Appendix A: Wind Loads

Basic Wind Information				(ASCE Ref)	
Basic Wind Speed	V =	90	mph	ASCE 7-05 Figure 6-1	
Directionality Factor	k_d =	0.85		ASCE 7-05 Table 6-4	
Importance Factor	I =	1.0		ASCE 7-05 Table 6-1	
Exposure Category		B		ASCE 7-05 6.5.6	
Topgraphic Factor	k_{zt} =	1.0		ASCE 7-05 6.5.7	
	z_g =	1200	ft		
	α =	7			
Velocity Presure Expusure Coefficient evaluated at Height z	K_z =	Varies			
Velocity Presure Expusure Coefficient evaluated at Mean Roof Height	K_h =	0.8930			
Velocity Pressure at Height z	q_z =	Varies			
Velocity Pressure at Mean Roof Height	q_h =	15.7			
Equivalent height of Structure	h =	70.1			
Intensity of turbulance	I_z =	0.3			
Integral Length Scale of Turbulence	L_z =	347.0			
Background Response Factor (N/S)	Q =	0.778			
Background Reponse Factor (E/W)	Q =	0.829			
Gust Effect Factor (N/S)	G =	0.850			
Gust Effect Factor (E/W)	G =	0.850			
Internal Pressure Coefficients	G_{cpi} =	± 0.18			
External Pressure Coefficient (Windward)	C_p =	0.8			
External Pressure Coefficient (N/S Leeward)	C_p =	-0.3			
External Pressure Coefficient (E/W Leeward)	C_p =	-0.5			
External Pressure Coefficient (Sidewall)	C_p =	-0.7			
External Pressure Coefficient (Roof Section 1)	C_p =	-0.9	(From Windward Edget to 70.14 ft.)		
External Pressure Coefficient (Roof Section 2)	C_p =	-0.5	(From 70.14 to 140.28 ft.)		
External Pressure Coefficient (Roof Section 3)	C_p =	-0.3	(From 140.28 to 297.53 ft.)		
Basic Building Information					
Mean Building Height	h =	21379	mm		
		70.14	ft		
N-S	L =	137.44	ft		
	B =	297.55	ft		
E-W	L =	297.55	ft		
	B =	137.44	ft		

Design Wind Pressures p in N-S Direction							
Location	Story Height		Level Height		K_z	q_z (psf)	External Pressure qGCp (psf)
	(mm)	(ft)	(mm)	(ft)			
Windward	0	0	0	0			
	4700	15.420	4700	15.4199	0.5793	10.210	6.943
	3930	12.894	8630	28.3136	0.6891	12.146	8.259
	3930	12.894	12560	41.2073	0.7671	13.521	9.194
	3930	12.894	16490	54.1010	0.8291	14.614	9.938
	3930	12.894	20420	66.9948	0.8814	15.535	10.564
	959	3.146	21379	70.1411	0.8930	15.740	10.703
Leeward				All	0.8930	15.740	-4.014
Side				All	0.8930	15.740	-9.365
Roof	(From Windward Edget to 70.14 ft.)			70.1411	0.8930	15.740	-12.041
	(From 70.14 to 140.28 ft.)			70.1411	0.8930	15.740	-6.689

Design Wind Pressures p in E-W Direction							
Location	Story Height		Level Height		K_z	q_z (psf)	External Pressure qGCp (psf)
	(mm)	(ft)	(mm)	(ft)			
Windward	0	0	0	0			
	4700	15.420	4700	15.4199	0.5793	10.210	6.943
	3930	12.894	8630	28.3136	0.6891	12.146	8.259
	3930	12.894	12560	41.2073	0.7671	13.521	9.194
	3930	12.894	16490	54.1010	0.8291	14.614	9.938
	3930	12.894	20420	66.9948	0.8814	15.535	10.564
	959	3.146	21379	70.1411	0.8930	15.740	10.703
Leeward				All	0.8930	15.740	-4.014
Side				All	0.8930	15.740	-9.365
Roof	(From Windward Edget to 70.14 ft.)			70.1411	0.8930	15.740	-12.041
	(From 70.14 to 140.28 ft.)			70.1411	0.8930	15.740	-6.689
	(From 140.28 to 297.53 ft.)			70.1411	0.8930	15.740	-4.014

Design Wind Loads in N-S Direction				
	External Windward Load (kips)	External Leeward Loads (kips)	Base Shear (kips)	
			1.0W	1.6W
Level 1	0	0		
Level 2	31.771	16.907	**48.678**	**77.884**
Level 3	33.480	15.398	**48.878**	**78.205**
Level 4	36.700	15.398	**52.098**	**83.356**
level 5	39.327	15.398	**54.725**	**87.560**
Roof	25.274	9.578	**34.851**	**55.762**
Parapet	5.010	1.879	**6.889**	**11.022**
Base Shear			**246.119**	**393.790**

Design Wind Loads in E-W Direction				
	External Windward Load (kips)	External Leeward Loads (kips)	Base Shear (kips)	
			1.0W	1.6W
Level 1	0	0		
Level 2	14.675	7.809	**22.484**	**35.974**
Level 3	15.464	7.112	**22.576**	**36.122**
Level 4	16.951	7.112	**24.064**	**38.502**
level 5	18.165	7.112	**25.277**	**40.443**
Roof	11.674	4.424	**16.098**	**25.756**
Parapet	2.314	0.868	**3.182**	**5.091**
Base Shear			**113.680**	**181.888**

Wind Loads

1. Determine basic wind speed V from Fig 6-1 ASCE 7-05

   From Fig 6-1 ⇒ V = 40 m/s (90 mph)

2. Determine wind directionality factor K_d from Table 6-4

   By Table 6-4
   Buildings
   MWFRS K_d = .85
   C-C K_d = .85

3. Determine importance factor I from Table 6-1

   By Table 6-I, Occupancy II : I = 1.00

4. Determine exposure category (6.5.6)

   By ASCE 7-5 5.5.6.2 Surface Roughness Categories
   Surface Roughness B

   By ASCE 7-5 6.5.6.3 Exposure Categories
   Exposure B applies

5. Topographic Factor: ASCE 7-5 6.5.7

   Not all 5 conditions are met K_{zt} = 1.0

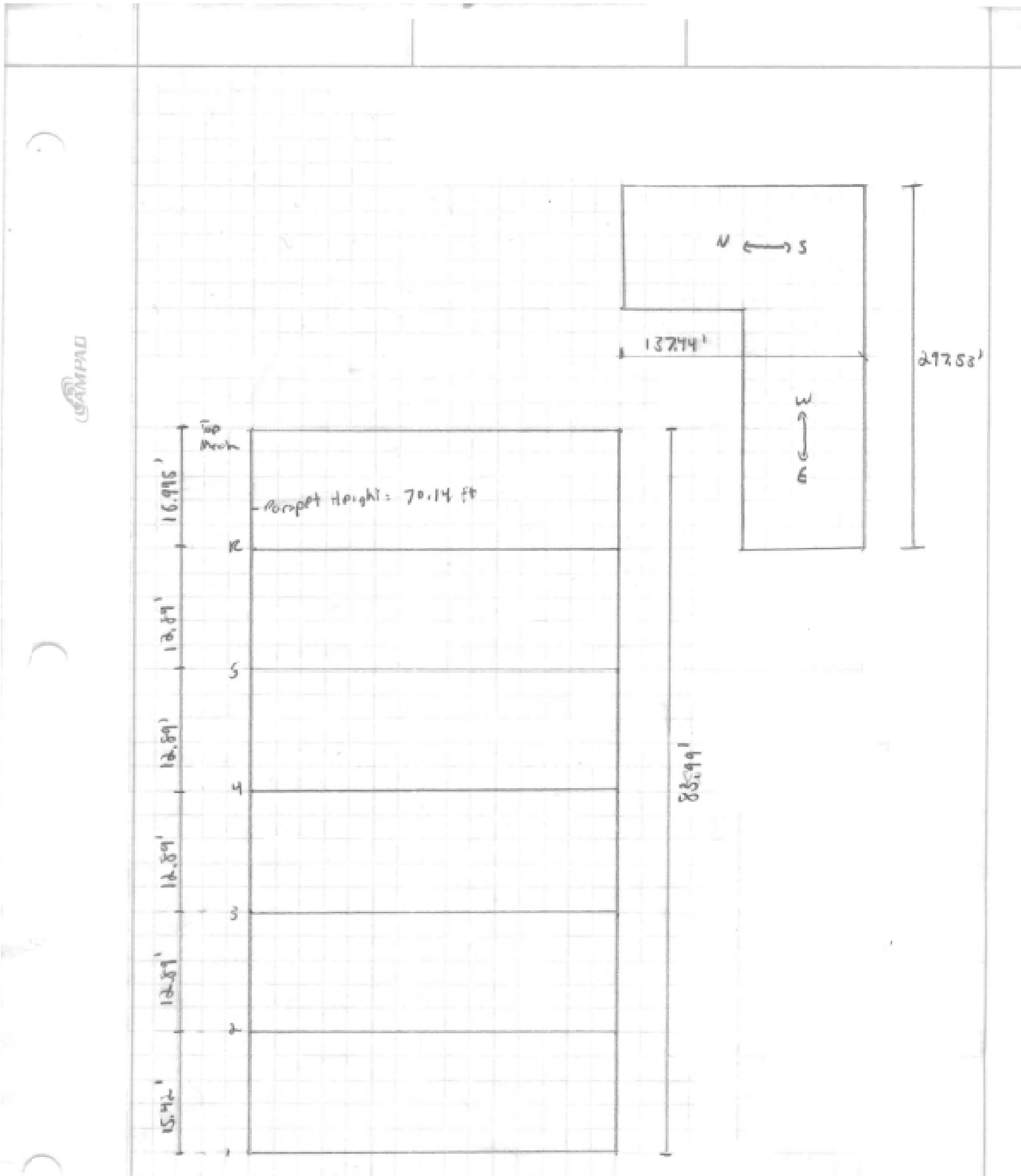
6. Determine velocity pressure exposure coefficients K_z and K_h
   from Table 6-2 w/ Exposure B α = 7, Z_g = 1200 ft

   From Table 6-3

   $$K_z = 2.01\left(\frac{15.4199}{1200}\right)^{2/7} = .5793$$

   $$q_z = .00256(.5793)(.85)(1)(90)^2(1) = 10.21$$

   See excel for complete table


N S
W E
137.44'
297.58'
Top Mech
Parapet Height = 70.14 ft
R
5
4
3
2
16.995'
13.89'
13.89'
13.89'
13.89'
15.42'
83.99'

Method 2: Gust Effect Factors G and Gf

B = 137.436'
L = 297.55'
h = 70.14'

* Rigid is assume to be rigid, design as concentric braced frames

By 6.5.8.1
G = .85 is more conservative if h/L < 4 $\frac{70.14}{137.44}$ = .51 < 4

G = .85

Determine Pressure coefficient Cp for the walls and roof from fig 6-6

For wind in the E-W direction

Windward wall : Cp = .8 for use w/ q_z
Leeward wall : Cp = -.5 for use w/ q_h
Sidewall : Cp = -.7 for use with q_h

For wind in the N-S direction

Windward wall : Cp = .8 for use w/ q_z
Leeward wall : $\frac{297.53}{137.436}$ = 2.16 Cp = -.2 for use w/ q_h
Sidewall : Cp = -.7 for use with q_h

For Roof E-W, N-S

E-S h/L : $\frac{70.14}{137.44}$ = .51 ≈ .5   N-S $\frac{h}{L} = \frac{70.14}{297.53}$ = .24 < .25

Cp: -.9, ±.18 for windward edge to 70.14'
Cp = -.5, ±.18 from 70.14 to 140.28'
Cp: -.3, ±.18 from 140.28 to 297.55'

- Determine design wind press P_z and P_h

$P_z = q_z G C_p$ - GCpi

Appendix B: Seismic Loads

Seismic Design Variables			(ASCE 7-05 Ref.)
Soil Classification		C	
Occupancy		II	(Table 1-1)
Structural System		Building Frame System: Ordinary reinforce concrete shear walls	(Table 12.2-1)
Spectral Response Acceleration, short	S_s	0.155	(USGS)
Spectral Response Acceleration, 1 s	S_1	0.05	(USGS)
Site Coefficient	Fa	1.2	(Table 11.4-1)
Site Coefficient	Fv	1.7	(Table 11.4-2)
Soil Modified Accelerationd, short	S_{ms}	0.186	(Eq. 11.4-1)
Soil Modified Accelerationd, 1 s	S_{m1}	0.085	(Eq. 11.4-2)
Design Spectral Acceleration, short	S_{DS}	0.124	(Eq. 11.4-3)
Design Spectral Acceleration, 1 s	S_{D1}	0.057	(Eq. 11.4-4)
Approximate Period Parameter	C_t	0.002	(Table 12.8-2)
Approximate Period Parameter	x	0.750	(Table 12.8-2)
Building height (above grade)	h_n	70.14 ft	
Approximate Fundamental Period	T_a	0.485	(Eq. 12.8-7)
Fundamental Period	T_s	0.460	
80% of Fundamental Period	$.8T_s$	0.368	
Seismic Design Category	S_{DC}	A	(Table 11.6-1)
Seismic Response Coefficient	C_s	0.012	(Eq 12.8-3)
Structure Period Exponet	k	1.250	(Sec. 12.8.3)
Seismic Base Shear	V	270.3 kips	(Eq. 12.8-1)

Seismic Loads					
Level	Story Weight w_x (kips)	Height h_x (ft)	Lateral Force F_x (Kips)	Base Shear (kips)	
2	1711.82	15.82	17.12		
3	1696.03	28.31	16.96		
4	1696.03	41.2	16.96		
5	1696.03	54.09	16.96		
Roof	2680.3	66.98	26.80		
			$\Sigma F_x = V_x =$	**95**	**kips**

Seismic Loads

Design Data

Location: Silver Spring, Md
Soil Classification: Site Class C
Occupancy: Office where less than 300 people congregate
Material: Structural A992 steel
Structural System: Concentric Braced Frames, Moment Frames

Seismic Ground Motion Values

1. Determine the mapped acceleration S_s and S_1
   From the USGS Ground Motion Parameter Calculator

   $S_s = .155$ $S_1 = .05$

2. Determine soil-modified acceleration S_{ms} and S_{m1}

   $F_a = 1.2$ By Table 11.4-1
   $F_v = 1.7$ By Table 11.4-2

   $S_{ms} = 1.2(.155) = .186$
   $S_{m1} = 1.7(.05) = .085$

3. Determine design acceleration

   $S_{Ds} = 2/3(.186) = .124$
   $S_{D1} = 2/3(.085) = .057$

Determine the SDC

- $S_s = .124 \le .15$ $S_1 = .057 > .04$ ∴ cannot be automatically assigned into SDC A
- Occupancy Category: II
- Conditions in 11.6 are not met
  ↳ Determine SDC by Table 11.6-1, 11.6-2
- From Table 11.6-1 with $S_{Ds} = .124$, Occupancy II ⇒ SDC = A
- From Table 11.6-2 with $S_{D1} = .057$, Occupancy II ⇒ SDC = A

Seismic Cont

11.7 Design Requirements for Seismic Design Category A

by 11.7.2 Lateral forces

$F_x = .01 w_x$ F_x: design lateral force applied at story x
w_x = portion of the total Dead Load of the structure at level x

Calculation of typical Floor self weight

Floor Area = 20824 SF
Floor Perimeter = 809.674 Ft

Floor Heights: 2nd - 14.16 ft
3rd-Re - 12.89 ft

Atrium Walls Area: 602 SF
Atrium Roof Area: 3711 SF

Weight of 2nd floor

Equiv weight of steel on metal deck floor System ≈ 50 psf
Includes slab, decking, Beams, Colums, Girders

DL: 50psf
SDL: 15 psf

2nd Floor weight = 20824(65)/1000 = 1353.56 kips
Wall Perimeter = $809.674\left(\frac{14.16}{2} + \frac{12.86}{2}\right)$ 30 psf /1000 = 328.16 kips

Atrium Weight: 602(50)/1000 = 30.1 k

W_2 = 1711.82 kips

F_2 = 17.12 kips

Weight of 3rd-5th Floors Typical

Floor weight = 20824(65)/1000 = 1353.56 kips
Wall Perimeter = 809.674(12.86)30/1000 = 312.37 kips
Atrium weight = 602(50)/1000 30.1 k

W_{3-5} = 1696.03 kips

F_{3-5} = 16.96 kips

Weight of Roof

Roof weight = 20824(65 + 20 [Snow])/1000 = 1770.04 kips
Wall Perimeter = $809.674\left(\frac{12.86}{2} + 3.146\right)$ 30/1000 = 232.60 kips

Atrium Roof weight = 3711(65+20)/1000 = 315.44 kips

W_R = 2318.08 kips

Seismic Cont.

Weight Cont:

- Mechanical Penthouse 1

W Roof : 457.5(40)/1000 = 18.3 kips

walls : 88.582(14.659)(20) = 25.97 kips

Colums/Beams = 4.49 kips

W_{P1} = 48.76 kips

- Mechanical Penthouse 2

W_{roof} : 356.96(40) = 14.278 kips

walls = 82.02(15.637)(20) = 25.63 kips

Colums/Beams = 4.725 kips

W_{P2} = 44.63 kips

- Mechanical Penthouse 3

W_{roof} = 392.6(40) = 15.704 kips

walls = 79.4(15.28)(20) = 24.265 kips

columns/Beams = 3.887 kips

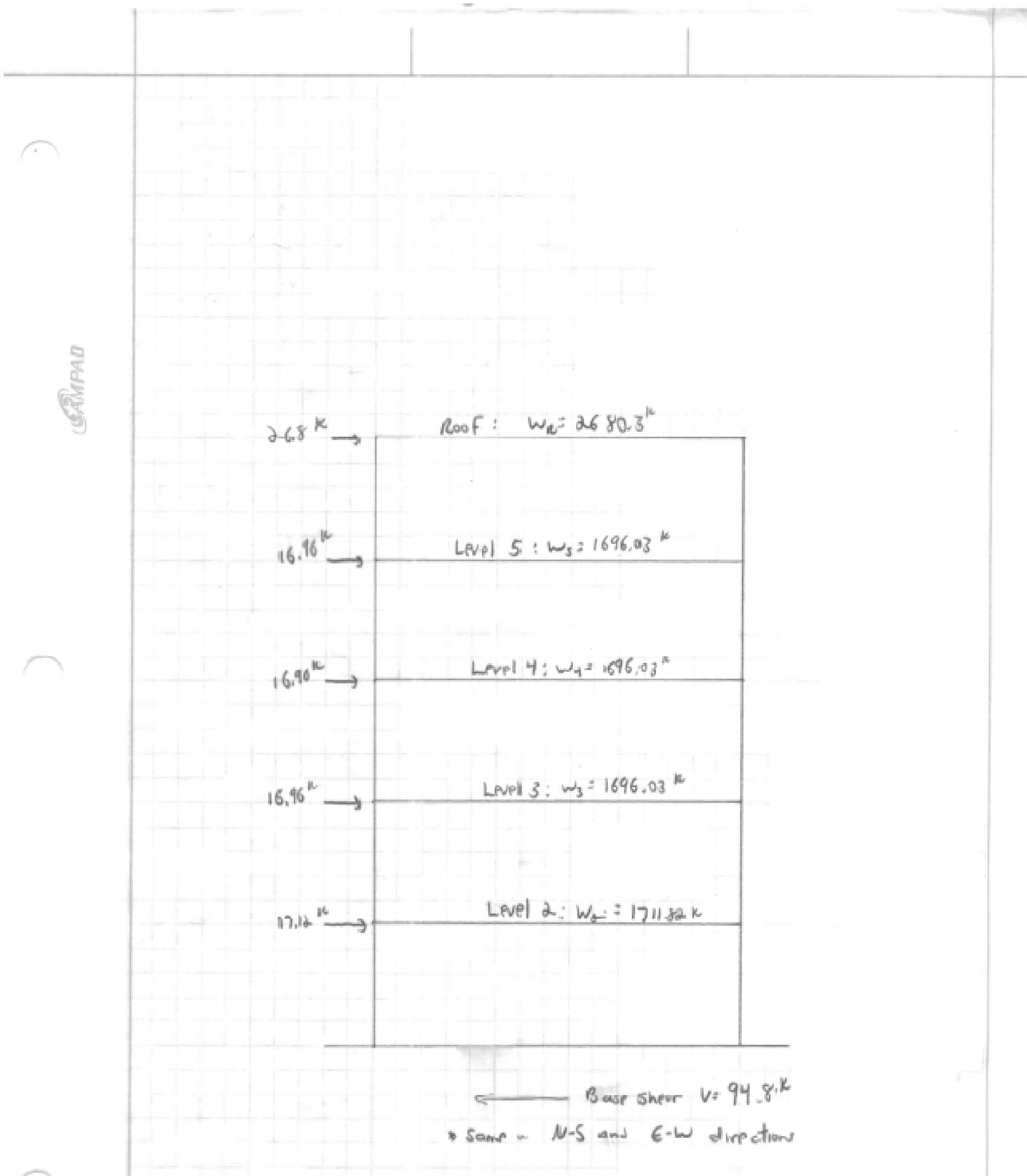
W_{P3} = 43.86 kips

- Mechanical Unit

W = 1500(150)/1000 = 225 kips

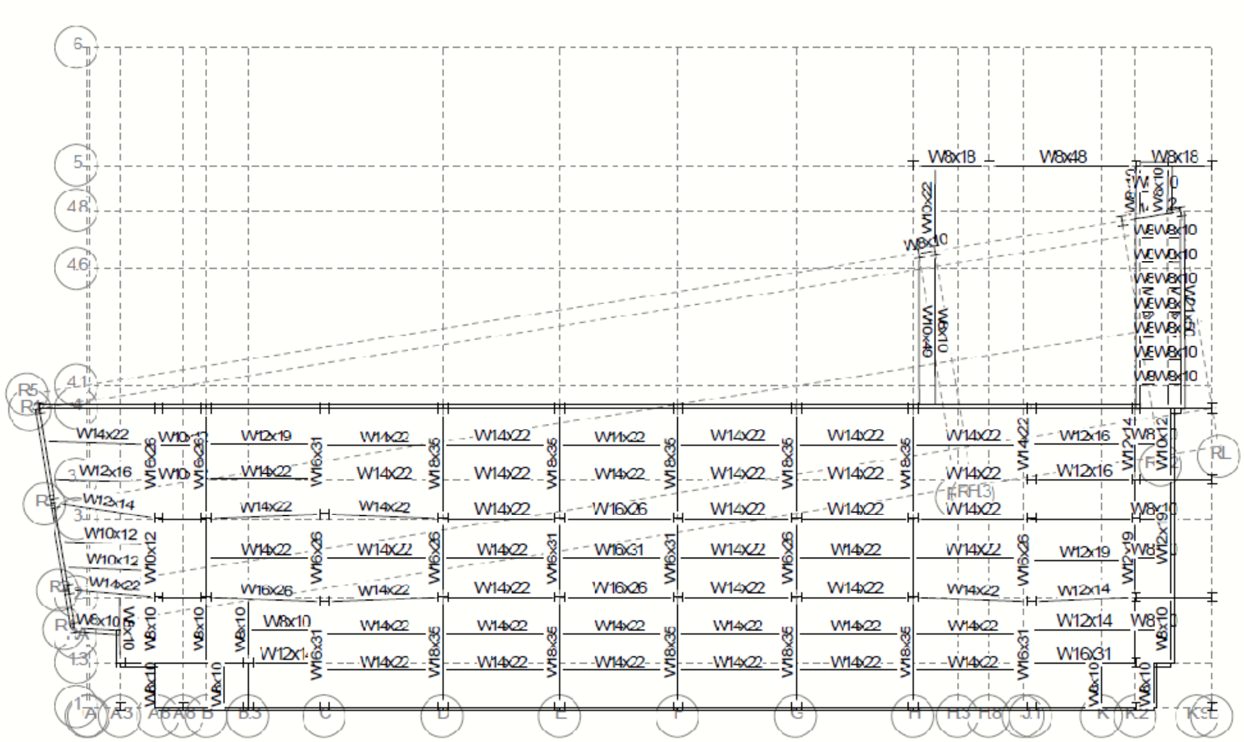
Total Roof = 2680.3 kips

F_R = 26.80 kips


26.8 k
Roof : W_R = 2680.3 k
16.96 k
Level 5 : W_5 = 1696.03 k
16.96 k
Level 4 : W_4 = 1696.03 k
16.96 k
Level 3 : W_3 = 1696.03 k
17.14 k
Level 2 : W_2 = 1711.82 k
Base Shear V = 94.8 k
* Same in N-S and E-W directions

