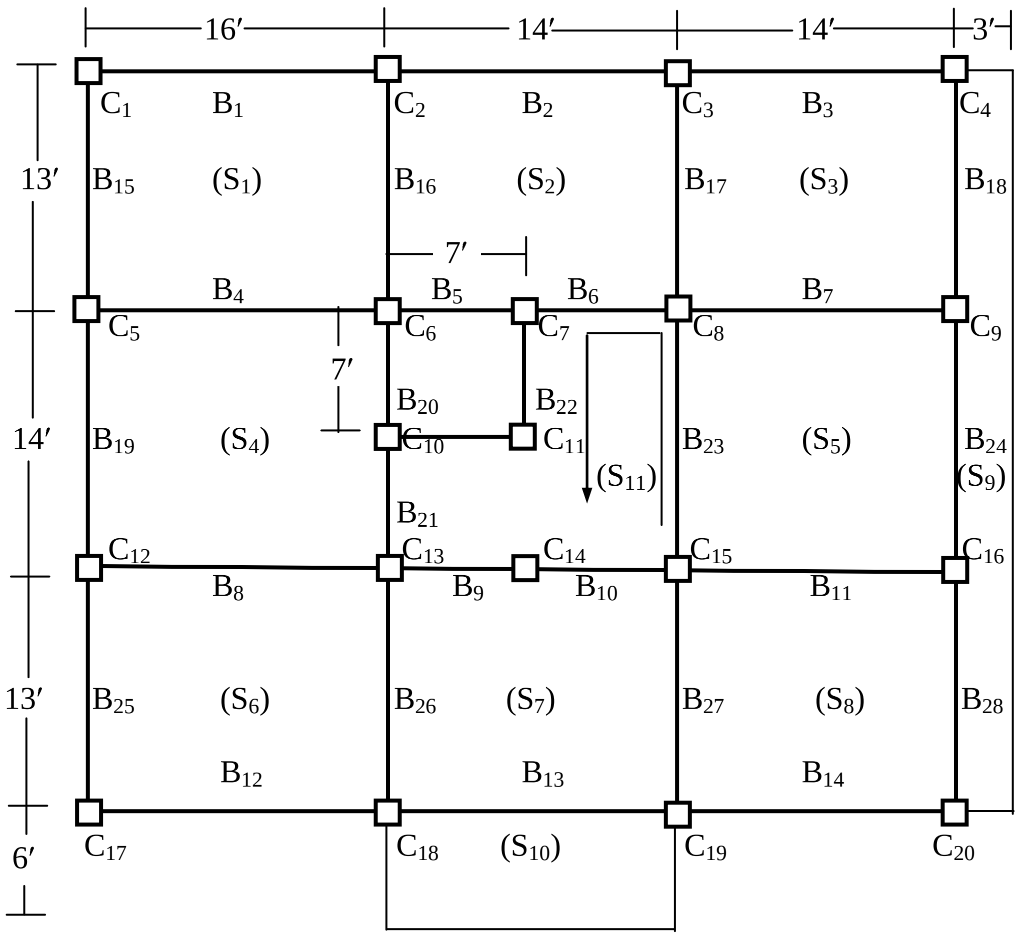
Design of a multi-storied steel building



Building Plan

Building Height = 4@10′ = 40′

Loads: LL = 40 psf, FF = 20 psf, RW = 20 psf

Seismic Coefficients: Z = 0.15, I = 1.0, S = 1.0, R = 6.0 [i.e., R = 4.0]

Material Properties: f'_c = 3 ksi, f_y = 40 ksi, Allowable Bearing Capacity of soil = 2 ksf

1. Load Calculation for Slabs

Slab (S_1) to Slab (S_9):

Assumed slab thickness, t = 4.5″

$\therefore$ Self Weight of slab = 4.5×150/12 = 56.25 psf

$\Rightarrow$ DL = 56.25 + 20 + 20 = 96.25 psf = 0.096 ksf

LL = 40 psf = 0.04 ksf

$\therefore$ Total Wt./slab area = 0.096 + 0.04 = 0.136 ksf

Slab (S_{10}):

Assumed slab thickness, t = 5″

$\therefore$ Self Weight of slab = 5×150/12 = 62.5 psf

FF = 20 psf, but there is no random wall and LL is less (i.e., assumed to be 20 psf)

$\therefore$ Total Wt./slab area, w = 62.5 + 20 + 20 = 102.5 psf = 0.103 ksf

Slab (S_{11}):

Assumed slab thickness, t = 7″

$\therefore$ Self Weight of slab = 7×150/12 = 87.5 psf

FF = 20 psf, but there is no random wall and LL is high (i.e., assumed to be 100 psf)

$\therefore$ Total Wt./slab area, w = 87.5 + 20 + 100 = 207.5 psf = 0.208 ksf

Additional weight on flights due to 6″ high stairs = ½ × (6/12) × 150 psf = 37.5 psf

$\therefore$ Total Wt./slab area, w = 207.5 + 37.5 = 245 psf = 0.245 ksf

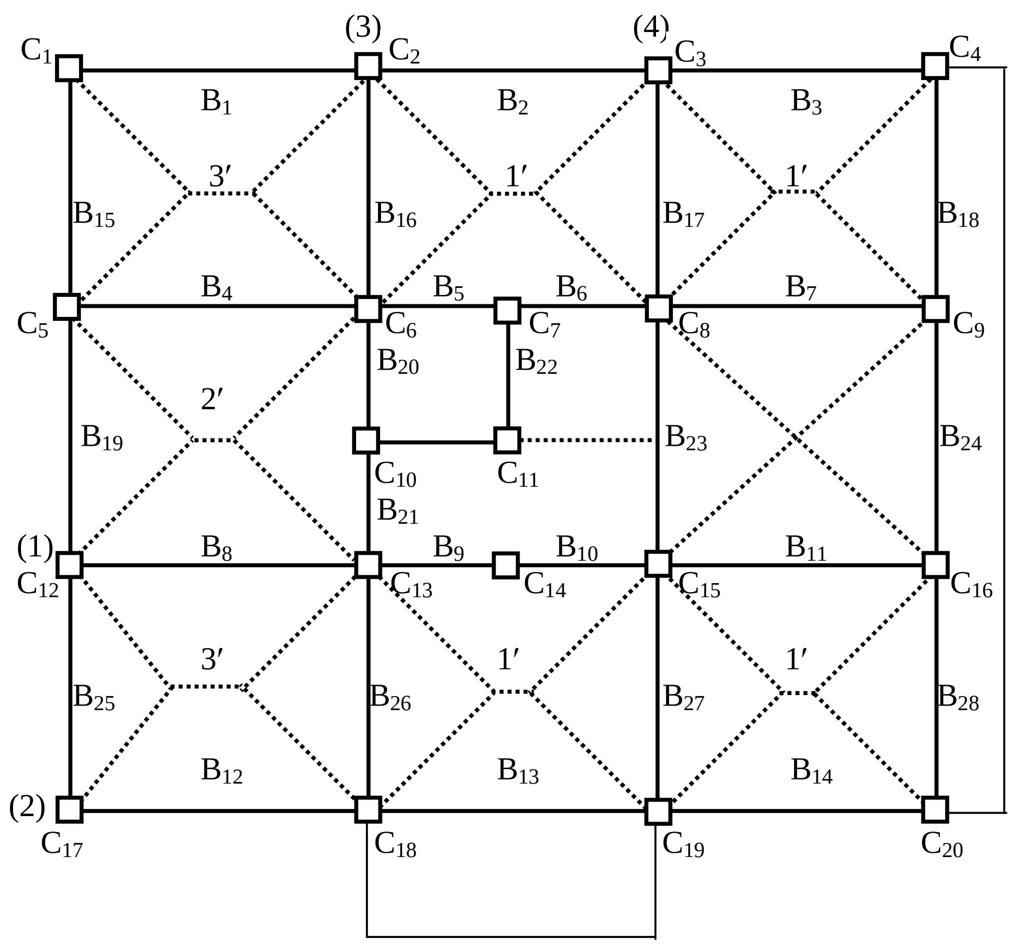
2. Vertical Load Analysis of Beams and Columns

Beams are assumed to have self-weights of about 30 lb/ft; i.e., 0.03 k/′

Partition Walls (PW) are assumed to be 5″ thick and Exterior Walls (EW) 10″ thick

Weight of 5″ PW = (5″/12)×9′×120 = 450 lb/′ = 0.45 k/′

Weight of 10″ EW = (10″/12)×9′×120 = 900 lb/′ = 0.90 k/′



Load Distribution from Slab to Beam

Frame (1) [$B_{8\text{-}9\text{-}10\text{-}11}$]:

Slab-load on B_8 = [13/2×(16+3)/2+14/2×(16+2)/2]× 0.136 = 16.97 ^{k}

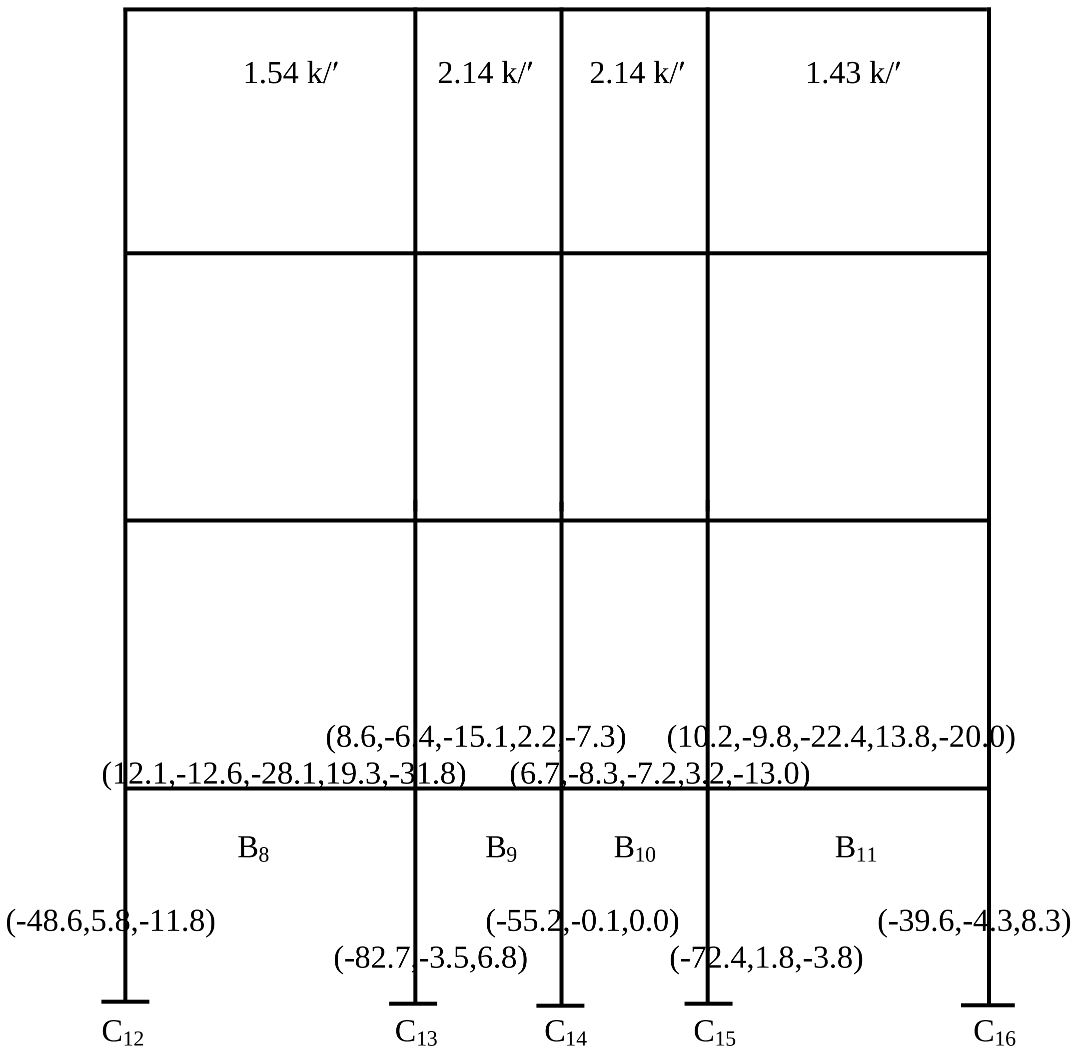
∴ Equivalent UDL (+ Self Wt. and PW) ≅ 16.97/16 + 0.03 + 0.45 = 1.54 k/′

