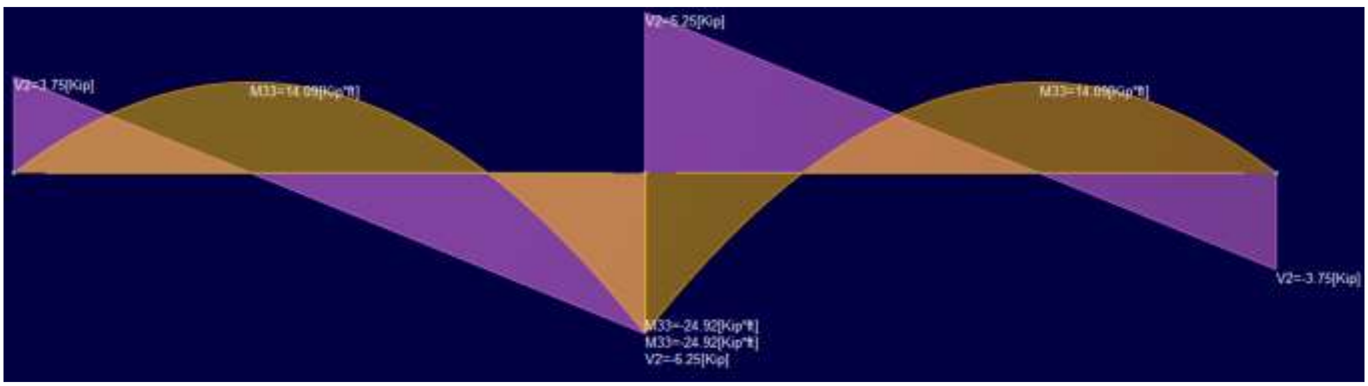
$\boxed{M_b := \frac{w_{TL} \cdot L_b^2}{8} = 2833 \cdot \text{ft} \cdot \text{lb}} \qquad \boxed{V_b := \frac{w_{TL} \cdot L_b}{2} = 666.497\,\text{lbf}}$ **part c**

$\boxed{f_b := \frac{M_b}{S_x} = 696.1\,\text{psi}}$ **part d**

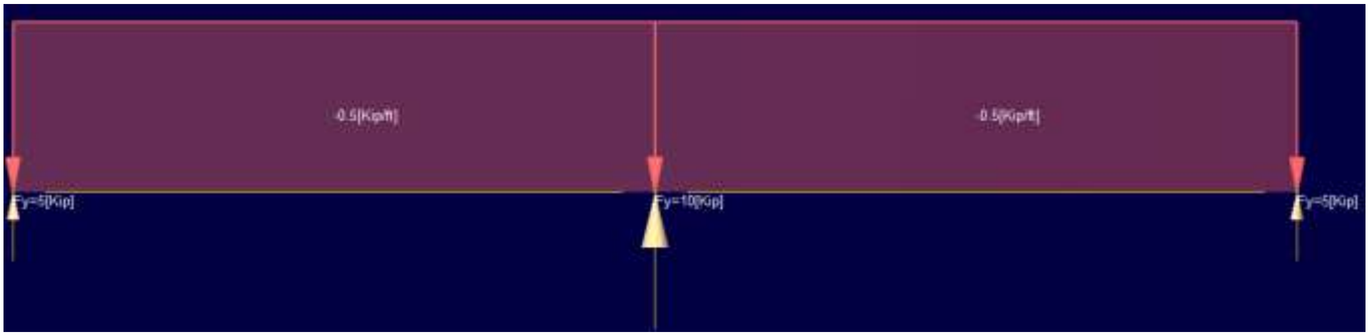
2-9. w = 500plf; L_1 = 20ft

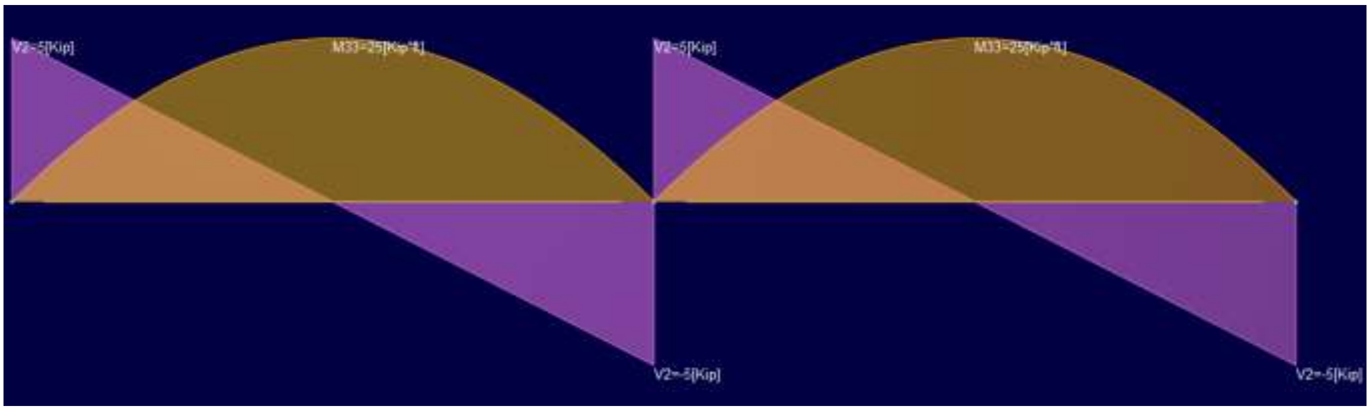
Case 1: continuous over support





Case 2: hinged over support





Case 1 would have less deflection

Case 2 is easier to build; 40ft section might be hard to get or ship/handle on-site

2-10

Job	Truss	Truss Type	Qty	Ply	
VESTAL ROOF	M01	Monopitch Truss	1	1	Job Reference (optional)