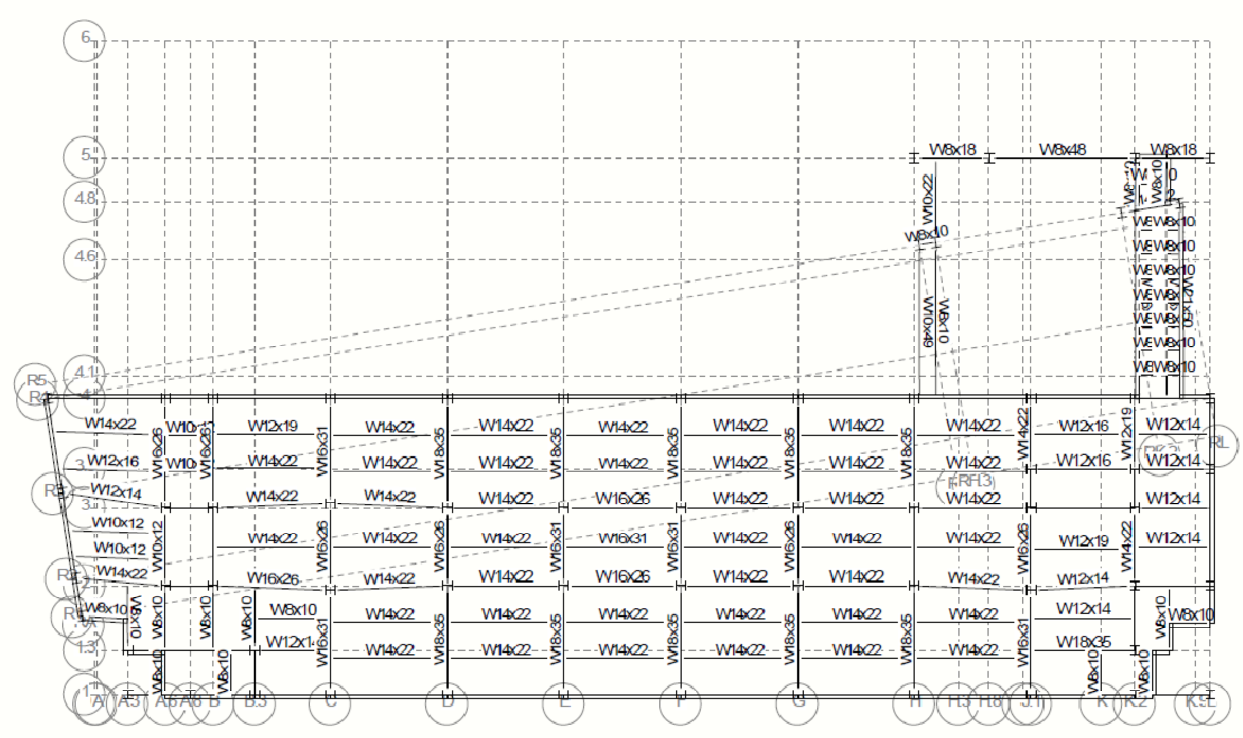
Appendix C: Steel Framing

Floor Type: 2nd

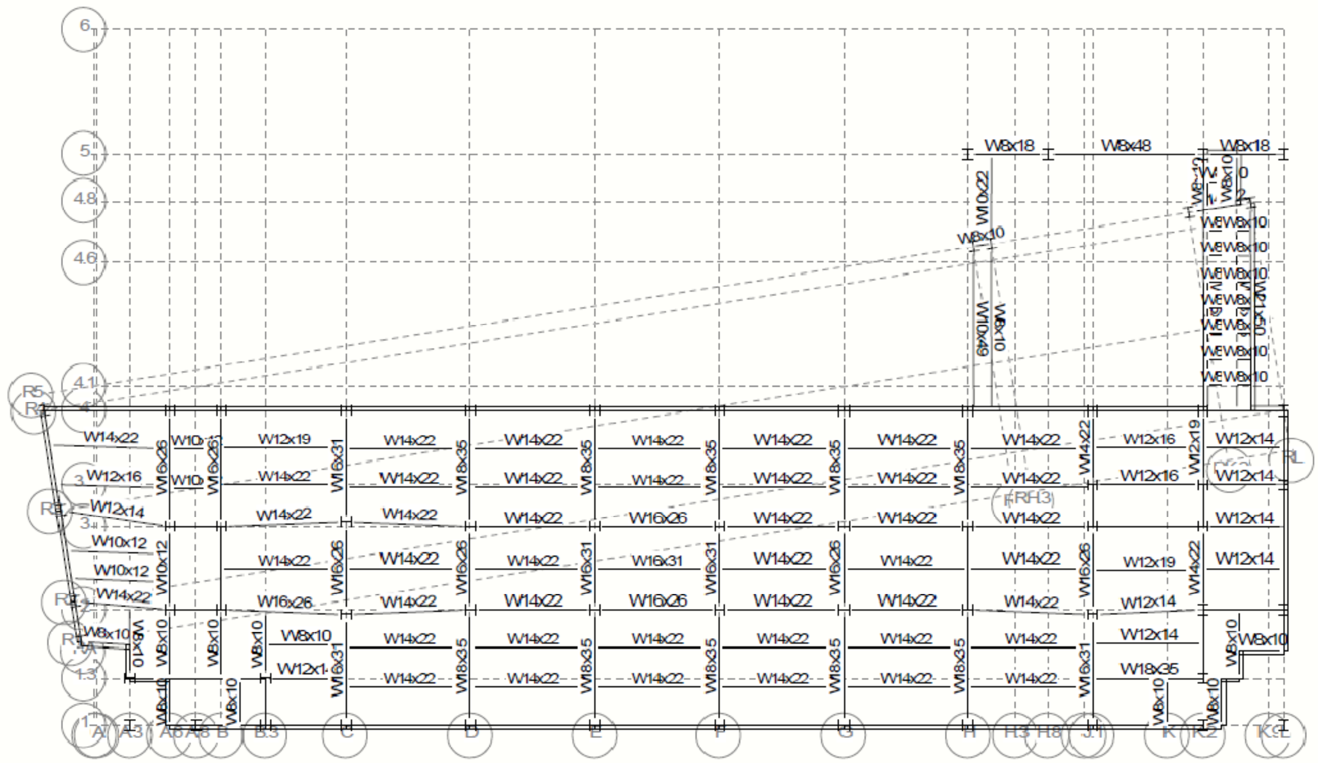


Final Report

Floor Type: 3rd

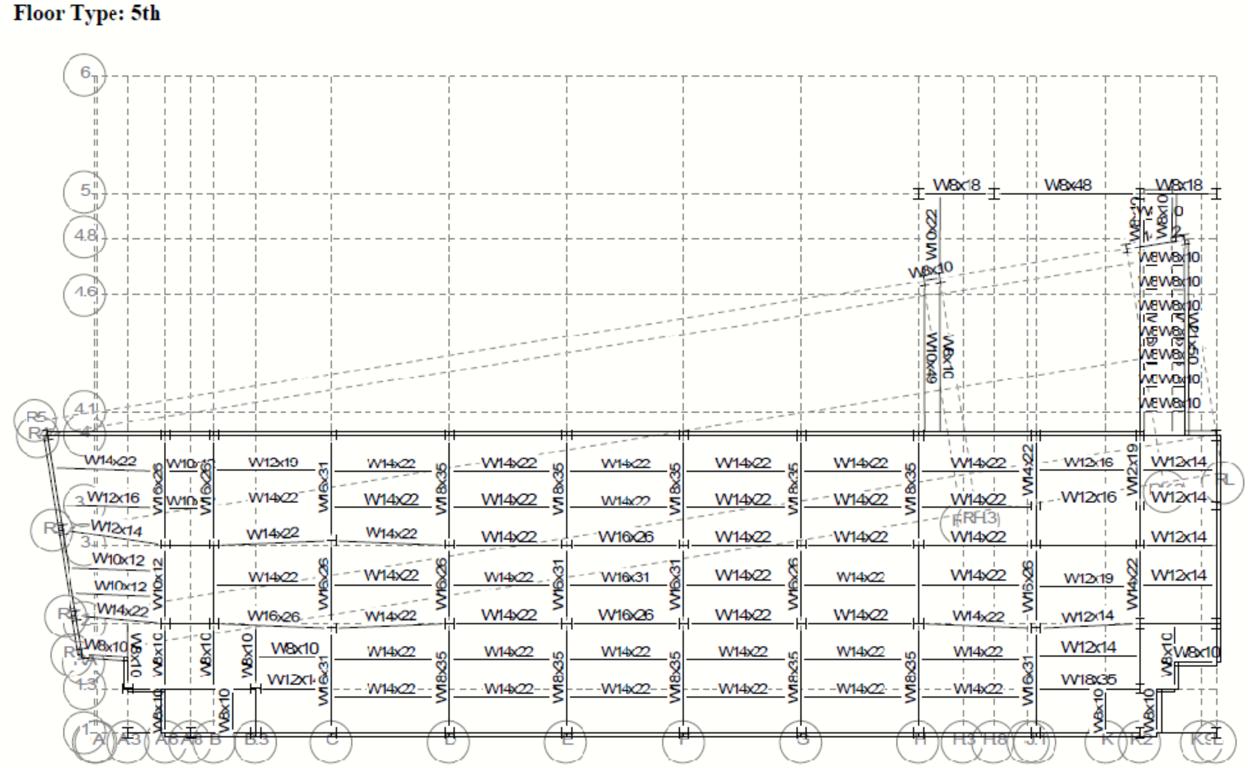


Floor Type: 4th

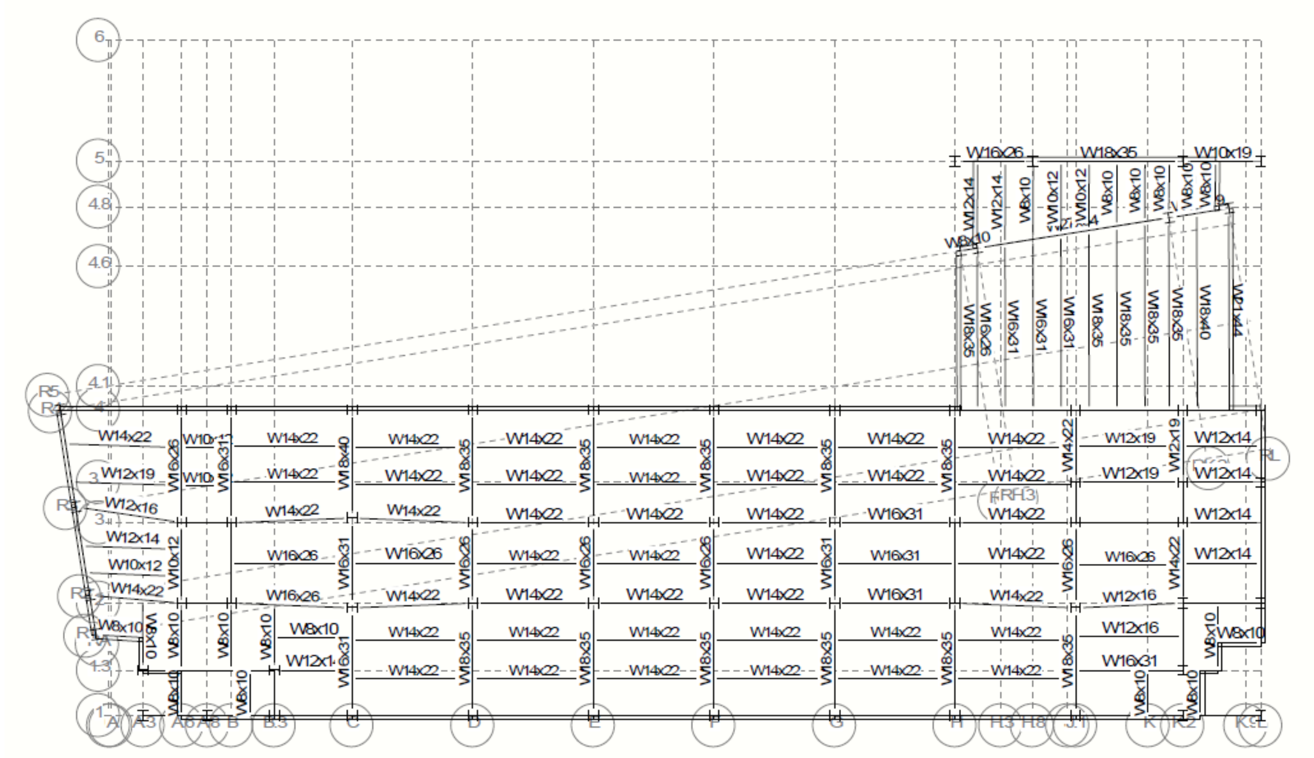


Final Report

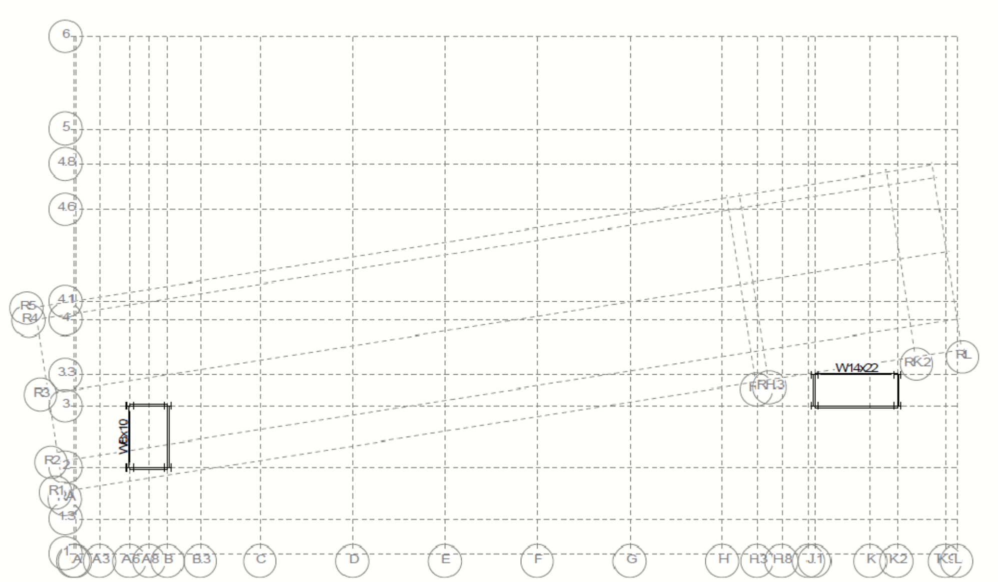
Floor Type: 5th



Floor Type: Roof



Floor Type: Penthouse



Gravity Column Design Summary

Column Line 59.66ft-26.41ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	63.8	2.3	7.6	6	0.57 Eq (H1-1a)	0.0	50	W10X33
Floor 5	131.9	2.0	3.4	3	0.69 Eq (H1-1a)	0.0	50	W10X39
Floor 4	186.5	1.9	4.6	6	0.96 Eq (H1-1a)	0.0	50	W10X39
Floor 3	252.3	1.9	3.8	3	0.75 Eq (H1-1a)	0.0	50	W10X54
Floor 2	310.6	0.4	4.1	6	0.96 Eq (H1-1a)	0.0	50	W10X54

Column Line 59.66ft-48.39ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	66.8	2.0	9.9	11	0.63 Eq (H1-1a)	0.0	50	W10X33
Floor 5	137.4	1.7	4.5	4	0.62 Eq (H1-1a)	0.0	50	W10X45
Floor 4	199.8	1.6	4.3	4	0.86 Eq (H1-1a)	0.0	50	W10X45
Floor 3	263.7	1.7	5.1	4	0.71 Eq (H1-1a)	0.0	50	W10X60
Floor 2	324.8	0.2	5.1	10	0.90 Eq (H1-1a)	0.0	50	W10X60

Column Line D-2

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	65.7	1.8	10.2	6	0.64 Eq (H1-1a)	0.0	50	W10X33
Floor 5	134.8	1.6	4.6	3	0.72 Eq (H1-1a)	0.0	50	W10X39
Floor 4	191.1	1.4	5.5	6	0.99 Eq (H1-1a)	0.0	50	W10X39
Floor 3	258.4	1.5	5.2	3	0.78 Eq (H1-1a)	0.0	50	W10X54
Floor 2	318.5	0.0	5.7	6	1.00 Eq (H1-1a)	0.0	50	W10X54

Column Line D-3

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	65.7	1.8	10.2	11	0.64 Eq (H1-1a)	0.0	50	W10X33
Floor 5	134.8	1.6	4.6	4	0.72 Eq (H1-1a)	0.0	50	W10X39
Floor 4	191.1	1.4	5.5	11	0.99 Eq (H1-1a)	0.0	50	W10X39
Floor 3	258.4	1.5	5.2	4	0.78 Eq (H1-1a)	0.0	50	W10X54
Floor 2	318.5	0.0	5.7	10	1.00 Eq (H1-1a)	0.0	50	W10X54

Column Line E-2

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	65.7	2.6	10.4	7	0.65 Eq (H1-1a)	0.0	50	W10X33
Floor 5	138.5	2.2	5.0	2	0.64 Eq (H1-1a)	0.0	50	W10X45
Floor 4	202.7	2.0	4.8	2	0.88 Eq (H1-1a)	0.0	50	W10X45
Floor 3	268.0	2.0	5.7	2	0.73 Eq (H1-1a)	0.0	50	W10X60
Floor 2	328.8	0.5	5.7	6	0.92 Eq (H1-1a)	0.0	50	W10X60

Column Line E-3

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	65.7	2.6	10.4	10	0.65 Eq (H1-1a)	0.0	50	W10X33
Floor 5	138.5	2.2	5.0	5	0.64 Eq (H1-1a)	0.0	50	W10X45

Floor 4	202.7	2.0	4.8	5	0.88 Eq (H1-1a)	0.0	50	W10X45
Floor 3	268.0	2.0	5.7	5	0.73 Eq (H1-1a)	0.0	50	W10X60
Floor 2	328.8	0.5	5.7	10	0.92 Eq (H1-1a)	0.0	50	W10X60

Column Line F-2

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	65.7	2.6	10.4	6	0.65 Eq (H1-1a)	0.0	50	W10X33
Floor 5	138.5	2.2	5.0	3	0.64 Eq (H1-1a)	0.0	50	W10X45
Floor 4	202.7	2.0	4.8	3	0.88 Eq (H1-1a)	0.0	50	W10X45
Floor 3	268.0	2.0	5.7	3	0.73 Eq (H1-1a)	0.0	50	W10X60
Floor 2	328.8	0.5	5.7	6	0.92 Eq (H1-1a)	0.0	50	W10X60

Column Line F-3

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	65.7	2.6	10.4	11	0.65 Eq (H1-1a)	0.0	50	W10X33
Floor 5	138.5	2.2	5.0	4	0.64 Eq (H1-1a)	0.0	50	W10X45
Floor 4	202.7	2.0	4.8	4	0.88 Eq (H1-1a)	0.0	50	W10X45
Floor 3	268.0	2.0	5.7	4	0.73 Eq (H1-1a)	0.0	50	W10X60
Floor 2	328.8	0.5	5.7	10	0.92 Eq (H1-1a)	0.0	50	W10X60

Column Line G-2

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	80.7	8.0	5.4	8	0.65 Eq (H1-1a)	0.0	50	W10X33
Floor 5	150.4	1.6	4.8	2	0.68 Eq (H1-1a)	0.0	50	W10X45
Floor 4	211.9	1.5	4.5	2	0.91 Eq (H1-1a)	0.0	50	W10X45
Floor 3	273.5	1.5	5.4	2	0.74 Eq (H1-1a)	0.0	50	W10X60
Floor 2	333.7	0.0	5.5	6	0.93 Eq (H1-1a)	0.0	50	W10X60

Column Line G-3

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	80.7	8.0	5.4	9	0.65 Eq (H1-1a)	0.0	50	W10X33
Floor 5	150.4	1.6	4.8	5	0.68 Eq (H1-1a)	0.0	50	W10X45
Floor 4	211.9	1.5	4.5	5	0.91 Eq (H1-1a)	0.0	50	W10X45
Floor 3	273.5	1.5	5.4	5	0.74 Eq (H1-1a)	0.0	50	W10X60
Floor 2	333.7	0.0	5.5	10	0.93 Eq (H1-1a)	0.0	50	W10X60

Column Line RH-R5

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	18.6	10.4	0.7	1	0.18 Eq (H1-1b)	99.0	50	W10X33
Floor 5	29.2	2.8	0.2	1	0.16 Eq (H1-1b)	99.0	50	W10X33
Floor 4	39.7	2.8	0.2	1	0.25 Eq (H1-1a)	99.0	50	W10X33
Floor 3	50.3	3.0	0.3	1	0.31 Eq (H1-1a)	99.0	50	W10X33
Floor 2	60.9	2.7	0.2	1	0.40 Eq (H1-1a)	99.0	50	W10X33

Column Line RH.3-R5

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	86.0	1.4	34.4	6	0.95 Eq (H1-1a)	99.0	50	W10X45
Floor 5	92.4	1.4	0.2	1	0.52 Eq (H1-1a)	99.0	50	W10X33
Floor 4	98.7	1.4	0.2	1	0.56 Eq (H1-1a)	99.0	50	W10X33
Floor 3	104.9	1.5	0.3	1	0.59 Eq (H1-1a)	99.0	50	W10X33
Floor 2	111.3	1.3	0.3	1	0.69 Eq (H1-1a)	99.0	50	W10X33

Column Line H.8-5

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	43.1	0.0	13.1	6	0.69 Eq (H1-1a)	90.0	50	W14X43
Floor 5	61.7	0.0	0.9	1	0.62 Eq (H1-1a)	90.0	50	W14X43
Floor 4	76.2	0.0	0.9	1	0.76 Eq (H1-1a)	90.0	50	W14X43
Floor 3	90.8	0.0	1.0	1	0.79 Eq (H1-1a)	90.0	50	W14X48
Floor 2	105.4	0.0	0.9	1	0.92 Eq (H1-1a)	90.0	50	W14X48

Column Line 236.82ft-26.41ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	61.1	2.4	9.7	6	0.60 Eq (H1-1a)	0.0	50	W10X33
Floor 5	125.2	2.1	4.4	3	0.68 Eq (H1-1a)	0.0	50	W10X39
Floor 4	181.8	2.0	4.1	3	0.93 Eq (H1-1a)	0.0	50	W10X39
Floor 3	238.7	2.0	4.9	3	0.73 Eq (H1-1a)	0.0	50	W10X54
Floor 2	294.2	0.7	5.0	6	0.92 Eq (H1-1a)	0.0	50	W10X54

Column Line RK.2-R5

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	93.3	9.5	35.4	10	0.91 Eq (H1-1a)	99.0	50	W14X68
Floor 5	111.4	13.0	4.4	1	0.81 Eq (H1-1a)	99.0	50	W14X61
Floor 4	125.0	15.8	4.3	1	0.91 Eq (H1-1a)	99.0	50	W14X61
Floor 3	142.0	16.4	4.6	1	0.91 Eq (H1-1a)	99.0	50	W14X68
Floor 2	155.8	20.6	4.2	1	1.00 Eq (H1-1a)	99.0	50	W14X68

Column Line K.2-5

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	36.1	0.0	11.6	10	0.59 Eq (H1-1a)	90.0	50	W14X43
Floor 5	57.7	0.0	0.5	4	0.57 Eq (H1-1a)	90.0	50	W14X43

Floor 4	76.1	0.0	0.5	4	0.75 Eq (H1-1a)	90.0	50	W14X43
Floor 3	94.4	0.0	0.5	4	0.82 Eq (H1-1a)	90.0	50	W14X48
Floor 2	112.8	0.0	0.3	1	0.97 Eq (H1-1a)	90.0	50	W14X48

Column Line RL-R5

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	44.7	16.0	9.8	1	0.64 Eq (H1-1a)	99.0	50	W10X33
Floor 5	82.8	5.6	6.1	1	0.66 Eq (H1-1a)	99.0	50	W10X33
Floor 4	120.0	5.5	5.8	1	0.86 Eq (H1-1a)	99.0	50	W10X33
Floor 3	156.9	5.8	6.0	1	0.75 Eq (H1-1a)	99.0	50	W10X45
Floor 2	193.5	5.3	7.7	1	0.99 Eq (H1-1a)	99.0	50	W10X45

Steel Beam Design Summary

Floor Type: Penthouse

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
	ft	kip-ft	kip-ft	kip-ft	ksi		
1	19.69	31.4	0.0	68.3	50.0	W8X10	6
7	26.25	145.0	0.0	210.9	50.0	W14X22	14

Floor Type: Roof

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
	ft	kip-ft	kip-ft	kip-ft	ksi		
3	28.53	186.6	0.0	220.2	50.0	W14X22	18
4	27.05	160.3	0.0	191.1	50.0	W12X19	21
6	26.36	143.6	0.0	173.3	50.0	W12X16	23
7	24.58	116.8	0.0	137.6	50.0	W12X14	16
8	23.55	92.7	0.0	113.9	50.0	W10X12	17
10	22.74	154.6	0.0	211.9	50.0	W14X22	7, 7
11	11.59	28.9	0.0	61.8	50.0	W8X10	6
12	17.22	54.3	0.0	68.3	50.0	W8X10	3, 3
15	11.15	3.4	0.0	68.0	50.0	W8X10	7
16	16.40	0.5	0.0	68.7	50.0	W8X10	6
17	19.69	116.2	0.0	137.8	50.0	W10X12	9, 2, 10
19	27.56	317.9	0.0	378.7	50.0	W16X26	19, 2, 21
21	12.14	36.1	0.0	52.5	50.0	W10X12	
22	12.14	36.2	0.0	52.5	50.0	W10X12	
25	16.40	0.5	0.0	68.7	50.0	W8X10	6
27	29.55	253.5	0.0	317.6	50.0	W16X26	11, 19
28	29.53	222.6	0.0	371.5	50.0	W16X26	44
29	27.56	339.1	0.0	408.0	50.0	W16X31	15, 2, 3, 1, 13
30	29.55	213.4	0.0	254.7	50.0	W14X22	28
31	29.53	199.5	0.0	239.5	50.0	W14X22	23
32	29.53	194.6	0.0	233.8	50.0	W14X22	20
34	11.15	0.2	0.0	68.2	50.0	W8X10	6
37	16.00	64.5	0.0	100.8	50.0	W8X10	16
38	19.09	93.1	0.0	114.5	50.0	W12X14	8
39	19.09	75.9	0.0	89.5	50.0	W8X10	18
40	26.41	365.8	0.0	433.0	50.0	W16X31	18, 4, 8, 2, 13
42	29.53	201.2	0.0	239.5	50.0	W14X22	22
43	29.53	196.2	0.0	233.8	50.0	W14X22	21
44	21.98	338.4	0.0	403.5	50.0	W16X31	34
45	29.55	206.9	0.0	244.7	50.0	W14X22	24
46	29.53	222.6	0.0	371.5	50.0	W16X26	44
47	26.41	417.4	0.0	563.1	50.0	W18X40	10, 1, 10, 1, 20
48	29.55	206.9	0.0	244.7	50.0	W14X22	24

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
49	29.53	196.2	0.0	233.8	50.0	W14X22	21
50	29.53	201.2	0.0	239.5	50.0	W14X22	22
52	27.56	442.9	0.0	525.0	50.0	W18X35	25, 2, 24
54	29.53	201.2	0.0	239.5	50.0	W14X22	22
55	29.53	201.2	0.0	239.5	50.0	W14X22	22
56	19.69	289.3	0.0	341.2	50.0	W16X26	30
57	29.53	206.7	0.0	244.7	50.0	W14X22	24
58	29.53	212.3	0.0	254.7	50.0	W14X22	28
59	27.56	442.9	0.0	525.0	50.0	W18X35	24, 2, 25
60	29.53	206.7	0.0	244.7	50.0	W14X22	24
61	29.53	201.2	0.0	239.5	50.0	W14X22	22
62	29.53	201.2	0.0	239.5	50.0	W14X22	22
64	27.56	444.5	0.0	529.4	50.0	W18X35	25, 2, 25
66	29.53	201.2	0.0	239.5	50.0	W14X22	22
67	29.53	201.2	0.0	239.5	50.0	W14X22	22
68	19.69	284.6	0.0	336.0	50.0	W16X26	28
69	29.53	206.7	0.0	244.7	50.0	W14X22	24
70	29.53	212.3	0.0	254.7	50.0	W14X22	28
71	27.56	444.5	0.0	529.4	50.0	W18X35	25, 2, 25
72	29.53	206.7	0.0	244.7	50.0	W14X22	24
73	29.53	201.2	0.0	239.5	50.0	W14X22	22
74	29.53	201.2	0.0	239.5	50.0	W14X22	22
76	27.56	444.5	0.0	529.4	50.0	W18X35	25, 2, 25
78	29.53	201.2	0.0	239.5	50.0	W14X22	22
79	29.53	201.2	0.0	239.5	50.0	W14X22	22
80	19.69	284.6	0.0	336.0	50.0	W16X26	28
81	29.53	206.7	0.0	244.7	50.0	W14X22	24
82	29.53	212.3	0.0	254.7	50.0	W14X22	28
83	27.56	444.5	0.0	529.4	50.0	W18X35	25, 2, 25
84	29.53	206.7	0.0	244.7	50.0	W14X22	24
85	29.53	201.2	0.0	239.5	50.0	W14X22	22
86	29.53	201.2	0.0	239.5	50.0	W14X22	22
88	27.56	444.5	0.0	529.4	50.0	W18X35	25, 2, 25
90	29.53	201.2	0.0	239.5	50.0	W14X22	22
91	29.53	201.2	0.0	239.5	50.0	W14X22	22
92	19.69	333.4	0.0	396.0	50.0	W16X31	32
93	29.53	241.7	0.0	376.5	50.0	W16X31	30
94	29.53	292.1	0.0	447.1	50.0	W16X31	50
95	27.56	444.5	0.0	529.4	50.0	W18X35	25, 2, 25
96	29.53	241.7	0.0	376.5	50.0	W16X31	30
97	29.53	201.2	0.0	239.5	50.0	W14X22	22
98	29.53	201.2	0.0	239.5	50.0	W14X22	22
100	27.56	442.9	0.0	525.0	50.0	W18X35	25, 2, 24
102	29.53	201.2	0.0	239.5	50.0	W14X22	22
103	29.53	196.2	0.0	233.8	50.0	W14X22	21

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
105	29.55	206.9	0.0	244.7	50.0	W14X22	24
106	29.53	217.2	0.0	254.7	50.0	W14X22	29
107	27.56	444.5	0.0	529.4	50.0	W18X35	25, 2, 25
108	29.53	206.7	0.0	244.7	50.0	W14X22	24
109	29.53	201.2	0.0	239.5	50.0	W14X22	22
110	29.53	201.2	0.0	239.5	50.0	W14X22	22
112	18.91	120.2	0.0	184.2	50.0	W16X26	
113	38.95	225.8	0.0	392.2	50.0	W18X35	14
114	4.12	1.6	0.0	58.6	50.0	W8X10	5
115	39.59	228.5	0.0	292.6	50.0	W16X26	20
116	21.28	82.4	0.0	114.3	50.0	W12X14	7
117	47.54	1355.0	0.0	1600.0	50.0	W27X84	90
118	40.68	286.7	0.0	377.5	50.0	W16X31	32
119	20.19	74.5	0.0	114.6	50.0	W12X14	8
120	36.86	381.4	0.0	485.1	50.0	W18X35	40
121	41.76	300.5	0.0	387.1	50.0	W16X31	36
122	19.11	66.5	0.0	79.5	50.0	W8X10	12
123	42.84	314.7	0.0	445.5	50.0	W16X31	56
124	18.03	59.2	0.0	89.7	50.0	W10X12	8
125	26.41	390.0	0.0	531.6	50.0	W18X35	22, 5, 13, 2, 12
127	26.25	124.5	-186.5	368.5	50.0	W16X31	31
	8.67	0.0	-186.5				
128	26.25	144.7	0.0	173.3	50.0	W12X16	23
129	20.83	288.8	0.0	342.3	50.0	W16X26	15, 1, 15
130	26.27	156.2	0.0	183.0	50.0	W12X16	26
131	26.25	176.2	0.0	276.8	50.0	W16X26	17
134	17.72	224.4	0.0	269.1	50.0	W14X22	14, 14
135	26.25	164.8	0.0	196.1	50.0	W12X19	22
136	26.25	158.1	0.0	191.0	50.0	W12X19	20
138	43.92	330.3	0.0	437.0	50.0	W18X35	28
139	16.95	52.3	0.0	89.6	50.0	W10X12	8
140	45.01	345.1	0.0	450.9	50.0	W18X35	32
141	15.86	45.8	0.0	68.6	50.0	W8X10	8
142	46.09	341.9	0.0	466.6	50.0	W18X35	38
143	14.78	37.2	0.0	62.1	50.0	W8X10	6
144	11.15	0.2	0.0	68.2	50.0	W8X10	6
145	47.03	354.5	0.0	471.1	50.0	W18X35	40
146	13.84	32.5	0.0	62.1	50.0	W8X10	6
147	14.87	179.2	0.0	214.9	50.0	W12X19	13, 7, 6
150	19.69	226.5	0.0	271.0	50.0	W14X22	14, 1, 13
152	19.03	95.0	0.0	114.5	50.0	W12X14	8
154	19.03	95.0	0.0	114.5	50.0	W12X14	8
155	17.72	190.8	0.0	225.8	50.0	W12X19	28
156	19.03	90.3	0.0	114.5	50.0	W12X14	8

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
157	19.03	85.6	0.0	114.5	50.0	W12X14	8
159	19.03	63.9	0.0	90.0	50.0	W10X19	
160	48.11	413.6	0.0	563.2	50.0	W18X40	52
161	12.76	24.6	0.0	62.0	50.0	W8X10	6
162	11.15	3.4	0.0	68.0	50.0	W8X10	7
163	12.04	21.8	0.0	68.0	50.0	W8X10	6
164	16.40	45.3	0.0	68.2	50.0	W8X10	3, 3
165	10.35	23.4	0.0	61.8	50.0	W8X10	6
166	49.36	487.9	0.0	680.6	50.0	W21X44	46

Floor Type: 5th

Bm #	Length ft	+Mu kip-ft	-Mu kip-ft	Mn kip-ft	Fy ksi	Beam Size	Studs
3	28.53	172.0	0.0	212.4	50.0	W14X22	15
4	27.05	147.7	0.0	178.4	50.0	W12X16	25
6	26.36	132.6	0.0	159.1	50.0	W12X14	23
7	24.58	107.8	0.0	128.8	50.0	W10X12	23
8	23.55	85.8	0.0	103.3	50.0	W10X12	13
10	22.74	143.8	0.0	211.9	50.0	W14X22	7, 7
11	11.59	27.3	0.0	61.8	50.0	W8X10	6
12	17.22	51.9	0.0	68.3	50.0	W8X10	3, 3
15	11.15	3.2	0.0	68.0	50.0	W8X10	7
16	16.40	0.5	0.0	68.7	50.0	W8X10	6
17	19.69	107.4	0.0	132.1	50.0	W10X12	8, 2, 9
19	27.56	291.1	0.0	346.8	50.0	W16X26	13, 2, 15
21	12.14	33.4	0.0	52.5	50.0	W10X12	
22	12.14	33.5	0.0	52.5	50.0	W10X12	
25	16.40	0.5	0.0	68.7	50.0	W8X10	6
27	29.55	232.3	0.0	277.1	50.0	W16X26	8, 9
28	29.53	203.9	0.0	244.7	50.0	W14X22	25
29	27.56	309.9	0.0	368.7	50.0	W16X26	17, 2, 3, 1, 16
30	29.55	196.1	0.0	233.8	50.0	W14X22	20
31	29.53	183.7	0.0	220.2	50.0	W14X22	17
32	29.53	178.8	0.0	211.7	50.0	W12X19	28
34	11.15	0.2	0.0	68.2	50.0	W8X10	6
37	16.00	59.7	0.0	88.9	50.0	W8X10	12
38	19.09	86.1	0.0	114.5	50.0	W12X14	8
39	19.09	70.2	0.0	84.6	50.0	W8X10	15
40	26.41	334.1	0.0	401.6	50.0	W16X31	13, 3, 6, 1, 9
42	29.53	185.1	0.0	220.2	50.0	W14X22	16
43	29.53	180.7	0.0	220.2	50.0	W14X22	17
44	21.98	309.7	0.0	367.5	50.0	W16X26	40
45	29.55	190.3	0.0	227.4	50.0	W14X22	18
46	29.53	203.9	0.0	244.7	50.0	W14X22	25
47	26.41	379.0	0.0	449.7	50.0	W16X31	21, 2, 3, 2, 24