Slab-load on $B_{9\text{-}10}$ ≅ 13/2×(14+1)/2×0.136 +(14×14–7×7)/2×(0.208+0.245)/2 = 23.28 ^{k}

∴ Equivalent UDL (+ Self Wt. and PW) ≅ 23.28/14 + 0.03 + 0.45 = 2.14 k/′

Load from Slabs to B_{11} = [13/2×(14+1)/2+14/2×(14)/2]× 0.136 = 13.29 ^{k}

∴ Equivalent UDL (+ Self Wt. and PW) ≅ 13.29/14 + 0.03 + 0.45 = 1.43 k/′



Beam (SF_1, SF_2 (k), BM_1, BM_0, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame (1) from Vertical Load Analysis

Frame (2) [$B_{12\text{-}13\text{-}14}$]:

Slab-load on B_{12} = [13/2×(16+3)/2]× 0.136 = 8.40 ^{k}

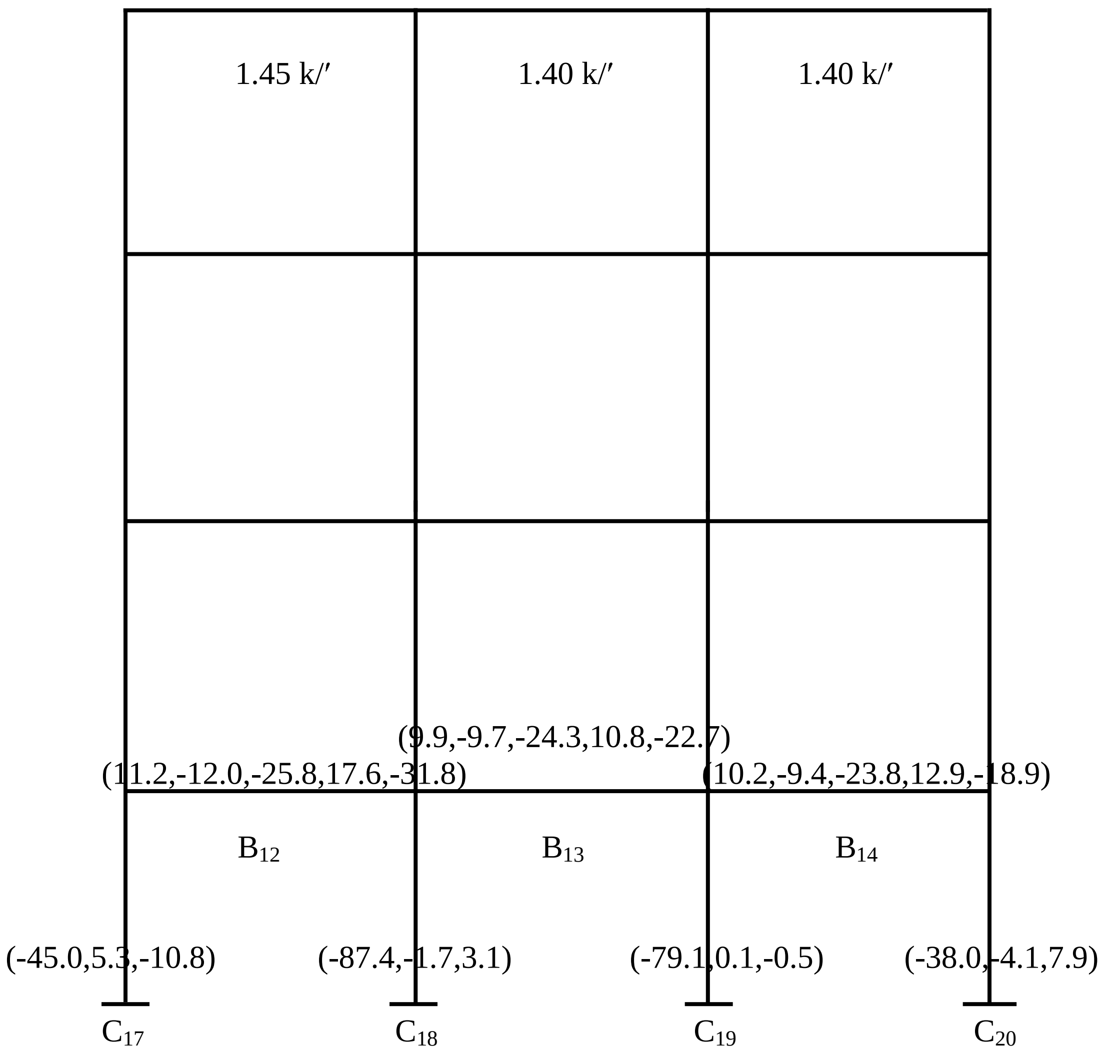
∴ Equivalent UDL (+ Self Wt. and EW) ≅ 8.40/16 + 0.03 + 0.90 = 1.45 k/′

Slab-load on B_{13} = [13/2×(14+1)/2]×0.136 = 6.63^{k}

∴ Equivalent UDL (+ Self Wt. and EW) ≅ 6.63/14 + 0.03 + 0.90 = 1.40 k/′

Load from Slabs to B_{14} = [13/2×(14+1)/2]× 0.136 = 6.63^{k}

∴ Equivalent UDL (+ Self Wt. and EW) ≅ 6.63/14 + 0.03 + 0.90 = 1.40 k/′



Beam (SF_1, SF_2 (k), BM_1, BM_0, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame (2) from Vertical Load Analysis

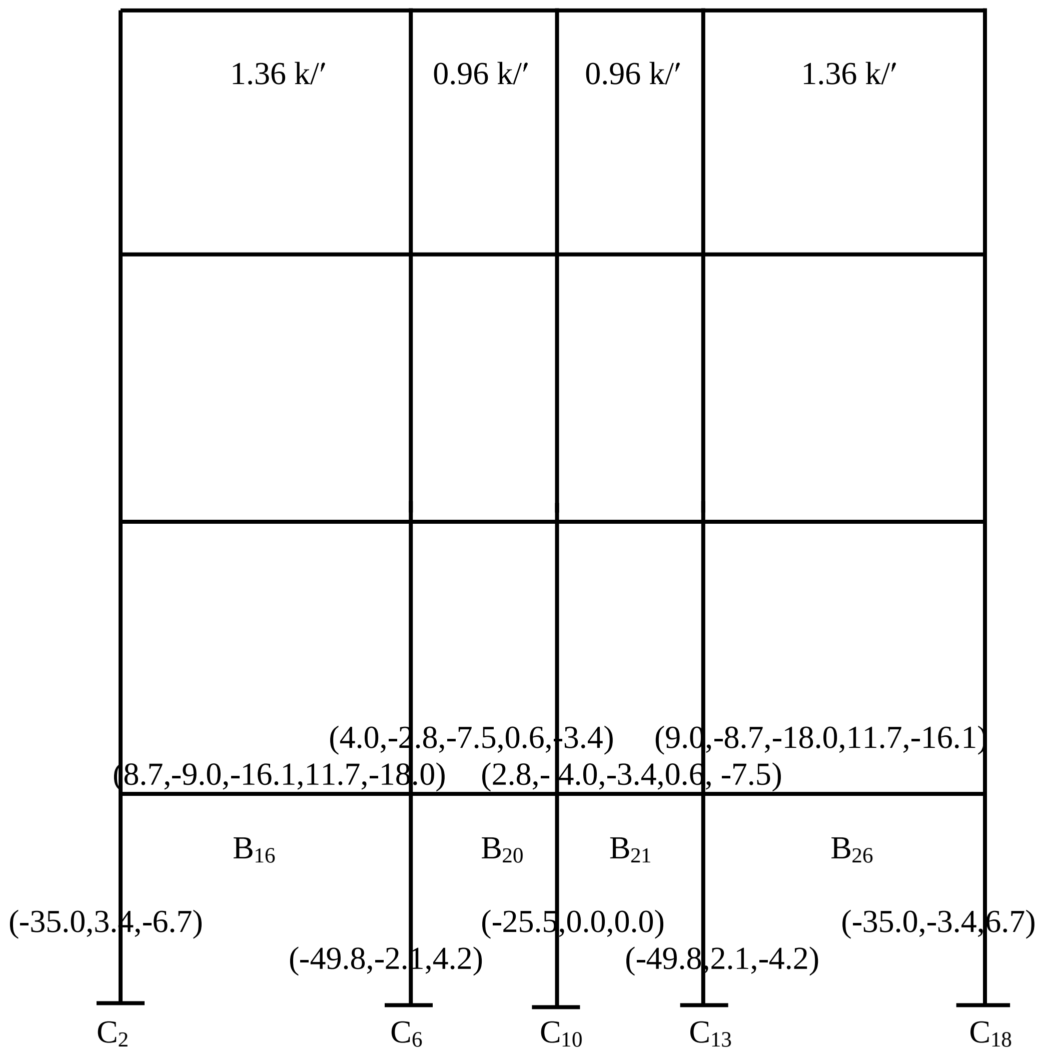
Frame (3) [$B_{16-20-21-26}$]:

Slab-load on B_{16} and B_{26} = [13/2×(13)/2+13/2×(13)/2]× 0.136 = 11.49 ^{k}

$\therefore$ Equivalent UDL (+ Self Wt. and PW) ≅ 11.49/13 + 0.03 + 0.45 = 1.36 k/′

Slab-load on B_{20-21} ≅ [14/2×(14)/2]× 0.136 = 6.66 ^{k}

$\therefore$ Equivalent UDL (+ Self Wt. and PW) ≅ 6.66/14 + 0.03 + 0.45 = 0.96 k/′



Beam (SF_1, SF_2 (k), BM_1, BM_0, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame (3) from Vertical Load Analysis

Frame (4) [$B_{17\text{-}23\text{-}27}$]:

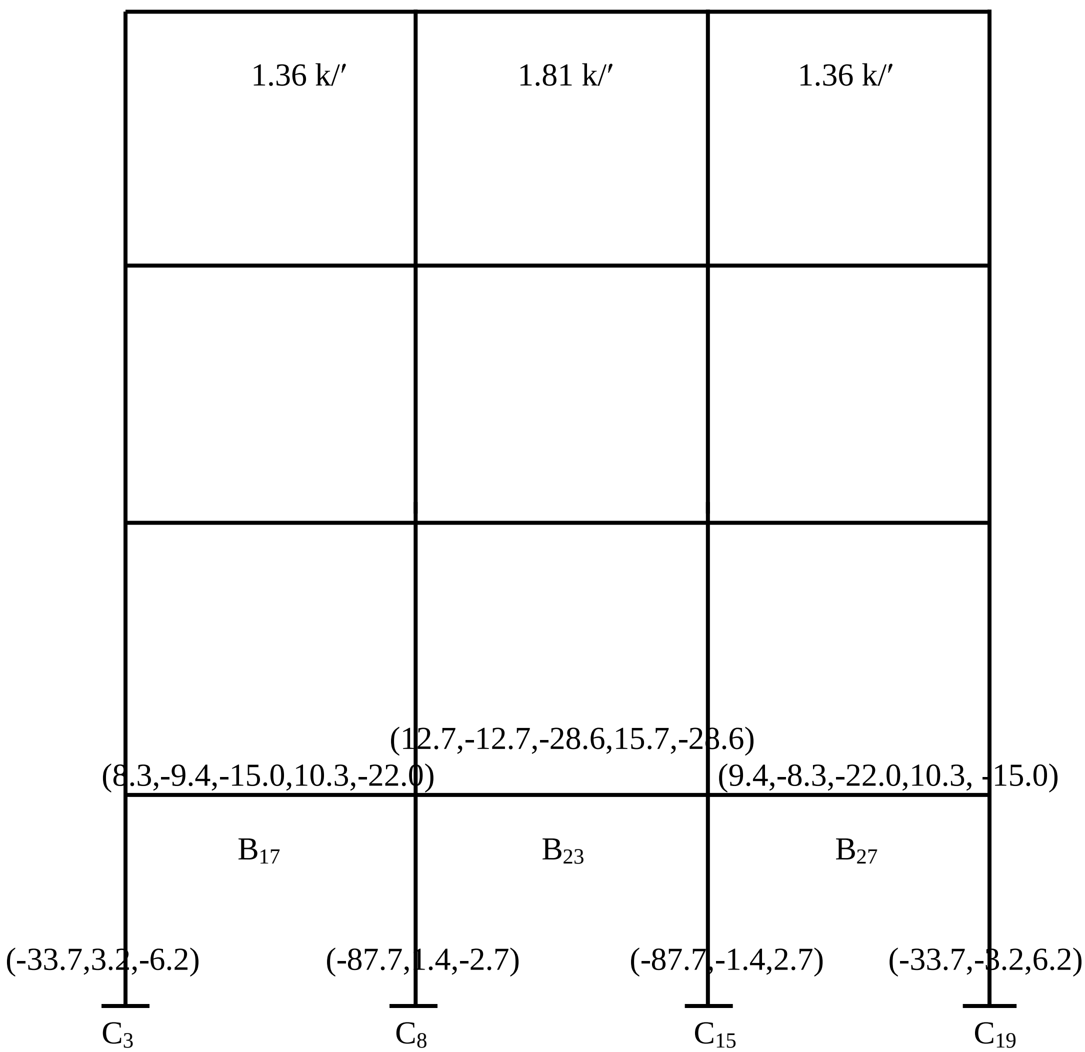
Slab-load on B_{17} and B_{27} = [13/2×(13)/2+13/2×(13)/2]× 0.136 = 11.49 ^{k}