Stark Truss Inc., Canton, OH

7.250 s Mar 23 2011 MiTek Industries, Inc. Tue Oct 11 16:17:21 2011 Page 1

ID:OtU7J6iS03dRR9ldxfYFreyUmUO-wrT9RJYWRQwW2ppZjT1KqbCJbQ25NWRF9iPwhTyUS2i

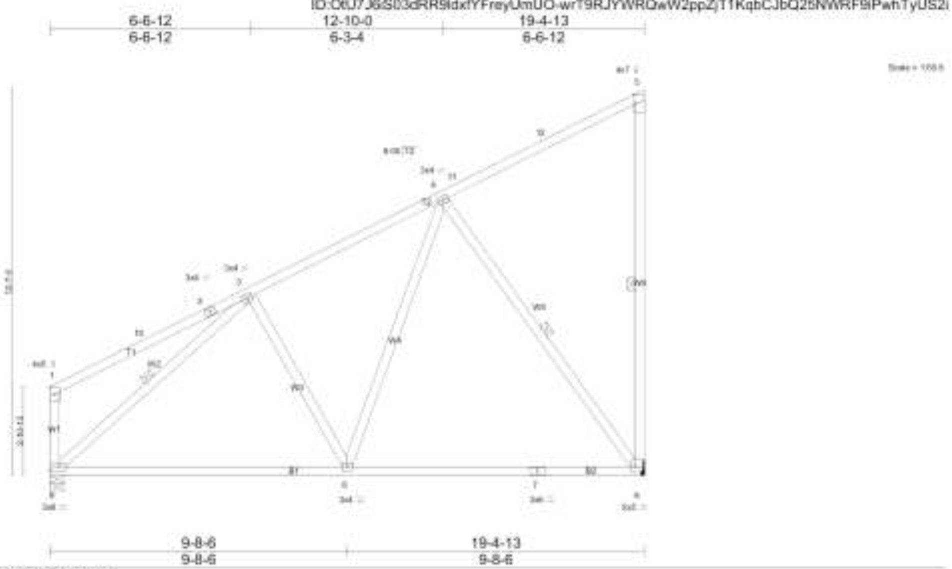


Plate Offsets (X,Y): [5:0-3-11,Edge], [6:Edge,0-2-4], [9:Edge,0-1-8]

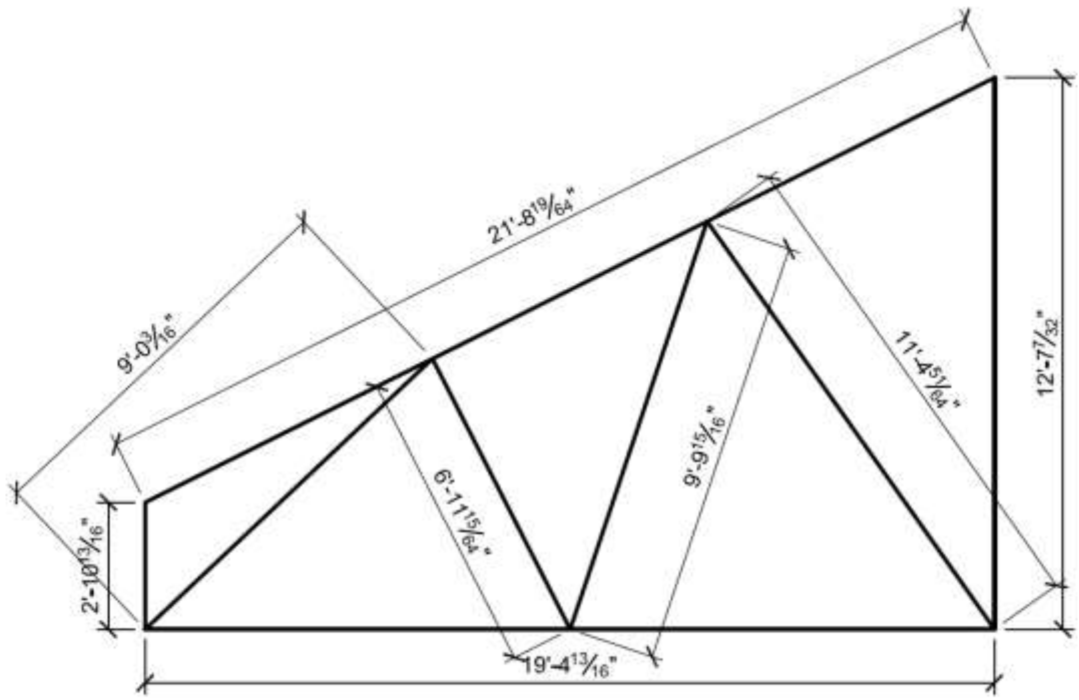
LOADING (psf)		SPACING		CSI		DEFL	in	(loc)	l/defl	L/d	PLATES	GRIP
TCLL(roof)	20.0	SPACING	2-0-0	TC	0.91	Vert(LL)	0.27	6-8	>834	360	MT20	197/144
Snow (Pf/Pg)	30.8/40.0	Plates Increase	1.15	BC	0.64	Vert(TL)	-0.42	6-8	>543	240		
TCDL	10.0	Lumber Increase	1.15	WB	0.61	Horz(TL)	0.02	6	n/a	n/a		
BCLL	0.0	Rep Stress Incr	YES	(Matrix)								
BCDL	10.0	Code IBC2006/TPI2002									Weight: 103 lb	FT = 10%

LUMBER
TOP CHORD 2 X 4 SPF No.2
BOT CHORD 2 X 4 SPF No.2
WEBS 2 X 4 SPF Stud *Except*
W6: 2 X 4 SPF 2100F 1.8E, W1: 2 X 4 SPF No.2

BRACING
TOP CHORD Structural wood sheathing directly applied or 2-2-0 oc purlins, except end verticals.
BOT CHORD Rigid ceiling directly applied or 8-7-5 oc bracing.
WEBS 1 Row at midpt 5-6, 4-6, 3-9

REACTIONS (lb/size) 6=971/Mechanical, 9=971/0-5-8 (min. 0-1-9)
Max Horz 9=510(LC 9)
Max Uplift6=-246(LC 10), 9=-92(LC 10)
Max Grav6=1181(LC 3), 9=1009(LC 3)

FORCES (lb) - Max. Comp./Max. Ten. - All forces 250 (lb) or less except when shown.
TOP CHORD 1-10=-278/121, 3-4=-865/219, 5-6=-354/145, 1-9=-331/133
BOT CHORD 8-9=-417/778, 7-8=-286/551, 6-7=-286/551
WEBS 4-8=-111/439, 4-6=-899/285, 3-9=-860/142



a) List each truss member in a table (B1, B2, T1, T2, W1 to W6) and list the following: size, length, species, grade, density, weight).

b) Calculate the total weight of the truss using the table in (a).