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
48	29.55	190.3	0.0	227.4	50.0	W14X22	18
49	29.53	180.7	0.0	220.2	50.0	W14X22	17
50	29.53	185.1	0.0	220.2	50.0	W14X22	16
52	27.56	403.2	0.0	478.2	50.0	W18X35	16, 2, 16
54	29.53	185.1	0.0	220.2	50.0	W14X22	16
55	29.53	185.1	0.0	220.2	50.0	W14X22	16
56	19.69	265.6	0.0	312.9	50.0	W16X26	20
57	29.53	190.2	0.0	227.4	50.0	W14X22	18
58	29.53	195.2	0.0	233.8	50.0	W14X22	20
59	27.56	403.2	0.0	478.2	50.0	W18X35	16, 2, 16
60	29.53	190.2	0.0	227.4	50.0	W14X22	18
61	29.53	185.1	0.0	220.2	50.0	W14X22	16
62	29.53	185.1	0.0	220.2	50.0	W14X22	16
64	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
66	29.53	185.1	0.0	220.2	50.0	W14X22	16
67	29.53	185.1	0.0	220.2	50.0	W14X22	16
68	19.69	339.7	0.0	400.9	50.0	W16X31	34
69	29.53	249.2	0.0	384.0	50.0	W16X26	46
70	29.53	311.7	0.0	421.9	50.0	W16X31	42
71	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
72	29.53	249.2	0.0	384.0	50.0	W16X26	46
73	29.53	185.1	0.0	220.2	50.0	W14X22	16
74	29.53	185.1	0.0	220.2	50.0	W14X22	16
76	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
78	29.53	185.1	0.0	220.2	50.0	W14X22	16
79	29.53	185.1	0.0	220.2	50.0	W14X22	16
80	19.69	339.7	0.0	400.9	50.0	W16X31	34
81	29.53	190.2	0.0	227.4	50.0	W14X22	18
82	29.53	195.2	0.0	233.8	50.0	W14X22	20
83	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
84	29.53	190.2	0.0	227.4	50.0	W14X22	18
85	29.53	185.1	0.0	220.2	50.0	W14X22	16
86	29.53	185.1	0.0	220.2	50.0	W14X22	16
88	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
90	29.53	185.1	0.0	220.2	50.0	W14X22	16
91	29.53	185.1	0.0	220.2	50.0	W14X22	16
92	19.69	261.7	0.0	312.9	50.0	W16X26	20
93	29.53	190.2	0.0	227.4	50.0	W14X22	18
94	29.53	195.2	0.0	233.8	50.0	W14X22	20
95	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
96	29.53	190.2	0.0	227.4	50.0	W14X22	18
97	29.53	185.1	0.0	220.2	50.0	W14X22	16
98	29.53	185.1	0.0	220.2	50.0	W14X22	16
100	27.56	403.2	0.0	478.2	50.0	W18X35	16, 2, 16
102	29.53	185.1	0.0	220.2	50.0	W14X22	16

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
103	29.53	180.7	0.0	220.2	50.0	W14X22	17
105	29.55	190.3	0.0	227.4	50.0	W14X22	18
106	29.53	199.5	0.0	239.5	50.0	W14X22	23
107	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
108	29.53	190.2	0.0	227.4	50.0	W14X22	18
109	29.53	185.1	0.0	220.2	50.0	W14X22	16
110	29.53	185.1	0.0	220.2	50.0	W14X22	16
112	18.91	43.3	0.0	70.8	50.0	W8X18 2	
113	38.95	125.8	0.0	251.7	50.0	W10X49	
114	4.12	1.3	0.0	37.0	50.0	W8X10 2	
115	39.59	2.8	0.0	37.0	50.0	W8X10	
116	21.28	35.4	0.0	108.3	50.0	W10X22 2	
117	36.86	112.5	0.0	204.2	50.0	W8X48 2	
118	26.41	353.3	0.0	423.2	50.0	W16X31	16, 4, 6, 2, 12
120	26.25	111.8	-177.1	402.5	50.0	W18X35	24
	8.67	0.0	-177.1				
121	26.25	133.6	0.0	159.1	50.0	W12X14	23
122	20.83	265.2	0.0	313.4	50.0	W16X26	10, 1, 10
123	26.27	144.0	0.0	168.7	50.0	W12X14	26
124	26.25	161.5	0.0	191.0	50.0	W12X19	21
127	17.72	206.7	0.0	247.6	50.0	W14X22	10, 10
128	26.25	151.6	0.0	183.0	50.0	W12X16	26
129	26.25	145.5	0.0	173.3	50.0	W12X16	22
131	11.15	0.2	0.0	68.2	50.0	W8X10	6
132	14.87	118.8	0.0	138.3	50.0	W14X22	
135	19.69	208.7	0.0	248.5	50.0	W14X22	10, 1, 10
137	19.03	87.9	0.0	114.5	50.0	W12X14	8
139	19.03	87.9	0.0	114.5	50.0	W12X14	8
140	17.72	176.3	0.0	210.3	50.0	W12X19	22
141	19.03	83.6	0.0	114.5	50.0	W12X14	8
142	19.03	79.2	0.0	114.5	50.0	W12X14	8
144	19.03	56.8	0.0	70.8	50.0	W8X18 2	
145	47.74	259.1	0.0	388.6	50.0	W16X31	32
146	13.13	18.6	0.0	68.1	50.0	W8X10	6
147	6.89	6.8	0.0	37.0	50.0	W8X10	
148	6.89	7.2	0.0	37.0	50.0	W8X10	
149	6.89	7.2	0.0	37.0	50.0	W8X10	
150	6.89	7.2	0.0	37.0	50.0	W8X10	
151	6.88	7.3	0.0	37.0	50.0	W8X10	
152	6.89	7.2	0.0	37.0	50.0	W8X10	
153	6.89	7.2	0.0	37.0	50.0	W8X10	
154	6.89	7.1	0.0	37.0	50.0	W8X10	
155	11.15	3.2	0.0	68.0	50.0	W8X10	7
156	48.83	307.0	0.0	461.8	50.0	W18X35	28
157	12.04	25.4	0.0	68.0	50.0	W8X10	3, 3

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
158	3.35	1.7	0.0	37.0	50.0	W8X10	
159	3.35	1.7	0.0	37.0	50.0	W8X10	
160	3.35	1.7	0.0	37.0	50.0	W8X10	
161	3.35	1.7	0.0	37.0	50.0	W8X10	
162	3.35	1.7	0.0	37.0	50.0	W8X10	
163	3.35	1.7	0.0	37.0	50.0	W8X10	
164	3.35	1.7	0.0	37.0	50.0	W8X10	
165	16.40	43.3	0.0	68.2	50.0	W8X10	3, 3
166	10.35	22.1	0.0	61.8	50.0	W8X10	6
167	49.36	332.6	0.0	646.9	50.0	W21X50	23

Floor Type: 4th

Bm #	Length ft	+Mu kip-ft	-Mu kip-ft	Mn kip-ft	Fy ksi	Beam Size	Studs
3	28.53	172.0	0.0	212.4	50.0	W14X22	15
4	27.05	147.7	0.0	178.4	50.0	W12X16	25
6	26.36	132.6	0.0	159.1	50.0	W12X14	23
7	24.58	107.8	0.0	128.8	50.0	W10X12	23
8	23.55	85.8	0.0	103.3	50.0	W10X12	13
10	22.74	143.8	0.0	211.9	50.0	W14X22	7, 7
11	11.59	27.3	0.0	61.8	50.0	W8X10	6
12	17.22	51.9	0.0	68.3	50.0	W8X10	3, 3
14	11.15	3.2	0.0	68.0	50.0	W8X10	7
16	16.40	0.5	0.0	68.7	50.0	W8X10	6
17	19.69	107.4	0.0	132.1	50.0	W10X12	8, 2, 9
19	27.56	291.1	0.0	346.8	50.0	W16X26	13, 2, 15
21	12.14	33.4	0.0	52.5	50.0	W10X12	
22	12.14	33.5	0.0	52.5	50.0	W10X12	
24	16.40	0.5	0.0	68.7	50.0	W8X10	6
26	29.55	232.3	0.0	277.1	50.0	W16X26	8, 9
27	29.53	203.9	0.0	244.7	50.0	W14X22	25
28	27.56	309.9	0.0	368.7	50.0	W16X26	17, 2, 3, 1, 16
29	29.55	196.1	0.0	233.8	50.0	W14X22	20
30	29.53	183.7	0.0	220.2	50.0	W14X22	17
31	29.53	178.8	0.0	211.7	50.0	W12X19	28
33	11.15	0.2	0.0	68.2	50.0	W8X10	6
36	16.00	59.7	0.0	88.9	50.0	W8X10	12
37	19.09	86.1	0.0	114.5	50.0	W12X14	8
38	19.09	70.2	0.0	84.6	50.0	W8X10	15
39	26.41	334.1	0.0	401.6	50.0	W16X31	13, 3, 6, 1, 9
41	29.53	185.1	0.0	220.2	50.0	W14X22	16
42	29.53	180.7	0.0	220.2	50.0	W14X22	17
43	21.98	309.7	0.0	367.5	50.0	W16X26	40
44	29.55	190.3	0.0	227.4	50.0	W14X22	18
45	29.53	203.9	0.0	244.7	50.0	W14X22	25

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
46	26.41	379.0	0.0	449.7	50.0	W16X31	21, 2, 3, 2, 24
47	29.55	190.3	0.0	227.4	50.0	W14X22	18
48	29.53	180.7	0.0	220.2	50.0	W14X22	17
49	29.53	185.1	0.0	220.2	50.0	W14X22	16
51	27.56	403.2	0.0	478.2	50.0	W18X35	16, 2, 16
53	29.53	185.1	0.0	220.2	50.0	W14X22	16
54	29.53	185.1	0.0	220.2	50.0	W14X22	16
55	19.69	265.6	0.0	312.9	50.0	W16X26	20
56	29.53	190.2	0.0	227.4	50.0	W14X22	18
57	29.53	195.2	0.0	233.8	50.0	W14X22	20
58	27.56	403.2	0.0	478.2	50.0	W18X35	16, 2, 16
59	29.53	190.2	0.0	227.4	50.0	W14X22	18
60	29.53	185.1	0.0	220.2	50.0	W14X22	16
61	29.53	185.1	0.0	220.2	50.0	W14X22	16
63	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
65	29.53	185.1	0.0	220.2	50.0	W14X22	16
66	29.53	185.1	0.0	220.2	50.0	W14X22	16
67	19.69	339.7	0.0	400.9	50.0	W16X31	34
68	29.53	249.2	0.0	384.0	50.0	W16X26	46
69	29.53	311.7	0.0	421.9	50.0	W16X31	42
70	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
71	29.53	249.2	0.0	384.0	50.0	W16X26	46
72	29.53	185.1	0.0	220.2	50.0	W14X22	16
73	29.53	185.1	0.0	220.2	50.0	W14X22	16
75	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
77	29.53	185.1	0.0	220.2	50.0	W14X22	16
78	29.53	185.1	0.0	220.2	50.0	W14X22	16
79	19.69	339.7	0.0	400.9	50.0	W16X31	34
80	29.53	190.2	0.0	227.4	50.0	W14X22	18
81	29.53	195.2	0.0	233.8	50.0	W14X22	20
82	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
83	29.53	190.2	0.0	227.4	50.0	W14X22	18
84	29.53	185.1	0.0	220.2	50.0	W14X22	16
85	29.53	185.1	0.0	220.2	50.0	W14X22	16
87	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
89	29.53	185.1	0.0	220.2	50.0	W14X22	16
90	29.53	185.1	0.0	220.2	50.0	W14X22	16
91	19.69	261.7	0.0	312.9	50.0	W16X26	20
92	29.53	190.2	0.0	227.4	50.0	W14X22	18
93	29.53	195.2	0.0	233.8	50.0	W14X22	20
94	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
95	29.53	190.2	0.0	227.4	50.0	W14X22	18
96	29.53	185.1	0.0	220.2	50.0	W14X22	16
97	29.53	185.1	0.0	220.2	50.0	W14X22	16
99	27.56	403.2	0.0	478.2	50.0	W18X35	16, 2, 16

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
101	29.53	185.1	0.0	220.2	50.0	W14X22	16
102	29.53	180.7	0.0	220.2	50.0	W14X22	17
104	29.55	190.3	0.0	227.4	50.0	W14X22	18
105	29.53	199.5	0.0	239.5	50.0	W14X22	23
106	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
107	29.53	190.2	0.0	227.4	50.0	W14X22	18
108	29.53	185.1	0.0	220.2	50.0	W14X22	16
109	29.53	185.1	0.0	220.2	50.0	W14X22	16
111	18.91	43.3	0.0	70.8	50.0	W8X18 2	
112	38.95	125.8	0.0	251.7	50.0	W10X49	
113	4.12	1.3	0.0	37.0	50.0	W8X10 2	
114	39.59	2.8	0.0	37.0	50.0	W8X10	
115	21.28	35.4	0.0	108.3	50.0	W10X22 2	
116	36.86	112.5	0.0	204.2	50.0	W8X48 2	
117	26.41	353.3	0.0	423.2	50.0	W16X31	16, 4, 6, 2, 12
119	26.25	111.8	-177.1	402.5	50.0	W18X35	24
	8.67	0.0	-177.1				
120	26.25	133.6	0.0	159.1	50.0	W12X14	23
121	20.83	265.2	0.0	313.4	50.0	W16X26	10, 1, 10
122	26.27	144.0	0.0	168.7	50.0	W12X14	26
123	26.25	161.5	0.0	191.0	50.0	W12X19	21
126	17.72	206.7	0.0	247.6	50.0	W14X22	10, 10
127	26.25	151.6	0.0	183.0	50.0	W12X16	26
128	26.25	145.5	0.0	173.3	50.0	W12X16	22
130	11.15	0.2	0.0	68.2	50.0	W8X10	6
131	14.87	118.8	0.0	138.3	50.0	W14X22	
133	19.69	208.7	0.0	248.5	50.0	W14X22	10, 1, 10
135	19.03	87.9	0.0	114.5	50.0	W12X14	8
137	19.03	87.9	0.0	114.5	50.0	W12X14	8
138	17.72	176.3	0.0	210.3	50.0	W12X19	22
139	19.03	83.6	0.0	114.5	50.0	W12X14	8
140	19.03	79.2	0.0	114.5	50.0	W12X14	8
142	19.03	56.8	0.0	70.8	50.0	W8X18 2	
143	47.74	259.1	0.0	388.6	50.0	W16X31	32
144	13.13	18.6	0.0	68.1	50.0	W8X10	6
145	6.89	6.8	0.0	37.0	50.0	W8X10	
146	6.89	7.2	0.0	37.0	50.0	W8X10	
147	6.89	7.2	0.0	37.0	50.0	W8X10	
148	6.89	7.2	0.0	37.0	50.0	W8X10	
149	6.88	7.3	0.0	37.0	50.0	W8X10	
150	6.89	7.2	0.0	37.0	50.0	W8X10	
151	6.89	7.2	0.0	37.0	50.0	W8X10	
152	6.89	7.1	0.0	37.0	50.0	W8X10	
153	11.15	3.2	0.0	68.0	50.0	W8X10	7
154	48.83	307.0	0.0	461.8	50.0	W18X35	28

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
46	29.53	203.9	0.0	244.7	50.0	W14X22	25
47	26.41	379.0	0.0	449.7	50.0	W16X31	21, 2, 3, 2, 24
48	29.55	190.3	0.0	227.4	50.0	W14X22	18
49	29.53	180.7	0.0	220.2	50.0	W14X22	17
50	29.53	185.1	0.0	220.2	50.0	W14X22	16
52	27.56	403.2	0.0	478.2	50.0	W18X35	16, 2, 16
54	29.53	185.1	0.0	220.2	50.0	W14X22	16
55	29.53	185.1	0.0	220.2	50.0	W14X22	16
56	19.69	265.6	0.0	312.9	50.0	W16X26	20
57	29.53	190.2	0.0	227.4	50.0	W14X22	18
58	29.53	195.2	0.0	233.8	50.0	W14X22	20
59	27.56	403.2	0.0	478.2	50.0	W18X35	16, 2, 16
60	29.53	190.2	0.0	227.4	50.0	W14X22	18
61	29.53	185.1	0.0	220.2	50.0	W14X22	16
62	29.53	185.1	0.0	220.2	50.0	W14X22	16
64	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
66	29.53	185.1	0.0	220.2	50.0	W14X22	16
67	29.53	185.1	0.0	220.2	50.0	W14X22	16
68	19.69	339.7	0.0	400.9	50.0	W16X31	34
69	29.53	249.2	0.0	384.0	50.0	W16X26	46
70	29.53	311.7	0.0	421.9	50.0	W16X31	42
71	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
72	29.53	249.2	0.0	384.0	50.0	W16X26	46
73	29.53	185.1	0.0	220.2	50.0	W14X22	16
74	29.53	185.1	0.0	220.2	50.0	W14X22	16
76	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
78	29.53	185.1	0.0	220.2	50.0	W14X22	16
79	29.53	185.1	0.0	220.2	50.0	W14X22	16
80	19.69	339.7	0.0	400.9	50.0	W16X31	34
81	29.53	190.2	0.0	227.4	50.0	W14X22	18
82	29.53	195.2	0.0	233.8	50.0	W14X22	20
83	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
84	29.53	190.2	0.0	227.4	50.0	W14X22	18
85	29.53	185.1	0.0	220.2	50.0	W14X22	16
86	29.53	185.1	0.0	220.2	50.0	W14X22	16
88	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
90	29.53	185.1	0.0	220.2	50.0	W14X22	16
91	29.53	185.1	0.0	220.2	50.0	W14X22	16
92	19.69	261.7	0.0	312.9	50.0	W16X26	20
93	29.53	190.2	0.0	227.4	50.0	W14X22	18
94	29.53	195.2	0.0	233.8	50.0	W14X22	20
95	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
96	29.53	190.2	0.0	227.4	50.0	W14X22	18
97	29.53	185.1	0.0	220.2	50.0	W14X22	16
98	29.53	185.1	0.0	220.2	50.0	W14X22	16

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
100	27.56	403.2	0.0	478.2	50.0	W18X35	16, 2, 16
102	29.53	185.1	0.0	220.2	50.0	W14X22	16
103	29.53	180.7	0.0	220.2	50.0	W14X22	17
105	29.55	190.3	0.0	227.4	50.0	W14X22	18
106	29.53	199.5	0.0	239.5	50.0	W14X22	23
107	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
108	29.53	190.2	0.0	227.4	50.0	W14X22	18
109	29.53	185.1	0.0	220.2	50.0	W14X22	16
110	29.53	185.1	0.0	220.2	50.0	W14X22	16
112	18.91	43.3	0.0	70.8	50.0	W8X18 2	
113	38.95	125.8	0.0	251.7	50.0	W10X49	
114	4.12	1.3	0.0	37.0	50.0	W8X10 2	
115	39.59	2.8	0.0	37.0	50.0	W8X10	
116	21.28	35.4	0.0	108.3	50.0	W10X22 2	
117	36.86	112.5	0.0	204.2	50.0	W8X48 2	
118	26.41	353.3	0.0	423.2	50.0	W16X31	16, 4, 6, 2, 12
120	26.25	111.8	-177.1	402.5	50.0	W18X35	24
	8.67	0.0	-177.1				
121	26.25	133.6	0.0	159.1	50.0	W12X14	23
122	20.83	265.2	0.0	313.4	50.0	W16X26	10, 1, 10
123	26.27	144.0	0.0	168.7	50.0	W12X14	26
124	26.25	161.5	0.0	191.0	50.0	W12X19	21
127	17.72	206.7	0.0	247.6	50.0	W14X22	10, 10
128	26.25	151.6	0.0	183.0	50.0	W12X16	26
129	26.25	145.5	0.0	173.3	50.0	W12X16	22
131	11.15	0.2	0.0	68.2	50.0	W8X10	6
132	14.87	118.8	0.0	138.3	50.0	W14X22	
135	19.69	208.7	0.0	248.5	50.0	W14X22	10, 1, 10
137	19.03	87.9	0.0	114.5	50.0	W12X14	8
139	19.03	87.9	0.0	114.5	50.0	W12X14	8
140	17.72	176.3	0.0	210.3	50.0	W12X19	22
141	19.03	83.6	0.0	114.5	50.0	W12X14	8
142	19.03	79.2	0.0	114.5	50.0	W12X14	8
144	19.03	56.8	0.0	70.8	50.0	W8X18 2	
145	47.74	259.1	0.0	388.6	50.0	W16X31	32
146	13.13	18.6	0.0	68.1	50.0	W8X10	6
147	6.89	6.8	0.0	37.0	50.0	W8X10	
148	6.89	7.2	0.0	37.0	50.0	W8X10	
149	6.89	7.2	0.0	37.0	50.0	W8X10	
150	6.89	7.2	0.0	37.0	50.0	W8X10	
151	6.88	7.3	0.0	37.0	50.0	W8X10	
152	6.89	7.2	0.0	37.0	50.0	W8X10	
153	6.89	7.2	0.0	37.0	50.0	W8X10	
154	6.89	7.1	0.0	37.0	50.0	W8X10	
155	11.15	3.2	0.0	68.0	50.0	W8X10	7

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
156	48.83	307.0	0.0	461.8	50.0	W18X35	28
157	12.04	25.4	0.0	68.0	50.0	W8X10	3, 3
158	3.35	1.7	0.0	37.0	50.0	W8X10	
159	3.35	1.7	0.0	37.0	50.0	W8X10	
160	3.35	1.7	0.0	37.0	50.0	W8X10	
161	3.35	1.7	0.0	37.0	50.0	W8X10	
162	3.35	1.7	0.0	37.0	50.0	W8X10	
163	3.35	1.7	0.0	37.0	50.0	W8X10	
164	3.35	1.7	0.0	37.0	50.0	W8X10	
165	16.40	43.3	0.0	68.2	50.0	W8X10	3, 3
166	10.35	22.1	0.0	61.8	50.0	W8X10	6
167	49.36	332.6	0.0	646.9	50.0	W21X50	23

Floor Type: 2nd

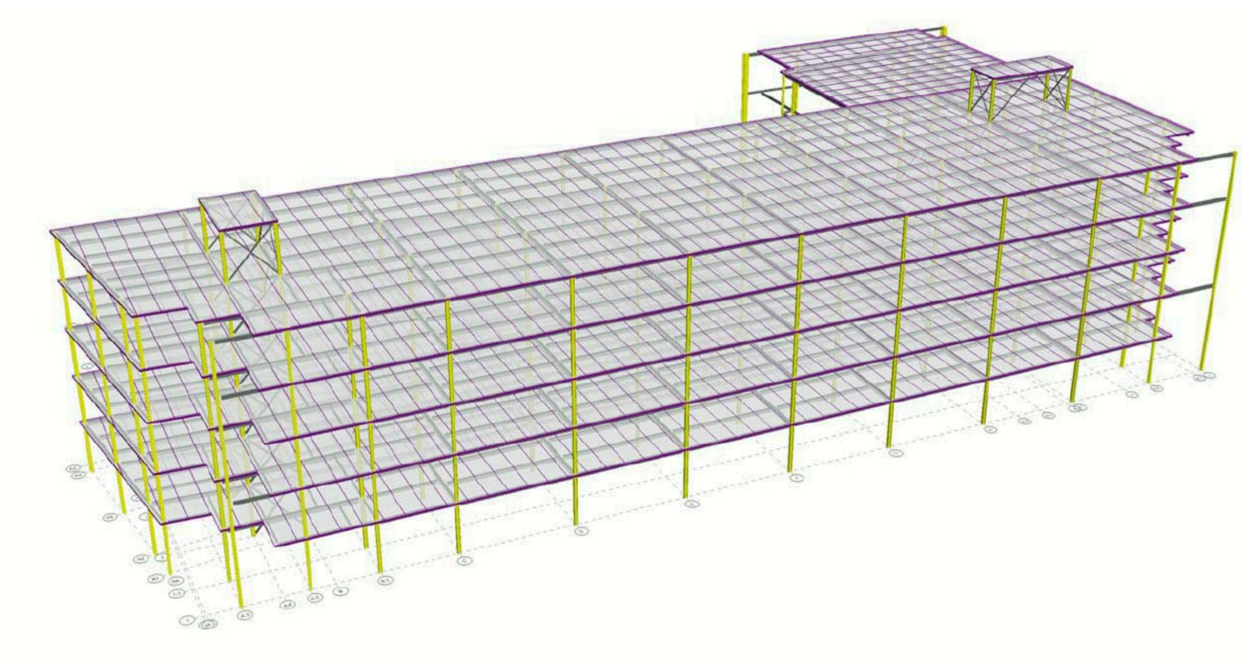
Bm #	Length ft	+Mu kip-ft	-Mu kip-ft	Mn kip-ft	Fy ksi	Beam Size	Studs
3	28.53	172.0	0.0	212.4	50.0	W14X22	15
4	27.05	147.7	0.0	178.4	50.0	W12X16	25
6	26.36	132.6	0.0	159.1	50.0	W12X14	23
7	24.58	107.8	0.0	128.8	50.0	W10X12	23
8	23.55	85.8	0.0	103.3	50.0	W10X12	13
10	22.74	143.8	0.0	211.9	50.0	W14X22	7, 7
11	11.59	27.3	0.0	61.8	50.0	W8X10	6
12	17.22	51.9	0.0	68.3	50.0	W8X10	3, 3
14	11.15	3.2	0.0	68.0	50.0	W8X10	7
16	16.40	0.5	0.0	68.7	50.0	W8X10	6
17	19.69	107.4	0.0	132.1	50.0	W10X12	8, 2, 9
19	27.56	291.1	0.0	346.8	50.0	W16X26	13, 2, 15
21	12.14	33.4	0.0	52.5	50.0	W10X12	
22	12.14	33.5	0.0	52.5	50.0	W10X12	
24	16.40	0.5	0.0	68.7	50.0	W8X10	6
26	29.55	232.3	0.0	277.1	50.0	W16X26	8, 9
27	29.53	203.9	0.0	244.7	50.0	W14X22	25
28	27.56	309.9	0.0	368.7	50.0	W16X26	17, 2, 3, 1, 16
29	29.55	196.1	0.0	233.8	50.0	W14X22	20
30	29.53	183.7	0.0	220.2	50.0	W14X22	17
31	29.53	178.8	0.0	211.7	50.0	W12X19	28
33	11.15	0.2	0.0	68.2	50.0	W8X10	6
36	16.00	59.7	0.0	88.9	50.0	W8X10	12
37	19.09	86.1	0.0	114.5	50.0	W12X14	8
38	19.09	70.2	0.0	84.6	50.0	W8X10	15
39	26.41	334.1	0.0	401.6	50.0	W16X31	13, 3, 6, 1, 9
41	29.53	185.1	0.0	220.2	50.0	W14X22	16
42	29.53	180.7	0.0	220.2	50.0	W14X22	17
43	21.98	309.7	0.0	367.5	50.0	W16X26	40

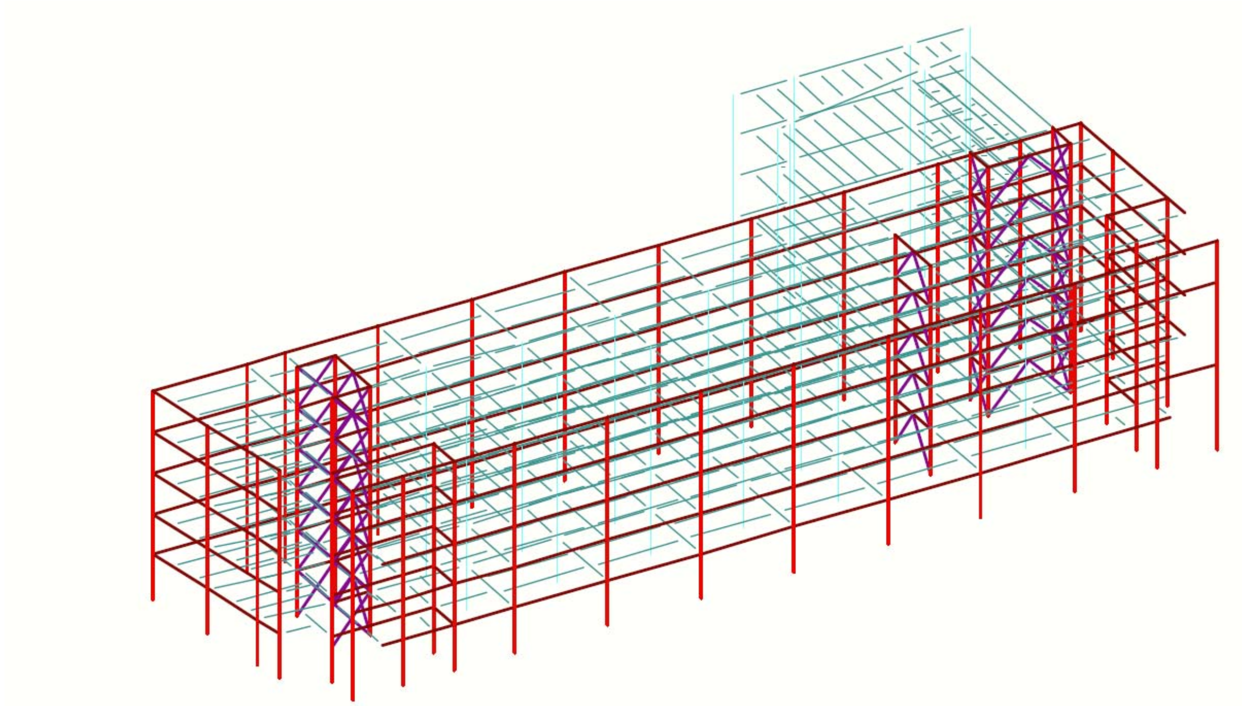
Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
44	29.55	190.3	0.0	227.4	50.0	W14X22	18
45	29.53	203.9	0.0	244.7	50.0	W14X22	25
46	26.41	379.0	0.0	449.7	50.0	W16X31	21, 2, 3, 2, 24
47	29.55	190.3	0.0	227.4	50.0	W14X22	18
48	29.53	180.7	0.0	220.2	50.0	W14X22	17
49	29.53	185.1	0.0	220.2	50.0	W14X22	16
51	27.56	403.2	0.0	478.2	50.0	W18X35	16, 2, 16
53	29.53	185.1	0.0	220.2	50.0	W14X22	16
54	29.53	185.1	0.0	220.2	50.0	W14X22	16
55	19.69	265.6	0.0	312.9	50.0	W16X26	20
56	29.53	190.2	0.0	227.4	50.0	W14X22	18
57	29.53	195.2	0.0	233.8	50.0	W14X22	20
58	27.56	403.2	0.0	478.2	50.0	W18X35	16, 2, 16
59	29.53	190.2	0.0	227.4	50.0	W14X22	18
60	29.53	185.1	0.0	220.2	50.0	W14X22	16
61	29.53	185.1	0.0	220.2	50.0	W14X22	16
63	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
65	29.53	185.1	0.0	220.2	50.0	W14X22	16
66	29.53	185.1	0.0	220.2	50.0	W14X22	16
67	19.69	339.7	0.0	400.9	50.0	W16X31	34
68	29.53	249.2	0.0	384.0	50.0	W16X26	46
69	29.53	311.7	0.0	421.9	50.0	W16X31	42
70	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
71	29.53	249.2	0.0	384.0	50.0	W16X26	46
72	29.53	185.1	0.0	220.2	50.0	W14X22	16
73	29.53	185.1	0.0	220.2	50.0	W14X22	16
75	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
77	29.53	185.1	0.0	220.2	50.0	W14X22	16
78	29.53	185.1	0.0	220.2	50.0	W14X22	16
79	19.69	339.7	0.0	400.9	50.0	W16X31	34
80	29.53	190.2	0.0	227.4	50.0	W14X22	18
81	29.53	195.2	0.0	233.8	50.0	W14X22	20
82	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
83	29.53	190.2	0.0	227.4	50.0	W14X22	18
84	29.53	185.1	0.0	220.2	50.0	W14X22	16
85	29.53	185.1	0.0	220.2	50.0	W14X22	16
87	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
89	29.53	185.1	0.0	220.2	50.0	W14X22	16
90	29.53	185.1	0.0	220.2	50.0	W14X22	16
91	19.69	261.7	0.0	312.9	50.0	W16X26	20
92	29.53	190.2	0.0	227.4	50.0	W14X22	18
93	29.53	195.2	0.0	233.8	50.0	W14X22	20
94	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
95	29.53	190.2	0.0	227.4	50.0	W14X22	18
96	29.53	185.1	0.0	220.2	50.0	W14X22	16