$\therefore$ Equivalent UDL (+ Self Wt. and PW) $\cong$ 11.49/13 + 0.03 + 0.45 = 1.36 k/′

Slab-load on B_{23} = [14/2×(14)/2]× 0.136 = 6.66^{k}

$\therefore$ Equivalent UDL (+ Self Wt., EW, S_9) $\cong$ 6.66/14 + 0.03 + 0.90 + 3 × 0.136 = 1.81 k/′

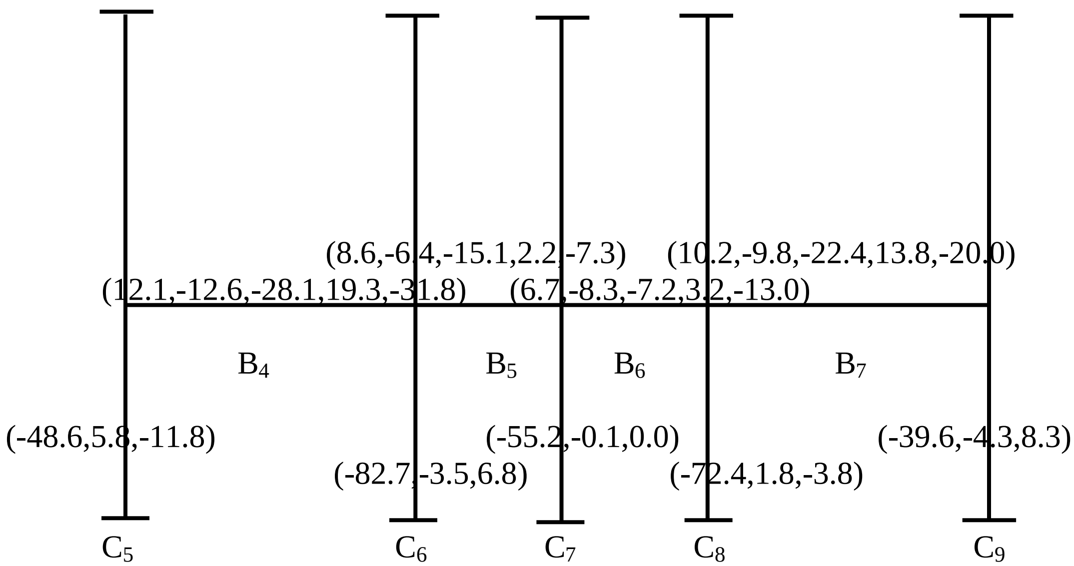
[Here, the EW is considered because the exterior beam B_{24} is more critical. It has the same slab load as B_{23} in addition to self-weight and EW]



Beam (SF_1, SF_2 (k), BM_1, BM_0, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame (4) from Vertical Load Analysis

Frame [$B_{4-5-6-7}$]:

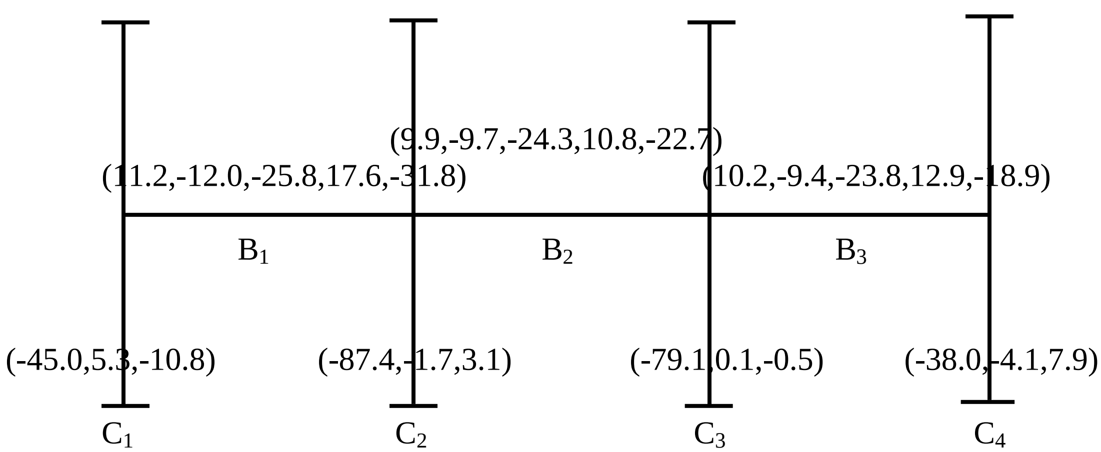
Similar to Frame (1) [$B_{8-9-10-11}$].



Beam (SF_1, SF_2 (k), BM_1, BM_0, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame [$B_{4-5-6-7}$] from Vertical Load Analysis

Frame [B_{1-2-3}]:

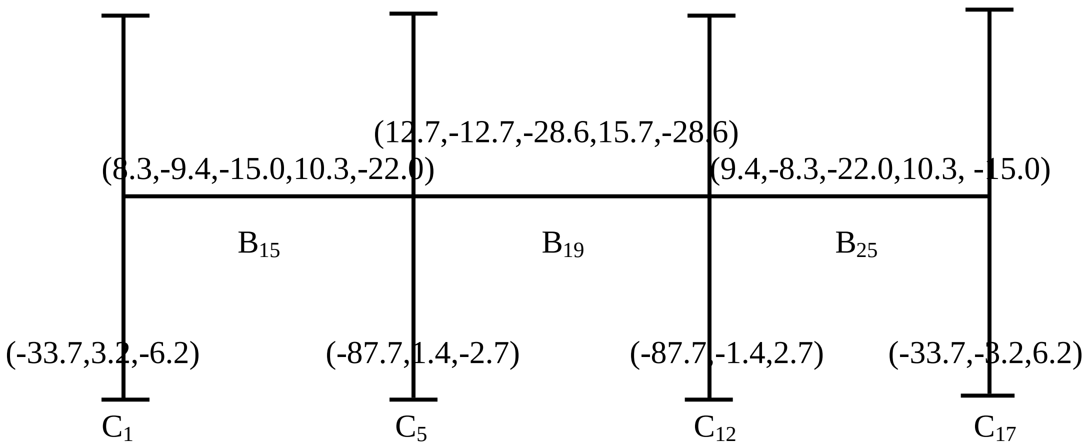
Similar to Frame (2) [$B_{12-13-14}$].



Beam (SF_1, SF_2 (k), BM_1, BM_0, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame [B_{1-2-3}] from Vertical Load Analysis

Frame [$B_{15\text{-}19\text{-}25}$]:

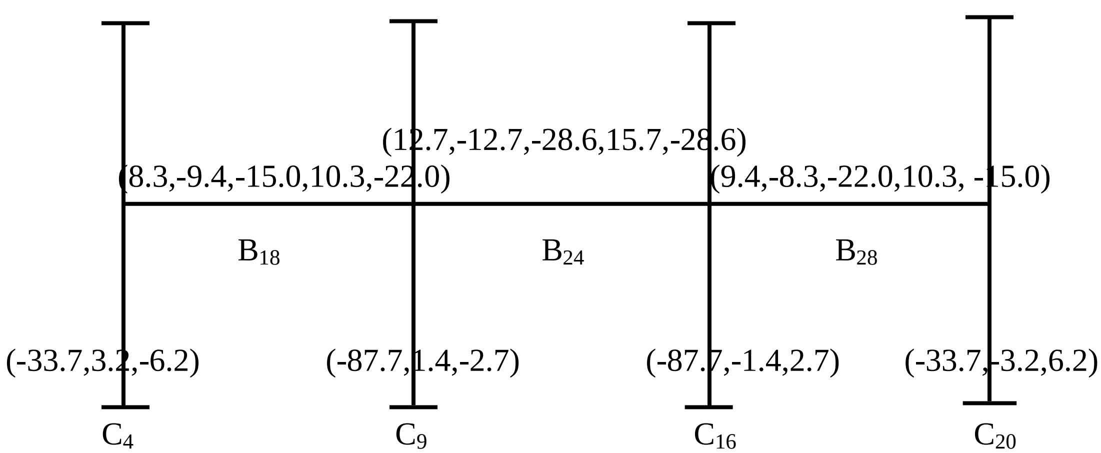
Similar to Frame (4) [$B_{17\text{-}23\text{-}27}$].



Beam (SF_1, SF_2 (k), BM_1, BM_0, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame [$B_{15\text{-}19\text{-}25}$] from Vertical Load Analysis

Frame [$B_{18\text{-}24\text{-}28}$]:

Similar to Frame (4) [$B_{17\text{-}23\text{-}27}$].



Beam (SF_1, SF_2 (k), BM_1, BM_0, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame [$B_{18\text{-}24\text{-}28}$] from Vertical Load Analysis

3. Lateral Load Analysis of Beams and Columns

Seismic Coefficients: $Z = 0.15$, $I = 1.0$, $S = 1.0$, $R = 6.0$, $C = 1.25\ S/T^{2/3} \leq 2.75$

where T = Fundamental period of vibration = $0.083\times h^{3/4}$ [h = Building Height in meters]

For the moment resisting steel frame, $T = 0.083\times(40/3.28)^{3/4} = 0.542$ sec

$\therefore C = 1.25\ S/T^{2/3} = 1.25\times1.0/(0.542)^{2/3} = 1.88$

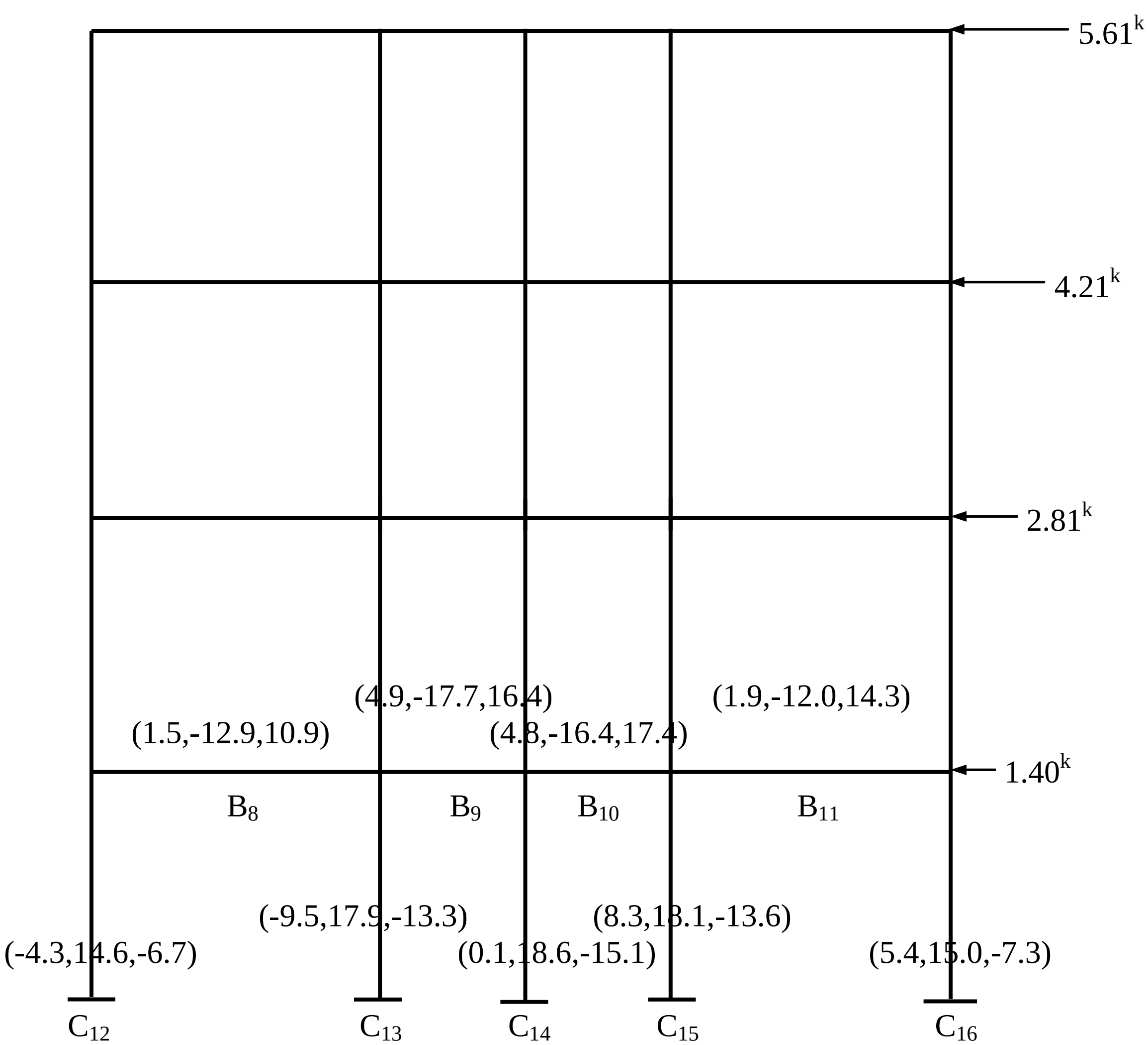
$\therefore$ Base Shear, $V = (ZIC/R)\ W = 0.15\times1.0\times1.88/6.0\ W = 0.0470W$