Member	Size	Area (in.^{2})	Length (ft.)	Species / Grade	γ	density (pcf)	weight (lb.)	
W1	2x4	5.25	2.91	SPF / #2	0.42	26.2	2.8	
W2	2x4	5.25	9.02	SPF / stud	0.42	26.2	8.6	
W3	2x4	5.25	6.94	SPF / stud	0.42	26.2	6.6	
W4	2x4	5.25	9.83	SPF / stud	0.42	26.2	9.4	
W5	2x4	5.25	11.4	SPF / stud	0.42	26.2	10.9	
W6	2x4	5.25	12.61	SPF / 2100F-1.8	0.46	28.7	13.2	
B1/B2	2x4	5.25	19.41	SPF / #2	0.42	26.2	18.5	
T1/T2	2x4	5.25	21.7	SPF / #2	0.42	26.2	20.7	
						$\Sigma =$	**90.8**	lb.

Plates	l (in.)	w (in.)	t (in.)	vol (in.^{3})	γ	density (pcf)	weight (lb.)	
3x6	3	6	0.125	2.25	7.85	490	0.638	
3x4	3	4	0.125	1.5	7.85	490	0.425	
3x6	3	6	0.125	2.25	7.85	490	0.638	
5x5	5	5	0.125	3.125	7.85	490	0.886	
4x5	4	5	0.125	2.5	7.85	490	0.709	
3x4	3	4	0.125	1.5	7.85	490	0.425	
3x4	3	4	0.125	1.5	7.85	490	0.425	
3x4	3	4	0.125	1.5	7.85	490	0.425	
4x7	4	7	0.125	3.5	7.85	490	0.992	
						$\Sigma =$	5.563	lb.
						2x =	**11.1**	**lb.**

Total truss wt. =	**101.9**	**lb.**

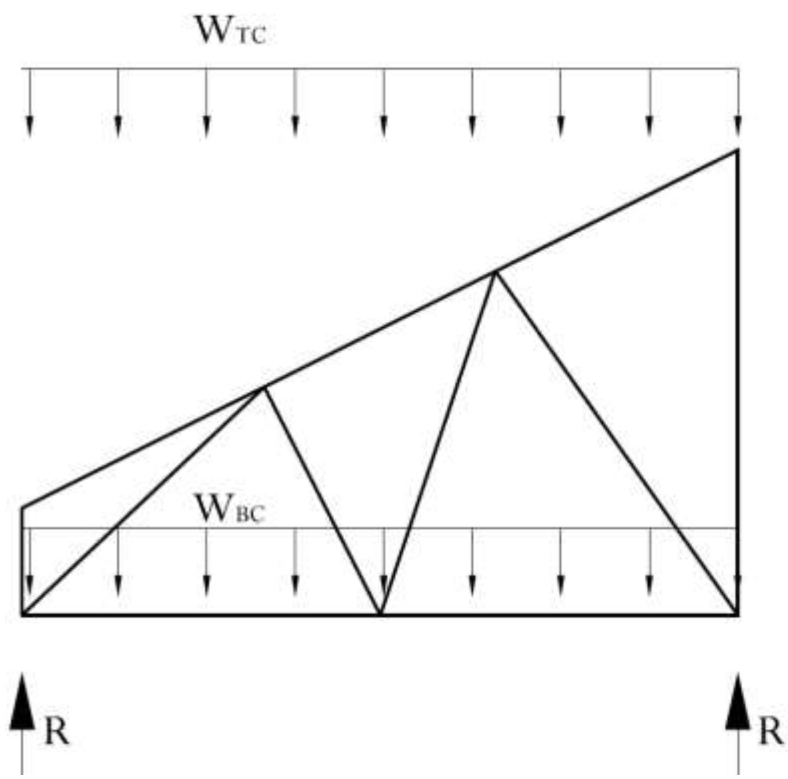
c) Draw a free-body diagram of the truss and indicate the uniformly distributed loads to the top and bottom chords in pounds per lineal foot (plf) and indicate the supports.
d) Calculate the maximum possible reaction using the controlling load case Dead + Snow.

Trib width = 2 ft.

<u>*Top chord:*</u>
w_D = (2)(10) = **<u>20plf</u>**
w_{Lr} = (2)(20) = **<u>40plf</u>**
w_S = (2)(30.8) = **<u>61.6plf</u>**

<u>*Bot. chord:*</u>
w_D = (2)(10) = **<u>20plf</u>**

w_D = 20+20 = 40 plf
w_S = 61.6 plf
R_{max} =
$$\frac{(20+20+61.6)(19.41)}{2} = 986 \text{ lb.}$$



d) What are the maximum compression loads to W2, W5, and W6 and what is the purpose of the single row of bracing at midpoint?