Bm #	Length	+Mu	Mu	Mn	Fy	Beam Size	Studs
97	29.53	185.1	0.0	220.2	50.0	W14X22	16
99	27.56	403.2	0.0	478.2	50.0	W18X35	16, 2, 16
101	29.53	185.1	0.0	220.2	50.0	W14X22	16
102	29.53	180.7	0.0	220.2	50.0	W14X22	17
104	29.55	190.3	0.0	227.4	50.0	W14X22	18
105	29.53	199.5	0.0	239.5	50.0	W14X22	23
106	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
107	29.53	190.2	0.0	227.4	50.0	W14X22	18
108	29.53	185.1	0.0	220.2	50.0	W14X22	16
109	29.53	185.1	0.0	220.2	50.0	W14X22	16
111	18.91	43.3	0.0	70.8	50.0	W8X18 2	
112	38.95	125.8	0.0	251.7	50.0	W10X49	
113	4.12	1.3	0.0	37.0	50.0	W8X10 2	
114	39.59	2.8	0.0	37.0	50.0	W8X10	
115	21.28	35.4	0.0	108.3	50.0	W10X22 2	
116	36.86	112.5	0.0	204.2	50.0	W8X48 2	
117	26.41	355.3	0.0	423.2	50.0	W16X31	16, 4, 7, 2, 12
119	26.25	115.4	-151.8	329.9	50.0	W16X31	22
	8.67	0.0	-151.8				
120	26.25	133.6	0.0	159.1	50.0	W12X14	23
121	20.83	265.2	0.0	313.4	50.0	W16X26	10, 1, 10
122	26.27	144.0	0.0	168.7	50.0	W12X14	26
123	26.25	161.5	0.0	191.0	50.0	W12X19	21
126	17.72	206.7	0.0	247.6	50.0	W14X22	10, 10
127	26.25	151.6	0.0	183.0	50.0	W12X16	26
128	26.25	145.5	0.0	173.3	50.0	W12X16	22
130	11.15	0.2	0.0	68.2	50.0	W8X10	6
131	14.87	118.8	0.0	138.3	50.0	W14X22	
133	8.67	15.2	0.0	37.0	50.0	W8X10	
134	19.69	167.8	0.0	199.4	50.0	W12X19	9, 1, 9
136	8.67	18.2	0.0	37.0	50.0	W8X10	
138	8.67	18.2	0.0	37.0	50.0	W8X10	
139	17.72	136.0	0.0	161.4	50.0	W12X14	18
141	8.67	16.4	0.0	37.0	50.0	W8X10	
143	19.03	56.8	0.0	70.8	50.0	W8X18 2	
144	47.74	259.1	0.0	388.6	50.0	W16X31	32
145	13.13	18.6	0.0	68.1	50.0	W8X10	6
146	6.89	6.8	0.0	37.0	50.0	W8X10	
147	6.89	7.2	0.0	37.0	50.0	W8X10	
148	6.89	7.2	0.0	37.0	50.0	W8X10	
149	6.89	7.2	0.0	37.0	50.0	W8X10	
150	6.88	7.3	0.0	37.0	50.0	W8X10	
151	6.89	7.2	0.0	37.0	50.0	W8X10	
152	6.89	7.2	0.0	37.0	50.0	W8X10	
153	6.89	7.1	0.0	37.0	50.0	W8X10	

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
154	11.15	3.2	0.0	68.0	50.0	W8X10	7
155	48.83	307.0	0.0	461.8	50.0	W18X35	28
156	12.04	25.4	0.0	68.0	50.0	W8X10	3, 3
157	3.35	1.7	0.0	37.0	50.0	W8X10	
158	3.35	1.7	0.0	37.0	50.0	W8X10	
159	3.35	1.7	0.0	37.0	50.0	W8X10	
160	3.35	1.7	0.0	37.0	50.0	W8X10	
161	3.35	1.7	0.0	37.0	50.0	W8X10	
162	3.35	1.7	0.0	37.0	50.0	W8X10	
163	3.35	1.7	0.0	37.0	50.0	W8X10	
164	16.40	52.8	0.0	68.3	50.0	W8X10	6
165	29.53	161.7	0.0	216.4	50.0	W12X19	24
166	17.72	61.6	0.0	89.3	50.0	W10X12	6
167	49.36	332.6	0.0	646.9	50.0	W21X50	23

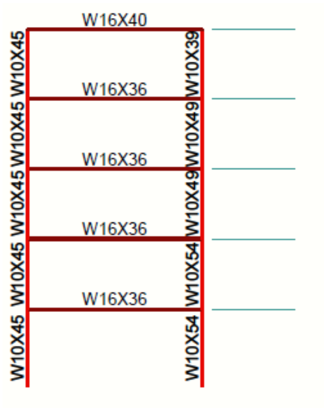
Appendix D: Lateral Framing



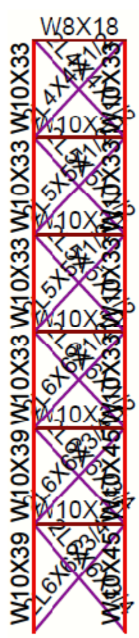


Moment Frame along Grid 1

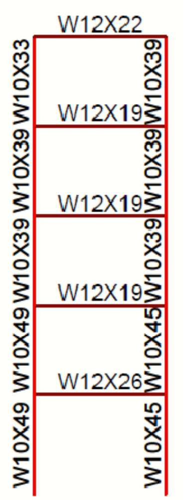
Moment Frame along Grid 1.3



Braced Frame along Grid 2

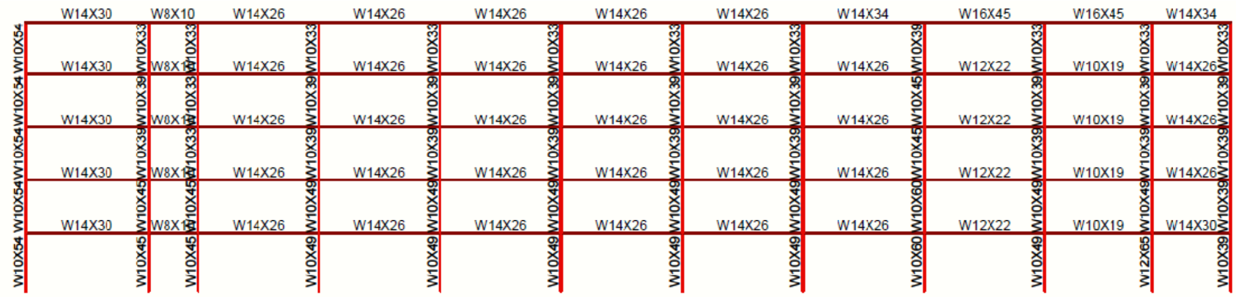


Moment Frame along Grid 2

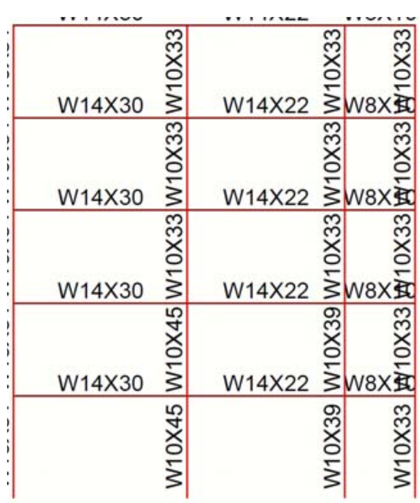


Braced Frames along Grid 3

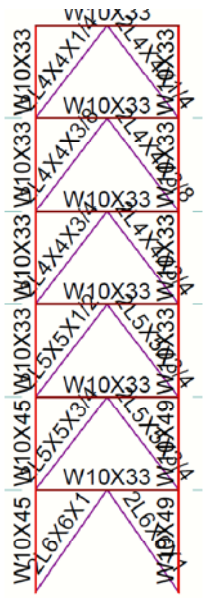
Moment Frame along Grid 4



Moment Frame along Grid RA

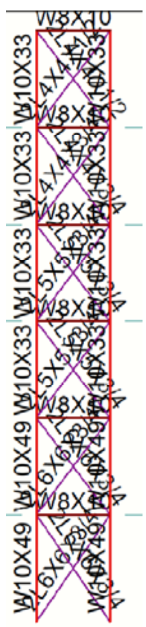


Braced Frame along Grid B

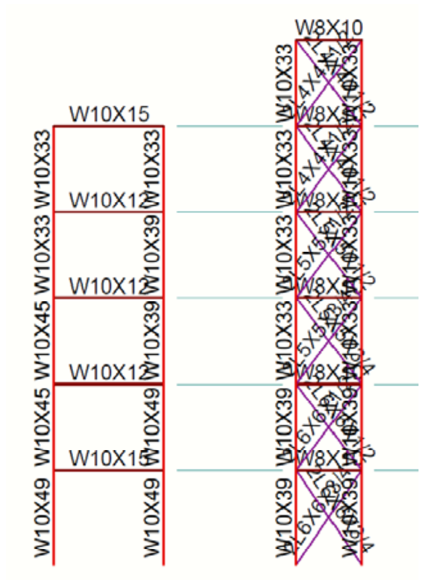


Brace Frame along Grid H

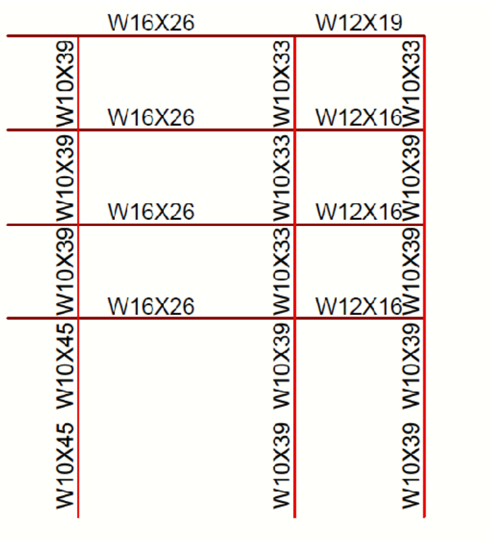
Brace Frame along Grid J



Moment Frame and Braced Frame along Grid K



Moment Frame along Grid L



Period of Vibration from RAM

FREQUENCIES AND PERIODS:

Mode	Period sec	Frequency Hz	Frequency rad/sec
1	1.4798	0.6757	4.2458
2	1.3521	0.7396	4.6470
3	1.1173	0.8950	5.6234
4	0.6283	1.5915	10.0000
5	0.6283	1.5915	10.0000
6	0.3930	2.5444	15.9869
7	0.3816	2.6207	16.4666

Center of Rigidity from RAM

Level	Diaph. #	Centers of Rigidity Xr ft	Centers of Rigidity Yr ft	Centers of Mass Xm ft	Centers of Mass Ym ft
Penthouse	1	38.62	37.64	24.12	37.41
Penthouse	2	238.65	44.44	249.95	52.17
Roof	1	122.97	40.40	154.37	50.00
Floor 5	1	126.11	40.10	142.64	41.10
Floor 4	1	134.75	40.00	142.66	41.10
Floor 3	1	146.66	39.83	142.65	41.10
Floor 2	1	166.28	40.03	138.90	41.01

Appendix E: Design Checks

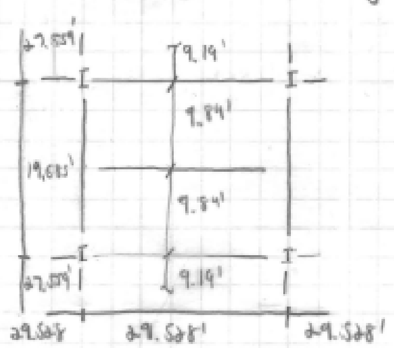
Gravity Loads

Live Loads:
- Office Live Load (Partions) : 80 psf
- Typical Roof : 20 psf
- Mechanical Room : 150 psf
- Pedestrian Bridge : 60 psf
- High Density Filing : 250 psf

Dead Loads:
- Super imposed Dead Load : 15 psf
- Roofing System : 40 psf
- Exterior Curtain Wall : 20 psf
- Atrium Curtain Wall : 20 psf
- Mechanical Unit : 150 psf

Snow Loads : 20 psf

Gravity System Design : Composite steel on metal deck



Given : LL = 80 psf
SDL : 15 psf
DL = Self weight

Assume
Normal weight concrete
use: United Steel Deck to determine deck size, and slab thickness

- By United Steel Deck determine deck size and slab thickness

<u>Deck Properties</u>
use 18 Gage Deck with the following properties 2" Lok-Floor

t = .0474 in	I : .560 in⁴	R_b : 1680 lb per 12" width
w : 2.4 psf	Sp : .523 in³	ØVn : 3180 lb per 12" width
As : .710 in²	Sn : .529 in³	Studs : .57 # studs per ft

<u>Composite Properties</u> - For 18 gage deck with 4 1/2" slab depth
Slab depth : 4.5 in
ØMn : 52.08 in-k
w : 42 psf

Max unshored spans
2 span : 10.48 ft

Uniform Live Loads (Including self weight) : 225 psf

* Exceeds required loading ∴ ok

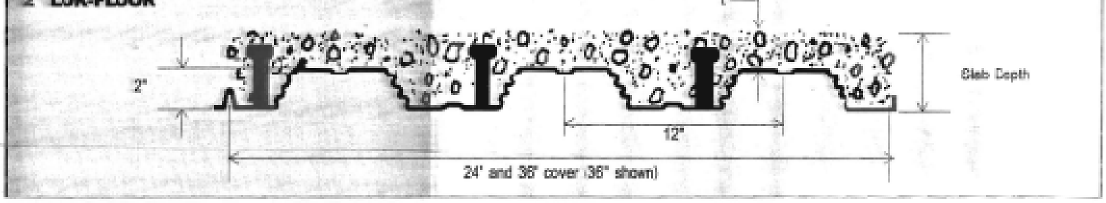
- Use: 18 gage 2" Lok-Floor Deck
with 4.5 in Normal Weight concrete
slab depth

2 x 12" DECK F_y = 33ksi f'_c = 3 ksi 145 pcf concrete

USD United Steel Deck, Inc.

LRFD

2" LOK-FLOOR



The **Deck Section Properties** are per foot of width. The I value is for postive bending (in.⁴); t is the gage thickness in inches; w is the weight in pounds per square foot; S_p and S_n are the section moduli for positive and negative bending (in.³); R_b and ϕV_n are the interior reaction and the shear in pounds (per foot of width); studs is the number of studs required per foot in order to obtain the full resisting moment, ϕM_{nf}.

DECK PROPERTIES									
Gage	t	w	As	I	S_p	S_n	R_b	ϕV_n	studs
22	0.0295	1.5	0.440	0.338	0.284	0.302	714	1990	0.36
20	0.0358	1.8	0.540	0.420	0.367	0.387	1010	2410	0.43
19	0.0418	2.1	0.630	0.490	0.445	0.458	1330	2810	0.51
18	0.0474	2.4	0.710	0.560	0.523	0.529	1680	3190	0.57
16	0.0598	3.1	0.900	0.700	0.654	0.654	2470	3890	0.72

The **Composite Properties** are a list of values for the composite slab. The **slab depth** is the distance from the bottom of the steel deck to the top of the slab in inches as shown on the sketch. U.L. ratings generally refer to the cover over the top of the deck so it is important to be aware of the difference in names. ϕM_{nf} is the factored resisting moment provided by the composite slab when the "full" number of studs as shown in the upper table are in place; inch kips (per foot of width). A_c is the area of concrete available to resist shear, in.² per foot of width. **Vol.** is the volume of concrete in ft.³ per ft.² needed to make up the slab; no allowance for frame or deck deflection is included. **W** is the concrete weight in pounds per ft.². S_c is the section modulus of the "cracked" concrete composite slab; in.³ per foot of width. I_{av} is the average of the "cracked" and "uncracked" moments of inertia of the transformed composite slab; in.⁴ per foot of width. The I_{av} transformed section analysis is based on steel; therefore, to calculate deflections the appropriate modulus of elasticity to use is 29.5 x 10⁶ psi. ϕM_{no} is the factored resisting moment of the composite slab if there are no studs on the beams (the deck is attached to the beams or walls on which it is resting) inch kips (per foot of width). ϕV_{nt} is the factored vertical shear resistance of the composite system; it is the sum of the shear resistances of the steel deck and the concrete but is not allowed to exceed $\phi 4(f'_c)^{1/2} A_c$; pounds (per foot of width). The next three columns list the **maximum unshored spans** in feet; these values are obtained by using the construction loading requirements of the SDI; combined bending and shear, deflection, and interior reactions are considered in calculating these values. A_{mesh} is the minimum area of welded wire fabric recommended for temperature reinforcing in the composite slab; square inches per foot.

COMPOSITE PROPERTIES													
Gage	Slab Depth	ϕM_{nf} in.k	A_c in²	Vol. ft³/ft²	W psf	S_c in³	I_{av} in⁴	ϕM_{no} in.k	ϕV_{nt} lbs.	Max. unshored spans, ft. 1span	2span	3span	A_{mesh}
22 gage	4.50	4027	326	0.292	42	1.05	5.9	29.40	5030	5.82	7.83	7.92	0.023
	5.00	4644	37.5	0.333	48	1.23	8.0	34.53	5480	5.54	7.47	7.56	0.027
	5.25	4953	40.0	0.354	51	1.32	9.2	37.16	5720	5.41	7.31	7.38	0.029
	5.50	5261	42.6	0.375	54	1.42	10.5	39.81	5960	5.30	7.16	7.24	0.032
	6.00	5878	48.0	0.417	60	1.61	13.5	45.21	6460	5.09	6.89	6.97	0.036
	6.25	6187	50.8	0.438	63	1.71	15.3	47.95	6720	5.03	6.76	6.84	0.038
	6.50	6495	53.6	0.458	66	1.81	17.1	50.70	6980	4.97	6.65	6.72	0.041
	7.00	7112	59.5	0.500	73	2.01	21.2	56.26	7530	4.85	6.43	6.51	0.045
	7.25	7421	61.9	0.521	76	2.11	23.5	59.07	7750	4.79	6.32	6.41	0.047
	7.50	7729	64.3	0.542	79	2.21	26.0	61.88	7970	4.74	6.22	6.31	0.050
20 gage	4.50	4860	326	0.292	42	1.26	6.3	35.43	5450	6.81	8.97	9.27	0.023
	5.00	5618	37.5	0.333	48	1.48	8.6	41.65	5900	6.47	8.55	8.83	0.027
	5.25	5996	40.0	0.354	51	1.60	9.8	44.84	6140	6.33	8.36	8.63	0.029
	5.50	6375	42.6	0.375	54	1.71	11.3	48.07	6380	6.18	8.18	8.45	0.032
	6.00	7132	48.0	0.417	60	1.95	14.5	54.63	6880	5.94	7.85	8.11	0.036
	6.25	7511	50.8	0.438	63	2.07	16.3	57.96	7140	5.86	7.70	7.95	0.038
	6.50	7890	53.6	0.458	66	2.19	18.2	61.31	7400	5.79	7.56	7.80	0.041
	7.00	8647	59.5	0.500	73	2.43	22.6	68.09	7950	5.65	7.29	7.53	0.045
	7.25	9026	61.9	0.521	76	2.55	25.0	71.50	8170	5.58	7.17	7.41	0.047
	7.50	9405	64.3	0.542	79	2.67	27.6	74.93	8390	5.52	7.05	7.28	0.050
19 gage	4.50	5585	32.6	0.292	42	1.45	6.7	40.69	5850	7.65	9.76	10.08	0.023
	5.00	6488	37.5	0.333	48	1.71	9.0	47.87	6300	7.26	9.30	9.61	0.027
	5.25	6930	40.3	0.354	51	1.84	10.4	51.56	6540	7.09	9.09	9.39	0.029
	5.50	7352	42.5	0.375	54	1.97	11.9	55.30	6780	6.93	8.90	9.19	0.032
	6.00	8235	48.3	0.417	60	2.24	15.2	62.90	7280	6.65	8.54	8.83	0.036
	6.25	8677	50.3	0.438	63	2.38	17.1	66.70	7540	6.56	8.38	8.66	0.038
	6.50	91.19	53.5	0.458	66	2.52	19.2	70.65	7800	6.48	8.23	8.50	0.041
	7.00	10003	59.5	0.500	73	2.80	23.8	78.50	8350	6.32	7.94	8.20	0.045
	7.25	10444	61.3	0.521	76	2.94	26.3	82.46	8570	6.24	7.81	8.07	0.047
	7.50	10886	64.3	0.542	79	3.08	29.0	86.45	8790	6.17	7.68	7.94	0.050
18 gage	4.50	62.18	32.5	0.292	42	1.62	7.0	45.34	6080	8.42	10.48	10.83	0.023
	5.00	72.04	37.5	0.333	48	1.90	9.5	53.36	6670	7.98	9.99	10.32	0.027
	5.25	77.02	40.0	0.354	51	2.05	10.9	57.48	6910	7.79	9.77	10.10	0.029
	5.50	82.00	42.6	0.375	54	2.20	12.4	61.66	7150	7.61	9.56	9.88	0.032
	6.00	91.15	48.0	0.417	60	2.50	15.9	70.18	7650	7.30	9.18	9.48	0.036
	6.25	96.93	50.4	0.438	63	2.66	17.9	74.50	7910	7.20	9.01	9.31	0.038
	6.50	101.91	53.6	0.458	66	2.81	20.0	78.85	8170	7.11	8.85	9.14	0.041
	7.00	111.87	59.5	0.500	73	3.13	24.8	87.66	8720	6.93	8.54	8.82	0.045
	7.25	116.85	61.9	0.521	76	3.28	27.4	92.10	8940	6.85	8.40	8.68	0.047
	7.50	121.83	64.3	0.542	79	3.44	30.2	96.57	9160	6.77	8.26	8.54	0.050
16 gage	4.50	62.88	32.6	0.292	42	1.99	7.7	45.34	6080	9.58	11.63	12.02	0.023
	5.00	72.84	37.3	0.333	48	2.35	10.4	53.36	6960	9.06	11.10	11.47	0.027
	5.25	77.82	40.8	0.354	51	2.53	11.9	57.48	7450	8.85	10.85	11.22	0.029
	5.50	82.80	42.6	0.375	54	2.72	13.6	61.66	7940	8.65	10.63	10.98	0.032
	6.00	91.95	48.0	0.417	60	3.10	17.4	70.18	8460	8.29	10.21	10.55	0.036
	6.25	96.93	50.8	0.438	63	3.29	19.5	74.50	8720	8.17	10.02	10.35	0.038
	6.50	101.91	53.6	0.458	66	3.48	21.8	78.85	8980	8.07	9.84	10.17	0.041
	7.00	111.37	59.5	0.500	73	3.88	27.8	87.66	9530	7.86	9.50	9.83	0.045
	7.25	116.85	61.9	0.521	76	4.08	29.8	92.10	9750	7.77	9.35	9.66	0.047
	7.50	121.33	64.3	0.542	79	4.28	32.8	96.57	9970	7.67	9.20	9.50	0.050

2" LOK-FLOOR

2 x 12" DECK F_y = 33ksi f'_c = 3 ksi 145 pcf concrete

LRFD

L_s Uniform Live Service Loads, psf *

Gage	Slab Depth	φMn in.k	6.00	6.50	7.00	7.50	8.00	8.50	9.00	9.50	10.00	10.50	11.00	11.50	12.00
22 gage	4.50	40.27	400	365	310	265	230	200	175	155	135	120	105	95	85
	5.00	46.44	400	400	360	305	265	230	204	175	155	140	125	110	95
	5.50	52.61	400	400	400	350	300	260	230	205	175	155	140	125	110
	6.00	58.78	400	400	400	390	335	295	255	221	200	175	155	140	125
	6.50	64.95	400	400	400	400	370	325	286	250	220	195	175	155	135
	7.00	71.12	400	400	400	400	400	355	318	275	240	215	190	170	150
	7.25	74.21	400	400	400	400	400	373	325	285	250	225	200	175	155
	7.50	77.29	400	400	400	400	400	395	348	295	260	230	205	185	165
20 gage	4.50	48.60	400	400	380	325	285	245	215	190	170	150	135	120	110
	5.00	56.18	400	400	400	380	330	285	250	220	195	175	155	140	125
	5.50	63.75	400	400	400	400	375	325	285	250	225	200	175	160	140
	6.00	71.32	400	400	400	400	400	365	320	285	250	225	200	180	160
	6.50	78.90	400	400	400	400	400	400	355	315	280	245	220	195	175
	7.00	86.47	400	400	400	400	400	400	390	345	305	270	240	215	195
	7.25	90.26	400	400	400	400	400	400	400	360	320	285	255	225	205
	7.50	94.05	400	400	400	400	400	400	400	375	330	295	265	235	210
19 gage	4.50	55.60	400	400	400	350	320	270	230	220	200	180	160	140	130
	5.00	64.68	400	400	400	400	365	335	295	260	230	205	185	165	150
	5.50	73.52	400	400	400	400	400	380	335	295	265	235	210	190	170
	6.00	82.35	400	400	400	400	400	400	375	335	295	265	235	215	190
	6.50	91.19	400	400	400	400	400	400	400	370	330	295	265	235	210
	7.00	100.03	400	400	400	400	400	400	400	400	360	320	290	260	235
	7.25	104.44	400	400	400	400	400	400	400	400	375	335	300	270	245
	7.50	108.86	400	400	400	400	400	400	400	400	395	350	315	280	255
18 gage	4.50	62.09	400	400	400	400	370	325	280	255	225	200	180	160	145
	5.00	72.04	400	400	400	400	400	375	335	295	260	235	210	190	170
	5.50	82.00	400	400	400	400	400	400	380	335	300	265	240	215	195
	6.00	91.95	400	400	400	400	400	400	400	375	335	300	270	245	220
	6.50	101.91	400	400	400	400	400	400	400	400	375	335	300	270	245
	7.00	111.87	400	400	400	400	400	400	400	400	400	365	330	295	270
	7.25	116.85	400	400	400	400	400	400	400	400	400	385	345	310	280
	7.50	121.83	400	400	400	400	400	400	400	400	400	400	360	325	290
16 gage	4.50	52.08	400	400	400	400	370	325	285	255	225	200	180	160	145
	5.00	72.04	400	400	400	400	400	375	335	295	260	235	210	190	170
	5.50	82.00	400	400	400	400	400	400	380	335	300	265	240	215	195
	6.00	91.95	400	400	400	400	400	400	400	375	335	300	270	245	220
	6.50	101.91	400	400	400	400	400	400	400	400	375	335	300	270	245
	7.00	111.87	400	400	400	400	400	400	400	400	400	365	330	295	270
	7.25	116.85	400	400	400	400	400	400	400	400	400	385	345	310	280
	7.50	121.83	400	400	400	400	400	400	400	400	400	400	360	325	290
22 gage	4.50	29.40	305	255	215	185	160	135	120	105	90	80	70	60	50
	5.00	34.53	360	300	250	210	180	160	140	120	105	90	80	70	60
	5.50	39.81	400	350	295	255	215	190	160	140	125	110	95	85	75
	6.00	45.21	400	400	340	290	250	215	185	160	140	125	110	95	85
	6.50	50.70	400	400	380	325	280	240	210	185	160	140	125	110	95
	7.00	56.26	400	400	400	360	310	270	235	205	180	155	140	120	105
	7.25	59.07	400	400	400	380	325	285	245	215	190	165	145	130	115
	7.50	61.88	400	400	400	400	345	295	260	225	200	175	155	135	120
20 gage	4.50	35.43	375	315	270	230	200	170	150	130	115	100	90	80	70
	5.00	41.65	400	375	315	270	235	205	175	155	135	120	105	95	85
	5.50	48.07	400	400	365	315	270	235	205	180	160	140	125	110	95
	6.00	54.63	400	400	400	360	310	270	235	205	180	160	140	125	110
	6.50	61.31	400	400	400	400	350	300	265	230	205	180	160	140	125
	7.00	68.09	400	400	400	400	390	335	295	260	230	200	180	160	140
	7.25	71.50	400	400	400	400	400	355	310	270	240	210	190	165	150
	7.50	74.93	400	400	400	400	400	370	325	285	250	225	200	175	155
19 gage	4.50	40.69	400	370	315	270	230	200	175	155	135	120	105	95	85
	5.00	47.87	400	400	370	315	275	240	210	185	160	145	125	115	100
	5.50	55.30	400	400	400	365	320	275	240	215	190	165	150	130	120
	6.00	62.90	400	400	400	400	360	315	275	245	215	190	170	150	135
	6.50	70.65	400	400	400	400	400	352	310	275	245	215	190	170	155
	7.00	78.50	400	400	400	400	400	395	350	305	270	240	215	190	170
	7.25	82.46	400	400	400	400	400	400	365	320	285	255	225	200	180
	7.50	86.45	400	400	400	400	400	400	385	340	300	265	235	210	190
18 gage	4.50	45.34	400	400	350	300	260	230	200	175	155	140	125	110	100
	5.00	53.36	400	400	400	355	310	270	235	210	185	165	145	130	115
	5.50	61.66	400	400	400	400	360	315	275	240	215	190	170	150	135
	6.00	70.18	400	400	400	400	400	360	315	275	245	220	195	175	155
	6.50	78.85	400	400	400	400	400	400	355	310	275	245	220	195	175
	7.00	87.66	400	400	400	400	400	400	395	350	310	275	245	220	195
	7.25	92.10	400	400	400	400	400	400	400	365	325	290	260	230	210
	7.50	96.57	400	400	400	400	400	400	400	385	340	305	270	245	220
16 gage	4.50	45.34	400	400	350	300	260	230	200	175	155	140	125	110	100
	5.00	53.36	400	400	400	355	310	270	235	210	185	165	145	130	115
	5.50	61.66	400	400	400	400	360	315	275	240	215	190	170	150	135
	6.00	70.18	400	400	400	400	400	360	315	275	245	220	195	175	155
	6.50	78.85	400	400	400	400	400	400	355	310	275	245	220	195	175
	7.00	87.66	400	400	400	400	400	400	395	350	310	275	245	220	195
	7.25	92.10	400	400	400	400	400	400	400	365	325	290	260	230	210
	7.50	96.57	400	400	400	400	400	400	400	385	340	305	270	245	220

☐ 1 STUD/FT.

