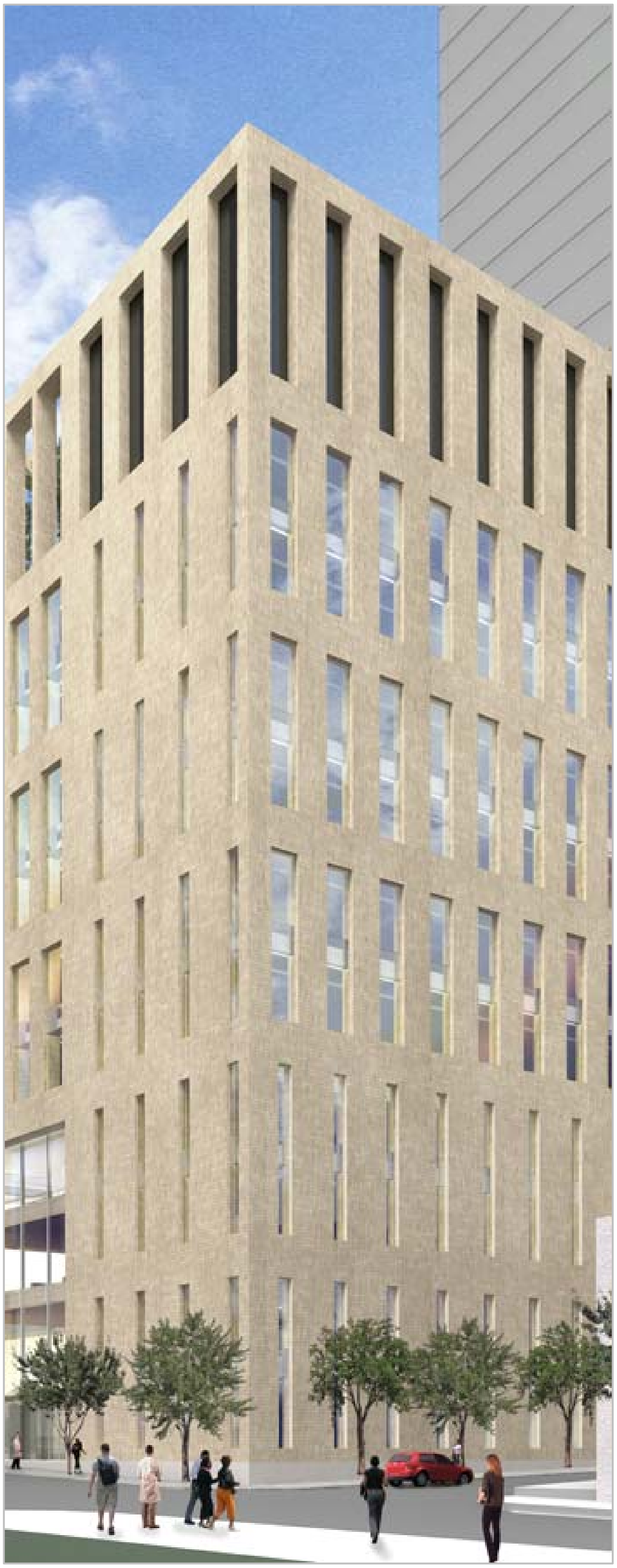
center for science & medicine

new york, ny



Technical Assignment 1

October 5, 2007

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Structural Option
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Table of Contents

Executive Summary 2

Structural Systems

- Foundation 3
- Floor System 3
- Roof System 4
- Columns 4
- Lateral System 4

Code and Design Requirements

- Codes and References 5
- Deflection Criteria 5
- Vibration Criteria 5

Gravity and Lateral Loads

- Dead / Live Loads 6
- Wind Loads 7
- Seismic Loads 9
- Simplified Analysis of Lateral System 10

Spot Checks

- Gravity Columns 10
- Composite Beam 10

Appendix 11

Executive Summary

The purpose of this report is to assess the existing conditions of the Center for Science & Medicine and to understand the procedures used in its structural design.

The Center for Science & Medicine is a research lab designed for the dual mission of investigation and discovery as well as treatment and healing. Located in New York City's Upper Manhattan, the building is organized and shaped by this thematic double program. On the north and south edges of the site, two linear lab bars encompass a core of support spaces. The building's east edge has been designed as an almost seamless extension of the busy street below, and rises from the public realm as an engaging 4-story Atrium. Situated within the building are 6 additional floors of wet lab research space, 1½ floors of clinical space, a clinical trial area, and space for research imaging. A 40-story residential tower will also rise on the site adjacent to the lab, but the buildings are clearly defined as two separate entities.

It is important to note that the Center for Science & Medicine, or CSM, is only at the 50% design development phase. Thus, the existing structural design and calculated quantities are not absolute or finalized.

The following report will examine existing structural designs as well as discuss the results of self-generated calculations. All diagrams, assumptions, code references, calculations, and computer outputs are included in the Appendix of this report.

Structural Systems

Foundation

The foundation will consist of reinforced concrete spread footings ranging from 4'x4'x2' to 8'x8'x4' (l x w x h) in size, with a concrete compressive strength of $f'_c = 5000$ psi. Maximum footing depth will be 49'-0" below grade, and all footings will bear on sound bedrock (Class 2-65 rock with bearing capacity 40TSF or Class 1-65 rock with bearing capacity 60TSF, according to New York City Building Code). Seven (7) of the total forty-three (43) footings will be designed to support columns from both the research center and the residential tower, as dictated by their location at the CSM / tower interface. Foundation loads vary from 400 to 3200 kips.