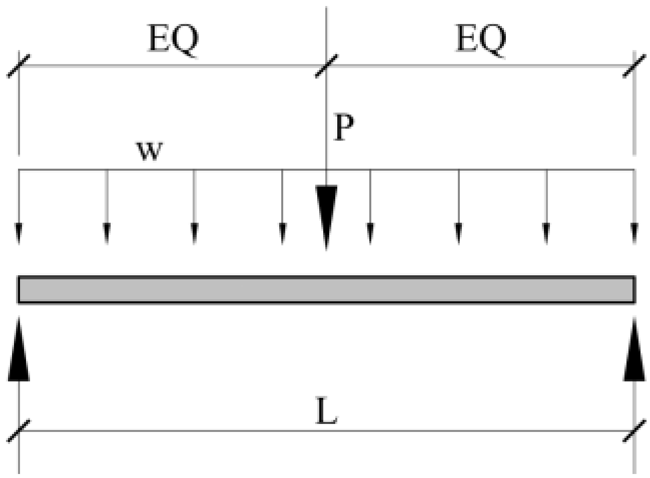
W2 – 860 lb
W5 – 899lb
W6 – 354 lb

The bracing limits the unbraced length of the members being braced and prevents buckling under compression.

2-11:

Given Loads:

Uniform load, w	Concentrated Load, P
D = 500plf	D = 11k
L = 800plf	S = 15k
S = 600plf	W = +12k or -12k
Beam length = 25 ft.	E = +8k or - 8k



Do the following:

a) Describe a practical framing scenario where these loads could all occur as shown.

b) Determine the maximum moment for each individual load effect (D, L, S, W, E)

c) Develop a spreadsheet to determine the worst-case bending moments for the code-required load combinations.

a) Transfer beam that has loads transferred from the roof down to a floor level.

Load Combinations

Uniform Loads | **Concentrated Loads** | $L_B := 25ft$

$w_D := 500plf$ $\quad P_D := 11kips$

$w_L := 800plf$ $\quad P_S := 15kips$

$w_S := 600plf$ $\quad P_W := 12kips$ $\quad P_{Wup} := -12kips$

$P_E := 8kips$ $\quad P_{Eup} := -8kips$

$$M_D := \frac{w_D \cdot L_B^2}{8} + \frac{P_D \cdot L_B}{4} = 108 \cdot ft \cdot kips \qquad M_W := \frac{P_W \cdot L_B}{4} = 75 \cdot ft \cdot kips \qquad M_{Wup} := \frac{P_{Wup} \cdot L_B}{4} = -75 \cdot ft \cdot kips$$

$$M_L := \frac{w_L \cdot L_B^2}{8} = 62 \cdot ft \cdot kips \qquad M_E := \frac{P_E \cdot L_B}{4} = 50 \cdot ft \cdot kips \qquad M_{Eup} := \frac{P_{Eup} \cdot L_B}{4} = -50 \cdot ft \cdot kips$$

$$M_S := \frac{w_S \cdot L_B^2}{8} + \frac{P_S \cdot L_B}{4} = 141 \cdot ft \cdot kips$$

$$LC1 := (1.4 \cdot M_D) = 151 \cdot ft \cdot kips$$

$$LC2 := (1.2 \cdot M_D) + (1.6 \cdot M_L) + (0.5 \cdot M_S) = 300 \cdot ft \cdot kips$$

$$LC3a := (1.2 \cdot M_D) + (1 \cdot M_L) + (1.6 \cdot M_S) = 417 \cdot ft \cdot kips$$

$$LC3b := (1.2 \cdot M_D) + (0.5 \cdot M_W) + (1.6 \cdot M_S) = 392 \cdot ft \cdot kips$$

$$LC4 := (1.2 \cdot M_D) + (1.6 \cdot M_W) + (M_L) + (0.5 \cdot M_S) = 382 \cdot ft \cdot kips$$

$$LC5 := (1.2 \cdot M_D) + (M_E) + (M_L) + (0.2 \cdot M_S) = 270 \cdot ft \cdot kips$$

$$LC6 := (0.9 \cdot M_D) + (1.0 \cdot M_{Wup}) = 22 \cdot ft \cdot kips$$

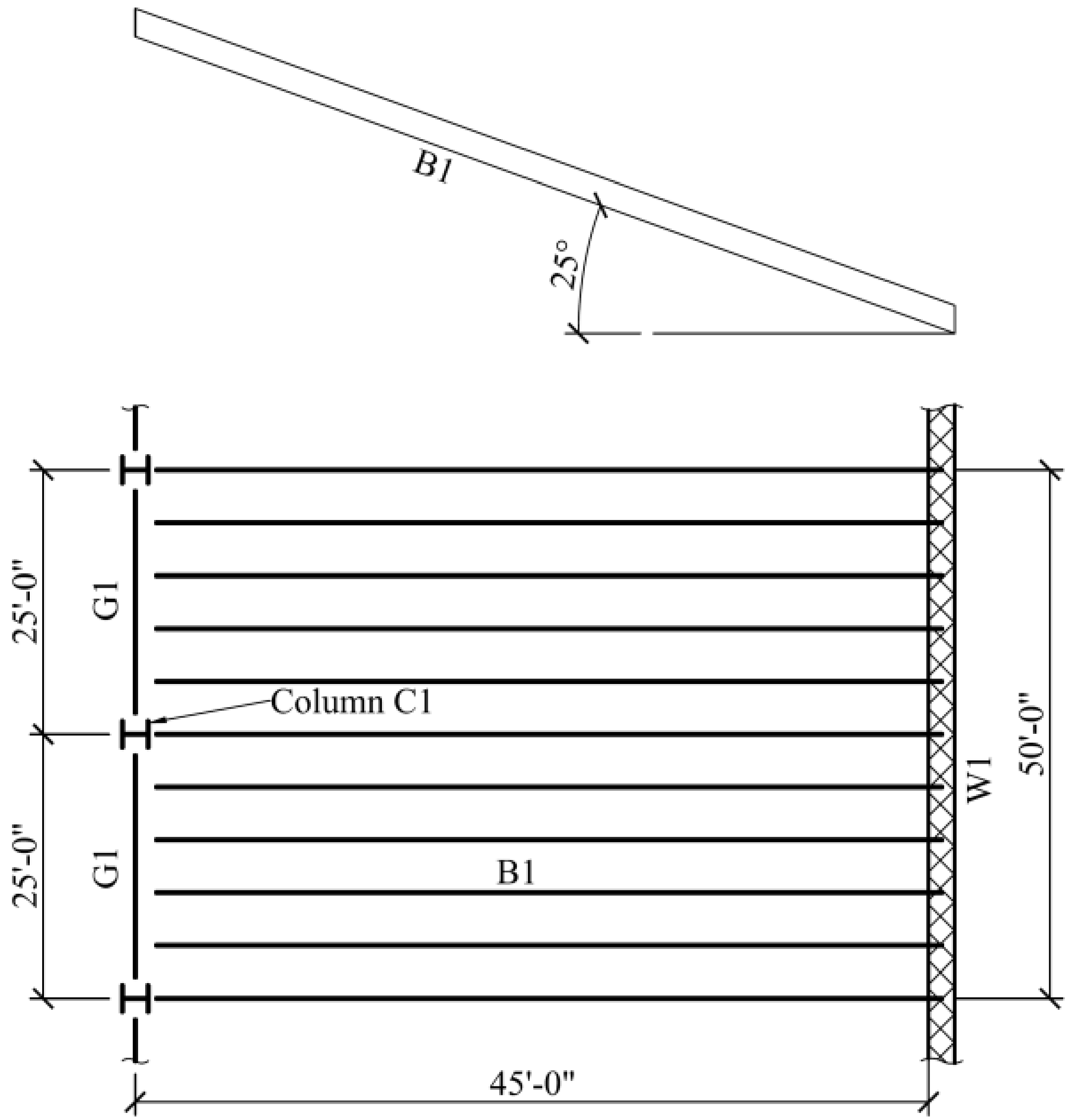
$$LC7 := (0.9 \cdot M_D) + (M_{Eup}) = 47 \cdot ft \cdot kips$$