☐ NO STUDS

* The Uniform Live Loads are based on the LRFD equation $\phi M_n = (1.6L + 1.2D)l^2/8$. Although there are other load combinations that may require investigation, this will control most of the time. The equation assumes there is no negative bending reinforcement over the beams and therefore each composite slab is a single span. Two sets of values are shown; ϕM_n is used to calculate the uniform load when the full required number of studs is present; ϕM_{no} is used to calculate the load when no studs are present. A straight line interpolation can be done if the average number of studs is between zero and the required number needed to develop the "full" factored moment. The tabulated loads are checked for shear controlling (it seldom does), and also limited to a live load deflection of 1/360 of the span.

An upper limit of 400 psf has been applied to the tabulated loads. This has been done to guard against equating large concentrated to uniform loads. Concentrated loads may require special analysis and design to take care of serviceability requirements not covered by simply using a uniform load value. On the other hand, for any load combination the values provided by the composite properties can be used in the calculations.

Welded wire fabric in the required amount is assumed for the table values. If welded wire fabric is not present, deduct 10% from the listed loads.

Refer to the example problems for the use of the tables.

2" LOK-FLOOR

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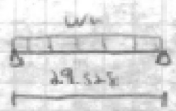
Gravity System Design

Typical Beam Design



Tributary width: 9.84'

Loads: DL: 44.4 psf (not including beams, increase by 10%)

DL: 48.84 psf
SDL: 15 psf
LL: 80 psf (Includes Partitions)

w_u: 1.2(48.84 + 15) + 1.6(80) = 204.61 psf (1.4D DNC)
w_u: 204.61(9.84)/1000 = 2.01 k/ft

$M_u = \frac{wl^2}{8} = \frac{2.01(29.528)^2}{8} = 219.43$ ft-k

$V_u = \frac{wl}{2} = \frac{2.01(29.528)}{2} = 29.7^k$

Assume a = 1.5 in ∴ y_2 = 4.5 − 1.5/2 = 3.75 in

Try W14x22 → ϕM_p = 125 ft-k
ϕM_n = 230 ft-k — use y_2 = 3.5 to be conservative
y_1 = 3"
ΣQ_n = 241 k
studs = $\frac{\Sigma Q_n}{Q_n} = \frac{241}{17.2}$ = 14.01 = 15 studs
across entire beam = 30 studs

Equiv weight: 29.528(22) + 30(10 lb/stud) = 949.6 lb = .95 k

$M_u = \frac{(.95/29.528)(29.528)^2}{8}$ = 3.5 ft-k

M_u = 219.43 + 3.5 = 222.9 ft-k < 230 ft-k ok

Check assumption
b_{eff} = 9.84 ≥ $\frac{29.528}{4}$ = 7.382

b_{eff} = 7.382(12) = 88.58

$a = \frac{\Sigma Q_n}{.85 f'_c\, b_{eff}} = \frac{241}{.85(3)(88.58)}$ = 1.07 in

a = 1.07 < 1.5 in conservative

Gravity System Design

Beam 1 cont

- check Load during construction

DL = 48.84 ; 48.84(9.84)/1000 = .437 k/ft

SW = .75/29.508 = .032 k/ft

$M_c = \frac{wl^2}{8} = \frac{(.437+.032)\,29.508^2}{8} = 51.12^{k} < \phi M_p = 125^{k}$ ok

- check deflection

$D_{cons} = \frac{5wl^4}{384EI}$ $w_D = .044(9.84) = .433$ k/ft, $w_{sw} = .032$ k/ft

$D_{cons} = \frac{5(.433+.032)(29.508)^4\,1728}{384(29000)(199)} = 1.38$ in

limit $l/240 = \frac{29.508(12)}{240} = 1.48$ in

$D_{cons} = 1.38 < 1.48$ ok

$D_{LL} = \frac{5wl^4}{384EI} = \frac{5(.08)(9.84)(29.508)^4\,1728}{384(29000)(496)} = .935$ in

limit $\frac{l}{360} = \frac{29.508(12)}{360} = .98$

$D_{LL} = .936 < .98$ ok

- Check shear

$V_u = 29.7^{k}$

$\phi V_n = 94.8$

$V_u = 29.7^{k} < \phi V_n = 94.8^{k}$ ok

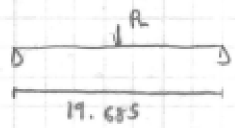
USE W14 x 22 (30) for int. Beams

d = 13.7 in

Gravity System Design

Girder Design

Reaction from Beam on one side
$R = V_u = 29.7^k$
$P_u = 2(29.7) = 59.4^k$



$M_u = \frac{Pl}{4} = \frac{59.4(19.685)}{4} = 292.3$ ft-k

$V_u = \frac{P}{2} = \frac{59.4}{2} = 29.7^k$

Assume a = 2 in ∴ $Y_2 = 4.5 - 2/2 = 3.5$ in

Try W16x26 $\phi M_p = 166$ ft-k
$\phi M_n = 301$ ft-k
$\Sigma Q_n = 289^k$
$Y_1 = 3$ in

By Table 3-21 Parallel Deck 3/4" stud $Q_n = 17.1^k$

$\#\text{studs} = \frac{\Sigma Q_n}{Q_n} = \frac{289}{17.7} = 16.9 = 17$

Total # studs = 34 studs
Beam weight = 19.685(26) + 34(10) = 851.8 lb = .852k

Check assumptions

$b_{eff} = 19.685/4 = 4.92 = 59.1$ in

$a = \frac{\Sigma Q_n}{.85 f'_c b_e} = \frac{289}{.85(3)(59.1)} = 1.92 < 2$ in ok

$P_D = [.059(9.84)(29.528) + .95]/2 = 9.05^k$

$P_L = .08(9.84)(29.528)/2 = 11.62$

Total $P_D = 2(9.05) = 18.09^k$
$P_L = 2(11.62) = 23.24^k$

Gravity System Design

Girder Design cont.

Check deflection
construct Loads

$$\Delta_{const} = \frac{Pl^3}{48EI} = \frac{18.08(19.685)^3(1728)}{48(29000)(301)} = .569 \text{ in}$$

$$\text{Limit} = l/360 = 19.685(12)/360 = .656 \text{ in}$$

$$\Delta_{const} = .569 < .656 \text{ ok}$$

Live Load Deflection

$$\Delta_{LL} = \frac{Pl^3}{48EI} = \frac{23.24(19.685)^3\,1728}{48(29,000)(1720)} = .303$$

$$\Delta_{LL} = l/360 = .656$$

$$\Delta_{LL} = .303 < .656 \text{ ok}$$

use W16×26 (34) for Girders
d = 15.7 in

Approx Equiv weight of floor system

Slab on metal Deck
42 psf
2.4 psf
total = 44.4 psf

Beams - 3 Beams

$$\frac{22\text{ plf }(29.528)}{29.528(19.685)} = 1.12 \text{ psf}$$

Girders - 2 Girders

$$\frac{26\text{ plf }(19.685)}{29.528(19.685)} = .88 \text{ psf}$$

Columns

$$\frac{100\text{ plf }(14.16)}{29.528(19.685)} = 2.44 \text{ psf}$$

Total = 44.4 + 1.12 + .88 + 2.44
= 48.84 psf

Gravity System Design

Column Design (Below 2nd Floor)
(Int colum)

Typical Floor - 4
Roof - 1

A_T = 697.51 sf

Loads: SDL = 15 psf
DL = 50 psf
RDL = 40 psf
LL = 80 psf
RLL = 20 psf
SL = 20 psf

Load Combination
1.4D - D.W.C.
* 1.2D + 1.6L + .5(Lr or S)
1.2D + 1.6(Lr or S) + (L) - check
1.2D + L + .5(Lr or S) - D.W.C.

Axial Load acting on column below 2nd floor

P_1 = 1.2(15+50) + 1.6(80) = 206 psf

P_2 = 1.2(40+15+50) + .5(20) = 131 psf

P_u = [4(206)(697.51) + 131(697.51)]/1000 = 666.1 k

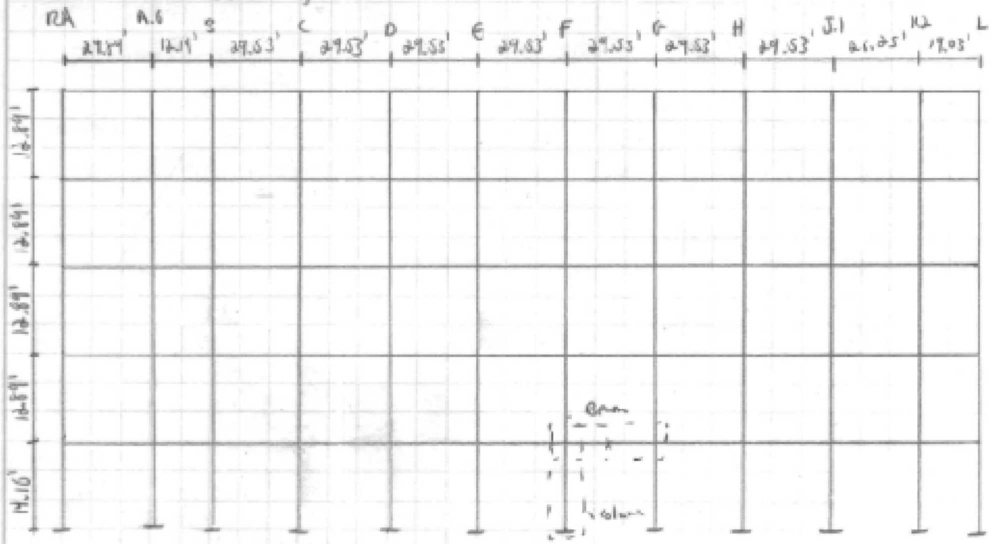
Assume K = 1 L_{eff} = 14.16'
By Table 4-1
W12x65 ϕP_n = 685 14'
ϕP_n = 662 15'
for 14.16' = 681.3 k

ϕP_n = 681.3 k > 666 k ok

Use W12x65

Lateral System Design / check

- Moment Frame along Grid 4
  - Design Controlled by 1.2D + 1.6L, not lateral load controlled, verified by RAM



FLL = 80 psf
SDL = 15 psf
DL = 50 psf
RLL = 20 psf
RDL = 40 psf
WDL = 425 plf

Trib width of beam = 4.593 ft
Trib area of column = 406.91 SF / floor

Design Beam

FLL = 80(4.593)/1000 = .3674 k/ft
FDL = 65(4.593)/1000 = .2985 k/ft
WDL = .425 k/ft

$w_u = 1.2(.2985 + .425) + 1.6(.3674) = 1.456$ k/ft

Fixed - Fixed $M_u = \frac{wl^2}{12} = \frac{1.456(29.53)^2}{12} = 105.8$ ft kips

Bracing at midspan - unbraced length = 14.77 ft

W14x40 $\phi M_p = 177^{1k}$ $L_p = 5.26$ ft
$\phi M_r = 110^{1k}$ $L_r = 14.9$ ft
$L_r = 14.9$ ft > 14.77 ft $\therefore \phi M_r = 110^{1k}$ is conservative for ϕM_n
$V_u = 21.5^k < \phi V_n = 112^k$

Use W14x30 for Lateral Beam in moment frame

Lateral System Design

Lateral Column

- Moments balance due to equal bays, load is primary axial

$[[1.2(65) + 1.6(80)]4 + [1.2(40+65) + .5(20)]]406.9/1000 = 361.37^k$

$1.2(.425)29.5(3)(5) = 75.3^k$

$P_u = 436.67^k$

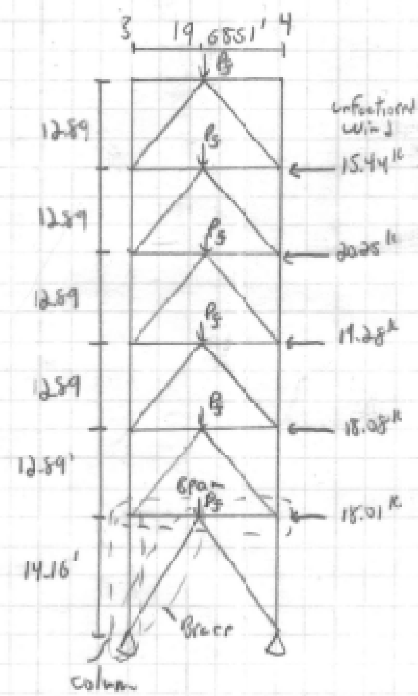
use W10x49 $\phi P_n = 450^k$ $L_p = 15'$

matches Ram Design

use W10x49 for lateral column

Lateral System Design

Braced Frame at Grid B



FLL: 80psf
FDL: 50psf
SDL: 15psf
RLL: 20psf
RDL = 40+FDL psf = 90psf
SL: 20psf

- P_f is reaction from floor beam
- Trib width of beam: $\left(\frac{5.9384}{2} + \frac{29.5275}{2}\right)$ = 17.73 ft
- Trib Area of column: $(17.733)\times\left(\frac{19.6851}{2}+\frac{27.559}{2}\right)$ = 418.59 sf
- From Etabs Model
  Brace takes 37% of wind Load in N-S Direction

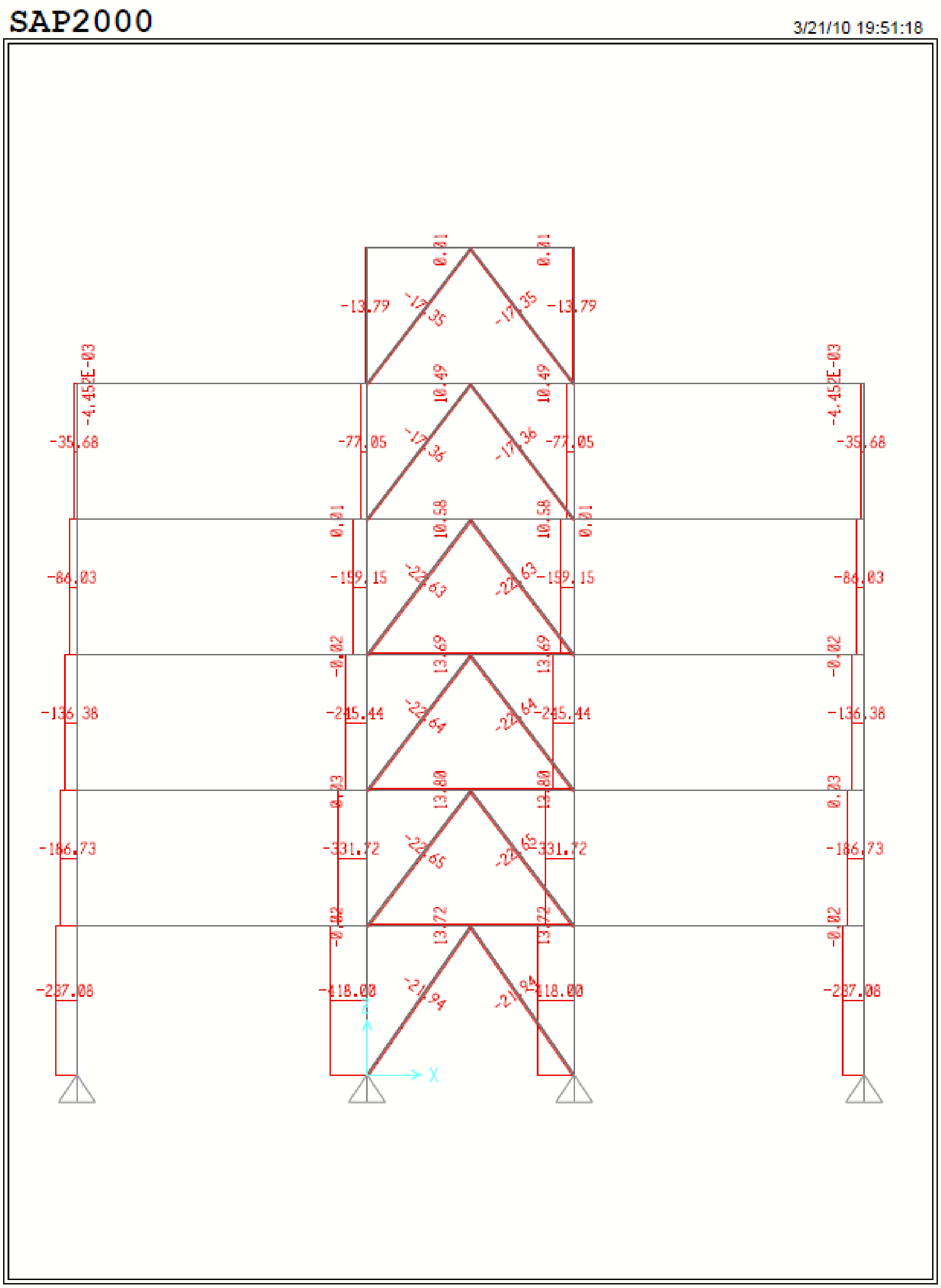
- Design Beam, Column, and Braces.
  - SAP 2000 2D Modeling was used to generate the axial forces in the braced frame.

  3 Load Combinations were looked at

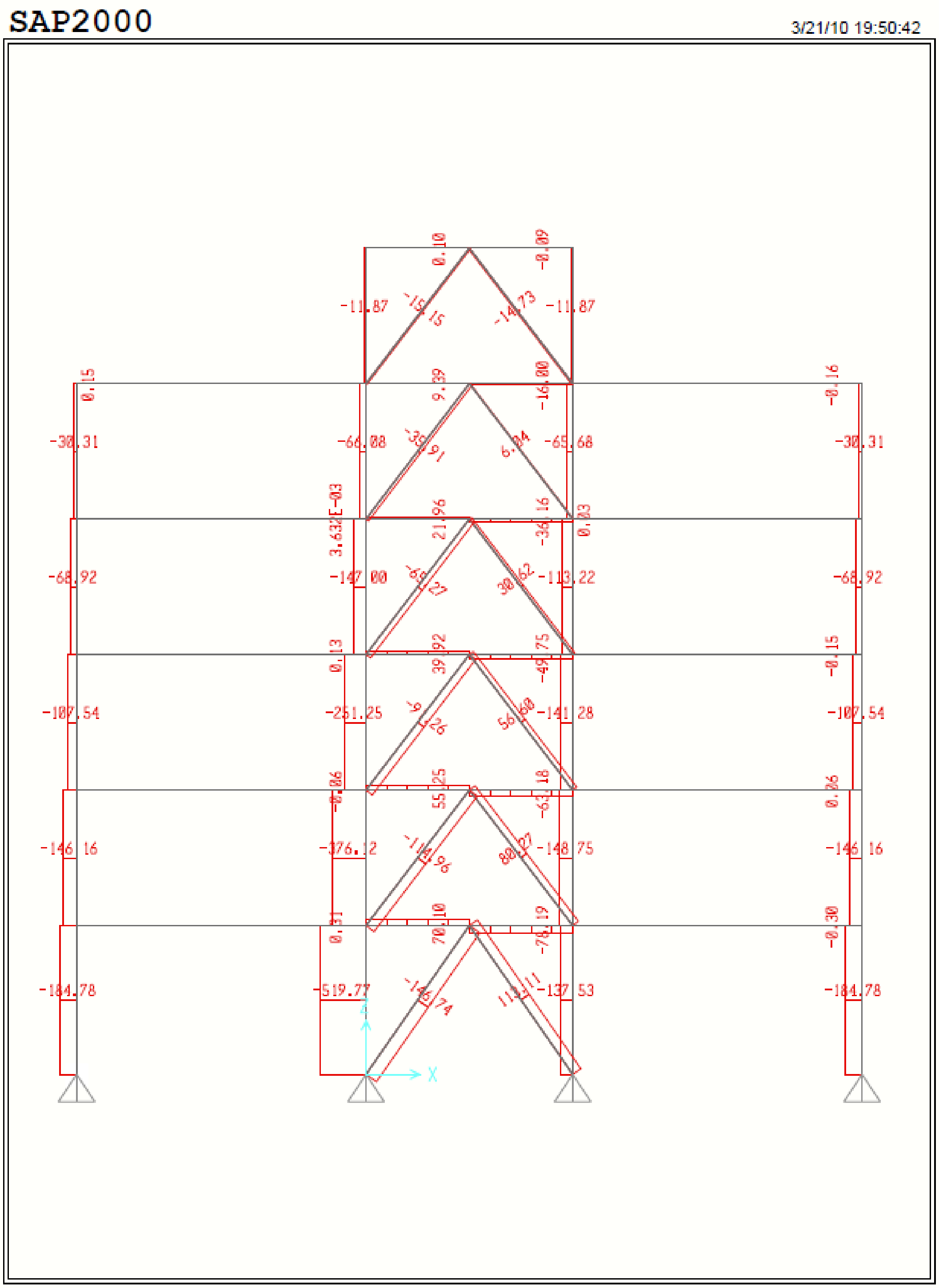
  1.2D + 1.6L + .5Lr

  1.2D + 1.6W + L + .5Lr

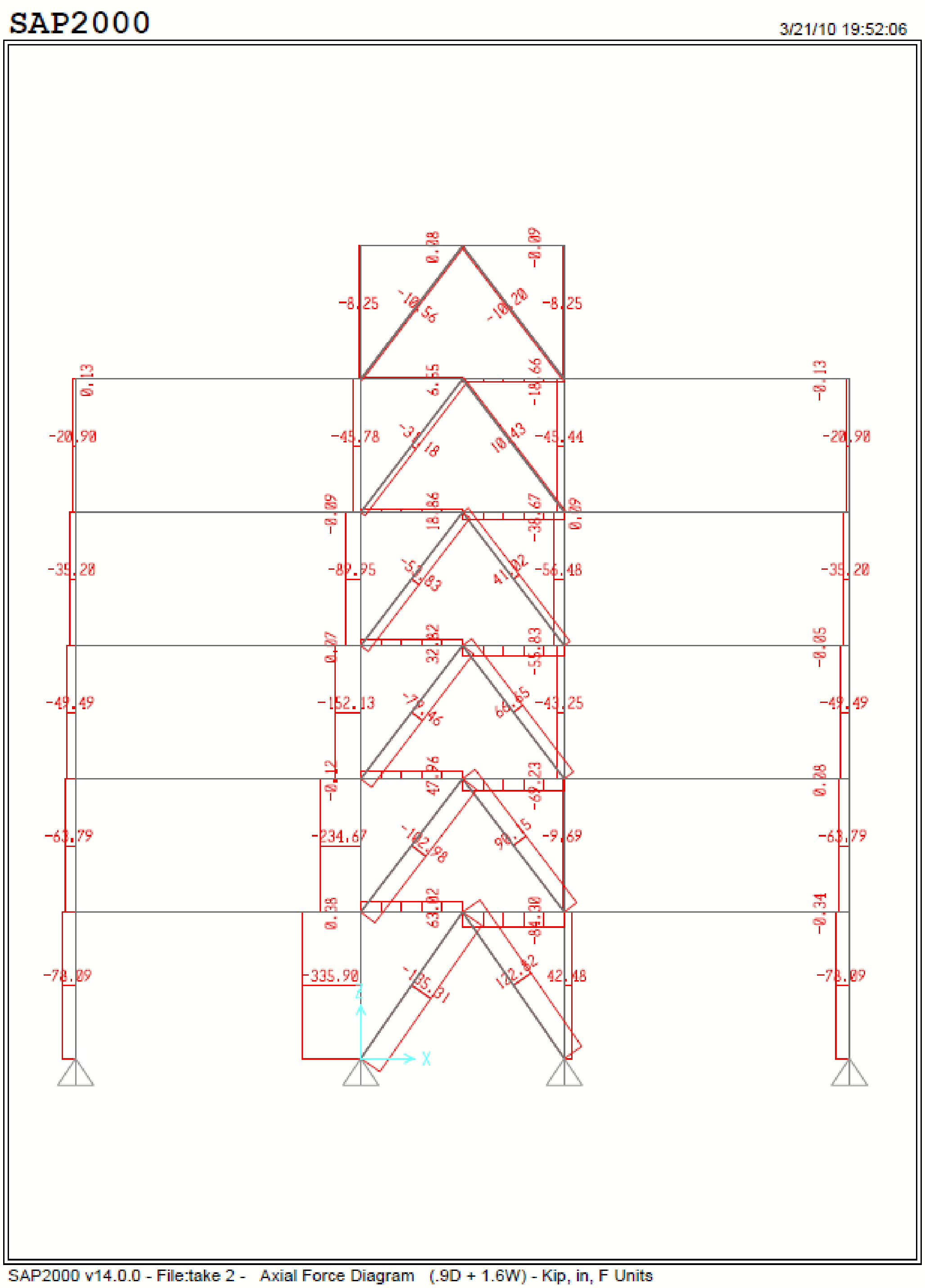
  .9D + 1.6W



SAP2000 v14.0.0 - File:take 2 - Axial Force Diagram (1.2D + 1.6L) - Kip, in, F Units



SAP2000 v14.0.0 - File:take 2 - Axial Force Diagram (1.2D +1.6W+ L .5Lr) - Kip, in, F Units


SAP2000
3/21/10 19:52:06
-8.25
-8.25
-45.78
-335.90
42.48
SAP2000 v14.0.0 - File:take 2 - Axial Force Diagram (.9D + 1.6W) - Kip, in, F Units

Lateral System Design

Braced Frame at Grid B

Use AISC 13th Table 4-1 - Axial Compression

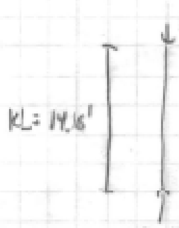
- Beam at 2nd floor

* The smallest W10 that meets slender ratio for compression is a W10x33 ϕP_n: 143 k > P_u: 78.19 k

Both $KL_x = KL_y$, so KL_y will control L_p.

Use W10x33

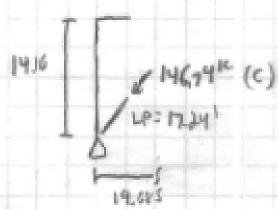
- Column below floor 2



Both $KL_x = KL_y$, so KL_y will control

Use W10x60 ϕP_n: 555 k L_p = 15' > 14.16' ok

- Brace below Floor 2



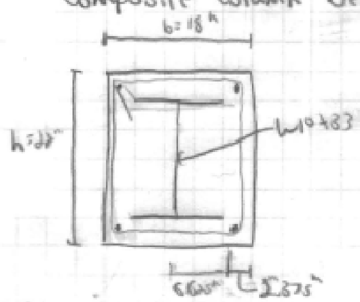
Both $KL_y = KL_x$ so KL_y will control design $KL = KL_y$: 12.24'

2L5x5x3/4" ϕP_n = 152 k L_r = 18' ok

* RAM Design use 2L6x6x3/4". Difference accounted for eccentric loading cases used in RAM that was not used in simple 2D model.