Below grade perimeter walls will consist of cast-in-place, reinforced concrete ($f'_c = 5000$ psi) braced by the below-grade floor slabs. These walls are designed to resist lateral loads from soil and surcharge in addition to the vertical loads transferred from perimeter columns above. On the north and south perimeter walls, reinforced concrete pilasters will support perimeter columns above. A continuous grade beam ($f'_c = 5000$ psi) will be constructed under these perimeter basement walls.

The lowest level basement floor will be an 8" concrete slab on grade with a compressive strength of $f'_c = 4000$ psi, typically reinforced with #5 bars@12" each way. At typical columns, additional slab reinforcement will be provided with (4)#4 bars oriented diagonally in the horizontal plane around the column base. At diagonal frame columns (located around the building core), the slab will be reinforced with (12)#5 bars oriented diagonally with additional longitudinal bars arranged in a grid pattern around the column base.

Floor System

The research center's floor slabs will typically consist of 3" metal deck with 4 ¾" normal-weight concrete topping, giving a total slab depth of 7 ¾". Thicker, normal-weight concrete slabs will be provided in spaces such as mechanical floors to meet acoustic and vibration criteria. These thickened slabs will be designed with 3" metal deck and 8" NWT concrete topping with reinforcement, giving a total slab depth of 11". Full composite action is created by 6" long, ¾" diameter shear studs, and concrete compressive strength is to be $f'_c = 4000$ psi. The composite metal deck is supported by wide flange steel beams ranging from W12x14 to W36x150 in size and spaced approximately 10'-6" on center. Typical bay sizes are roughly 21'x21' within the building core and approximately 21'x43' elsewhere.

Roof System

The flat roof system is similar to a typical floor slab, consisting of 3" metal roof deck with 4 ¾" normal weight reinforced concrete topping and 6"x ¾" shear studs. Supporting this deck are wide flange steel beams ranging from W12x14 to W36x150 in size and spaced approximately 10'-6" on center. It is also important to note that a portion of the roof will be a green roof, but design has not progressed enough to gather significant detail at this time.

Lateral System

Lateral resistance to wind and seismic loads is provided by a combination of braced and moment resisting steel frames. In the North/South direction, lateral loads are resisted by a system of diagonally-braced frames around the service core area of the building's interior. The core is made up of (6) column bays spaced at approximately 20'x20' and using W14 column sections. Heavy double tee bracing sections provide the lateral resistance at the core and vary from WT6x39.5 to WT6x68 in size.

In the East/West direction, lateral loads are taken by a dual system of perimeter beam/column moment frames and the diagonally-braced frame around the service core. Thus, it is assumed that the moment frames in this system are capable of resisting 25% of the design lateral forces. These moment frames have been designed to use W14 or W24 column sections spaced approximately 21'-0" on center and W30 wide flange beams. The frames first occur on the third level and then alternate levels up through the building's roof (a total of five floors with moment frames).

Columns

The research center's columns have been designated ASTM A992 Grade 50 steel and placed in a rectangular grid pattern. Typical gravity columns range from W14x61 to W14x311 in size. Columns acting as part of a moment frame are typically W24x117 to W36x256 in size.

Code & Design Requirements

Applicable design standards

New York City Building Code

International Building Code 2003

ACI 318-99 (Reinforced Concrete Design)

AISC ASD-89 (Structural Steel)

AISC LRFD-2002, 3rd Edition (Structural Steel)

ASCE 7-98

*Code substituted for thesis design: ASCE 7-05

Deflection Criteria

Floor to Floor Deflection

Typical live load deflection	L/360
Typical total deflection	L/240
Typical exterior spandrel deflection	½"

Lateral Deflection

Wind allowable inter-story drift	H/500
Seismic allowable story drift	H/400

Vibration Criteria

Imaging rooms / laboratories	2000 Micro inches / sec
Patient rooms	4000 Micro inches / sec
Offices / seminar rooms	8000 Micro inches / sec

Gravity Loads

Below is a table summarizing the load values of the structural designer and of IBC 2003 (which references ASCE 7-05).