$$M_{max} := \max(LC1, LC2, LC3a, LC3b, LC4, LC5) = 417 \cdot ft \cdot kips$$

$$M_{maxUp} := \min(LC6, LC7) = 22 \cdot ft \cdot kips$$

2-12 (see framing plan)
Assuming a roof dead load of 25 psf and a 25 degree roof slope, determine the following using the IBC factored load combinations. Neglect the rain load, R and assume the snow load, S is zero:

e. Determine the tributary areas of B1, G1, C1, and W1
f. The uniform dead and roof live load and the factored loads on B1 in PLF
g. The uniform dead and roof live load on G1 and the factored loads in PLF (Assume G1 is uniformly loaded)
h. The total factored axial load on column C1, in kips
i. The total factored uniform load on W1 in PLF (assume trib. length of 50 ft.)



Roof Framing Plan

$\text{Slope} := 25 \qquad F := 12\cdot\tan\left(\text{Slope}\cdot\frac{\pi}{180}\right) = 5.596 \qquad L_B := 45\text{ft} \qquad TW_B := 5\text{ft} \qquad D := 25\text{psf}$

$L_G := 25\text{ft} \qquad TW_W := 2\cdot L_G = 50\text{ ft}$

$R_2 := 1.2 - (0.05\cdot F) = 0.92$

Part (a):

$TA_{B1} := L_B\cdot TW_B = 225\text{ ft}^2 \qquad R_{1B1} := 1.2 - \frac{0.001\cdot TA_{B1}}{1\text{ft}^2} = 0.975$

$TA_{G1} := L_G\cdot\frac{L_B}{2} = 562.5\text{ ft}^2 \qquad R_{1G1} := 1.2 - \frac{0.001\cdot TA_{G1}}{1\text{ft}^2} = 0.638$

$TA_{C1} := L_G\cdot\frac{L_B}{2} = 563\text{ ft}^2 \qquad R_{1C1} := 1.2 - \frac{0.001\cdot TA_{C1}}{1\text{ft}^2} = 0.638$

$TW_{W1} := TW_W\cdot\frac{L_B}{2} = 1125\text{ ft}^2 \qquad R_{1W1} := 0.6$

Part (b):

$L_{rB1} := \max\left[0.6\cdot 20\text{psf}, \left(R_{1B1}\cdot R_2\cdot 20\text{psf}\right)\right] = 17.9\cdot\text{psf}$

$w_{DB1} := TW_B\cdot D = 125\cdot\text{plf} \qquad w_{LrB1} := TW_B\cdot L_{rB1} = 90\cdot\text{plf} \qquad w_{uB1} := \left(1.2\cdot w_{DB1}\right) + \left(1.6\cdot w_{LrB1}\right) = 294\cdot\text{plf}$

Part (c):

$L_{rG1} := \max\left[0.6\cdot 20\text{psf}, \left(R_{1G1}\cdot R_2\cdot 20\text{psf}\right)\right] = 12\cdot\text{psf}$

$w_{DG1} := \frac{L_B}{2}\cdot D = 563\cdot\text{plf} \qquad w_{LrG1} := \frac{L_B}{2}\cdot L_{rG1} = 270\cdot\text{plf} \qquad w_{uG1} := \left(1.2\cdot w_{DG1}\right) + \left(1.6\cdot w_{LrG1}\right) = 1107\cdot\text{plf}$

Part (d):

$L_{rC1} := \max\left[0.6\cdot 20\text{psf}, \left(R_{1C1}\cdot R_2\cdot 20\text{psf}\right)\right] = 12\cdot\text{psf}$

$P_{DC1} := TA_{C1}\cdot D = 14\cdot\text{kips} \qquad P_{LrC1} := TA_{C1}\cdot L_{rC1} = 7\cdot\text{kips} \qquad P_{uC1} := \left(1.2\cdot P_{DC1}\right) + \left(1.6\cdot P_{LrC1}\right) = 28\cdot\text{kips}$

Part (e):

$L_{rW1} := \max\left[0.6\cdot 20\text{psf}, \left(R_{1W1}\cdot R_2\cdot 20\text{psf}\right)\right] = 12\cdot\text{psf}$

$w_{DW1} := \frac{L_B}{2}\cdot D = 563\cdot\text{plf} \qquad w_{LrW1} := \frac{L_B}{2}\cdot L_{rW1} = 270\cdot\text{plf} \qquad w_{uW1} := \left(1.2\cdot w_{DW1}\right) + \left(1.6\cdot w_{LrW1}\right) = 1107\cdot\text{plf}$

2-13. A 3-story building has columns spaced at 25 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. Develop this table using a spreadsheet. Submit a hard copy that is properly formatted with your HW and submit the XLS file by e-mail.