Gravity stystem Design

Composite Column Design



- Determine area

$A_s = 9.71\ in^2$
$A_{sr} = 4(1) = 4\ in^2$
$A_c = 18(22) - 9.71 - 4 = 382.29\ in^2$

- Check min steel

$\rho_s = \frac{A_s}{A_g} = \frac{9.71}{22(18)} = .0245 > .001$

$\rho_{sr} = \frac{4}{22(18)} = .0101 > .004$

- Determine P_o and P_e

$C_2 = .85$

$C_{4} = .5$

$C_3 = .1 + 2\left(\frac{9.71}{382.29+9.71}\right) = .1495 < .3$

$I_s = I_y = 36.6\ in^4$
$I_{sr} = 4(1.0)(6.25)^2 = 176\ in^4$

$I_c = \frac{22(18)^3}{12} - 36.6 - 176 = 10,479\ in^4$

$P_o = 9.71(50) + 4(60) + .85(5)(382.29) = 1113.6^k$

$EI_{eff} = 29,000(36.6) + .5(29,000)(176) + .15(3900)(10,479) = 9.74 \times 10^6\ in\text{-}kips$

Gravity System Design

- Composite Column Design cont

$$P_e = \frac{\pi^2 EI_{eff}}{(KL)^2} = \frac{\pi^2 (9.74 \times 10^6)}{(65.72 \times 12)^2} = 154.62 \text{ kips}$$

$$\frac{P_e}{P_o} = \frac{154.62}{1113.6} = .139 < .44$$

$$P_n = .877 P_e = .877(154.62) = 135.6 \text{ kips}$$

$$\phi P_n = .75(135.6) = 101.7 \text{ k}$$

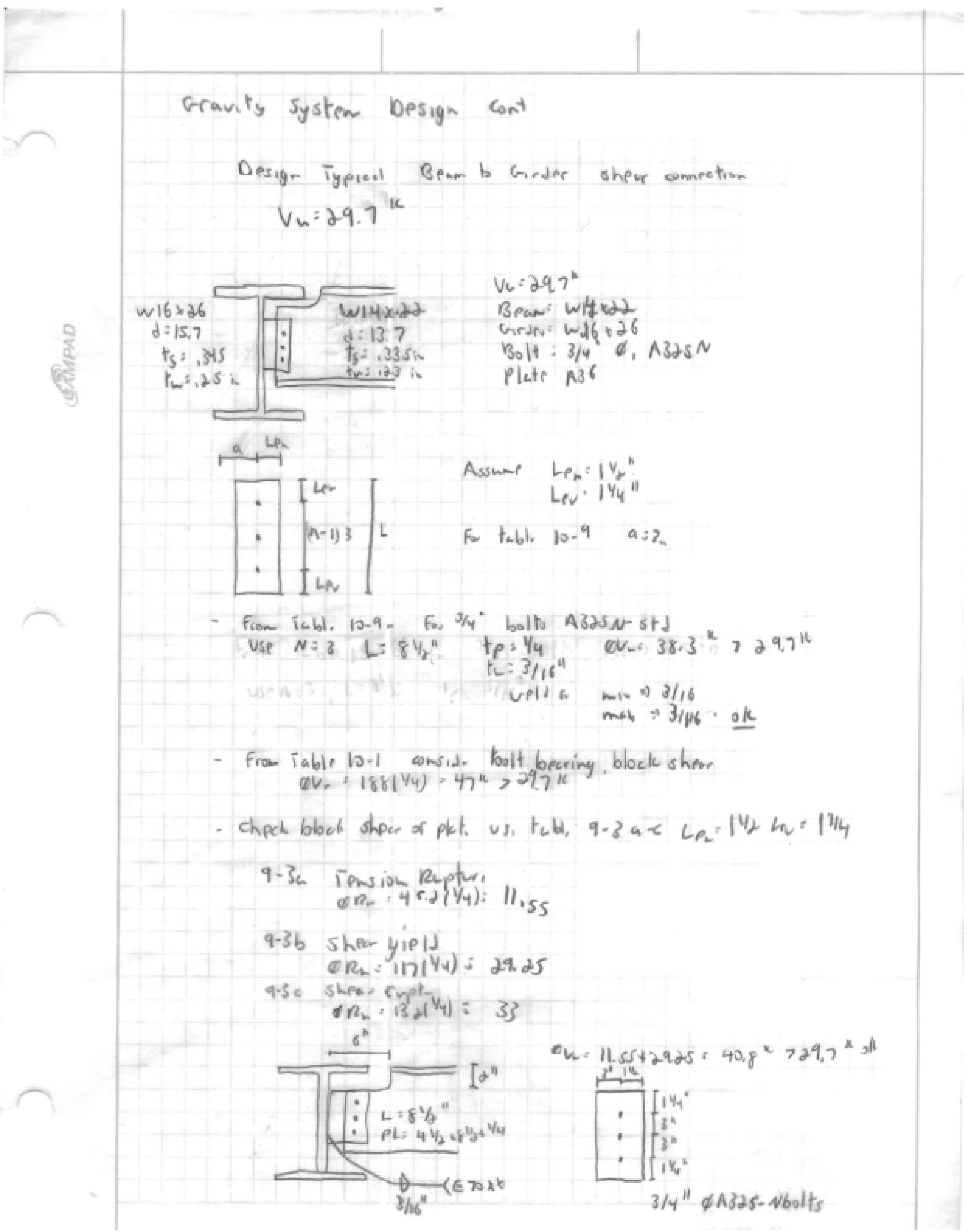
$$P_u = 1.4(65.72 \times 83) = 8036.3 \text{ lbs}$$

$$+ 1.4(15' \times 6 \times 6) = ?$$

$$P_u = 6.3 \text{ kips} \quad \underline{\text{ok}}$$

Appendix F: Connection Design

Typical Beam to Girder Connection

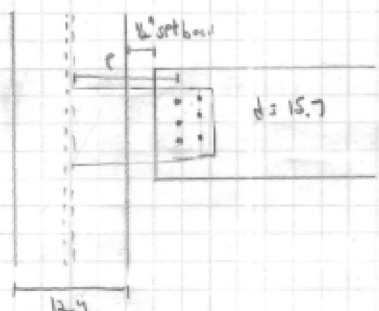


Typical Girder to Column Connection: Extended Shear Tab

Gravity System Design

Design Typical Girder to Column Connection (Shear
Extended Shear Tab Connection

$V_u = 30.2^k$ (includes self weight)



Given
V_u: 30.2^k
Beam: W16x26
Column: W12x79
Bolts: 3/4" Ø A325N
Plate: A36
$e: \frac{(12.4 - .47)}{2} + 1/2 + 1\,1/2 + 3/2 = 9.47"$
use $e = 9.5"$

- Required strength = V_u: 30.2^k
- Bolt A325 3/4" Ø bolt
  $ØR_n = C(ØR_n)$ $30.2 = C(15.9)$ $C: 1.9$

From Table 7-8 pg. 7-38
for $s = 3"$ $e_x = 8"$, 2 rows of 4 bolts $C = 2.54 > 1.9$ ok
$ØR_n = 2.54(15.9) = 40.4^k > 30.2^k$ ok

bearing on web
$Ør_n = .75(2.4)(65)(.25)(.875) = 25.59^k > 15.9^k$ Bolt shear controls

bearing on plate (Assume plate thickness of 3/8")
$Ør_n = .75(2.4)(58)(.375)(.875) = 34.3^k > 15.9^k$
$ØV_n = 46.6^k > 30.2^k$ ok

- Maximum plate thickness
  $1.25\,F_{nv} = 1.25(48) = 60$ ksi
  A_b: .601
  C': 26.0 Pg 7-38
  $M_n = 1.25\,F_{nv} A_b C' = 60(.601)(26) = 937.56$
  $t_{max} = \frac{6M_n}{F_y d^2} = \frac{6(937.6)}{36(11.5)^2} = 1.18" > .375"$ ok
- Shear yielding of plate
  $ØR_n = 1.0(.6F_y)A_g = 1.0(.6)(36)(11.5)(.375) = 93.15 > 30.2^k$ ok
- Shear rupture of
  $ØR_n = .75(.6F_u)A_{nv} = .75(.6(58))(.375)[11.5 - 4(3/4+1/8)] = 78.3^k$
- Block shear by Table 9-3 a-c L_{eh}: 1 1/4 L_{ev}: 1 1/4 n = 4
  TR = 46.2
  SY = 166
  SR 188
  $ØR_n = (46.2+166)(.375) = 79.6^k > 30.2^k$

Gravity System Design

Extended shear tab connection cont.

- check flexure using von mises shear reduction

$M_u = 30.2(9.5) = 286.9$ in-k

$f_v = \frac{30.2/.9}{9.5(.375)} = 9.42$ ksi

$F_{cr} = \sqrt{36^2 - 3(9.42)^2} = 32.09$ ksi

$\phi M_n = .9(32.1)\left(\frac{.375(11.5)^2}{4}\right) = 358.2 > 286.9$ in kip

- check plate flexure rupture

$Z_{net} = Z_p - Z_{holes}$

$Z_p = \frac{.375(11.5)^2}{4} = 12.4$ in^3

$Z_{ho} = .375[3/4 + 1/8][2(1.5) + 2(3 + 1.5)] = 3.94$ in^3

$\phi M_n = .75(58)(12.4 - 3.94) = 368.$ in-k > 286.9 in-k ok

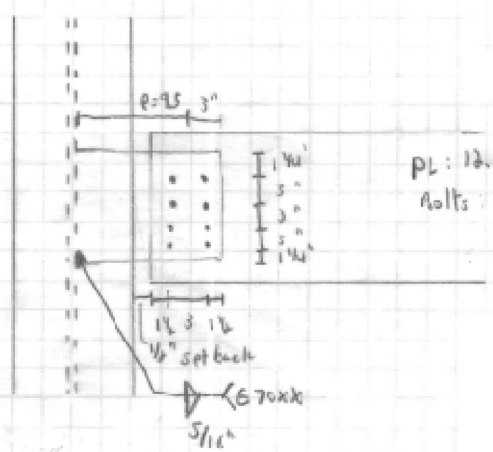
- check plate buckling

$F_{cr} = F_y Q$

$\lambda = \frac{h_o \sqrt{F_y}}{10 t_w \sqrt{475 + 280(h_o/c)^2}} = \frac{11.5\sqrt{36}}{10(.47)\sqrt{475 + 280\left(\frac{11.5}{8}\right)^2}} = .45$

$\lambda \leq .7 \simeq Q = 1$

$F_{cr} = F_y$ plate buckling O.K.



PL: 12.5" x 11.5" x 3/8"

Bolts: 2 Rows of 4 3/4"ø A325 N

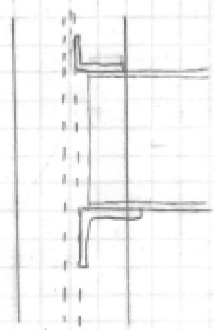
Typical Girder to Column Web Connection: Seated

Gravity System Design

Design Typical Girder to Column web Connection

Seated Connection

℄ Col. W



Given:
$V_u = 30.2^k$
Beam: W16x26
Column: W12x79
Bolts: 3/4" ∅ A325N

- Required strength: $V_u = 30.2^k$
- Bolts A325 3/4" ∅ bolts

- Constructability

T of Column: T = 9 1/8"
B_f of Beam: $B_f = 5.5"$ Beam fits inside column web.

- Try L6x6x1" ∙ 0'-5 1/2" A36

$R_n e = .9 F_y L t_a^2/4$ $e = \frac{N_{min}}{2} + 3/4 - t_a - 3/8 = \frac{N_{min}}{2} - 5/8$

$R_n = \frac{.9 f_y L t_a^2}{\left(\frac{N_{min}}{2} - 5/8\right)4} = \frac{.9(36)(5.5)(1)^2}{\left(\frac{N_m}{2} - 5/8\right)4} = \frac{44.55}{\left(\frac{N_m}{2} - 5/8\right)}$

① $N_{min} = \frac{R_u}{1 F_y t_w} - 2.5 k_d = \frac{R}{1(50)(.25)} - 2.5(.747)$

$N_{min} = \frac{44.55/\left(\frac{N}{2} - 5/8\right)}{1(50).25} - 2.5(.747) = 2.78$ in

② when $N/d \leq .2$

$N_m = \frac{d}{3}\left(\frac{t_f}{t_w}\right)^{1.5}\left[\frac{R_u}{.75(.4)t_w^2}\sqrt{\frac{t_w}{EF_y t_f}} - 1\right]$ $t_w = .25$ $d = 15.7$ in
$t_f = .345$

$\frac{15.7}{3}\left(\frac{.345}{.25}\right)^{1.5}\left[\frac{R_u}{.75(.4)(.25)^2}\sqrt{\frac{.25}{29{,}000(50)(.345)}} - 1\right] = 8.48[53.33 R_u(.000707) - 1]$

$N_{min} = .333\left[\frac{44.55}{N_m/2 - 5/8}\right] - 8.48$ $N_{min} = 3.69$ in $N/d = \frac{3.69}{15.7} = .235 > .2$

(2) cont w/ $N/d > .2$

$$N_{min} = \frac{d}{4}\left[\left(\frac{t_f}{t_w}\right)^{1.5}\left[\frac{R_u}{.75(.4)(t_w^2)}\sqrt{\frac{t_w}{EF_y t_f}}-1\right]+.2\right]$$

$$= \frac{15.7}{4}\left[\left(\frac{.345}{.25}\right)^{1.5}\left[\frac{R_u}{.75(.4)(.25)^2}\sqrt{\frac{.25}{29{,}000(50)(.345)}}-1\right]+.2\right]$$

$$N_{min} = 3.925\left[1.62\left[.0377R_u-1\right]+.2\right]$$

$$N_{min} = 3.925\left[.06107R_u-1.62+.2\right]$$

$$N_{min} = .2397R_u - 5.574 = .2397\left[\frac{44.55}{N_{min}/2-3/8}\right]-5.574$$

$N_{min} = 3.58$ in

(3) $N_{min} = K_{det} = 1\frac{1}{10}$

$$N_{min} = 3.58 \qquad N/d = \frac{3.58}{15.7} = .228 > .2$$

Limit states

Beam Local web yield

$$\phi R_n = \phi(2.5k_{des}+N)F_{yw}t_w$$

$$= 1(2.5(.747)+3.58)(50)(.25) = 68.09^K$$

Beam Local web Crippling

$$N/d = \frac{3.58}{15.7} = .228 > .2$$

$$R_n = .4t_w^2\left[1+\left(\frac{4N}{d}-.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}$$

$$= .4(.25)^2\left[1+\left(\frac{4(3.58)}{15.7}-.2\right)\left(\frac{.25}{.345}\right)^{1.5}\right]\sqrt{\frac{29{,}000(50)(.345)}{.25}} = 50.898^K$$

$$\phi R_n = .75(50.9) = 38.17^K$$

Seat Angle flexure

$$e = \frac{N_m}{2}+3/4-t_a-3/8 = \frac{3.58}{2}+3/4-1-3/8 = 1.165 \text{ in}$$

$$\phi R_n = \frac{.9F_y L_a t_a^2}{4e} = \frac{.9(36)(5.5)(1)^2}{4(1.165)} = 38.24^K$$

Angle shear yielding

$$\phi R_n = 1.0(.6F_y)L_a t_a = 1(.6)(36)(5.5)(1) = 118.8^K$$

Weld rupture use tab. 8-4

Angle = 0°

e = 3/4 + N/2 = 3/4 + $\frac{3.58}{2}$ = 2.54"

a = $\frac{e}{L}$ = $\frac{2.54}{5.5}$ = .462 → a = .4 c = 2.66; a = .5 c = 2.29; a = .462 c = 2.44

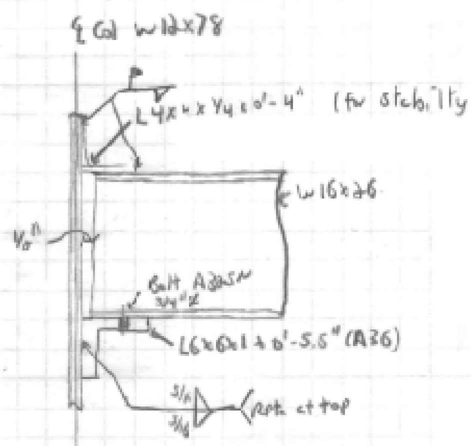
$\phi R_n = \phi C C_1 D l$ = .75(2.44)(1)(5)(6) = 54.9^k (5/16)

Base metal strength

ϕR_n = .75(.6F_y A_{nv}) A_{nv} = t_w...

= .75(.6)(50)(.47)(6) = 63.45^k

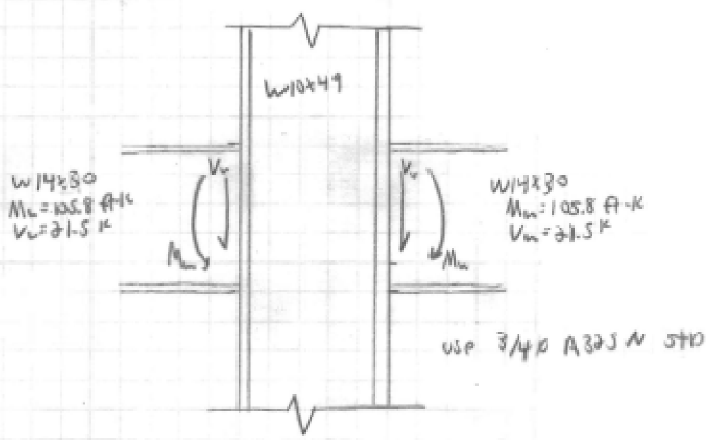
ϕV_n = 38.17^k
Web crippling controls



Typical Moment Connection: Beam to Column Flange

Lateral System Design

– Design Beam to Column Flange Moment connection



Beam: W14x30 F_y = 50 ksi F_u = 65 ksi
d = 13.8 in b_f = 6.73 in t_w = .27 in
A = 8.85 in² t_f = .385 in S_x = 42 in³

Column: W10x49 F_y = 50 ksi F_u = 65 ksi
d = 10 in b_f = 10 in t_w = .34 in
A = 14.4 in² t_f = .56 in S_x = 54.6 in³

– Check the beam available flexural strength
Assume two rows of bolts STD

$A_{fg} = b_f t_f = 6.73(.385) = 2.59 \text{ in}^2$

$A_{fn} = A_{fg} - 2(d_b + 1/8) t_f = 2.59 - 2(3/4 + 1/8).385 = 1.92 \text{ in}^2$

$\frac{F_y}{F_u} = \frac{50}{65} = .769 < .8 \therefore Y_t = 1.0$

$F_u A_{fn} = 65(1.92) = 124.56 \text{ k}$

$Y_t F_y A_{fg} = 1.0(50)(2.59) = 129.5 \text{ k} > 124.56 \text{ k}$

$M_n = \frac{F_u A_{fn} S_x}{A_{fg}} = \frac{65(1.92)(42)}{2.59} = 2019.81 \text{ k-in} = 168.32 \text{ k-ft}$

$\phi M_n = .9(168.32) = 151.5 \text{ k-ft}$

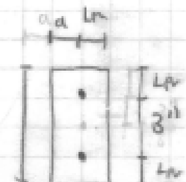
$\phi V_n = 112 \text{ k}$

Lateral System Design

- Moment Connection continued

  - Design Single Plate Connection

    $V_u = 21.5^k$



Assume $L_{eh} = 1\frac{1}{2}"$
$L_{ev} = 1\frac{1}{4}"$
For table 10-9 a = 3.5 anything < 3" is conservative

- From Table 10-9a for 3/4" bolt, A325 N-STD
  use N = 2, L = 5 1/2", t_p = 1/4", t_{weld} = 3/16" weld, min = 3/16, max = 3/16 ok

  $\phi V_n = 24.5^k > 21.5^k$

- From Table 10-1 consider bolt bearing, block shear

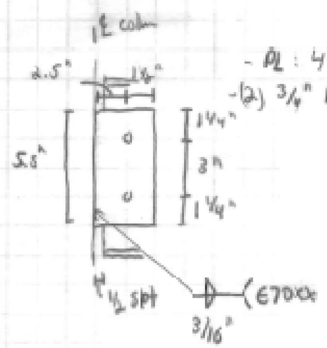
  $\phi V_n = 126(1/4) = 31.5^k > 21.5^k$

- Check block shear of plate w/ table 9-3 a-c $L_{eh} = 1\frac{1}{2}$ $L_{ev} = 1\frac{1}{4}$
  *controls over thicker web of beam

  9-3a Tension Rupture
  $\phi R_n = 46.2(1/4) = 11.55$

  9-3b Shear yield
  $\phi R_n = 68.8(1/4) = 17.2^k$

  9-3c Shear rupture
  $\phi R_n = 76.7(.25) = 19.175^k$ $\phi R_n = 11.55 + 17.2 = 28.75^k > 21.5^k$ ok



- PL: 4" x 5 1/2" x 1/4"
- (2) 3/4" A325N STD

Lateral System Design

- Moment Connection Cont.

  - Design tension flange and connection
    - Design Bolt

$P_{uf} = \frac{M_u}{d} = \frac{105.8(12)}{13.8} = 92^k$

what thickness would the plate have to be for tension yield

$\phi R = \phi A_g F_y \quad b_f = 6.73 \quad w_g = 6''$

$92 = 6\, t_p (36) = .426\ in \quad$ use $t_p = 1/2''$

Try PL $6'' \times 1/2''$ $\quad t_f = .56\ in$ A992
$t_p = .5$ A36 → plate controls

shear $\phi r_n = 15.9\ ^k/bolt$

bearing on flange $\phi r_n = 78.3\ ^k/bolt$ from table 7-5

tearout at $\phi r_n = \phi 1.2 F_u l_c t = .75(1.2)(58)(1.5 - 1/2(3/4 + 1/16))1/2$
$= 28.55\ ^k/bolt$

$n = \frac{92}{15.9} = 5.79 \quad$ use 2 rows of 3 bolts
A325 3/4'' ϕ

- Check plate tension yielding

$P_n = F_y A_g = 36(6)(.5) = 108^k$

$P_{uf} = \frac{M_u}{d + t_p} = \frac{105.8(12)}{13.8 + 1/2} = 88.78^k \qquad \phi P_n = .9(108) = 97.2^k > 88.78^k$ ok

- check plate tension rupture

$A_n = [w_g - 2(d_b + 1/8)]\, t_p = [6 - 2(3/4 + 1/8)]\, 1/2 = 2.125\ in^2$

$P_n = F_u A_n = 58(2.125) = 123.25 \qquad \phi P_n = .75(123.25) = 92.44^k > 88.78^k$

- Check flange block shear of plate $\quad L_{eh} = 1 1/2'' \quad L_{ev} = 1 1/2''$

9-3a $\quad$ TR $= 54.4^k (1/2)(2) = 54.4^k$

SY $= 121 (1/2)(2) = 121^k$

SR $= 139 (1/2)(2) = 139^k \qquad \phi R_n = 54.4 + 121 = 175.4^k > 88.78^k$

- Determine the req. of the fillet weld to support column flange
applied the req. is ⊥ (90°)

$D_{min} = \frac{88.78}{2(1.5)(1.392)(6)} = 3.54 \quad$ use 1/4'' weld

- check base metal strength

$\phi R_n = .75(.6)(65)(.56)(6) = 98.28^k > 88.78^k$ ok

Lateral System Design

- Moment Connection Cont.

- Design compression flange plate

  - Use PL 6" x 1/2"

    Assum K = .65 L = 2 1/4 = 1 1/2 + 1/2 SP

    local buckling

    $\frac{b_f}{t_p} \leq \frac{253}{\sqrt{36}}$ $\frac{6.73}{.5} \leq \frac{253}{\sqrt{50}}$ 13.46 < 46.17 ok

    Flexural buckling $\frac{KL}{r} = \frac{.65(2)}{\sqrt{\frac{.5^2}{12}}} = 9.01 < 25$ $F_{cr} = F_y$

    $P_n = \phi F_{cr} A_g$

    $.9(36)(6)(1/2) = 97.2^k > 88.78^k$ ok

    Comp flange plate to be design to match tension side for load reversal

- Column side limit states

  Local flange bending

  $T_u \leq \phi R$ $\phi = .9$

  $R_n = 6.25\, t_f^2 F_y = 6.25(.56)^2(50) = 98^k$

  $T_u = 88.78 < 98^k$ no stiffener req.

  Local web yielding

  $T_u = C_u \leq \phi R$

  $R_n = \phi F_{yc}(6k_c + N)\, t_{wc} = 50[5(1.06 + .5)].56 = 160.4 > T_u = C_u$ ok

  Local web crippling

  $C_u \leq \phi R$ $\phi = .75$

  $R_n = .8\, t_w^2 \left[1 + 3(N/d)\left(\frac{t_{wc}}{t_f}\right)^{1.5}\right]\sqrt{\frac{E F_{yw} t_f}{t_w}}$

  $.8(.34)^2\left[1 + 3(.5/10)\left(\frac{.34}{.56}\right)^{1.5}\right]\sqrt{29{,}000(50)\left(\frac{.56}{.36}\right)} = 167.3^k$

  $\phi R_n = .75(167.3) = 125.5^k > T_u = C_u$ ok

  web buckling

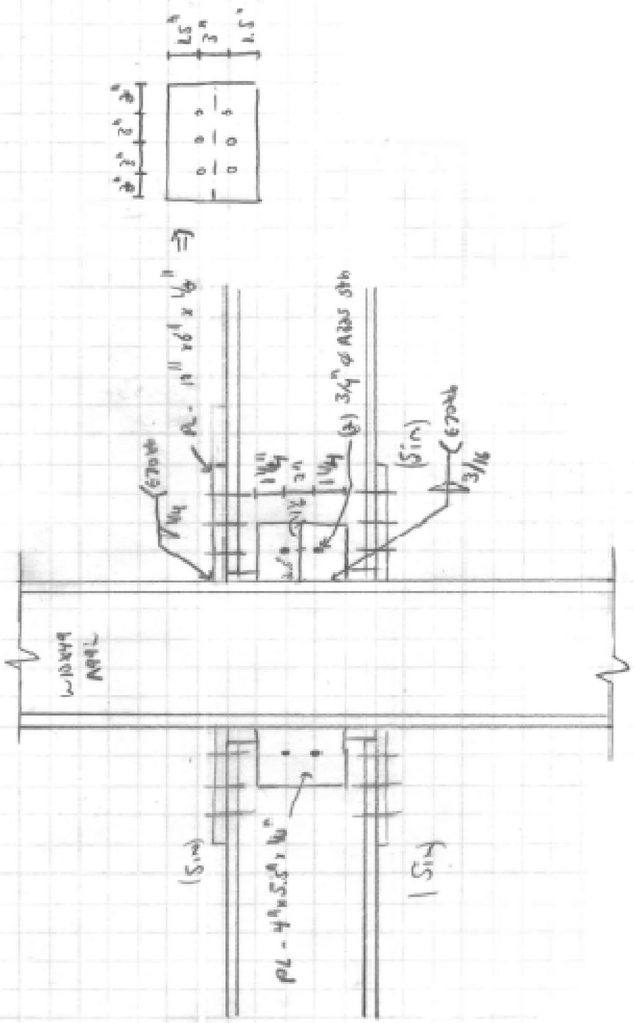
  $C_u \leq \phi R$ $\phi = .9$

  $R_n = 24\sqrt{E F_{yc}}\left(\frac{t_w^3}{h}\right) = 24\sqrt{29{,}000(50)}\left(\frac{.34^3}{7.5}\right) = 151.5$

  $\phi R_n = .9(151.5) = 136.2^k > C_u$ ok

Lateral System Design cont.

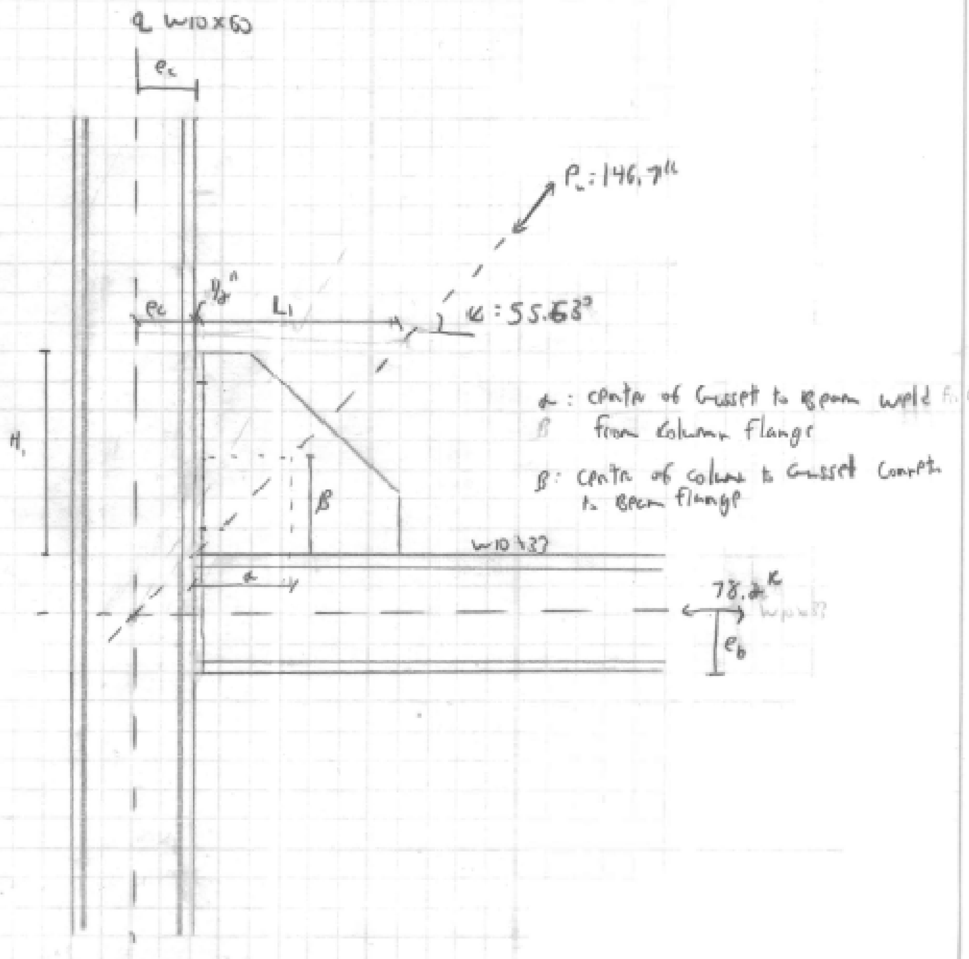
Moment Connection Design



Typical Heavy Braced Connection

Lateral System Design

Design a Braced Connection using General Uniform force



Column: W10x60

A: 17.6 in²	t_w: .42 in	t_f: .77 in
d: 10.2 in	b_f: 10.1 in	T: 7.5 in
e_c: 5.1 in		

Beam: W10x33

A: 9.71 in²	t_w: .29 in	t_f: .435 in
d: 9.73 in	b_f: 8 in	T: 7.5 in
e_b: 4.865 in		

Lateral System Design

- Limit States For Brace - 2L5"x5"x3/4"

Note $A_e = A_n U$ $U = 1 - \frac{\bar{x}}{L}$ $\bar{x} = 1.52 in$

L = Length of connection

*each angle needs to looked at separately

1. Tension Yield

Assume 1 row of bolts 3/4"Ø A325N std in Double Shear

$\phi R_n = \phi F_y A_g = .9(36)(14) = 453.6^k > 146.7^k$ ok

* Note compression controlled size of angle.