Floor / Description		Design Dead Load	Design Live Load	IBC Live Load	Vibration Velocity
SC1 & SC 2					
·	Vivarium	30 psf	50 psf	-	2000 μin/s
·	Stair	5 psf	100 psf	100 psf	-
SC1 & SC2 Interstitial					
·	Mechanical Service	10 psf	50 psf	-	-
·	Stair	5 psf	100 psf	100 psf	-
Level 1					
·	Lobbies, Corridors	110 psf	100 psf	100 psf	-
·	Office	30 psf	50 psf	50 psf	8000 μin/s
·	Glass Wash	10 psf	125 psf	-	2000 μin/s
·	Stair	5 psf	100 psf	100 psf	-
Level 2					
·	Wet Lab	25 psf	100 psf	-	2000 μin/s
·	Loading Dock	75 psf	250 psf	250 psf	-
·	Auditorium	40 psf	60 psf	60 psf	-
·	Stair	5 psf	100 psf	100 psf	-
Level 3					
·	Wet Lab	25 psf	100 psf	-	2000 μin/s
·	Stair	5 psf	100 psf	100 psf	-
Level 4					
·	Lobbies, Corridors	110 psf	100 psf	100 psf	-
·	Office	30 psf	50 psf	50 psf	8000 μin/s
·	Stair	5 psf	100 psf	100 psf	-
Levels 5 - 10					
·	Office	30 psf	50 psf	50 psf	8000 μin/s
·	Wet Lab	25 psf	100 psf	-	2000 μin/s
·	Stair	5 psf	100 psf	100 psf	-
Level 11					
·	Roof Terrace	235 psf	100 psf	100 psf	-
·	Mechanical	80 psf	125 psf	-	-
·	Stair	5 psf	100 psf	100 psf	-
Roof					
·	Green Roof	60 psf	100 psf	100 psf	-
·	Snow Load	-	30 psf	22 psf (see calcs)	-
Superimposed Loads					
·	Partitions	10-20 psf	-	-	-
·	CMEP	10 psf	-	-	-
·	Finishes / Screed	5-15 psf	-	-	-
·	Roofing Membrane / Insul.	10 psf	-	-	-

Wind Loads

Wind loads were calculated in accordance with ASCE 7-05, Chapter 6. I used the analytical method to examine lateral wind loads in the North/South direction as well as the East/West direction. Although a residential tower will eventually rise adjacent to the Center for Science & Medicine on its south side, I calculated wind pressures based on the absence of this tower to account for the time CSM will be standing alone on the site. I found the fundamental frequency of the building to be less than one, indicating that the structure is flexible rather than rigid. It is categorized as Exposure B due to its urban location. The building is not quite a square, with the N/S direction (200'-0") slightly longer than the E/W direction (172'-0"). Thus, wind controlled in the N/S direction. The Appendix contains loading diagrams and detailed hand calculations, which are summarized below.

Wind Loads N/S

B = 172'-0"

L = 200'-0"

Floor	hx	Pressures (psf)									Force (kips)	Shear (kips)	Moment (ft-k)
		N/S windward				N/S leeward				Total			
Roof	184	19.13	±	5.32 =	24.4	-11.00	±	5.32 =	-16.3	40.8	119.2	119.2	4,052.9
11	150	18.00	±	5.32 =	23.3	-11.00	±	5.32 =	-16.3	39.6	170.3	289.5	2,554.9
10	135	17.51	±	5.32 =	22.8	-11.00	±	5.32 =	-16.3	39.1	101.6	391.2	1,524.3
9	120	16.86	±	5.32 =	22.2	-11.00	±	5.32 =	-16.3	38.5	100.2	491.3	1,502.4
8	105	16.22	±	5.32 =	21.5	-11.00	±	5.32 =	-16.3	37.8	98.5	589.8	1,477.3
7	90	15.57	±	5.32 =	20.9	-11.00	±	5.32 =	-16.3	37.2	96.8	686.6	1,452.2
6	75	14.76	±	5.32 =	20.1	-11.00	±	5.32 =	-16.3	36.4	94.9	781.5	1,423.9
5	60	13.78	±	5.32 =	19.1	-11.00	±	5.32 =	-16.3	35.4	92.6	874.2	1,389.4
4	45	12.73	±	5.32 =	18.0	-11.00	±	5.32 =	-16.3	34.4	90.0	964.2	1,350.2
3	30	11.35	±	5.32 =	16.7	-11.00	±	5.32 =	-16.3	33.0	86.9	1051.0	1,303.1
2	15	9.24	±	5.32 =	14.6	-11.00	±	5.32 =	-16.3	30.9	82.4	1133.4	1,235.7
1	0										39.8	1173.3	0.0
										Base Shear =	1,173.3	M =	19,266.2

Wind Load E/W

B = 200'-0"

L = 172'-0"

Floor	hx	Pressures (psf)											Force (kips)	Shear (kips)	Moment (ft-k)
		E/W windward					E/W leeward					Total			
Roof	184	12.76	±	5.32	=	18.1	-7.33	±	5.32	=	-12.6	30.7	104.5	104.5	3,551.4
11	150	12.00	±	5.32	=	17.3	-7.33	±	5.32	=	-12.6	30.0	149.4	253.9	2,241.0
10	135	11.68	±	5.32	=	17.0	-7.33	±	5.32	=	-12.6	29.6	89.4	343.3	1,341.1
9	120	11.24	±	5.32	=	16.6	-7.33	±	5.32	=	-12.6	29.2	88.3	431.5	1,324.1
8	105	10.81	±	5.32	=	16.1	-7.33	±	5.32	=	-12.6	28.8	87.0	518.5	1,304.6
7	90	10.38	±	5.32	=	15.7	-7.33	±	5.32	=	-12.6	28.3	85.7	604.2	1,285.2
6	75	9.84	±	5.32	=	15.2	-7.33	±	5.32	=	-12.6	27.8	84.2	688.4	1,263.3
5	60	9.19	±	5.32	=	14.5	-7.33	±	5.32	=	-12.6	27.2	82.4	770.8	1,236.5
4	45	8.49	±	5.32	=	13.8	-7.33	±	5.32	=	-12.6	26.5	80.4	851.2	1,206.1
3	30	7.57	±	5.32	=	12.9	-7.33	±	5.32	=	-12.6	25.5	78.0	929.2	1,169.6
2	15	6.16	±	5.32	=	11.5	-7.33	±	5.32	=	-12.6	24.1	74.5	1003.7	1,117.3
1	0												36.2	1039.9	0.0
										Base Shear =			1,039.9	M =	17,040.2