Since $T < 0.7$ sec, $F_t = 0$; $\therefore$ For equally loaded stories without F_t, $F_i = (h_i/\sum h_i)V$

$\Rightarrow F_1 = 0.1V$, $F_2 = 0.2V$, $F_3 = 0.3V$, $F_4 = 0.4V$

Frame (1) [$B_{8\text{-}9\text{-}10\text{-}11}$]:

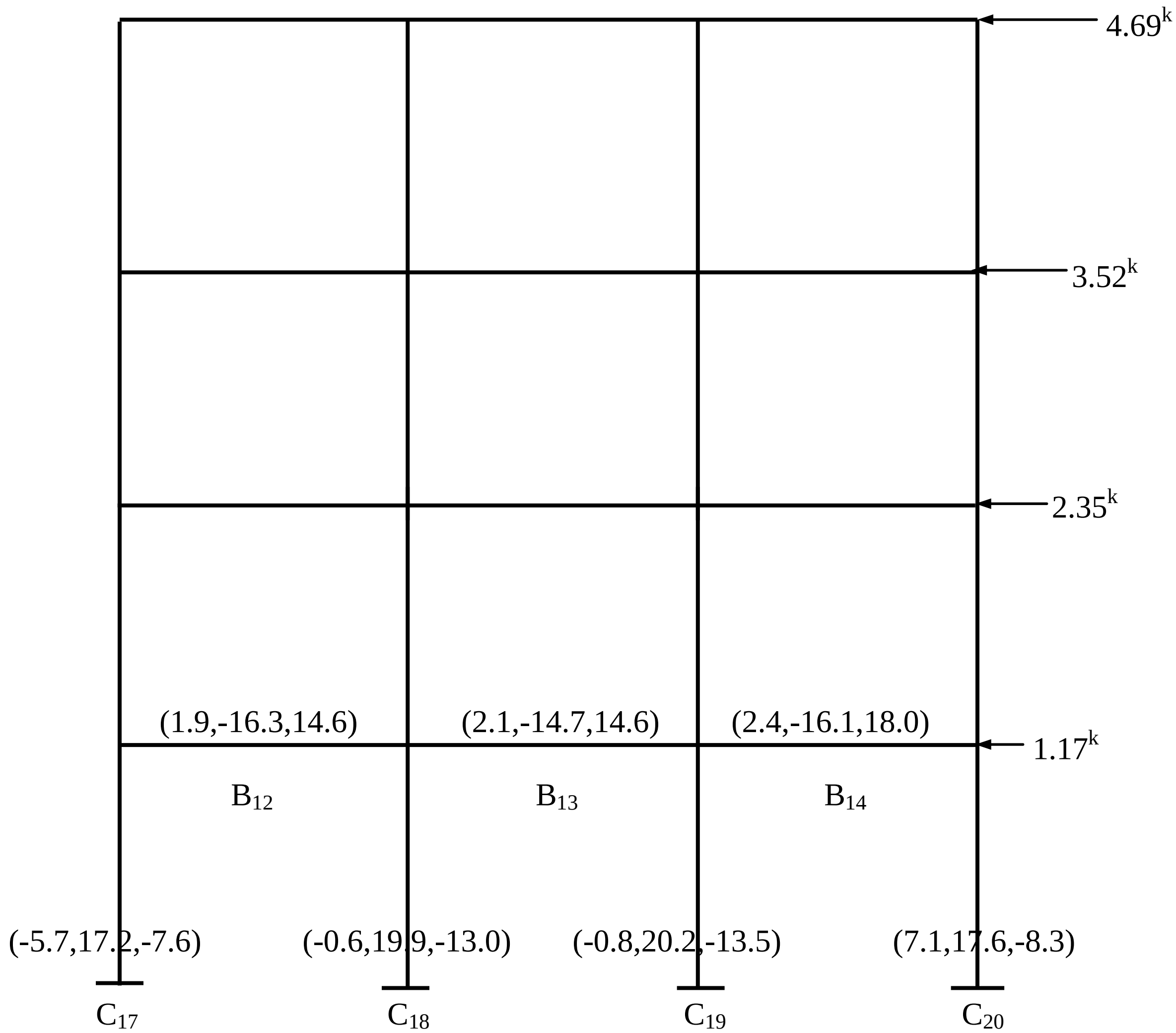
$W = 4\times(1.54\times16+2.14\times7+2.14\times7+1.43\times14) = 298^k \Rightarrow V = 0.0470W = 14.03^k$



Beam (SF(k), BM_1, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame (1) from Lateral Load Analysis

Frame (2) [$B_{12\text{-}13\text{-}14}$]:

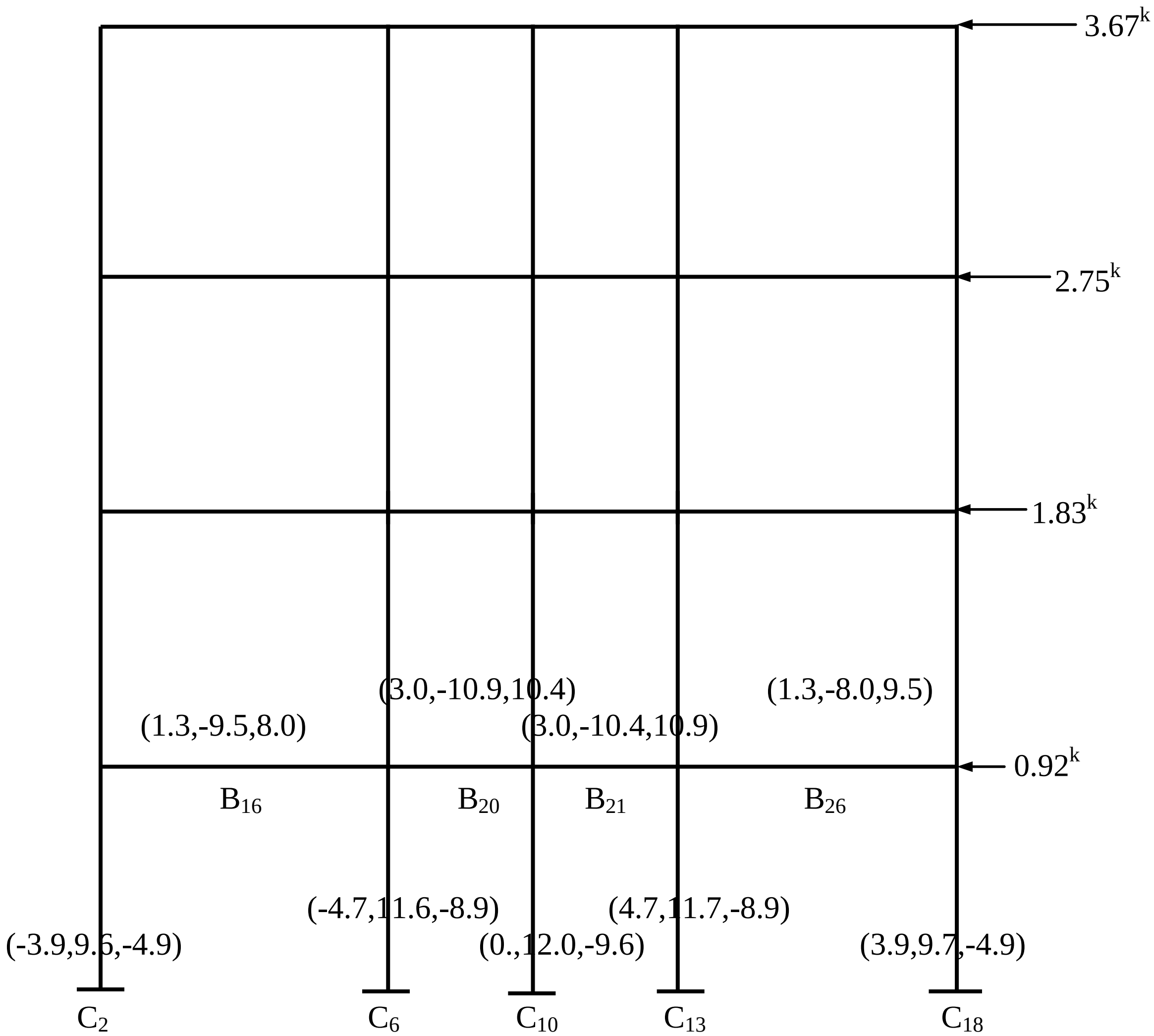
$W = 4\times(1.45\times16+1.40\times14+1.40\times14) = 249.60^k \Rightarrow V = 0.0470W = 11.73^k$



Beam (SF(k), BM_1, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame (2) from Lateral Load Analysis

<u>Frame (3) [$B_{16-20-21-26}$]:</u>

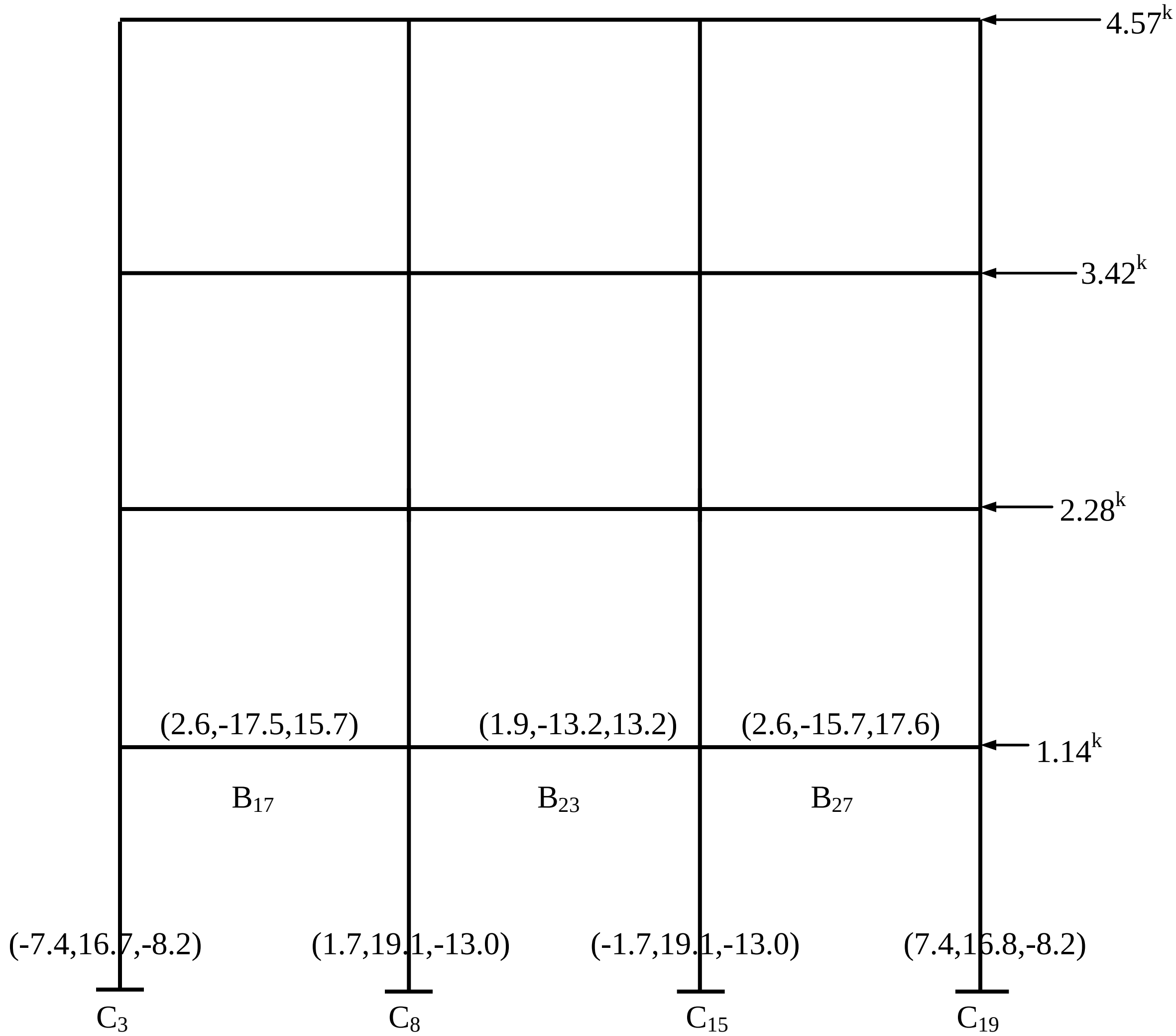
W = 4×(1.36×13+0.96×7+0.96×7+1.36×13) = 195.20^k ⇒ V = 0.0470W = 9.17^k