Roof Loads:	2nd & 3rd floor loads
Dead, D = 20psf	Dead, D = 60psf
Snow, S = 45psf	Live, L = 100psf

All other loads are 0

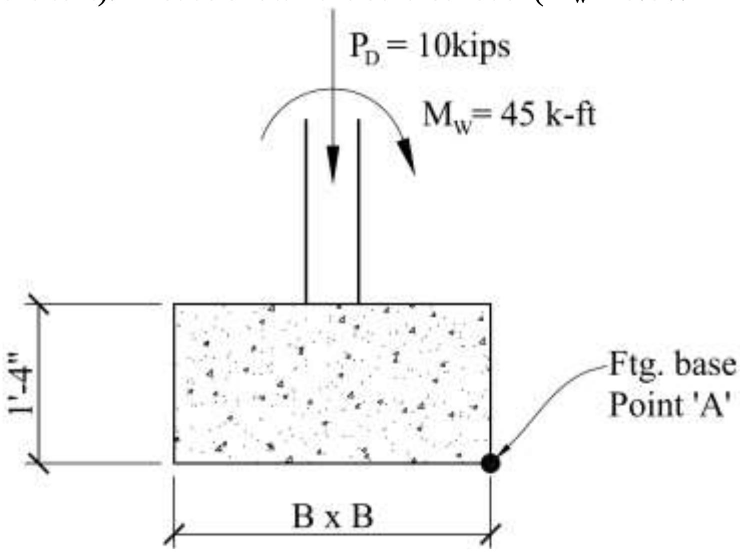
Column Load Table

L1 = 25 ft
L2 = 25 ft

Level	TA	D	S	L	wu1	wu2	Pu1	Pu2	Cumulative Pu1	Cumulative Pu2	Max. Load
	($ft.^2$)	(psf)	(psf)	(psf)	(psf)	(psf)	(kips)	(kips)	(kips)	(kips)	(kips)
Roof	625	20	45	0	46.5	96	29.06	60.00	29.06	60.00	60.00
3rd	625	60	0	100	232	122	145.00	76.25	174.06	136.25	174.06
2nd	625	60	0	100	232	122	145.00	76.25	319.06	212.50	319.06

Pu1, wu1 = 1.2D+1.6L+0.5S
Pu2, wu2 = 1.2D+0.5L+1.6S

2-14 Using only the loads shown and the weight of the concrete footing only (γ_{conc} = 150pcf), determine the required square footing size, BxB using the appropriate load combination to keep the footing from overturning about point A (i.e. - either load combination 6 or 15 Chapter 2 of the text). Loads shown are service level (M_w = 0.6W = 45k-ft)



$P_D := 10kips$ $\quad M_W := 45ft \cdot kips$ $\quad \gamma_{conc} := 150pcf$

$B := 7.67ft$ $\quad H := 1.333ft$ $\quad P_{ftg} := B \cdot B \cdot H \cdot \gamma_{conc} = 11.8 \cdot kips$

Overturning Moment **Resisting Moment**

$OM := M_W = 45 \cdot ft \cdot kips$ $\quad RM := (P_D + P_{ftg}) \cdot \frac{B}{2} = 83.5 \cdot ft \cdot kips$