Results:

Base Shear (N/S) = 1,173.3 k (controls)

Base Shear (E/W) = 1,039.9 k

Overturning Moment (N/S) = 19,266.2 'k (controls)

Overturning Moment (E/W) = 17,040.2 'k

Seismic Loads

Seismic loads were calculated in accordance with ASCE 7-05, Chapter 12. After careful study of the geotechnical report, I was able to conclude that the building subterranean site is primarily rock and falls under Site Class B. All other factors and accelerations were obtained from ASCE 7-05 figures, tables, and equations. To determine the effective weight of the building, I first calculated the weight of each of the building's twelve floors above grade. This included the exact weights of all slabs and columns, an approximation for beams / connections / bracing elements obtained from the construction documents, and the superimposed dead loads listed in the table on page (7). Summing the weights of each floor generated the building's effective weight, and in turn, seismic base shear. More extensive calculations and diagrams are shown in the Appendix.

Vertical Distribution of Seismic Forces

Floor	w_x (k)	h_x (ft)	h_x^k	$w_x h_x^k$	C_{vx}	F_x (k)	Moment at Floor (ft-k)
1							
2	4,018.5	15.0	74.1	297,886	0.005	9.2	137.5
3	3,214.5	30.0	223.2	717,353	0.011	22.1	662.2
4	2,983.0	45.0	425.2	1,268,417	0.020	39.0	1,756.4
5	3,461.6	60.0	671.8	2,325,622	0.037	71.6	4,293.9
6	3,457.2	75.0	958.0	3,311,892	0.052	101.9	7,643.5
7	3,453.9	90.0	1,280.1	4,421,378	0.070	136.1	12,244.9
8	3,450.7	105.0	1,635.7	5,644,135	0.089	173.7	18,236.6
9	3,427.6	120.0	2,022.5	6,932,432	0.109	213.3	25,599.0
10	3,423.5	135.0	2,439.1	8,350,167	0.131	257.0	34,688.5
11	5,154.2	150.0	2,883.9	14,864,371	0.234	457.4	68,611.1
Roof	3861.7611	184.0	3,990.8	15,411,530	0.243	474.2	87,261.0
		$\sum w_i h_i^k =$	63,545,182	$\sum F_x = V =$	1,955.4	$\sum M =$	261,134.7

Results:

Effective Seismic Weight = 39,906.4 k

Calculated Base Shear = 1,955.4 k

Thus, it is determined that seismic controls over wind.

Braced Frame Analysis

As previously discussed, the building's lateral system consists of diagonally braced frames in the North/South direction and a dual system in the East/West direction. I chose to analyze the North/South system of braced frames for simplicity. To carry out such an analysis, I built a model of the two N/S braced frames in RAM and applied a 1 kip load to every floor above ground level. After running the analysis, I used the calculated deflections to find the relative stiffness of each frame. Finally, these percentages were applied to previously calculated seismic story forces (which govern over wind loads) to determine how each frame will react to such lateral forces. Although this is an approximate method, I feel that it is a reasonable approach to analyzing the frames for my purposes.

A summary of lateral load distribution is displayed in the Appendix. RAM output is also included, along with elevations of each braced frame and the forces calculated at each level. Upon finishing the analysis, I was able to conclude that the selected WT members are satisfactory in resisting the design seismic load.

Spot Checks

The first spot-check performed was an evaluation of gravity columns located in one of the building's interior bays, from the lowest basement level (SC2) to the eleventh floor. Dead loads applied to each column were taken from earlier seismic calculations (weight of structural elements plus superimposed dead loads), and live loads were applied in accordance with IBC 2003 (which are equal to those specified by the original designer). It was assumed that the effective length, KL, of each column was equal to the column's floor-to-floor height. After performing the first calculation, I used the AISC LRDF Steel Manual to check all other columns (Table 4-1). Refer to calculations in the Appendix.

In general, I found that the columns seemed to be over-designed. Most of my calculations called for much smaller axial load capacities than what is provided by the current design. This may be due to personal error in load calculations, or it could be attributed to the stringent vibration criteria set up for the structure. My calculations did not take vibration into effect.

The second spot-check performed was an evaluation of a typical composite beam located in one of the building's interior bays. My calculations show that the beam is capable of supporting the applied factored moment, and the number of shear studs required for full composite action is equal to the number specified in the original design.