Beam (SF(k), BM_1, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame (3) from Lateral Load Analysis

Frame (4) [$B_{17\text{-}23\text{-}27}$]:

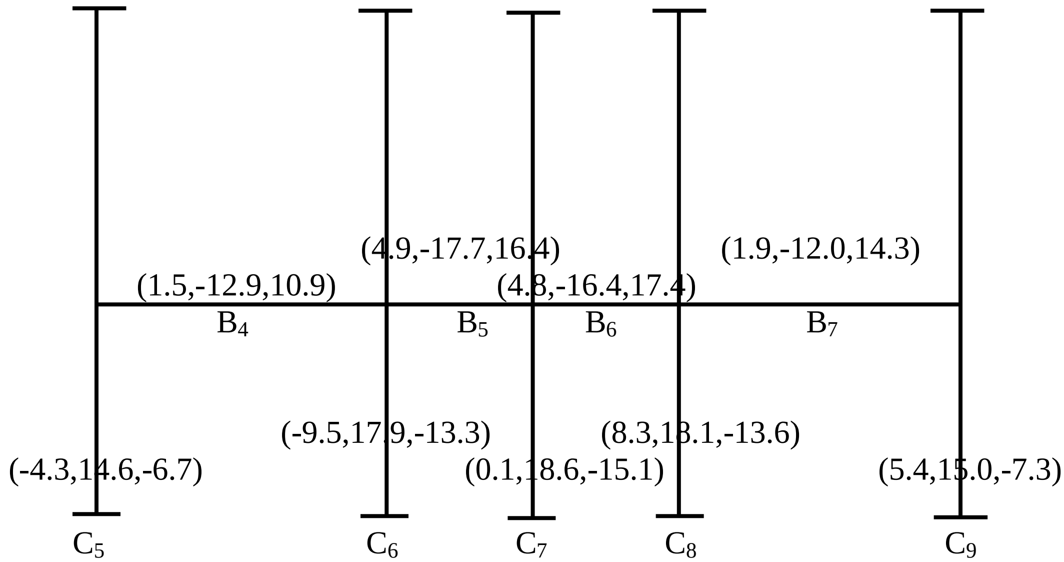
$W = 4\times(1.36\times13+1.81\times14+1.36\times13) = 242.80^k \Rightarrow V = 0.0470W = 11.41^k$



Beam (SF(k), BM_1, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame (4) from Lateral Load Analysis

Frame [$B_{4-5-6-7}$]:

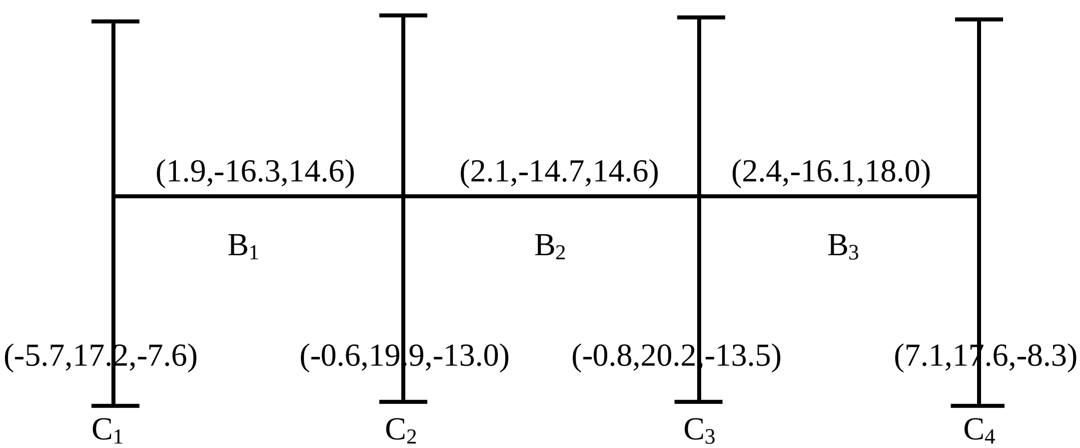
Similar to Frame (1) [$B_{8-9-10-11}$].



Beam (SF (k), BM_1, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame [$B_{4-5-6-7}$] from Lateral Load Analysis

Frame [B_{1-2-3}]:

Similar to Frame (2) [$B_{12-13-14}$].

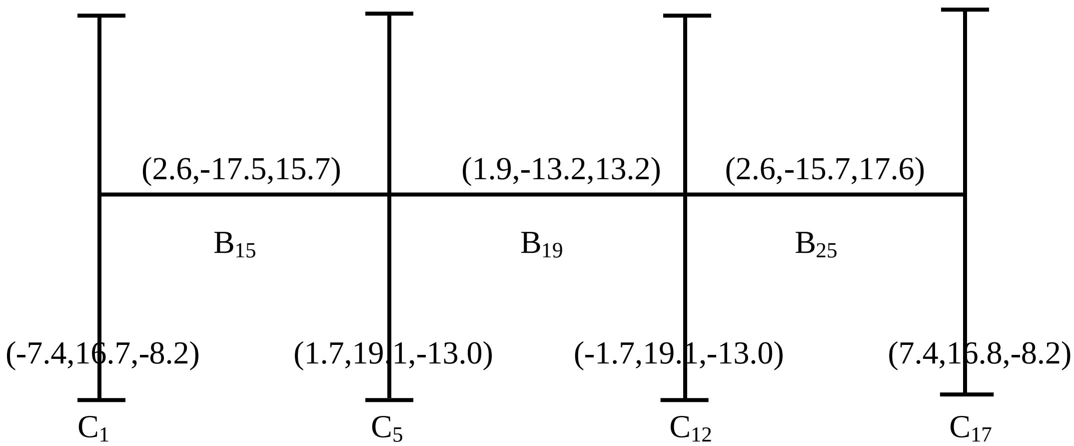


Beam (SF (k), BM_1, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in

Frame [B_{1-2-3}] from Lateral Load Analysis

Frame [$B_{15-19-25}$]:

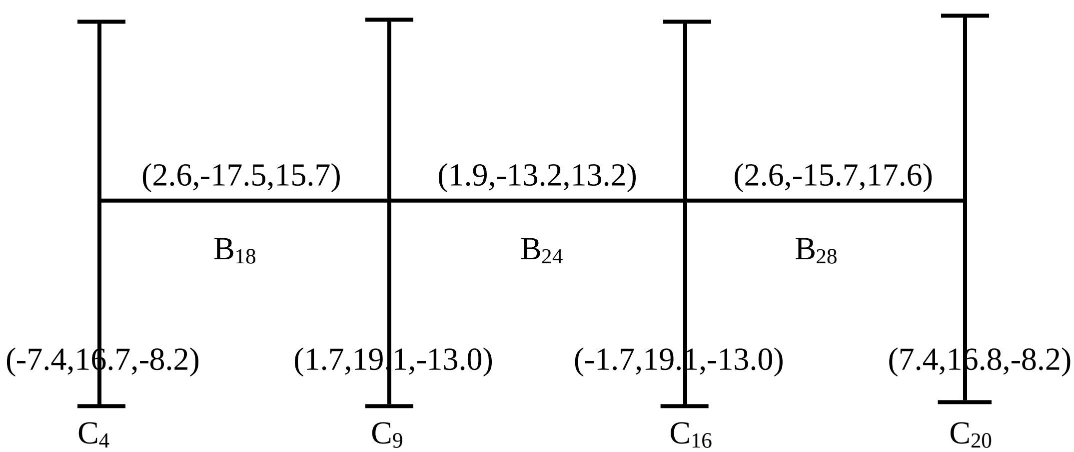
Similar to Frame (4) [$B_{17-23-27}$].



Beam (SF (k), BM_1, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame [$B_{15-19-25}$] from Lateral Load Analysis

Frame [$B_{18-24-28}$]:

Similar to Frame (4) [$B_{17-23-27}$].



Beam (SF (k), BM_1, BM_2 (k′)) and Column (AF (k), BM_1, BM_2 (k′)) in Frame [$B_{18-24-28}$] from Lateral Load Analysis

4. Combination of Vertical and Lateral Loads

The Design Force (i.e., AF, SF or BM) will be the maximum between the following two combinations

(i) Vertical Force = DL+LL

(ii) Combined Vertical and Lateral Force = 0.75 (DL+LL+EQ); i.e., 0.75 times the combined force from Vertical and Lateral Load Analysis.

The design Shear Forces and Bending Moments for various beams are calculated below using the two options mentioned above.

4.1 Load Combination for Beams

Frame (1) [$B_{4-5-6-7}$] and [$B_{8-9-10-11}$]:

Beams	$SF_1(V)$	$SF_1(L)$	$SF_1(D)$	$SF_2(V)$	$SF_2(L)$	$SF_2(D)$
B_4, B_8	12.1	±1.5	12.1	-12.6	±1.5	-12.6
B_5, B_9	8.6	±4.9	10.1	-6.4	±4.9	-8.5
B_6, B_{10}	6.7	±4.8	8.6	-8.3	±4.8	-9.8
B_7, B_{11}	10.2	±1.9	10.2	-9.8	±1.9	-9.8

Beams	$BM_1(V)$	$BM_1(L)$	$BM_1(D)$	$BM_0(V=D)$	$BM_2(V)$	$BM_2(L)$	$BM_2(D)$
B_4, B_8	-28.1	±12.9	-30.8	19.3	-31.8	±10.9	-32.0
B_5, B_9	-15.1	±17.7	-24.6	2.2	-7.3	±16.4	-17.8
B_6, B_{10}	-7.2	±16.4	-17.7	3.2	-13.0	±17.4	-22.8
B_7, B_{11}	-22.4	±12.0	-25.8	13.8	-20.0	±14.3	-25.7

Frame (2) [B_{1-2-3}] and [$B_{12-13-14}$]:

Beams	$SF_1(V)$	$SF_1(L)$	$SF_1(D)$	$SF_2(V)$	$SF_2(L)$	$SF_2(D)$
B_1, B_{12}	11.2	±1.9	11.2	-12.0	±1.9	-12.0
B_2, B_{13}	9.9	±2.1	9.9	-9.7	±2.1	-9.7
B_3, B_{14}	10.2	±2.4	10.2	-9.4	±2.4	-9.4

Beams	$BM_1(V)$	$BM_1(L)$	$BM_1(D)$	$BM_0(V=D)$	$BM_2(V)$	$BM_2(L)$	$BM_2(D)$
B_1, B_{12}	-25.8	±16.3	-31.6	17.6	-31.8	±14.6	-34.8
B_2, B_{13}	-24.3	±14.7	-29.3	10.8	-22.7	±14.6	-28.0
B_3, B_{14}	-23.8	±16.1	-29.9	12.9	-18.9	±18.0	-27.7

Frame (3) [$B_{16-20-21-26}$]:

Beams	$SF_1(V)$	$SF_1(L)$	$SF_1(D)$	$SF_2(V)$	$SF_2(L)$	$SF_2(D)$
B_{16}	8.7	±1.3	8.7	-9.0	±1.3	-9.0
B_{20}	4.0	±3.0	5.3	-2.8	±3.0	-4.4
B_{21}	2.8	±3.0	4.4	-4.0	±3.0	-5.3
B_{26}	9.0	±1.3	9.0	-8.7	±1.3	-8.7