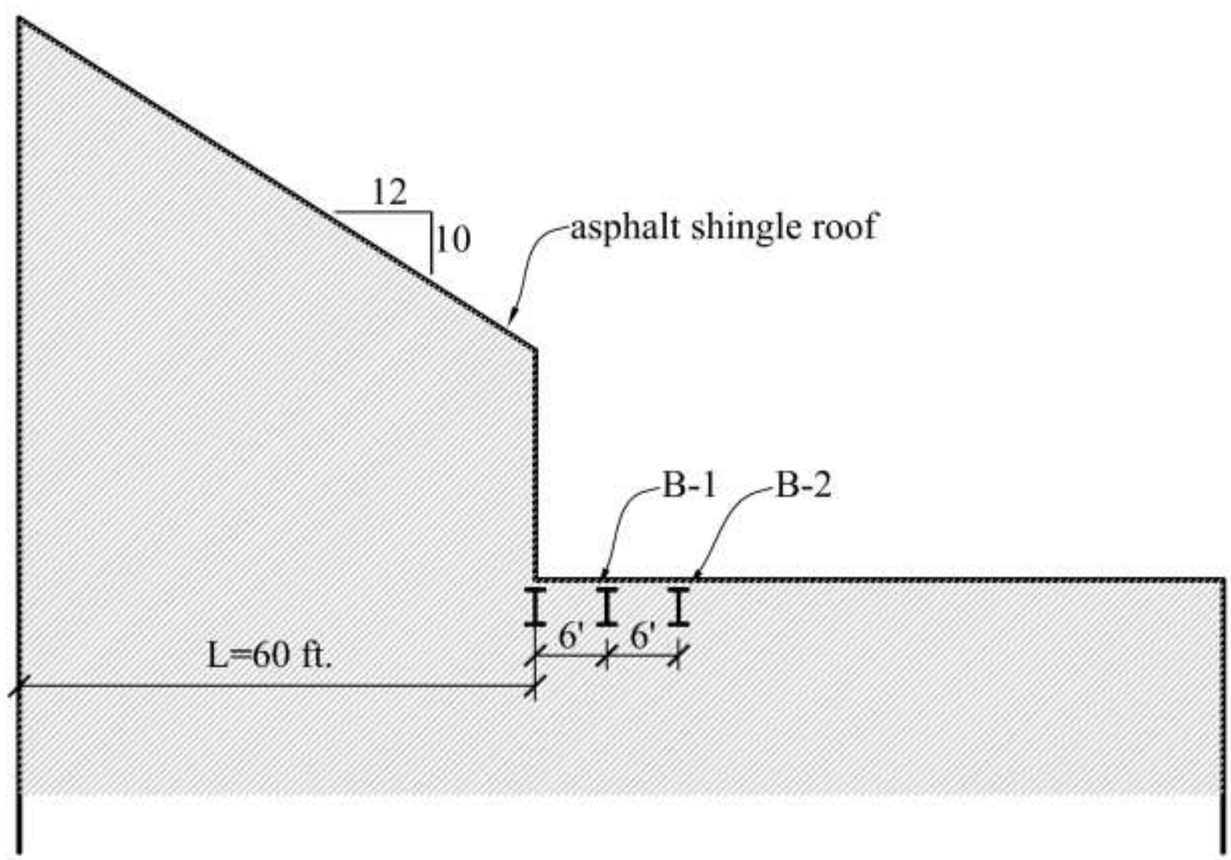
ASD Load Comb **LRFD Load Comb**

$Unity_{ASD} := \frac{(0.6 \cdot RM)}{OM} = 1.113$ $\quad Unity_{LRFD} := \frac{(0.9 \cdot RM)}{\left(\frac{OM}{0.6}\right)} = 1.002$ ***must be greater than 1.0***

Use B=7.28ft for ASD and 7.67ft for LRFD

2-15.
Given:
Location - Massena, NY; elevation is less than 1000 feet
Total roof DL = 25psf
Ignore roof live load; consider load combination 1.2D+1.6S only
Use normal occupancy, temperature, and exposure conditions
Length of B-1, B-2 is 30 ft.

Find:
a) Flat roof snow load and sloped roof snow load
b) Sliding snow load
c) Determine the depth of the balanced snow load and the sliding snow load on B-1 and B-2
d) Draw a free-body diagram of B-1 showing the service dead and snow loads in PLF
e) Find the factored Moment and Shear in B-1.



Problem 3-1

$p_g := 60psf \quad C_e := 1.0 \quad C_t := 1.0 \quad I_s := 1.0 \quad \theta := atan\left(\frac{10}{12}\right)\cdot\left(\frac{180}{\pi}\right) = 39.806$

$C_s := \frac{5}{3} - \frac{\theta}{45} = 0.782 \quad W_{SL} := 60ft$

$P_f := 0.7p_g \cdot C_e \cdot C_t \cdot I_s = 42\,psf \quad P_s := P_f \cdot C_s = 32.848\,psf$ ***part (a)***

$P_{SL} := \frac{0.4 \cdot P_f \cdot W_{SL}}{15ft} = 67.2\,psf$ ***part (b)***

$\gamma_{snow} := \frac{0.13}{1ft} \cdot p_g + 14pcf = 21.8\,pcf$

$h_{bal} := \frac{P_f}{\gamma_{snow}} = 1.927\,ft \quad h_{SL} := \frac{P_{SL}}{\gamma_{snow}} = 3.083\,ft$ ***part (c)***

$L_B := 30ft \quad TW := 6ft \quad D := 25psf$

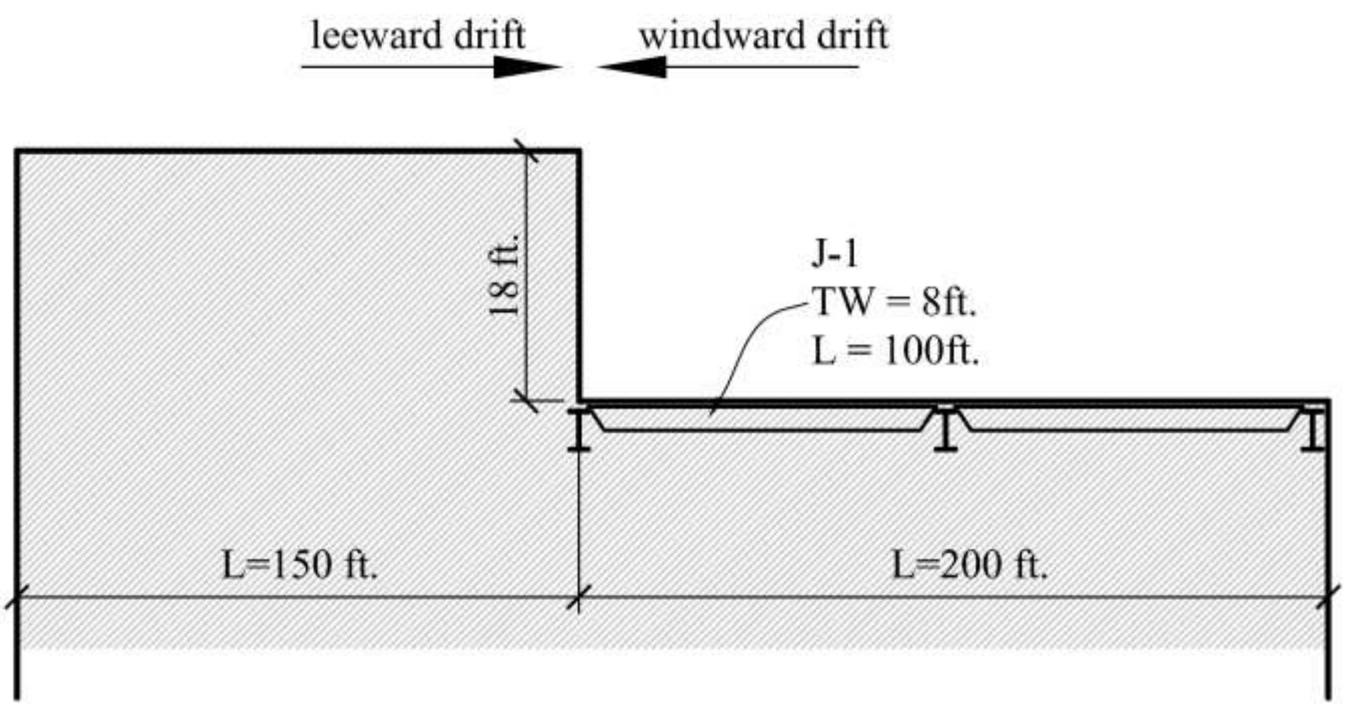
$w_D := TW \cdot D = 150\,plf \quad w_S := TW \cdot P_f = 252\,plf \quad w_{SL} := TW \cdot P_{SL} = 403.2\,plf$ ***part (d)***