Appendix

A) Wind Load Calculations

Reference: ASCE 7-05

Wind Load (North/South) B = 172' L = 200'

Floor	Height (ft)	hx	Kz	qz	Pressures (psf)											Force (kips)	Shear (kips)	Moment (ft-k)
					N/S windward					N/S leeward					Total			
Roof	34	184	1.18	29.53	19.13	±	5.32	=	24.4	-11.00	±	5.32	=	-16.3	40.8	119.2	119.2	4,052.9
11	15	150	1.11	27.78	18.00	±	5.32	=	23.3	-11.00	±	5.32	=	-16.3	39.6	170.3	289.5	2,554.9
10	15	135	1.08	27.03	17.51	±	5.32	=	22.8	-11.00	±	5.32	=	-16.3	39.1	101.6	391.2	1,524.3
9	15	120	1.04	26.02	16.86	±	5.32	=	22.2	-11.00	±	5.32	=	-16.3	38.5	100.2	491.3	1,502.4
8	15	105	1.00	25.02	16.22	±	5.32	=	21.5	-11.00	±	5.32	=	-16.3	37.8	98.5	589.8	1,477.3
7	15	90	0.96	24.02	15.57	±	5.32	=	20.9	-11.00	±	5.32	=	-16.3	37.2	96.8	686.6	1,452.2
6	15	75	0.91	22.77	14.76	±	5.32	=	20.1	-11.00	±	5.32	=	-16.3	36.4	94.9	781.5	1,423.9
5	15	60	0.85	21.27	13.78	±	5.32	=	19.1	-11.00	±	5.32	=	-16.3	35.4	92.6	874.2	1,389.4
4	15	45	0.785	19.64	12.73	±	5.32	=	18.0	-11.00	±	5.32	=	-16.3	34.4	90.0	964.2	1,350.2
3	15	30	0.70	17.52	11.35	±	5.32	=	16.7	-11.00	±	5.32	=	-16.3	33.0	86.9	1051.0	1,303.1
2	15	15	0.57	14.26	9.24	±	5.32	=	14.6	-11.00	±	5.32	=	-16.3	30.9	82.4	1133.4	1,235.7
1	0	0														39.8	1173.3	0.0
															Base Shear =	1,173.3	M =	19,266.2

Wind Load (East/West) B = 200' L = 172'

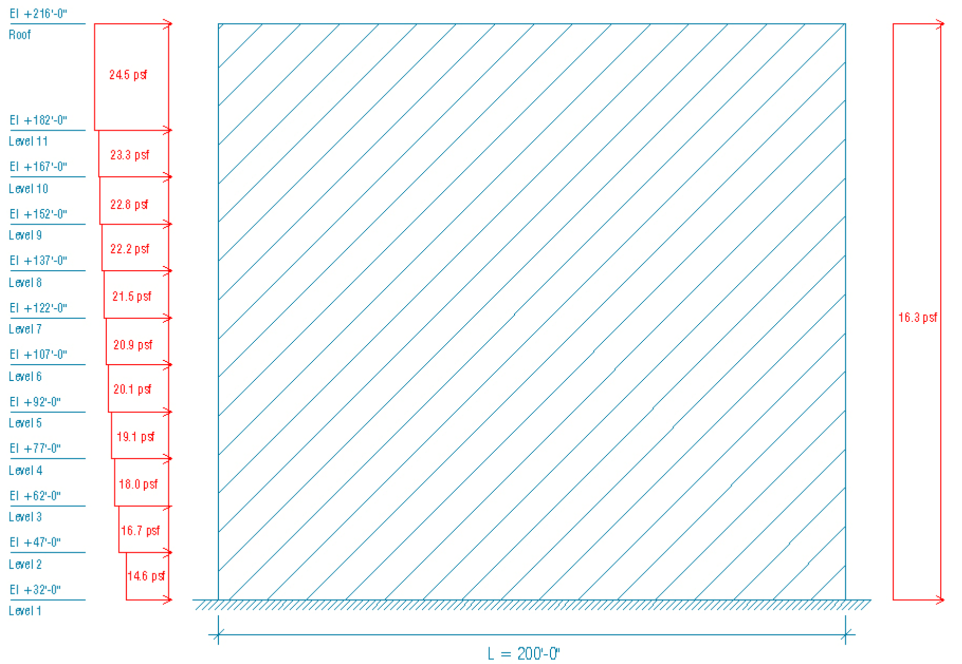
Floor	Height (ft)	hx	Kz	qz	Pressures (psf)											Force (kips)	Shear (kips)	Moment (ft-k)
					E/W windward					E/W leeward					Total			
Roof	34	184	1.18	29.53	12.76	±	5.32	=	18.1	-7.33	±	5.32	=	-12.6	30.7	104.5	104.5	3,551.4
11	15	150	1.11	27.78	12.00	±	5.32	=	17.3	-7.33	±	5.32	=	-12.6	30.0	149.4	253.9	2,241.0
10	15	135	1.08	27.03	11.68	±	5.32	=	17.0	-7.33	±	5.32	=	-12.6	29.6	89.4	343.3	1,341.1
9	15	120	1.04	26.02	11.24	±	5.32	=	16.6	-7.33	±	5.32	=	-12.6	29.2	88.3	431.5	1,324.1
8	15	105	1.00	25.02	10.81	±	5.32	=	16.1	-7.33	±	5.32	=	-12.6	28.8	87.0	518.5	1,304.6
7	15	90	0.96	24.02	10.38	±	5.32	=	15.7	-7.33	±	5.32	=	-12.6	28.3	85.7	604.2	1,285.2
6	15	75	0.91	22.77	9.84	±	5.32	=	15.2	-7.33	±	5.32	=	-12.6	27.8	84.2	688.4	1,263.3
5	15	60	0.85	21.27	9.19	±	5.32	=	14.5	-7.33	±	5.32	=	-12.6	27.2	82.4	770.8	1,236.5
4	15	45	0.785	19.64	8.49	±	5.32	=	13.8	-7.33	±	5.32	=	-12.6	26.5	80.4	851.2	1,206.1
3	15	30	0.70	17.52	7.57	±	5.32	=	12.9	-7.33	±	5.32	=	-12.6	25.5	78.0	929.2	1,169.6
2	15	15	0.57	14.26	6.16	±	5.32	=	11.5	-7.33	±	5.32	=	-12.6	24.1	74.5	1003.7	1,117.3
1	0															36.2	1039.9	0.0
															Base Shear =	1,039.9	M =	17,040.2