Beams	$BM_1(V)$	$BM_1(L)$	$BM_1(D)$	$BM_0(V=D)$	$BM_2(V)$	$BM_2(L)$	$BM_2(D)$
B_{16}	-16.1	±9.5	-19.2	11.7	-18.0	±8.0	-19.5
B_{20}	-7.5	±10.9	-13.8	0.6	-3.4	±10.4	-10.4
B_{21}	-3.4	±10.4	-10.4	0.6	-7.5	±10.9	-13.8
B_{26}	-18.0	±8.0	-19.5	11.7	-16.1	±9.5	-19.2

Frame (4) [$B_{15-19-25}$], [$B_{17-23-27}$] and [$B_{18-24-28}$]:

Beams	$SF_1(V)$	$SF_1(L)$	$SF_1(D)$	$SF_2(V)$	$SF_2(L)$	$SF_2(D)$
B_{15}, B_{17}, B_{18}	8.3	±2.6	8.3	-9.4	±2.6	-9.4
B_{19}, B_{23}, B_{24}	12.7	±1.9	12.7	-12.7	±1.9	-12.7
B_{25}, B_{27}, B_{28}	9.4	±2.6	9.4	-8.3	±2.6	-8.3

Beams	$BM_1(V)$	$BM_1(L)$	$BM_1(D)$	$BM_0(V=D)$	$BM_2(V)$	$BM_2(L)$	$BM_2(D)$
B_{15}, B_{17}, B_{18}	-15.0	±17.5	-24.4	10.3	-22.0	±15.7	-28.3
B_{19}, B_{23}, B_{24}	-28.6	±13.2	-31.4	15.7	-28.6	±13.2	-31.4
B_{25}, B_{27}, B_{28}	-22.0	±15.7	-28.3	10.3	-15.0	±17.6	-24.5

Other Beams:

1. Beam B_{22} -

Approximately designed as a simply supported beam under similar load as B_{20}.

$\therefore$ Maximum SF $\cong 0.96\times7/2 = 3.36$ k

and Maximum positive BM $\cong 0.96\times7^2/8 = 5.88$ k′

2. Edge Beam for S_{10} -

Uniformly distributed load on S_{10} = 0.103 ksf

Uniformly distributed load on Edge Beam = $0.103\times5' = 0.51$ k/′

$\therefore$ Clear Span = 13′ $\Rightarrow V_{max} \cong 0.51\times(13)/2 = 3.3$ k; $M^{\pm} \cong 0.51\times(13)^2/10 = 8.6$ k′

4.2 Load Combination for Columns

The column forces are shown below as [AF (k), BM_{1y}, BM_{1x} (k′)]

Columns	Frame	(V)	(L_x)	0.75(V+L_x)	0.75(V-L_x)	(L_y)	0.75(V+L_y)	0.75(V-L_y)
C_1, C_{17}	2, 4	-78.7, 5.3, 3.2	-5.7, 17.2, 0	-63.3, 16.9, 2.4	-54.8, -8.9, 2.4	-7.4, 0, 16.7	-64.6, 4.0, 14.9	-53.5, 4.0, -10.1
C_2, C_{18}	2, 3	-122.4, -1.7, 3.4	-0.6, 19.9, 0	-92.3, 13.7, 2.6	-91.4, -16.2, 2.6	-3.9, 0, 9.6	-97.4, -1.3, 15.1	-86.3, -1.3, 10.0
C_3, C_{19}	2, 4	-112.8, 0.1, -3.2	-0.8, 20.2, 0	-85.2, 15.2, -2.4	-84.0, 15.1, -2.4	-7.4, 0, 16.8	-90.2, 0.1, 10.2	-79.1, 0.1, -15.0
C_4, C_{20}	2, 4	-71.7, -4.1, -3.2	7.1, 17.6, 0			-7.4, 0, 16.7		
C_5, C_{12}	1, 4	-126.3, 5.8, 1.4	-4.3, 14.6, 0			1.7, 0, 19.1		
C_6, C_{13}	1, 3	-132.5, -3.5, -2.1	-9.5, 17.9, 0			-4.7, 0, 11.6		
C_7, C_{14}	1	-55.2, -0.1, 0	0.1, 18.6, 0			0., 0, 0.		
C_8, C_{15}	1, 4	-160.1, 1.8, 1.4	8.3, 18.1, 0	-113.9, 14.9, 1.1	-126.3, -12.2, 1.1	1.7, 0, 19.1	-118.8, 1.4, 15.4	-121.4, 1.4, -13.3
C_9, C_{16}	1, 4	-127.3, -4.3, 1.4	5.4, 15.0, 0			1.7, 0, 19.1		
C_{10}	3	-25.5, 0, 0	0, 0, 0			0., 0, 12.0		
C_{11}	-	-	-	-	-	-	-	-

In this work, only one size will be chosen for all the columns. For this purpose, the columns (C_8, C_{15}) are chosen as the model because they provide the most critical design conditions.

The designed column should therefore satisfy the following design conditions,

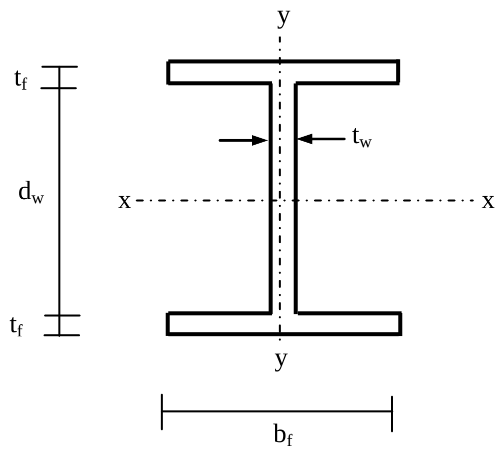
(1) Compressive Force = 160.1^k, Bending Moments BM_{1x} = 1.4 k′, BM_{1y} = 1.8 k′.

(2) Compressive Force = 113.9^k, Bending Moments BM_{1x} = 1.1 k′, BM_{1y} = 14.9 k′.

(3) Compressive Force = 118.8^k, Bending Moments BM_{1x} = 15.4 k′, BM_{1y} = 1.4 k′.

5. Design of Beams

The figure below shows an I-section, which has been chosen here for all the beams.



b_f = Width of flange
t_f = Thickness of flange
d_w = Depth of web
t_f = Thickness of web

The design of beams follows the following steps

1. Calculate the design moment M_d from vertical and lateral load analyses
2. Assume the allowable bending stress f_b and calculate $Z_{req} = M_d/f_b$

   If $f_b = 0.6f_y = 24$ ksi and M_d is in k′, then Z_{req} (in in^3) is $= M_d \times 12/24 = M_d/2$
3. Choose a beam section with $Z_{xx} \geq Z_{req}$
4. Check the chosen beam section against

   (i) local buckling of flange; i.e., $b_f/t_f \leq 190/\sqrt{f_y} \cong 30$ (for $f_y = 40$ ksi)

   (ii) local buckling of web; i.e., $d_w/t_w \leq 760/\sqrt{f_b} = 155$ (for $f_b = 24$ ksi)

   and vertical buckling of flange; i.e., $d_w/t_w \leq 2000/\sqrt{f_y} = 316$ (for $f_y = 40$ ksi)

   (iii) lateral torsional buckling; i.e., $kL/b_f \leq 76\sqrt{(C_b/f_y)}$

   $\Rightarrow L/b_f \leq 17$ (for $f_y = 40$ ksi, $C_b = 1$, $k = 0.70$ assuming hinged-fixed conditions)

   or take measures to prevent LTB; i.e., (a) embed the flange into slab, (b) provide stubs, (c) reduce allowable bending stress; i.e., if $kLd_w/A_f \geq 20000\ C_b/f_y$

   $\Rightarrow Ld_w/A_f \geq 28000/f_y$ (= 700) then $f_b = 17000/(Ld_w/A_f)$
5. Calculate the maximum shear stress $f_s = VQ/I_{xx}t_w$ [where $Q = t_w d_w^2/8 + A_f (d_w + t_f)/2$]
6. Check against shear buckling of web; i.e.,

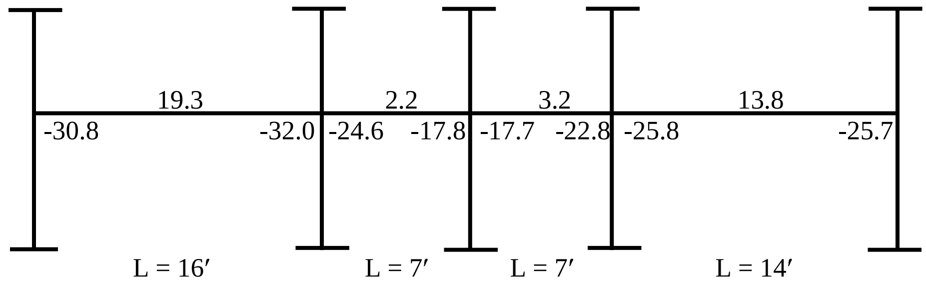
   $f_{s(all)} = 0.4f_y$ (= 16 ksi) if $d_w/t_w \leq 369/\sqrt{f_y}$ (= 58);

   $= 148\sqrt{f_y}/(d_w/t_w)$ if $369/\sqrt{f_y} \leq d_w/t_w \leq 532/\sqrt{f_y}$;

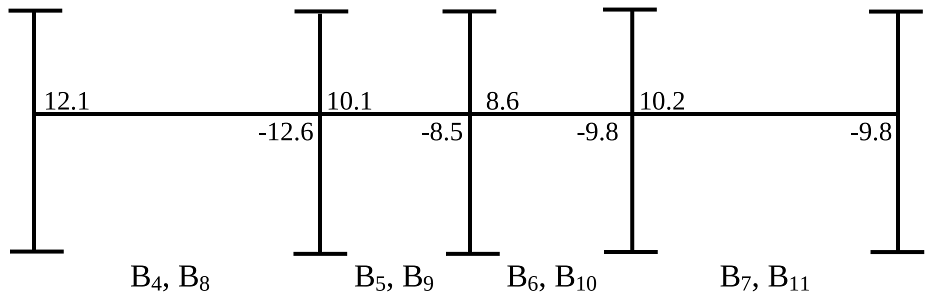
   $= 78000/(d_w/t_w)^2$ if $d_w/t_w \geq 532/\sqrt{f_y}$

Frame (1) [$B_{4\text{-}5\text{-}6\text{-}7}$] and [$B_{8\text{-}9\text{-}10\text{-}11}$]:

The design moments (k′) are



The design shear forces (k) are



Design Table for Beams (Moment)

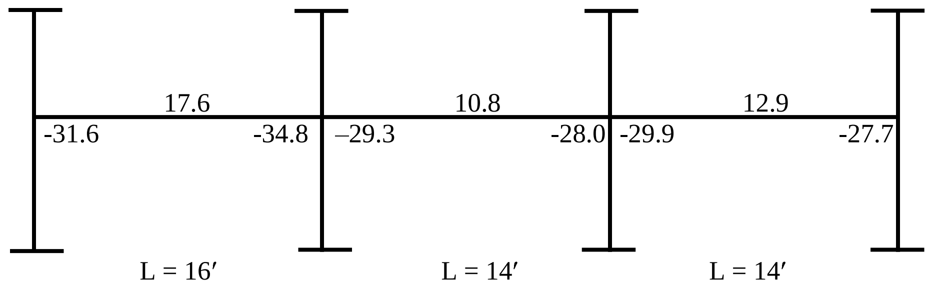
Beam	M_d (k′)	Z_{req} (in^3)	Z_{xx} (in^3)	b_f (in)	t_f (in)	d_w (in)	t_w (in)	A_f (in^2)	b_f/t_f	d_w/t_w	Ld_w/A_f	f_b (ksi)	Z_{req} (in^3)	OK
B_4, B_8	32.0	16.0	16.2	4.17	0.426	7.15	0.44	1.78	9.79	16.21	772	22.0	17.4	No
		17.4	24.7	4.66	0.491	9.02	0.31	2.29	9.49	29.00	757	22.5	17.1	Yes
B_5, B_9	24.6	12.3	14.4	4.00	0.426	7.15	0.27	1.70	9.39	26.38	352	24.0	12.3	Yes
B_6, B_{10}	22.8	11.4	14.4	4.00	0.426	7.15	0.27	1.70	9.39	26.38	352	24.0	11.4	Yes
B_7, B_{11}	25.8	12.9	14.4	4.00	0.426	7.15	0.27	1.70	9.39	26.38	705	24.0	12.9	Yes