$w_u := \left(1.2 \cdot w_D\right) + \left[1.6 \cdot \left(w_S + w_{SL}\right)\right] = 1228.3\,plf$

$M_u := \frac{w_u \cdot L_B^{\,2}}{8} = 138.2\,ft \cdot kips \quad V_u := \frac{w_u \cdot L_B}{2} = 18.4\,kips$ ***part (e)***

2-16.
Given:
Location - Pottersville, NY; elevation is 1500 feet
Total roof DL = 20psf
Ignore roof live load; consider load combination 1.2D+1.6S only
Use normal occupancy, temperature, and exposure conditions

Find:
a) Flat roof snow load
b) Depth and width of the leeward drift and windward drifts; which one controls the design of J-1?
c) Determine the depth of the balanced snow load and controlling drift snow load
d) Draw a free-body diagram of J-1 showing the service dead and snow loads in PLF



$p_g := 70psf + 10psf = 80 \cdot psf$ $\quad C_e := 1.0 \quad C_t := 1.0 \quad I_s := 1.0$

$P_f := 0.7 p_g \cdot C_e \cdot C_t \cdot I_s = 56 \cdot psf$ ***part (a)***

$$L_{uW} := 200ft \qquad h_{dW} := 0.75ft \cdot \left[0.43 \cdot \left(\frac{L_{uW}}{1ft} \right)^{\frac{1}{3}} \cdot \left[\left(\frac{p_g + 10psf}{1psf} \right)^{\frac{1}{4}} \right] - 1.5 \right] = 4.684\,ft$$

part (b)

$$L_{uL} := 150ft \qquad h_{dL} := 1ft \cdot \left[0.43 \cdot \left(\frac{L_{uL}}{1ft} \right)^{\frac{1}{3}} \cdot \left[\left(\frac{p_g + 10psf}{1psf} \right)^{\frac{1}{4}} \right] - 1.5 \right] = 5.537\,ft$$

$$\gamma_{snow} := \frac{0.13}{1ft} \cdot p_g + 14pcf = 24.4 \cdot pcf \qquad h_{bal} := \frac{P_f}{\gamma_{snow}} = 2.295\,ft$$

$w_W := 4 \cdot h_{dW} = 18.736\,ft$ $\qquad w_L := 4 \cdot h_{dL} = 22.148\,ft$ ***part (b)***

<u>The Leeward drift will control the design</u>

$SD := \gamma_{snow} \cdot h_{dL} = 135.1 \cdot psf$ ***part (c)***

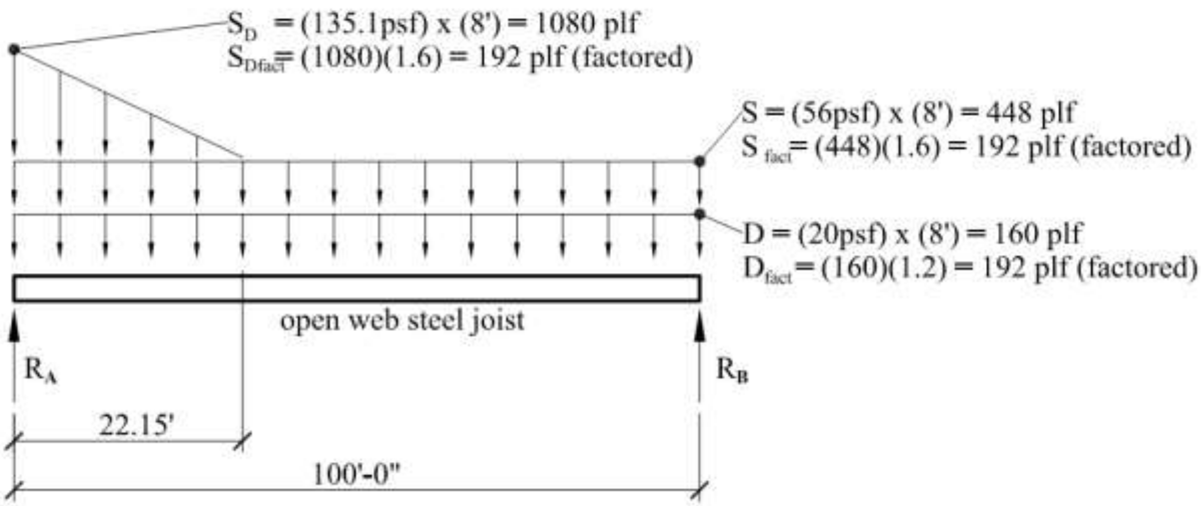
$L_B := 100ft \qquad TW := 8ft \qquad D := 20psf$

$w_D := TW \cdot D = 160 \cdot plf \qquad w_S := TW \cdot P_f = 448 \cdot plf \qquad w_{SD} := TW \cdot SD = 1081 \cdot plf$

$w_u := (1.2 \cdot w_D) + \left[1.6 \cdot (w_S + w_{SD}) \right] = 2638 \cdot plf$ ***part (d)***

$w_{uS} := 1.6 \cdot (w_S + w_{SD}) = 2446 \cdot plf$

Problem 2-16 Part (e)



Factored Reactions and Maximum Moment:

R_A = 63.18 kips
R_B = 46.85 kips

$M_{u,\,max}$ = 1207.6 ft-kips (occurs at 51.55 ft from B)