External Pressure Coefficients, CP
Windward 0.8
Leeward -0.46

Internal Pressure Coefficient, GCpi
±0.18

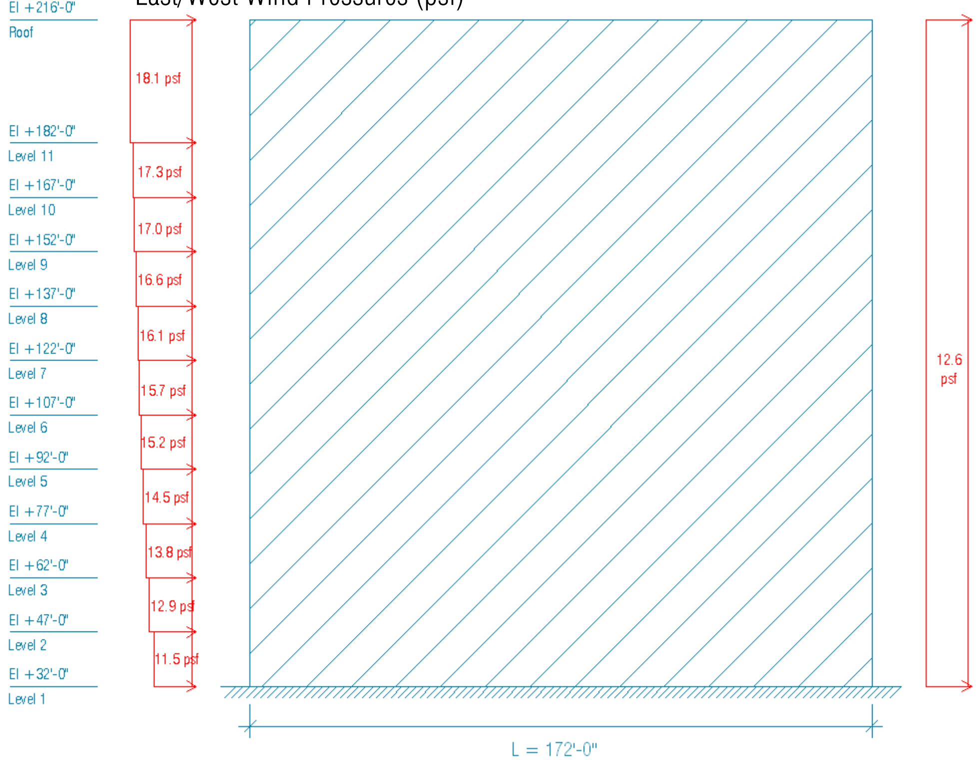
A) Wind Load Calculations (con)

North/South Wind Pressures (psf)



Gust Factor		
	N/S	E/W
L	200.00	172.00
B	172.00	200.00
n1	0.60	0.60
	Flexible	
h	184.00	184.00
0.6h	110.40	110.40
z_{min}	30.00	30.00
c	0.30	0.30
Iz	0.245	0.245
ℓ	320.00	320.00
z effective	110.40	110.40
Q	0.657	0.648
g_Q	3.40	3.40
g_V	3.40	3.40
R_n	0.067	0.067
R_h	0.16	0.16
R_B	0.17	0.15
R_L	0.047	0.055
β	0.05	0.05
R	0.142	0.134
G_f	0.81	0.54

East/West Wind Pressures (psf)



B) Seismic Calculations

Reference: ASCE 7-05

Seismic Design Values, ASCE 7-05

Occupancy	III	Table 1-1
Importance Factor	I = 1.25	Table 11.5-1
Site Class	B	Table 20.3-1
Spectral Response Acceleration, short	$S_s = 0.35$	Figure 22-1
Spectral Response Acceleration, 1 sec	$S_1 = 0.06$	Figure 22-2
Site Coefficient, F_a	$F_a = 1.0$	Table 11.4-1
Site Coefficient, F_v	$F_v = 1.0$	Table 11.4-2
MCE Spectral Response Acceleration, short	$S_{MS} = 0.35$	Eq. 11.4-1
MCE Spectral Response Acceleration, 1 sec	$S_{M1} = 0.06$	Eq. 11.4-2
Design Spectral Acceleration, short	$S_{DS} = 0.233$	Eq. 11.4-3
Design Spectral Acceleration, 1 sec	$S_{D1} = 0.04$	Eq. 11.4-4
Seismic Design Category	B	Table 11.6-1