Design Table for Beams (Shear)

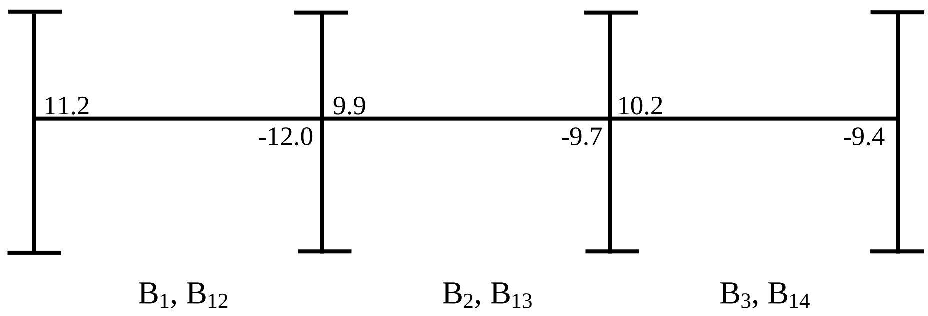
Beam	V (k)	I_{xx} (in^4)	Q (in^3)	f_s (ksi)	$f_{s(all)}$ (ksi)	OK	Section
B_4, B_8	12.6	124	14.05	4.59	16	Yes	S 10 × 25.4
B_5, B_9	10.1	57.6	8.17	5.29	16	Yes	S 8 × 18.4
B_6, B_{10}	9.8	57.6	8.17	5.13	16	Yes	S 8 × 18.4
B_7, B_{11}	10.2	57.6	8.17	5.34	16	Yes	S 8 × 18.4

Frame (2) [B_{1-2-3}] and [$B_{12-13-14}$]:

The design moments (k′) are



The design shear forces (k) are



Design Table for Beams (Moment)

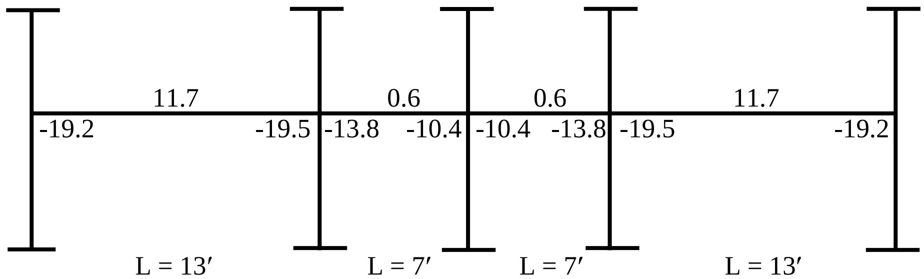
Beam	M_d (k′)	Z_{req} (in^3)	Z_{xx} (in^3)	b_f (in)	t_f (in)	d_w (in)	t_w (in)	A_f (in^2)	b_f/t_f	d_w/t_w	Ld_w/A_f	f_b (ksi)	Z_{req} (in^3)	OK
B_1, B_{12}	34.8	17.4												
B_2, B_{13}	29.3	14.7												
B_3, B_{14}	29.9	15.0												

Design Table for Beams (Shear)

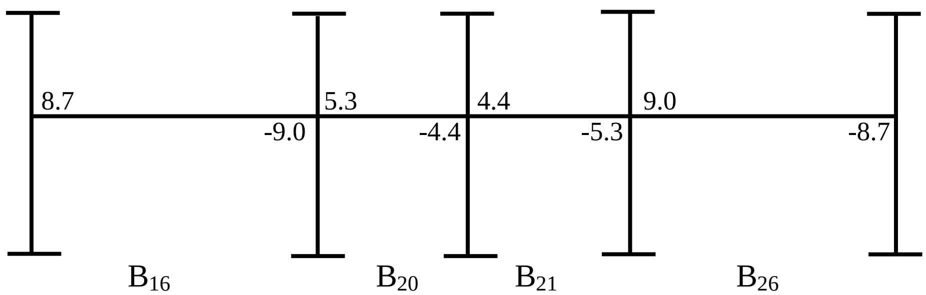
Beam	V (k)	I_{xx} (in^4)	Q (in^3)	f_s (ksi)	$f_{s(all)}$ (ksi)	OK	Section
B_1, B_{12}	12.0						
B_2, B_{13}	9.9						
B_3, B_{14}	10.2						

Frame (3) [$B_{16-20-21-26}$]:

The design moments (k′) are



The design shear forces (k) are



Design Table for Beams (Moment)

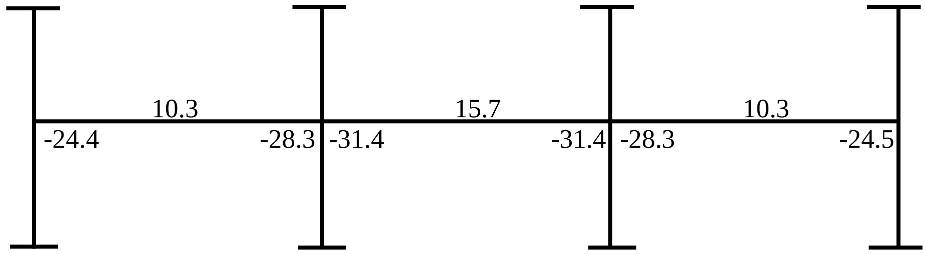
Beam	M_d (k′)	Z_{req} (in^3)	Z_{xx} (in^3)	b_f (in)	t_f (in)	d_w (in)	t_w (in)	A_f (in^2)	b_f/t_f	d_w/t_w	Ld_w/A_f	f_b (ksi)	Z_{req} (in^3)	OK
B_{16}	19.5	9.8												
B_{20}	13.8	6.9												
B_{21}	13.8	6.9												
B_{26}	19.5	9.8												

Design Table for Beams (Shear)

Beam	V (k)	I_{xx} (in^4)	Q (in^3)	f_s (ksi)	$f_{s(all)}$ (ksi)	OK	Section
B_{16}	9.0						
B_{20}	5.3						
B_{21}	5.3						
B_{26}	9.0						

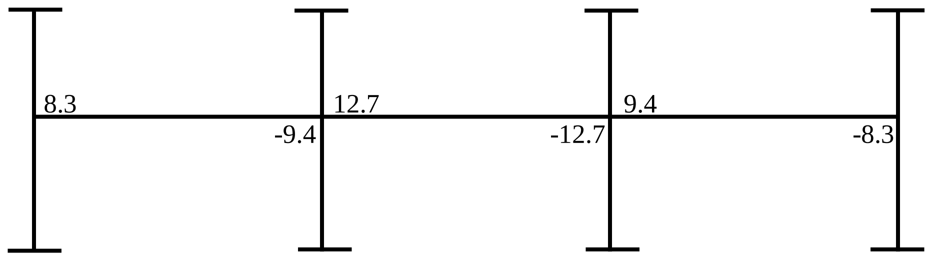
Frame (4) [$B_{15\text{-}17\text{-}18}$], [$B_{17\text{-}23\text{-}27}$] and [$B_{25\text{-}27\text{-}28}$]:

The design moments (k′) are



L = 13′ L = 14′ L = 13′

The design shear forces (k) are



B_{15}, B_{17}, B_{18} B_{19}, B_{23}, B_{24} B_{25}, B_{27}, B_{28}

Design Table for Beams (Moment)

Beam	M_d (k′)	Z_{req} (in^3)	Z_{xx} (in^3)	b_f (in)	t_f (in)	d_w (in)	t_w (in)	A_f (in^2)	b_f/t_f	d_w/t_w	Ld_w/A_f	f_b (ksi)	Z_{req} (in^3)	OK
B_{15}, B_{17}, B_{18}	28.3	14.2												
B_{19}, B_{23}, B_{24}	31.4	15.7												
B_{25}, B_{27}, B_{28}	28.3	14.2												

Design Table for Beams (Shear)

Beam	V (k)	I_{xx} (in^4)	Q (in^3)	f_s (ksi)	$f_{s(all)}$ (ksi)	OK	Section
B_{15}, B_{17}, B_{18}	9.4						
B_{19}, B_{23}, B_{24}	12.7						
B_{25}, B_{27}, B_{28}	9.4						

6. Design of Columns

Design Concept of Compression Members:

The design of steel compression members is carried out using the following equations.

(1) Members under pure compression –

If F_c = Compressive force, E = Modulus of elasticity

f_c = Compressive stress, $f_{all(c)}$ = Allowable compressive stress

A = Cross-sectional area, L_e = Effective length of member = kL (k is determined from alignment chart based on column end conditions), r_{min} = Minimum radius of gyration

$f_c = F_c/A$

Slenderness Ratio, $\lambda = L_e/r_{min}$, and $\lambda_c = \pi\sqrt{(2E/f_y)}$

If $\lambda \leq \lambda_c$, $f_{all(c)} = f_y\,[1-0.5\,(\lambda/\lambda_c)^2]/[5/3 + 3/8\,(\lambda/\lambda_c) - 1/8(\lambda/\lambda_c)^3]$

If $\lambda > \lambda_c$, $f_{all(c)} = 0.52\,(\pi^2 E/\lambda^2)$ ………………………(6.1(a)~6.1(d))

The acceptable design condition is $f_c \leq f_{all(c)}$; i.e., $f_c/f_{all(c)} \leq 1$

For the material properties used for design; i.e., E = 29000 ksi, f_y = 40 ksi

$\lambda_c = \pi\sqrt{(2E/f_y)} = 119.63$, $\rho = \lambda/\lambda_c = \lambda/119.63$

If $\rho \leq 1$, $f_{all(c)} = 40\,(1-0.5\rho^2)/(5/3 + 3\rho/8 - \rho^3/8)$

If $\rho > 1$, $f_{all(c)} = 149000/\lambda^2 = 10.4/\rho^2$ ………………………(6.2(a)~6.2(d))

(2) Members under combined compression and biaxial bending –

$f_c/f_{all(c)} + f_{bx}/f_{all(bx)} + f_{by}/f_{all(by)} \leq 1$ ……………………(6.3)

The stresses f_c and $f_{all(c)}$ are obtained from Eqs. (6.1) and (6.2)

f_{bx} = Bending stress about x-axis = $M_{x(mag)}/Z_{xx}$

Here $M_{x(mag)}$ is the bending moment about x-axis magnified by the axial force, and it is given by $M_{x(mag)} = M_x\,[C_{mx}/(1-f_c/f_{all(ex)})]$

where C_{mx} = 1.0 for unrestrained members, and

$f_{all(ex)}$ is the allowable Euler stress about x-axis; i.e., $f_{all(ex)} = 0.52\,(\pi^2 E/\lambda_x^2)$

$f_{all(bx)}$ = Allowable bending stress about x-axis = $0.6\,f_y$ …………...(6.4(a)~6.4(d))

$\therefore f_{all(bx)}$ = 24 ksi, in this case

Similar notations are used for f_{by} and $f_{all(by)}$; i.e.,

$f_{by} = M_y\,[C_{my}/(1-f_c/f_{all(ey)})]/Z_{yy}$, $f_{all(by)} = 0.6\,f_y$ ……………(6.5(a)~6.5(b))

Design Forces and Assumed Section:

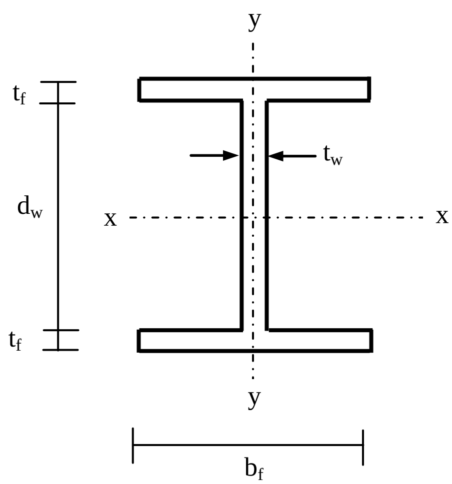
The designed column (C_8, C_{15}) should satisfy the three design conditions mentioned before (in the load combination for columns); i.e.,

(i) $F_c = 160.1^k$, $M_x = 1.4$ k′, $M_y = 1.8$ k′

(ii) $F_c = 113.9^k$, $M_x = 1.1$ k′, $M_y = 14.9$ k′

(iii) $F_c = 118.8^k$, $M_x = 15.4$ k′, $M_y = 1.4$ k′

To ensure larger moments of inertia and better flexural/buckling behavior, several built-up and closed sections can be used as steel columns. But as was done for all the beams, the I-section is chosen here for all the columns. Moreover the wide-flanged W-sections are chosen because they are more compact and therefore suitable as buckling members.



b_f = Width of flange
t_f = Thickness of flange
d_w = Depth of web
t_f = Thickness of web

Preliminary choice of column section –

The bending moments are very small for condition (i), so that the section can be chosen based on axial force only. Since $F_{all(c)} = 0.52(\pi^2 EI_{min}/L_e^2)$, one needs to know L_e in order to calculate I_{min} for a given load.