2. Tension Rupture

$\phi R_n = \phi F_u A_e$

$A_e = A_n U$ $U = 1 - \frac{1.52}{15} = .899$

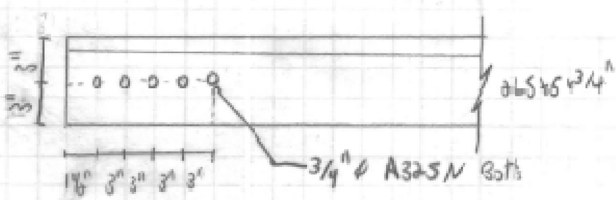
← assumes (5) 3/4"Ø Bolts in double shear

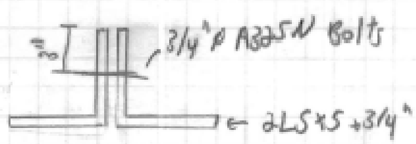
$A_g = 6.94 in^2$

$A_n = 6.94 - 1(3/4 + 1/8)3/4 = 6.28 in^2$

$A_e = .899(6.28) = 5.65 in^2 < .85 A_g = 5.899 in^2$

$\phi R_n = .75(58)(5.899)(2) = 513.2^k > 146.7^k$ ok





Lateral System Design

- Limit states for Brace Cont.

3. Block Shear

Using table 9-3 $L_{eh} = 3"$ $L_{ev} = 1\frac{1}{2}"$

9-3a $\phi F_u A_{nt} = 111$ k/in

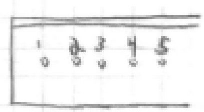
9-3b $\phi 0.6 F_y A_{gv} = 219$ k/in

9-3c $\phi 0.6 F_u A_{nv} = 250$ k/in

$\phi R_n = 2\left[.5(111).75 + .5(219)\frac{3}{4}\right] = 247.5^k > 146.7^k$

(2 = 2 Angles; .5 = non uniform load)

- Limit states for Bolts



- Bolt shear = $15.9 \times 2 = 31.8^k$

- Bearing on Angle = $\phi 2.4 F_u t d_b$
$= .75(2.4)(58)(2)(.75)(.75) = 117.45^k$

- Bearing on Plate (assume ½" thick)
$= .75(2.4)(58)(.5)(.75) = 39.15^k$

- tearout Angle edge (Bolt 1)
$= \phi 1.2 F_u L_c t$
$= .75(1.2)(58)\left[1.5 - \frac{1}{2}(\frac{3}{4} + \frac{1}{16})\right](2)(\frac{3}{4}) = 85.64^k$

- tearout Angle middle (Bolt 2-5)
$= .75(1.2)(58)\left[3 - 1(\frac{3}{4} + \frac{1}{16})\right]2(\frac{3}{4}) = 171.28^k$

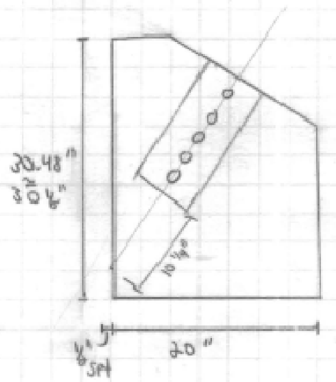
- tearout Plate edge (Bolt 5)
$= .75(1.2)(58)\left[1.5 - .5(\frac{3}{4} + \frac{1}{16})\right].75 = 28.55^k$

- tearout Plate middle
$= .75(1.2)(58)\left[3 - 1(\frac{3}{4} + \frac{1}{16})\right].75 = 57.09^k$

$\phi R_n = 4(31.8) + 28.55 = 155.75^k > 146.9^k$ ok

Lateral System Design

- Limits State for Gusset



* Gusset Dimmensions were designed using geometry of connection and the Uniform Force method.

PL: 30 1/2" x 20" x 3/4"

- Block shear

  * TR Does not occur

  Table 9-3b SY: $219(3/4) = 164^k$

  Table 9-3c SR: $250(3/4) = 187.5^k$

  $\phi R = 164^k > 146.5^k$ ok

- Gusset Yielding

  Whitmore section $12\tan(30) = 6.93''$

  $A_w = 2(6.93)(.75) = 10.39\ in^2$

  $R_n = A_n F_y = 36(10.39) = 374.12\ k$

  $\phi R_n = .9(374.12) = 336.71^k > 146.7^k$ ok

- Gusset Buckling

  $r = \frac{t}{\sqrt{12}} = \frac{.75}{\sqrt{12}} = .217$

  $l = 10.25\ in$ $\quad \frac{Kl}{r} = \frac{1.2(10.25)}{.217} = 56.68 < 134$ ok

  $F_e = \frac{\pi^2 E}{(\frac{KL}{r})^2} = \frac{\pi^2(29{,}000)}{(56.68)^2} = 89.1\ ksi$

  $F_{cr} = \left[.658^{F_y/F_e}\right] F_y = (.658^{36/89.1})36 = 30.4\ ksi$

  $P_n = F_{cr} A_w = 30.4(10.39) = 315.84^k$

  $\phi P_n = .9(315.84) = 284.3^k > 146.7^k$ ok

Lateral System Design

Uniform force method

$\alpha = ?$ $\quad e_b = 4.865"$ $\quad \phi = 90 - 52.64 = 37.36°$

$\beta = 30.48/2 =$ $\quad e_c = 5.1$ in

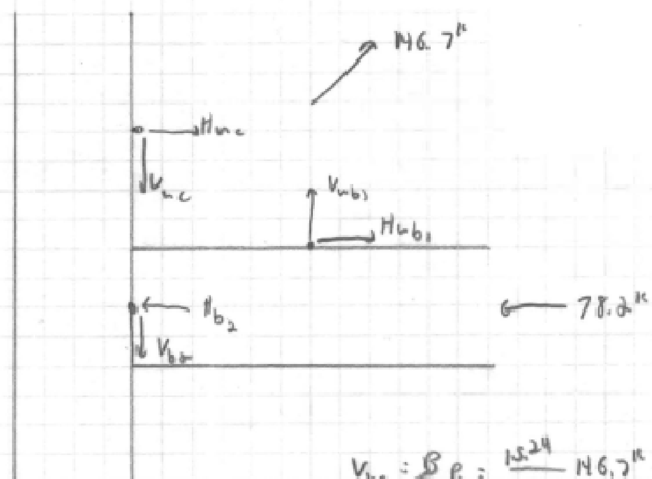
$\alpha = 4.865 \tan(37.36) - 5.1 + 15.24 \tan(37.36)$

$\alpha = 10.25$

$L_1 = 2\alpha - 1/2 = 2(10.25) - 1/2 = 20"$ good

$r = \sqrt{(10.25 + 5.1)^2 + (30.48/2 + 4.865)^2} = 25.29$ in

Gusset to



$V_{uc} = \frac{\beta}{r} P_u = \frac{15.24}{25.29} 146.7^k = 88.39^k$

$H_{uc} = \frac{e_c}{r} P = \frac{5.1}{25.29} 146.7 = 29.58^k$

$H_{ub_1} = \frac{10.25}{25.29} 146.7^k = 59.45^k$

$V_{ub_1} = \frac{4.865}{25.29} 146.7^k = 27.17^k$

$H_{b_2} = 78.2 - 59.45 = 18.75^k$

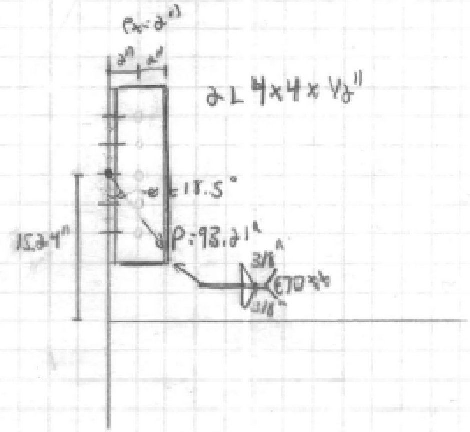
$V_{b_2} = -27.17^k$

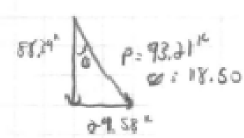
Lateral System Design

Design Double Angle Gusset to Column Connection

$V_{uc} = 88.34^k$
$H_{uc} = 29.58^k$ (Prying)

Assume 10 3/4" ϕ A325 Bolts, 2 rows of 5 on column side of a double angle connection





- check weld strength

$l = 15" = 4(6)+2(1.5) > 27$
$kl = 3.5" = 4 - 1/2 = 3.5"$
$al = \frac{0(15) + 2(3.5)(3.5/2)}{15+3.5+3.5} = .557$

$k = .233$
$a = .037$

Table 8-8 θ = 15°

	k=.2	k=.233	k=.3
a=0	3.07		3.56
a=.037			
a=.1	2.87		3.40

θ=30°

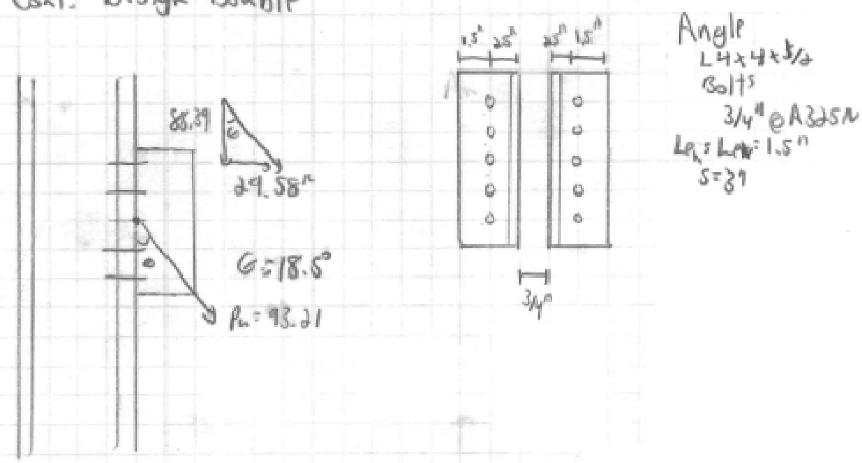
	k=.2	k=.237	k=.3
a=0	3.00		3.41
a=.037			
a=.1	3.13		3.62

C = 3.149

$\phi R_n = .75(3.149)(1.0)(6)(15) = 212.56^k > 93.21^k$ OK

Lateral System Design

Cont. Design Double Angle



1. Calculate shear stress in bolts

$$f_v = \frac{V_u}{\# A_b} = \frac{88.39}{10(.442)} = 19.998" \approx 20\text{ ksi}$$

2. calculate available tensile per bolt

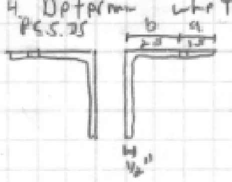
$$F'_t = \sqrt{90^2 - 6.25(20)^2} = 74.84\text{ ksi}$$

$$\phi r_{nt} = .75(74.84)(.442) = 24.808\text{ k}$$

3. Calculate r_{nt}

$$r_{nt} = \frac{29.58}{10} = 2.958\text{ k}$$

4. Determine whether prying force will occur



$$p \le \frac{15}{5} = 3" \quad (5.75)$$

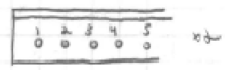
$$r_{ut} > \frac{\phi F_u p t^2}{4b'} = \frac{.9(58)(3)(.5)^2}{4(1.875)} = 5.22\text{ k}$$

a = 1.5"
a' = 1.5 + .75/2 = 1.88"
b = 2.5 - (1/2)/2 = 2.25"
b' = 2.25 - (3/4)/2 = 1.875"

$r_{ut} \le 5.22$" Prying
no prying

Lateral System Design

- Gusset to Column Double Angle
  - no prying,
  - check shear load $V_u = 88.39^k$
- Bolt shear, bearing, tearout
  - Bolt shear = $\underline{15.9^k}$



  - Bolt bearing on Angle
    $= .75(2.4)(58)(^5/_{16})(3/4) = 39.15^k$
  - Bearing on column flange
    $= .75(2.4)(65)(.42)(.75) = 36.9^k$
  - tearout Angle edge (Bolt 1)
    $= .75(1.2)(58)[1.5 - ½(3/4 + 1/16)].51 = 28.55^k$
  - tearout Angle middle (Bolt 2-5)
    $= .75(1.2)(58)[3 - (3/4 + 1/16)].51 = 57.09^k$
  - tearout column middle
    $= .75(1.2)(65)[3 - (3/4 + 1/16)].42 = 53.75^k$

$\phi R_n = 5(15.9)2 = \underline{159^k} > 88.39^k$

- Block shear

  Using Tables 9-3 $L_{ev} = 1\frac{1}{2}"$ $L_{eh} = 1\frac{1}{2}"$
  9-3a = 46.2 (.5) = 23.1
  9-3b = 219 (.5) = 109.5
  9-3c = 250 (.5) = 125

  $\phi R_n = 2(23.1 + 109.5) = \underline{161.8}^k$

- Shear yield of Angles
  $A_g = 7.49\ in^2$
  $\phi .6F_yA_g = 1.0(.6F_yA_{gv} = .6(36)(7.49) = 262.7^k$

- shear rupture of Angles
  $A_n = 7.49 - 2(3/4 + 1/8).5 = 6.62\ in^2$

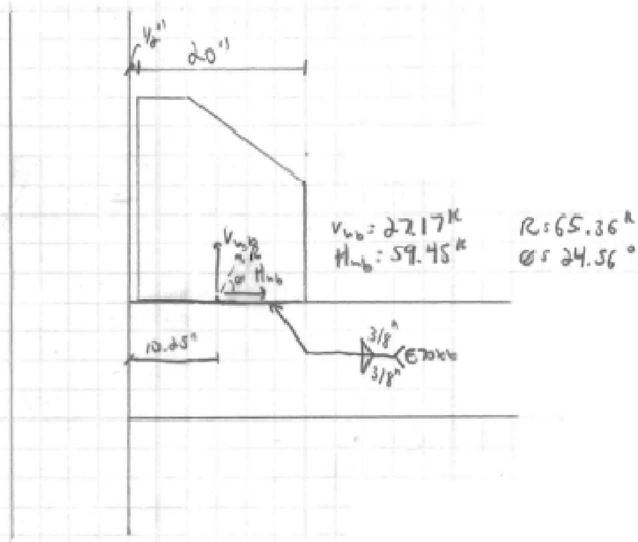
  $\phi .6F_uA_{nv} = .75(.6)(58)(6.62) = \underline{172.78}^k$

Connection over design.
ok

8

Lateral System Design

Braced Connection Design Continued



- Determine strength of weld group
  use Table 8-4

$al = 0"$ $kl = .75"$
$l = 20"$ $k = .0375"$

$\theta = 15°$

	k = .0	k = .0375	k = .1
a = 0	3.84		3.84

$\theta = 30°$

a = 0	4.18		4.18

$\theta = 24.56$ $C = 4.057$

$\phi R_n = .75(4.057)(1.0)(6)(20) = \underline{365.13^k} > 65.36^k$ ok

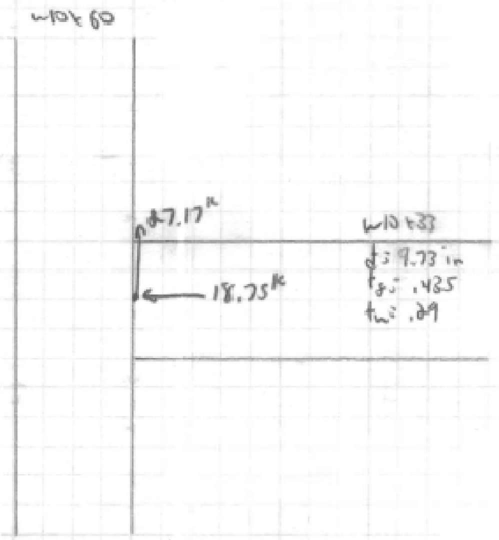
- check Base metal strength

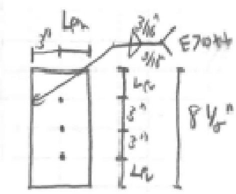
$\phi R_n = .75(.6) F_y A_{nw} = .75(.6)(50)(.435)(20)(2) = 391.5^k$ ok (t_f beam)

Lateral System Design

Braced Connection Design Cont.

- Design Beam to Column Connection





Leh = 1 1/2"
Lev = 1 1/4"
For table 10-9 a = 3

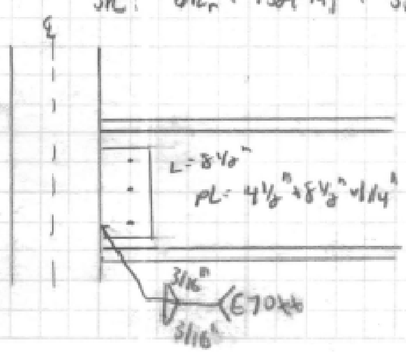
- From Table 10-9 f. 3/4" bolts A325N-st?
  Use N = 3 L = 8 1/2 t_p = 1/4" ϕV_n = 38.8k > 27.17k
  t_w = 3/16"
  $t_{w\,min}$ = 3/16" ok

- From Table 10-1 a=3 bolt bearing, block shear
  ϕV_n = 188 (1/4) = 47k > 27.17k

- Check block shear of plate us 9-3

9-3c TR: ϕR_n = 46.2(1/4) = 11.55k
SY: ϕR_n = 117 (1/4) = 29.25k
SR: ϕR_n = 132(1/4) = 33k

ϕR_n = 40.8k > 27.17k



Appendix G: Foundation Design

Gravity System Design

Foundation Design

$P_D = [(15+50)4(697.51) + (40+15+50)(697.51)]/1000 = 254.59^k$

$P_L = [80(4)697.51]/1000 = 223.2^k$

$P_{Lr} = 20(697.51)/1000 = 13.95^k$

Allowable Bearing Capacity = 5994 psf

- Steel column to 24"x24" Concrete Pier to Spread Footing
  Design and Detail spread footing

$P = P_D + P_L = 254.59 + 223.2 + 13.95 = 491.74^k$

$q_a \geq \frac{P}{A} = \quad 5.99\ ksf \geq \frac{491.75}{B^2}$ $\quad B \geq 9.06'$

use $B = 10\ ft$

$P_u = 1.2(254.59) + 1.6(223.2) + .5(13.95) = 669.6^k$

$q = \frac{P_u}{A} = \frac{669.6}{10^2} = 6.696\ ksf$
$= 46.5\ psi$

$v_c = \phi 4\sqrt{f'_c} = .75(4)\sqrt{3000} = 164\ psi$

The equation for two way shear controls by inspection

$d^2[v_c + q/4] + d[v_c + q/2]w = [q/4(B^2 - w^2)]$

$d^2[164 + \frac{46.5}{4}] + d[164 + \frac{46.5}{2}]24 = \frac{46.5}{4}(120^2 - 24^2)$

$175.63d^2 + 4494d = 16,070$ $\quad d = 20.05"$

$h = 20.05 + 3 + .75 = 23.7996"$ $\quad$ use $h = 24"$ $\quad d = 20.25$

Gravity System Design Cont.

Use $h = 24''$

$d = 24 - 3 - .75 = 20.25''$

$l = \frac{10' - 2'}{2} = 4'$

$M_u = \frac{q l^2}{2} = \frac{6.696(4)^2}{2} = 53.568 \text{ ft·k}$

$a = \frac{A_s f_y}{.85 f'_c b} = 1.96 A_s$

$M_u = \phi M_n = \phi A_s f_y (d - a/2)$

$53.568(12) = .9 A_s (60)\left[20.25 - \frac{1.96 A_s}{2}\right]$ $A_s = .606 \text{ in}^2$

use #7 @ 11" o.c. < 18" ok
$A_s = .655 \text{ in}^2$

$\rho = \frac{A_s}{bh} = \frac{.655}{(12)24} = .0023 \geq .0018$

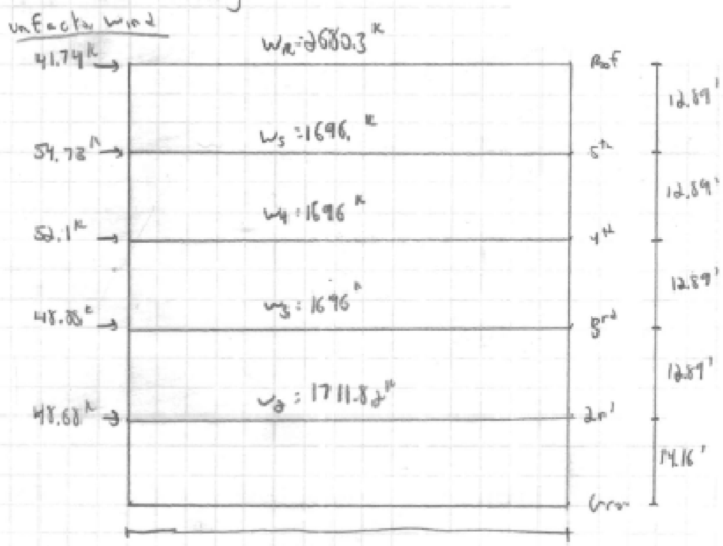
$a = 1.96 A_s = 1.96(.655) = 1.284''$ $c = \frac{1.284}{.85} = 1.51 \text{ in}$

$\varepsilon_s = \frac{.003}{1.51}(20.25 - 1.51) = .037 \text{ in/in} > .005 \text{ in/in} \therefore \phi = .9$

use (11) #7 both ways

Fondation check

-overturning



check overturning - general stability of structure

overturning moment

1.6[41.74(65.72)+54.73(52.83)+52.1(39.94)+48.88(27.05)+48.68(14.16)] = 15,563.1 ft-k

uplift T = 15563.1 / 74.8 = 208.1 k

Gravity Load to resist uplift => .9D

.9[2680.3 + 1696 + 1696 + 1696 + 1711.82] = 8532.1 >> 208.1 k

* Building passes general stability check

* lateral baseplates should be designed to take tension

Appendix H: Cost and Schedule Takeoffs

Orignal Design Takeoff

Floor	Slab Concrete		Slab Reinforcing	Slab Finishing	Beam Concrete		Beam Reinforcing	Column Concrete		Column Reinforcing
	CY	SFCA	Ton	SF	CY	SFCA	Ton	CY	SFCA	Ton
Ground	385.63	0	120.42	20824						
2nd	586.15	17407.28	116.23	20098.55	196.75	7100.31	65.02	168.32	4574.06	33.38
3rd	607.30	18132.73	120.42	20824	196.75	7100.31	65.02	153.30	4165.95	30.40
4th	607.30	18132.73	120.42	20824	196.75	7100.31	65.02	153.30	4165.95	30.40
5th	607.30	18132.73	120.42	20824	196.75	7100.31	65.02	153.30	4165.95	30.40
Roof	607.30	18132.73	120.42	20824	196.75	7100.31	65.02	107.60	2931.85	21.34

Original Concrete Design Cost and Schedule Breakdown

Roof	Item #	Decreption	Unit	Quantity	Total Cost per Unit	Cost		# of Crews	Daily output	# of Days
	03 11 13.20 1050	Beam Formwork	S.F.C.A	7100.31	7.92	56234.46		3	275	9
	03 11 13.25 6550	Column Formwork	S.F.C.A	2931.85	6.56	19232.96		2	216	7
	03 11 13.35 2050	Slab Formwork	S.F.C.A	18132.73	6.08	110247.01		3	509	12
	03 21 10.60 0100	Beam Reinf.	Ton	65.02	1720	111838.48		3	1.6	14
	03 31 10.60 0200	Column Reinf.	Ton	21.34	1775	37869.91		3	1.5	5
	03 21 10.60 0400	Slab Reinf.	Ton	120.42	1405	169191.19		3	2.9	14
	03 31 05.35 0300	4000 psi conc.	CY	911.65	108	98458.26		N/A		0
	03 31 05.70 0200	Placing Beams	CY	196.75	30.35	5971.40		1	90	3
	03 31 05.70 0800	Placing Columns	CY	107.60	29.65	3190.25		1	92	2
	03 31 05.70 1600	Placing Slab	CY	607.30	15.22	9243.14		1	180	4
	03 35 29.30 0150	Finishing	S.F.	20824	0.45	9370.80	630847.86	3	630	12

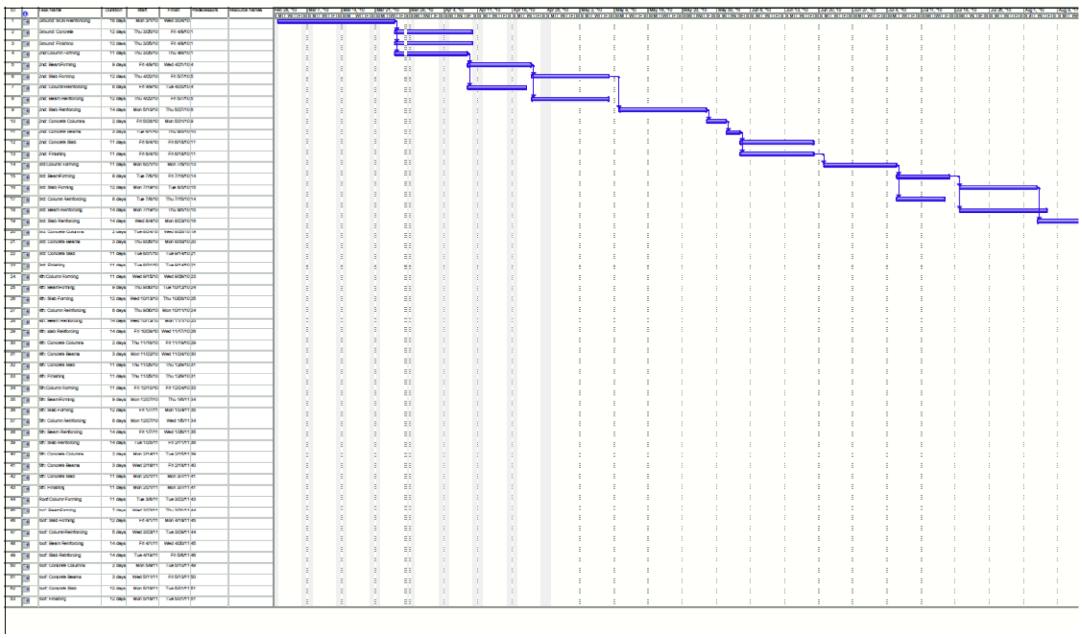
5th	Item #	Decreption	Unit	Quantity	Total Cost per Unit	Cost		# of Crews	Daily output	# of Days
	03 11 13.20 1050	Beam Formwork	S.F.C.A	7100.31	7.92	56234.46		3	275	9
	03 11 13.25 6550	Column Formwork	S.F.C.A	4165.95	6.56	27328.60		2	216	10
	03 11 13.35 2050	Slab Formwork	S.F.C.A	18132.73	6.08	110247.01		3	509	12
	03 21 10.60 0100	Beam Reinf.	Ton	65.02	1720	111838.48		3	1.6	14
	03 31 10.60 0200	Column Reinf.	Ton	30.40	1775	53957.02		3	1.5	7
	03 21 10.60 0400	Slab Reinf.	Ton	120.42	1405	169191.19		3	2.9	14
	03 31 05.35 0300	4000 psi conc.	CY	957.36	108	103394.62		N/A		0
	03 31 05.70 0200	Placing Beams	CY	196.75	30.35	5971.40		1	90	3
	03 31 05.70 0800	Placing Columns	CY	153.30	29.65	4545.46		1	92	2
	03 31 05.70 1600	Placing Slab	CY	607.30	15.22	9243.14		1	180	4
	03 35 29.30 0150	Finishing	S.F.	20824	0.45	9370.80	661322.18	3	630	12

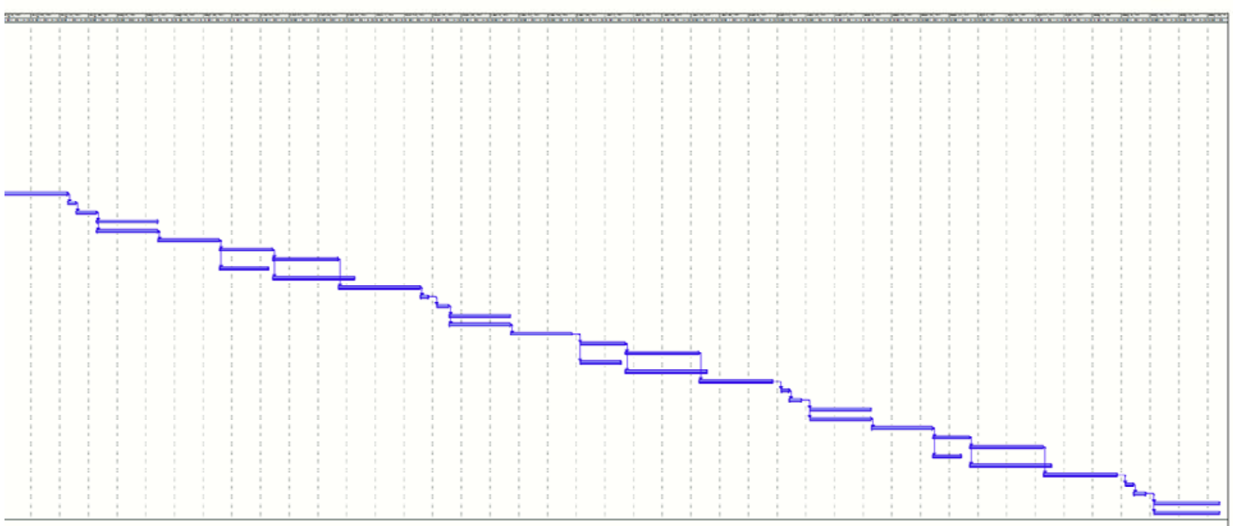
4th	Item #	Decreption	Unit	Quantity	Total Cost per Unit	Cost		# of Crews	Daily output	# of Days
	03 11 13.20 1050	Beam Formwork	S.F.C.A	7100.31	7.92	56234.46		3	275	9
	03 11 13.25 6550	Column Formwork	S.F.C.A	4165.95	6.56	27328.60		2	216	10
	03 11 13.35 2050	Slab Formwork	S.F.C.A	18132.73	6.08	110247.01		3	509	12
	03 21 10.60 0100	Beam Reinf.	Ton	65.02	1720	111838.48		3	1.6	14
	03 31 10.60 0200	Column Reinf.	Ton	30.40	1775	53957.02		3	1.5	7
	03 21 10.60 0400	Slab Reinf.	Ton	120.42	1405	169191.19		3	2.9	14
	03 31 05.35 0300	4000 psi conc.	CY	957.36	108	103394.62		N/A		0
	03 31 05.70 0200	Placing Beams	CY	196.75	30.35	5971.40		1	90	3
	03 31 05.70 0800	Placing Columns	CY	153.30	29.65	4545.46		1	92	2
	03 31 05.70 1600	Placing Slab	CY	607.30	15.22	9243.14		1	180	4
	03 35 29.30 0150	Finishing	S.F.	20824	0.45	9370.80	661322.18	3	630	12