North/South Direction: Concentrically Braced Fames (Special)	Response Modification Coefficient	R = 6	Table 12.2-1
	Coefficient C_u	$C_u = 1.7$	Table 12.8-1
	Fundamental Period, T	T = 1.68	Sec. 12.8.2
	Seismic Respose Coefficient	$C_s = 0.049$	Eq. 12.8-3
	Building Height (above grade)	h = 184'	
East/West Direction: Dual System (on odd # floors only)	Response Modification Coefficient	R = 7	Table 12.2-1
	Coefficient C_u	$C_u = 1.7$	Table 12.8-1
	Fundamental Period, T	T = 1.68	Sec. 12.8.2
	Seismic Respose Coefficient	$C_s = 0.042$	Eq. 12.8-3
	Building Height (above grade)	h = 184'	

Weight of each floor calculated as followed:

Floor 10					
Approx Area:	28,663	ft^2	Floor to Floor Height:	15 ft	
Slab:					
thickness =	4.75	in			
unit weight =	150	pcf			
total weight =	1,701.9	kips			
Columns:					
Shape	Quantity	Unit Weight (lb/ft)	Column Height (ft)	Total Weight	
W14x61	9	61	15	8.2	kips
W14x68	1	68	15	1.0	kips
W14x90	6	90	15	8.1	kips
W14x74	3	74	15	3.3	kips
W14x109	1	109	15	1.6	kips
W14x120	4	120	15	7.2	kips
W14x145	1	145	15	2.2	kips
W14x176	1	176	15	2.6	kips
W14x211	10	211	15	31.7	kips
W24x117	9	117	15	15.8	kips
W24x146	7	146	15	15.3	kips
W36x135	4	135	15	8.1	kips
W36x150	5	150	15	11.3	kips
total weight =	116.5	kips			
Beams, Connections, Bracing, etc:					
allowance =	11.0	psf			
total weight =	315.3	kips			
Super-Imposed:					
partitions =	20	psf			
CMEP =	10	psf			
Finishes =	15	psf			
total weight =	1,289.8	kips			
TOTAL FLOOR WEIGHT:			3,423.5 kips	or	119 psf

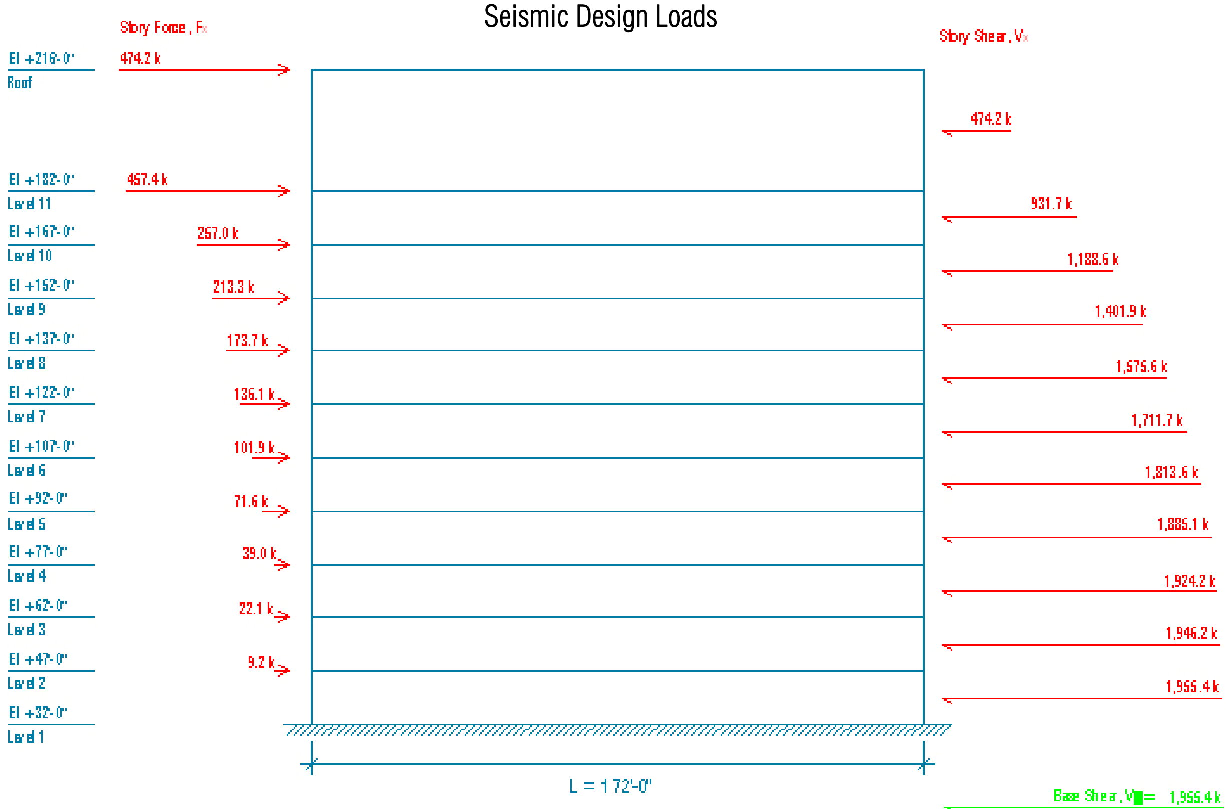
Floor 11					
Approx Area:	28,663	ft^2	Floor to Floot Height:	34 ft	
(Mezzanine additional 4,580 ft^2)					
Slab (Flr 11):					
thickness =	8	in			
unit weight =	150	pcf			
total weight =	2,866.3	kips			
Slab (Mezz):					
thickness =	8	in			
unit weight =	150	pcf			
total weight =	458.0	kips			
Columns:					
Shape	Quantity	Unit Weight (lb/ft)	Column Height (ft)	Total Weight	
W14x61	18	61	34	37.3	kips
W14x82	1	82	34	2.8	kips
W14x120	5	120	34	20.4	kips
W14x145	1	145	34	4.9	kips
W14x176	1	176	34	6.0	kips
W14x211	10	211	34	71.7	kips
W24x117	2	117	34	8.0	kips
W24x146	6	146	34	29.8	kips
W36x135	4	135	34	18.4	kips
W36x150	5	150	34	25.5	kips
total weight =	224.8	kips			
Beams, Connections, Bracing, etc:					
allowance =	11.0	psf			
total weight =	315.3	kips			
Super-Imposed:					
partitions =	20	psf			
CMEP =	10	psf			
Finishes =	15	psf			
total weight =	1,289.8	kips			
TOTAL FLOOR WEIGHT:			5,154.2 kips	or	180 psf