Assuming (a) unrestrained column, (b) critical axis is y-axis ($\therefore$ consider Frame1), (c) column base is fixed, (d) two similar beam and column sections at the other joint; i.e.,

$G_A = 0$, $G_B = (1/10 + 1/10)/(1/7 + 1/14) = 0.93 \Rightarrow k = 1.12$

$\therefore 160.1 = 0.52 \times \pi^2 \times 29000 \times I_{min}/(1.12\times10\times12)^2 \Rightarrow I_{min} = 19.43 \text{ in}^4$

$\therefore$ Choose the smallest section with the required I_{min}, i.e., W 8 × 28

Design Table for Columns

Design condition (i); i.e., $F_c = 160.1^k$, $M_x = 1.4$ k′ = 16.8 k″, $M_y = 1.8$ k′ = 21.6 k″

Assumed section W 8 × 28 ⇒ $A = 8.25$ in^2, $I_{xx} = 98.0$ in^4, $Z_{xx} = 24.3$ in^3, $r_{xx} = 3.45''$

$I_{min} = I_{yy} = 21.7$ in^4, $Z_{yy} = 6.63$ in^3, $r_{yy} = 1.62''$

$f_c = F_c/A = 160.1/8.25 = 19.41$ ksi, $\rho = (10 \times 12/1.62)/119.63 = 77.78/119.63 = 0.65$

$\Rightarrow f_{all(c)} = 40\,(1-0.5\rho^2)/(5/3 + 3\rho/8 - \rho^3/8) = 16.81$ ksi

About x-axis (axis of I_{max}); i.e., in the y-direction,

$G_{Ax} = 0$, $G_{Bx} = (98.0/10 + 98.0/10)/(57.6/13 + 57.6/14) = 2.29 \Rightarrow k_x = 1.30$

$L_{ey} = 1.30 \times 120 = 156''$, $\lambda_x = L_{ex}/r_{xx} = 156/3.45 = 45.22$

$f_{all(ex)} = 0.52(\pi^2 E/\lambda_x^2) = 0.52\,(\pi^2 \times 29000/45.22^2) = 72.79$ ksi

$f_{bx} = M_x\,[C_{mx}/(1-f_c/f_{all(ex)})]/Z_{xx} = 16.8\,[1.0/(1-19.41/72.79)]/24.3 = 0.94$ ksi

About y-axis (axis of I_{min}); i.e., in the x-direction,

$G_{Ay} = 0$, $G_{By} = (21.7/10 + 21.7/10)/(57.6/7 + 57.6/14) = 0.35 \Rightarrow k_y = 1.05$

$L_{ey} = 1.05 \times 120 = 126''$, $\lambda_y = L_{ey}/r_{yy} = 126/1.62 = 77.78$

$f_{all(ey)} = 0.52(\pi^2 E/\lambda_y^2) = 0.52\,(\pi^2 \times 29000/77.78^2) = 24.60$ ksi

$f_{by} = M_y\,[C_{my}/(1-f_c/f_{all(ey)})]/Z_{yy} = 21.6\,[1.0/(1-19.41/24.60)]/6.63 = 15.42$ ksi

$\therefore f_c/f_{all(c)} + f_{bx}/f_{all(bx)} + f_{by}/f_{all(by)} = 19.41/16.81 + 0.94/24 + 15.42/24 = 1.16 + 0.04 + 0.64$

$= 1.84 > 1$, i.e., not OK

Section	k_x	k_y	L_{ex} (in)	L_{ey} (in)	λ_x	λ_y	f_c (ksi)	$f_{all(c)}$ (ksi)	$f_{all(ex)}$ (ksi)	f_{bx} (ksi)	$f_{all(ey)}$ (ksi)	f_{by} (ksi)	LS
W 8 × 28	1.30	1.05	156.0	126.0	45.22	77.78	19.41	16.81	72.79	0.94	24.60	15.42	1.84
W 8 × 35	1.38	1.10	165.6	132.0	47.18	65.02	15.54	18.42	66.86	0.70	35.20	3.65	1.02
W 8 × 40	1.41	1.13	169.2	135.6	47.93	66.47	13.68	18.25	64.78	0.60	33.69	2.98	0.90

∴ The section W 8 × 40 is OK for design condition (i).

Design condition (ii); i.e., $F_c = 113.9^k$, $M_x = 1.1\ k' = 13.2\ k''$, $M_y = 14.9\ k' = 178.8\ k''$

Section	k_x	k_y	L_{ex} (in)	L_{ey} (in)	λ_x	λ_y	f_c (ksi)	$f_{all(c)}$ (ksi)	$f_{all(ex)}$ (ksi)	f_{bx} (ksi)	$f_{all(ey)}$ (ksi)	f_{by} (ksi)	LS
W 8×40	1.41	1.13	169.2	135.6	47.93	66.47	9.74	18.25	64.78	0.44	33.69	20.61	1.41
W 8×58	1.52	1.18	182.4	141.6	49.97	67.43	6.66	18.13	59.60	0.29	32.74	12.27	0.89

$\therefore$ The section W 8×58 is OK for design condition (ii)

Design condition (iii); i.e., $F_c = 118.8^k$, $M_x = 15.4\ k' = 184.8\ k''$, $M_y = 1.4\ k' = 16.8\ k''$

Section	k_x	k_y	L_{ex} (in)	L_{ey} (in)	λ_x	λ_y	f_c (ksi)	$f_{all(c)}$ (ksi)	$f_{all(ex)}$ (ksi)	f_{bx} (ksi)	$f_{all(ey)}$ (ksi)	f_{by} (ksi)	LS
W 8×58	1.52	1.18	182.4	141.6	49.97	67.43	6.95	18.13	59.60	4.02	32.74	1.17	0.60

$\therefore$ The section W 8×58 is also OK for design condition (iii)

The section is the strongest about x-axis and weakest about y-axis; therefore case (ii) provides the most critical design condition here [in this case, it is even more critical than case (i)].

This needs to be kept in consideration while choosing the orientation of the column section in the building plan, so that bending moment about the weakest axis is kept as small as possible. In case of the building designed, with an almost square-shaped plan, this does not affect the design significantly. But for highly elongated plans (high L/B ratio), weakest axis of the section should not coincide with the weakest axis of the plan.

7. Design of Connections

The following sample connections will be designed for illustration.

1. A moment-resisting connection between column C_8 and beam B_7 will be designed for shear force 10.2^k and negative bending moment of 25.8 k′.
2. A base slab for column C_8 and the footing underneath will be designed for column axial force of 160.1^k

<u>Moment-resisting connection between column C_8 and beam B_7</u>:

Column C_8 is a W 8 × 58 section and beam B_7 is an S 8 × 18.4 section

(i) Tension Connection

Tensile Force (T) in cover plate due to moment = 25.8 × 12 k″/8″ = 38.7^k

Allowable bending stress = 24 ksi

∴ Area of cover plate required = 38.7^k/24 ksi = 1.61 in^2

For beam B_7, width of the flange b_f = 4.00″

∴ Width of cover plate = 3″, in order to allow for welding and clearance on both sides

∴ Thickness of cover plate = 1.61 in^2/3″ = 0.54″

Use cover plate of size 3″ × 5/8″, to be butt-welded to column flange and fillet-welded to tension flange of the beam.

Also, 5/8″ thick stiffeners may be used between column flanges on both sides of the web

Connect cover plate to tension flange of beam by 0.25″ fillet welds

Strength of weld = f_v × 0.707t = 12 × 0.707× 0.25 = 2.12 k/″

∴ Total length of weld required, L_w = T/2.12 = 38.7/2.12 = 18.25″

∴ Provide (8″ + 3″+ 8″ =) 19″ long 0.25 ″ thick weld around the cover plate.

Also keep an additional 4″ unwelded on both sides of the cover plate

∴ Total length of cover plate = 8″ + 4″ = 12″

(ii) Compression Connection

Compressive Force (C) in cover plate due to moment is also = 38.7^k

To transfer this compressive force to the column, the compressive flange of the beam is fillet-welded to a horizontal seat plate, dividing the weld length (thickness = 0.25″) into two 9.5″ parts on both sides of the beam. Providing an additional 0.5″ as clearance between beam and column, the length of the plate = 9.5″ + 0.5″ = 10″.

If the width of the plate is chosen to be = 5″ (considering beam flange width of 4″), its thickness = 1.61/5 = 0.32″; i.e., provide 3/8″ thickness.

Provide 10″ × 5″ × 3/8″ horizontal seat plate, which is butt-welded to the column flange.

For the shear force of 10.2^k, which is not considered too high, an unstiffened seat connection is designed at the bottom of the beam.

The length of web (B) required to transfer the force without web crippling of beam is

$B = R/\sigma_p\, t_w = 10.2/(0.75 \times 40 \times 0.27) = 1.26''$

Thickness of beam flange, $t_f = 0.426''$.

Effective bearing width is the greater of ($B - h_2 \cot 30°$) and $B/2$

Assuming a fillet of similar diameter, depth below root of fillet $h_2 = 0.852''$

$\Rightarrow b = B - h_2 \cot 30°$ is negative

However $b \geq B/2 \Rightarrow b = 0.63''$

$\therefore$ Assuming 0.5″ clearance, the minimum width of the horizontal leg of angle

= 0.63 + 0.5 = 1.13″

Selecting equal angle section of L 4 × 4 × 1/4, $t_a = 0.25''$, assumed $r_1 = 0.25''$

$\Rightarrow$ Overturning moment $M = R\,(b/2 + 0.5 - t_a - r_1)$

$= 10.2 \times (0.31 + 0.5 - 0.25 - 0.25) = 3.16$ k″

$\therefore$ Length of seating angle across the flange-width, $L_a = 5''$ (in order to accommodate beam-flange width and welding).

$\therefore$ Thickness t_a (required) $= \sqrt{(6M/\sigma_b L_a)} = \sqrt{(6 \times 3.16/(24 \times 5))} = 0.40''$

Selecting equal angle section of L 4 × 4 × 5/16, $t_a = 0.31''$, assumed $r_1 = 0.31''$

$\Rightarrow$ Overturning moment $M = R\,(b/2 + 0.5 - t_a - r_1) = 1.94$ k″

$\therefore$ Thickness t_a (required) $= \sqrt{(6M/\sigma_b L_a)} = \sqrt{(6 \times 1.94/(24 \times 5))} = 0.31$″

Select equal angle section of L $4 \times 4 \times 5/16$

$\therefore$ Moment at welds $M_w = R\,(b/2 + 0.5) = 10.2 \times (0.31 + 0.5) = 8.26$ k″

$\therefore$ Horizontal shear force per length $V_h = 3M/d^2 = 3 \times 8.26/4^2 = 1.56$ k/″

$\therefore$ Vertical shear force per length $V_v = V/2d = 10.2/8 = 1.28$ k/″

$\therefore$ Resultant shear force per unit length, $V_r = \sqrt{(V_h^2 + V_v^2)} = 2.02$ k/″

Strength of weld $= f_v \times 0.707t = 12 \times 0.707t = 8.48\,t$

$\therefore$ Required thickness of weld, $t = 2.02/8.48 = 0.24$″ $\Rightarrow$ Provide $t = 1/4$″

<u>A base slab for column C_8 and the footing underneath</u>:

Axial load on column = 160.1^k and the additional moments are small enough to be neglected for the design of the base plate.

Assuming the base plate area = A_p and allowable bearing pressure = $0.35\,f_c' = 1.05$ ksi

$$1.05A_p = 160.1 \Rightarrow A_p = 160.1/1.05 = 152.5 \text{ in}^2$$

$\therefore$ Provide 12″ × 13″ base plate

$\therefore$ Bearing pressure $q = 160.1/(12 \times 13) = 1.03$ ksi

Thickness of the plate is given by the

$t = \sqrt{\{3q(a^2 - \nu b^2)/f_b\}}$

$\;= \sqrt{\{3 \times 1.03\,(2.39^2 - 0.25 \times 2^2)/24\}} = 0.78$″

$\therefore$ Provide a 7/8″ thick base plate

Although fastenings are not required to transmit any load, connect each flange of the column to the base plate by 0.5″ welds.

Also connect the base plate to the footing below using 4 #6 bolts and anchor them their development lengths.