3rd	Item #	Decreption	Unit	Quantity	Total Cost per Unit	Cost		# of Crews	Daily output	# of Days
	03 11 13.20 1050	Beam Formwork	S.F.C.A	7100.31	7.92	56234.46		3	275	9
	03 11 13.25 6550	Column Formwork	S.F.C.A	4165.95	6.56	27328.60		2	216	10
	03 11 13.35 2050	Slab Formwork	S.F.C.A	18132.73	6.08	110247.01		3	509	12
	03 21 10.60 0100	Beam Reinf.	Ton	65.02	1720	111838.48		3	1.6	14
	03 31 10.60 0200	Column Reinf.	Ton	30.40	1775	53957.02		3	1.5	7
	03 21 10.60 0400	Slab Reinf.	Ton	120.42	1405	169191.19		3	2.9	14
	03 31 05.35 0300	4000 psi conc.	CY	957.36	108	103394.62		N/A		0
	03 31 05.70 0200	Placing Beams	CY	196.75	30.35	5971.40		1	90	3
	03 31 05.70 0800	Placing Columns	CY	153.30	29.65	4545.46		1	92	2
	03 31 05.70 1600	Placing Slab	CY	586.15	15.22	8921.14		1	180	4
	03 35 29.30 0150	Finishing	S.F.	20824	0.45	9370.80	661000.18	3	630	12

2nd	Item #	Decreption	Unit	Quantity	Total Cost per Unit	Cost		# of Crews	Daily output	# of Days
	03 11 13.20 1050	Beam Formwork	S.F.C.A	7100.31	7.92	56234.46		3	275	9
	03 11 13.25 6550	Column Formwork	S.F.C.A	4574.06	6.56	30005.83		2	216	11
	03 11 13.35 2050	Slab Formwork	S.F.C.A	17407.28	6.08	105836.27		3	509	12
	03 21 10.60 0100	Beam Reinf.	Ton	65.02	1720	111838.48		3	1.6	14
	03 31 10.60 0200	Column Reinf.	Ton	33.38	1775	59242.89		3	1.5	8
	03 21 10.60 0400	Slab Reinf.	Ton	116.23	1405	163297.02		3	2.9	14
	03 31 05.35 0300	4000 psi conc.	CY	951.22	108	102731.66		N/A		0
	03 31 05.70 0200	Placing Beams	CY	196.75	30.35	5971.40		1	90	3
	03 31 05.70 0800	Placing Columns	CY	168.32	29.65	4990.75		1	92	2
	03 31 05.70 1600	Placing Slab	CY	586.15	15.22	8921.14		1	180	4
	03 35 29.30 0150	Finishing	S.F.	20099	0.45	9044.35	658114.25	3	630	11

Ground	Item #	Decreption	Unit	Quantity	Total Cost per Unit	Cost		# of Crews	Daily output	# of Days
	03 21 10.60 0600	SOG Reinf	Ton	120.42	1425	171599.61		3	2.3	18
	03 31 05.35 0300	4000 psi conc.	CY	385.63	108	41648.00		N/A		0
	03 31 05.70 4650	Slab on Grade	CY	385.63	14.81	5711.17		1	185	3
	03 35 29.30 0150	Finishing	S.F.	20824	0.45	9370.80	228329.58	3	630	12
							$3,500,936.23			



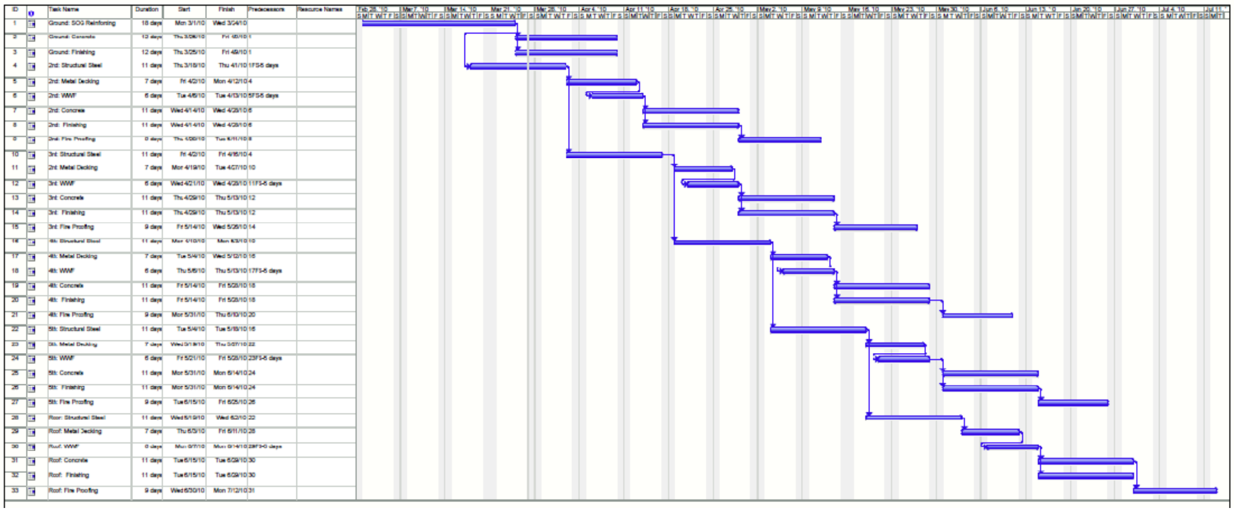


Steel Redesign Takeoff

	Beam ton	columns ton	frame ton	decking sf	concrete cy	wwf sf	fireproofing sf
Roof	85.44	7.51	55.78	20824	224.95	20824	20824
5	70.97	7.51	54.99	20824	224.95	20824	20824
4	70.97	9.78	61.60	20824	224.95	20824	20824
3	70.97	9.78	64.25	20824	224.95	20824	20824
2	70.63	12.26	73.77	20099	217.11	20098.55	20098.55

Steel Redesign Cost and Schedule Breakdown

Roof	Item #	Decreption	Unit	Quantity	Total Cost per Unit	Cost		# of Crews	Daily output per crew	# of Days	
	05 12 23.77 0800	Structural Steel	Ton	148.74	2591	385374.64		1	14.4	11	
	05 31 13.35 3400	18 Gage Decking	SF	20824	3.65	76007.60		1	3200	7	
	03 31 05.35 0300	4000 psi Concrete	CY	224.95	108	24294.67		1	N/A	0	
	03 22 04.40 0100	WWF	CSF	208.24	31.65	6590.80		1	35	6	
	03 31 05.70 1400	Elevated Slab	CY	224.95	19.55	4397.78		1	140	2	
	03 35 29.30 0150	Broom Finish	SF	20824	0.45	9370.80		3	630	12	
	07 81 16.10 0500	Fireproofing	SF	20824	1.35	28112.40	**534148.69**	2	1250	9	**47.00**
5th	Item #	Decreption	Unit	Quantity	Total Cost per Unit	Cost		# of Crews	Daily output per crew	# of Days	
	05 12 23.77 0800	Structural Steel	Ton	133.48	2591	345846.34		1	14.4	10	
	05 31 13.35 3400	18 Gage Decking	SF	20824	3.65	76007.60		1	3200	7	
	03 31 05.35 0300	4000 psi Concrete	CY	224.95	108	24294.67		1	N/A	0	
	03 22 04.40 0100	WWF	CSF	208.24	31.65	6590.80		1	35	6	
	03 31 05.70 1400	Elevated Slab	CY	224.95	19.55	4397.78		1	140	2	
	03 35 29.30 0150	Broom Finish	SF	20824	0.45	9370.80		3	630	12	
	07 81 16.10 0500	Fireproofing	SF	20824	1.35	28112.40	**494620.39**	2	1250	9	**46.00**
4th	Item #	Decreption	Unit	Quantity	Total Cost per Unit	Cost		# of Crews	Daily output per crew	# of Days	
	05 12 23.77 0800	Structural Steel	Ton	142.35	2591	368837.94		1	14.4	10	
	05 31 13.35 3400	18 Gage Decking	SF	20824	3.65	76007.60		1	3200	7	
	03 31 05.35 0300	4000 psi Concrete	CY	224.95	108	24294.67		1	N/A	0	
	03 22 04.40 0100	WWF	CSF	208.24	31.65	6590.80		1	35	6	
	03 31 05.70 1400	Elevated Slab	CY	224.95	19.55	4397.78		1	140	2	
	03 35 29.30 0150	Broom Finish	SF	20824	0.45	9370.80		3	630	12	
	07 81 16.10 0500	Fireproofing	SF	20824	1.35	28112.40	**517611.99**	2	1250	9	**46.00**
3rd	Item #	Decreption	Unit	Quantity	Total Cost per Unit	Cost		# of Crews	Daily output per crew	# of Days	
	05 12 23.77 0800	Structural Steel	Ton	145.01	2591	375714.46		1	14.4	11	
	05 31 13.35 3400	18 Gage Decking	SF	20824	3.65	76007.60		1	3200	7	
	03 31 05.35 0300	4000 psi Concrete	CY	224.95	108	24294.67		1	N/A	0	
	03 22 04.40 0100	WWF	CSF	208.24	31.65	6590.80		1	35	6	
	03 31 05.70 1400	Elevated Slab	CY	224.95	19.55	4397.78		1	140	2	
	03 35 29.30 0150	Broom Finish	SF	20824	0.45	9370.80		3	630	12	
	07 81 16.10 0500	Fireproofing	SF	20824	1.35	28112.40	**524488.51**	2	1250	9	**47.00**
2nd	Item #	Decreption	Unit	Quantity	Total Cost per Unit	Cost		# of Crews	Daily output per crew	# of Days	
	05 12 23.77 0800	Structural Steel	Ton	156.66	2591	405910.10		1	14.4	11	
	05 31 13.35 3400	18 Gage Decking	SF	20099	3.65	73359.71		1	3200	7	
	03 31 05.35 0300	4000 psi Concrete	CY	217.11	108	23448.31		1	N/A	0	
	03 22 04.40 0100	WWF	CSF	200.9855	31.65	6361.19		1	35	6	
	03 31 05.70 1400	Elevated Slab	CY	217.11	19.55	4244.58		1	140	2	
	03 35 29.30 0150	Broom Finish	SF	20099	0.45	9044.35		3	630	11	
	07 81 16.10 0500	Fireproofing	SF	20098.55	1.35	27133.04	**549501.28**	2	1250	9	**46.00**
Ground	Item #	Decreption	Unit	Quantity	Total Cost per U	Cost		# of Crews	Daily output per crew	# of Days	
	03 21 10.60 0600	SOG Reinf	Ton	120.4208	1425.00	171599.6077		3.00	2.3	18	
	03 31 05.35 0300	4000 psi conc.	CY	385.6296	108.00	41648			N/A	0	
	03 31 05.70 4650	Slab on Grade	CY	385.6296	14.81	5711.174815		1	185	3	
	03 35 29.30 0150	Finishing	S.F.	20824	0	9370.8	**228329.58**	3.00	630	12	**33.00**
						Total Cost	**$2,848,700.43**			**Days**	**265**



ID	Task Name	Duration	Start	Finish	Predecessors	Resource Names
1	Ground: SOG Reinforcing	18 days	Mon 3/1/10	Wed 3/24/10		
2	Ground: Concrete	[illegible]	[illegible]	Fri 4/9/10	1	
3	Ground: Finishing	12 days	Thu 3/25/10	Fri 4/9/10	1	
4	2nd: Structural Steel	11 days	Thu 3/18/10	Thu 4/1/10	1FS-5 days	
5	2nd: Metal Decking	7 days	Fri 4/2/10	Mon 4/12/10	4	
6	2nd: WWF	6 days	Tue 4/6/10	Tue 4/13/10	5FS-5 days	
7	2nd: Concrete	11 days	Wed 4/14/10	Wed 4/28/10	6	
8	2nd: Finishing	11 days	Wed 4/14/10	Wed 4/28/10	6	
9	2nd: Fire Proofing	[illegible]	[illegible]	[illegible]	8	
10	3rd: Structural Steel	11 days	Fri 4/2/10	Fri 4/16/10	4	
11	2nd: Metal Decking	7 days	Mon 4/19/10	Tue 4/27/10	10	
12	3rd: WWF	6 days	Wed 4/21/10	Wed 4/28/10	11FS-5 days	
13	3rd: Concrete	11 days	Thu 4/29/10	Thu 5/13/10	12	
14	3rd: Finishing	11 days	Thu 4/29/10	Thu 5/13/10	12	
15	3rd: Fire Proofing	9 days	Fri 5/14/10	Wed 5/26/10	14	
16	4th: Structural Steel	11 days	[illegible]	[illegible]	[illegible]	
17	4th: Metal Decking	7 days	Tue 5/4/10	Wed 5/12/10	16	
18	4th: WWF	6 days	Thu 5/6/10	Thu 5/13/10	17FS-5 days	
19	4th: Concrete	11 days	Fri 5/14/10	Fri 5/28/10	18	
20	4th: Finishing	11 days	Fri 5/14/10	Fri 5/28/10	18	
21	4th: Fire Proofing	9 days	Mon 5/31/10	Thu 6/10/10	20	
22	5th: Structural Steel	11 days	Tue 5/4/10	Tue 5/18/10	16	
23	5th: Metal Decking	7 days	[illegible]	[illegible]	22	
24	5th: WWF	6 days	Fri 5/21/10	Fri 5/28/10	23FS-5 days	
25	5th: Concrete	11 days	Mon 5/31/10	Mon 6/14/10	24	
26	5th: Finishing	11 days	Mon 5/31/10	Mon 6/14/10	24	
27	5th: Fire Proofing	9 days	Tue 6/15/10	Fri 6/25/10	26	
28	Roof: Structural Steel	11 days	Wed 5/19/10	Wed 6/2/10	22	
29	Roof: Metal Decking	7 days	Thu 6/3/10	Fri 6/11/10	28	
30	Roof: WWF	[illegible]	[illegible]	[illegible]	[illegible]	
31	Roof: Concrete	11 days	Tue 6/15/10	Tue 6/29/10	30	
32	Roof: Finishing	11 days	Tue 6/15/10	Tue 6/29/10	30	
33	Roof: Fire Proofing	9 days	Wed 6/30/10	Mon 7/12/10	31	

Appendix I: Mechanical Coordination

- Duct 1: 20(47 L/s)
  3(6 L/s)
  2(1 L/s)
  2(147 L/s)
  2(52 L/s)
  3(28 L/s)
  4(99 L/s)
  1598 L/s → 3385.97 cfm

- Duct 2: 5(4)(47 L/s)
  5(2 L/s)
  (4 L/s)
  (38 L/s)
  992 L/s → 2101.93 cfm

- Velocity choice
  4 m/s: $4\ m/s\ (3.2808\ ft/m)\left(\frac{60\ s}{1\ min}\right) = 787.39\ ft/s$
  5 m/s: $5\ m/s\ (3.2808\ ft/m)(60) = 984.2\ ft/s$
  6 m/s: $6\ m/s\ (3.2808)(60) = 1181.1\ ft/s$

- Req'd Duct Area
  Duct 1: $A = Q/V = 3385.97/984.2 = 3.44\ ft^2$ → 5 m/s
  $= 3385.97/1181.1 = 2.867\ ft^2$ → 6 m/s

  Duct 2: $A = Q/V = 2101.93/984.2 = 2.14\ ft^2$ → 5 m/s
  $= 2101.93/1181.1 = 1.78\ ft^2$ → 6 m/s

Mechanical Cordination Breadth

Current Sizes

Duct 1 - (750mm x 400mm), (29.53" x 15.75")

Duct 2 - (650mm x 300mm), (25.59" x 11.81")

air flow takeoff

Duct 1 - 3385.97 cfm

Duct 2 2101.93 cfm

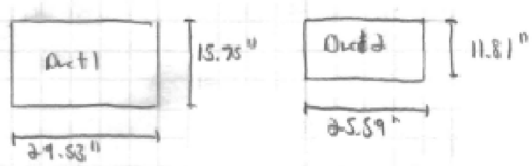
required area

Duct 1 : 2.867 ft²

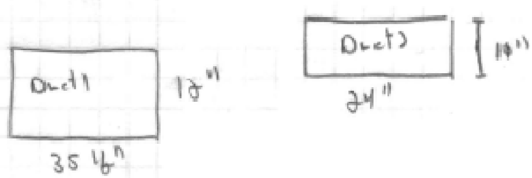
Duct 2 : 1.78 ft²

required height by new W18x35 d=17.7" floor system

- Original Design



- redesign



Appendix J: Progressive Collapse

Table 3-4. Load Increase Factors for Linear Static Analysis

Material	Structure Type	Ω_{LD}, Deformation-controlled	Ω_{LF}, Force-controlled
Steel	Framed	$0.9\ m_{LIF} + 1.1$	2.0
Reinforced Concrete	Framed[A]	$1.2\ m_{LIF} + 0.80$	2.0
	Load-bearing Wall	$2.0\ m_{LIF}$	2.0
Masonry	Load-bearing Wall	$2.0\ m_{LIF}$	2.0
Wood	Load-bearing Wall	$2.0\ m_{LIF}$	2.0
Cold-formed Steel	Load-bearing Wall	$2.0\ m_{LIF}$	2.0

[A] Note that, per ASCE 41, reinforced concrete beam-column joints are treated as force-controlled; however, the hinges that form in the beam near the column are deformation-controlled and the appropriate m-factor from Chapter 4 of this UFC shall be applied to the calculation of the deformation-controlled load increase factor Ω_{LD}.

Table 3-5. Dynamic Increase Factors for Nonlinear Static Analysis

Material	Structure Type	Ω_N
Steel	Framed	$1.08 + 0.76/(\theta_{pra}/\theta_y + 0.83)$
Reinforced Concrete	Framed	$1.04 + 0.45/(\theta_{pra}/\theta_y + 0.48)$
	Load-Bearing Wall	2
Masonry	Load-bearing Wall	2
Wood	Load-bearing Wall	2
Cold-formed Steel	Load-bearing Wall	2

Connection Type	Linear Acceptance Criteria	
	m-factors	
	Primary[(1)]	Secondary[(1)]
Fully Restrained Moment Connections		
Improved WUF with Bolted Web	2.3 – 0.021d	4.9 – 0.048d
Reduced Beam Section (RBS)	4.9 – 0.025d	6.5 – 0.025d
WUF	4.3 – 0.083d	4.3 -0.048d
SidePlate®	6.7 – 0.039d[(2)]	11.1 – 0.062d
Partially Restrained Moment Connections (Relatively Stiff)		
Double Split Tee		
a. Shear in Bolt	4	6
b. Tension in Bolt	1.5	4
c. Tension in Tee	1.5	4
d. Flexure in Tee	5	7
Partially Restrained Simple Connections (Flexible)		
Double Angles		
a. Shear in Bolt	5.8 – 0.107d_{bg}[(3)]	8.7 – 0.161d_{bg}
b. Tension in Bolt	1.5	4
c. Flexure in Angles	8.9 – 0.193d_{bg}	13.0 – 0.290d_{bg}
Simple Shear Tab	5.8 – 0.107d_{bg}	8.7 – 0.161d_{bg}

[(1)] Refer to Section 3-2.4 for determination of Primary and Secondary classification
[(2)] d = depth of beam, inch
[(3)] d_{bg} = depth of bolt group, inch

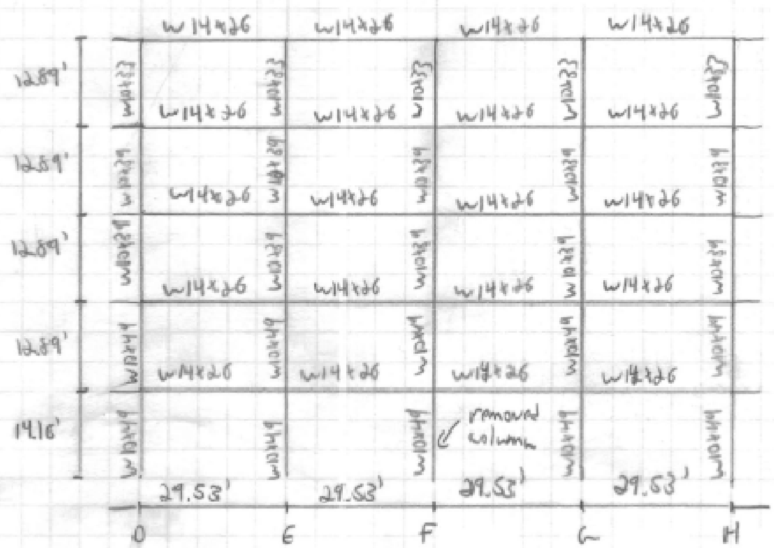
Table 5-2. Modeling Parameters and Acceptance Criteria for Nonlinear Modeling of Steel Frame Connections

Connection Type	Nonlinear Modeling Parameters[1]			Nonlinear Acceptance Criteria	
	Plastic Rotation Angle, radians		Residual Strength Ratio	Plastic Rotation Angle, radians	
	a	*b*	*c*	Primary[2]	Secondary[2]
Fully Restrained Moment Connections					
Improved WUF with Bolted Web	0.021 - 0.0003d	0.050 - 0.0006d	0.2	0.021 - 0.0003d	0.050 - 0.0006d
Reduced Beam Section (RBS)	0.050 - 0.0003d	0.070 - 0.0003d	0.2	0.050 - 0.0003d	0.070 - 0.0003d
WUF	0.0284 - 0.0004d	0.043 - 0.0006d	0.2	0.0284 - 0.0004d	0.043 - 0.0006d
SidePlate®	0.089 - 0.0005d[3]	0.169 - 0.0001d	0.6	0.089 - 0.0005d	0.169 - 0.0001d
Partially Restrained Moment Connections (Relatively Stiff)					
Double Split Tee					
a. Shear in Bolt	0.036	0.048	0.2	0.03	0.040
b. Tension in Bolt	0.016	0.024	0.8	0.013	0.020
c. Tension in Tee	0.012	0.018	0.8	0.010	0.015
d. Flexure in Tee	0.042	0.084	0.2	0.035	0.070
Partially Restrained Simple Connections (Flexible)					
Double Angles					
a. Shear in Bolt	$0.0502 - 0.0015d_{bg}$[4]	$0.072 - 0.0022d_{bg}$	0.2	$0.0502 - 0.0015d_{bg}$	$0.0503 - 0.0011d_{bg}$
b. Tension in Bolt	$0.0502 - 0.0015d_{bg}$	$0.072 - 0.0022d_{bg}$	0.2	$0.0502 - 0.0015d_{bg}$	$0.0503 - 0.0011d_{bg}$
c. Flexure in Angles	$0.1125 - 0.0027d_{bg}$	$0.150 - 0.0036d_{bg}$	0.4	$0.1125 - 0.0027d_{bg}$	$0.150 - 0.0036d_{bg}$
Simple Shear Tab	$0.0502 - 0.0015d_{bg}$	$0.072 - 0.0022d_{bg}$	0.2	$0.0502 - 0.0015d_{bg}$	$0.1125 - 0.0027d_{bg}$

(1) Refer to Figure 3-6 for definition of nonlinear modeling parameters *a*, *b*, and *c*
(2) Refer to Section 3-2.4 for determination of Primary and Secondary classification
(3) d = depth of beam, inch
(4) d_{bg} = depth of bolt group, inch

Appendix I: Progressive Collapse Design

Progressive Collaps Design
GSA - AP Method



Load Comb: 2(1.0DL + .25LL)

DD Floor: 65 psf
Roof: 95 psf
LL Floor: 80 psf
Roof: 20 psf

Extend of collapse

2(29.53)27.559 = 1627.6 ft² ≤ 1800 ft   use 1627 → 1680 ft²

Loads:

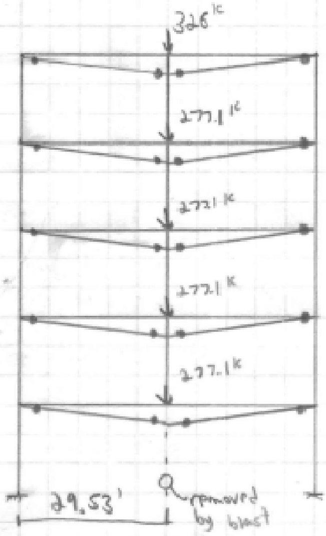
Typical Floor

2[.065 + .25(.08)] = .17 ksf (1680) = 277.1 k

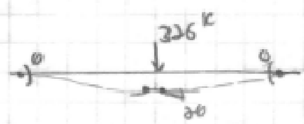
Roof

2[.095 + .25(.02)] = .2 ksf (1680) = 326 k

Progressive Collapse Design



Level Roof - Plastic Analysis (Virtual Work)



$M_p\theta + 2\theta M_p + 2M_p\theta + M_p\theta = P(29.53(\theta))$

$6M_p\theta = 29.53(326)\theta$

$M_p = 1604.5$ ft-k

Acceptance Criteria $DCR = \frac{Q_{UD}}{Q_{CE}} = \frac{Acting}{Expe} = 3$

$Q_{CE} = \frac{1604.5}{3} = 534.8$ ft-k ⇒ W21x62 $\phi M_p = 540$ ft-k $L_p = 6.25'$

W21x62 $b_f = 8.24"$, $t_f = .615$, $h/t_w = 46.9$

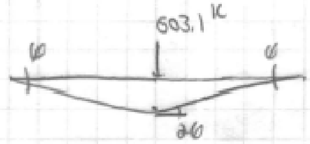
Flange slender $\frac{8.24}{2(.615)} \leq \frac{52}{\sqrt{50}}$ $6.7 \leq 7.0$ ok

Web slender $46.9 \leq \frac{418}{\sqrt{55}} = 56.36$ ok

W21x62 adequate

2

Level 5- Plastic Analysis



$6M_p\theta = P(29.53\,\theta)$

$6M_p\theta = (603.1)(29.53\,\theta)$

$M_p = 2968.3$ 'k

Acceptance Criteria DCR = 3

$Q_{CE} = \frac{2968.3}{3} = 989.4$ 'k ⇒ W30x90 $\phi M_p = 1060$ 'k $L_p = 7.38$'

W30x90 $b_f = 10.4$ $t_f = .61$ $h/t_w = 57.5$

Flange slenderness: $\frac{10.24}{2(.61)} = 8.39 > 7.07$ no good ∴ DCR = 2

$Q_{CE} = \frac{2968.3}{2} = 1484.1$ 'k ⇒ W33x118 $\phi M_p = 1560$ 'k $L_p = 8.19$'

W33x118 $t_f = .74$ $b_f = 11.5$ $h/t_w = 54.5$

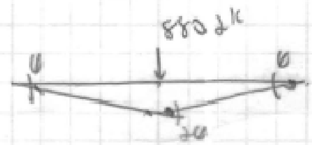
Flange slenderness: $\frac{11.5}{2(.74)} = 7.85 < 8.76$ no good

web slender $54.5 \geq \frac{640}{\sqrt{55}} = 86.3$ no good

* interpolate for DCR, DCR = 2 conservative

W33x118 adequate

Level 4: Plastic Analysis



$6M_p\theta = 880.2(29.53)(\theta)$

$\phi M_p = 4298.3$ 'k

$\theta_{C6} = \frac{4298.3}{3} = 1432.8$ 'k → W24x146 → $\phi M_p = 1570$ 'k $L_p = 10.6$'

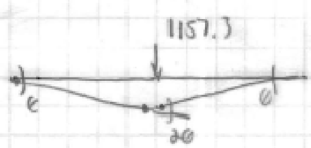
W24x146 $b_f = 12.9$ $t_f = 1.09$ $h/t_w = 33.2$

flange $\frac{12.9}{2(1.09)} = 5.92 < \frac{52}{\sqrt{55}} = 7.01$

web $33.2 < \frac{418}{\sqrt{55}} = 56.4$ ok

W24x146 adequate

Level 3: Plastic Analysis



$6M_p\theta = 1157.3(29.53)\theta$

$M_p = 5695.8$ 'k

$\theta_{C6} = \frac{5695.8}{3} = 1898.6$ 'k → W33x141 $\phi M_p = 1930$ $L_p = 8.58$'

$b_f = 11.5$ $t_f = .96$ $h/t_w = 49.6$

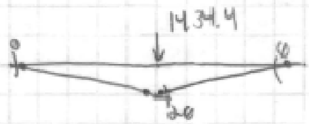
flange $\frac{b_f}{2t_f} = \frac{11.5}{2(.96)} = 5.99 \leq \frac{52}{\sqrt{55}} = 7.01$ ok

web $\frac{h}{t_w} = 49.6 < \frac{418}{\sqrt{55}} = 56.4$ ok

W33x141 adequate

Progressive Collapse Design

Level 2: Plastic Analysis



$6M_p\theta = 1434.4\,(29.53\,\theta)$

$M_p = 7059.6$ ft-k

$Q_{CE} = \frac{7059.6}{3} = 2353.2$ ft-k → W33x169 $\phi M_p = 2360$ ft-k $L_p = 8.83$

$b_f = 11.5$ $t_f = 1.22$ $h/t_w = 44.7$

Flange $\frac{b_f}{2t_f} = \frac{11.5}{2(1.22)} = 4.71 < 7.21$ ok

Web $\frac{h}{t_w} = 44.7 < 56.4$ ok

W33x169 adequate

For moment connection @ Level 2

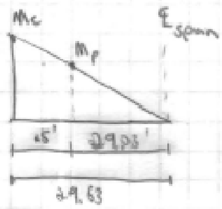
DCR = 2

$M_p = 7059.6$ $Q_{CE} = \frac{7059.6}{2} = 3529.8$ ft-k

Progressive Collapse Design - GSA

Column Design / check - 2nd Floor

- Moment transferred from Beam



$\frac{M_p}{29.03} = \frac{M_c}{29.53}$

$M_c = \frac{29.53}{29.03}(7059.6) = 7181.2^{\prime k}$

Assume DCR = 1, $P/P_c > .5$

Axial Load in column ≈ 1434.4/2 = 717.2 k from collapse loading

DCR = 2

W14x600 $p \times 10^3 = .171$ $b_x \times 10^3 = .226$

$\frac{.171(717.2)}{1000} = .123 < .2$ H1-a

7181.2/2

$.5(.123) + 9/8(.226/1000)(3590.6) = .974 \leq 1.0$

$b_f = 17$ $t_f = 3.50$ h/t_w 5.21

Flange: $\frac{b_f}{2t_f}$, $\frac{17}{2(3.50)} = 2.43 < \frac{52}{\sqrt{55}} = 7.01$ ok

$\frac{h}{t_w} = 5.21 < \frac{300}{\sqrt{F_y}} = \frac{300}{\sqrt{55}} = 40.5$ ok

DCR of 2 ok

use W14x500