B) Seismic Calculations (con)

Vertical Distribution of Seismic Forces

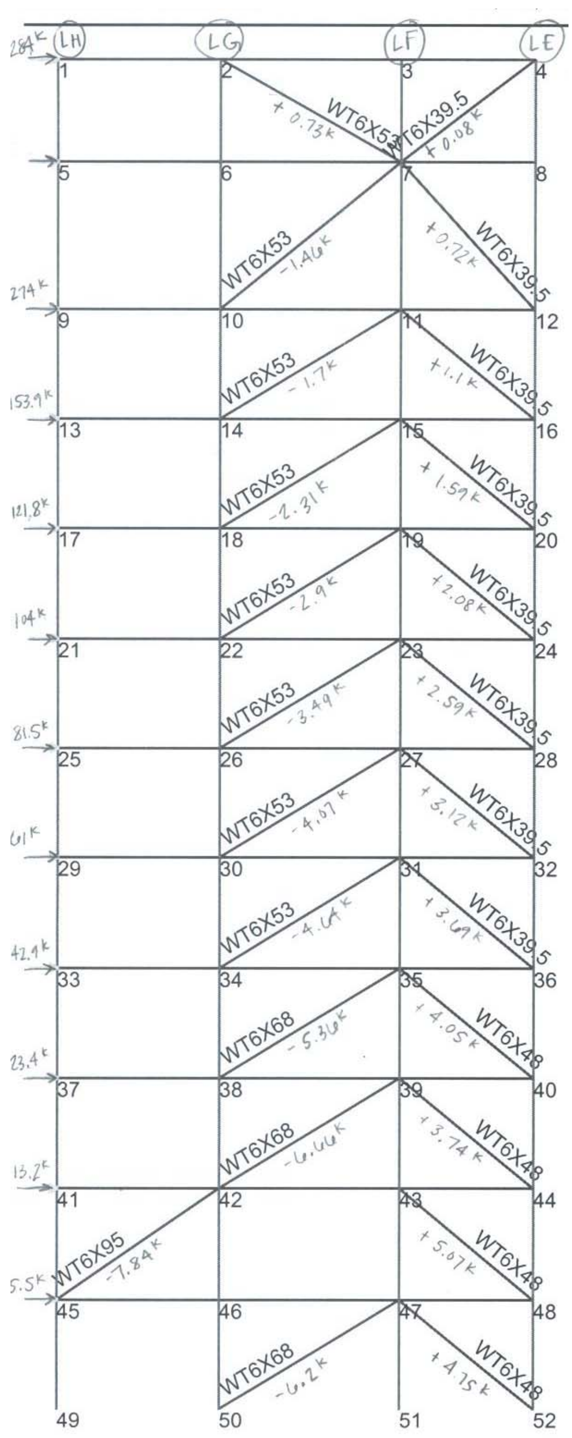
Floor	w_x (k)	h_x (ft)	h_x^k	$w_x h_x^k$	C_{vx}	Story Force F_x (k)	Story Shear V_x (k)	Moment at Floor (ft-k)
1							1,955.4	
2	4,018.5	15.0	74.1	297,886	0.005	9.2	1,946.2	137.5
3	3,214.5	30.0	223.2	717,353	0.011	22.1	1,924.2	662.2
4	2,983.0	45.0	425.2	1,268,417	0.020	39.0	1,885.1	1,756.4
5	3,461.6	60.0	671.8	2,325,622	0.037	71.6	1,813.6	4,293.9
6	3,457.2	75.0	958.0	3,311,892	0.052	101.9	1,711.7	7,643.5
7	3,453.9	90.0	1,280.1	4,421,378	0.070	136.1	1,575.6	12,244.9
8	3,450.7	105.0	1,635.7	5,644,135	0.089	173.7	1,401.9	18,236.6
9	3,427.6	120.0	2,022.5	6,932,432	0.109	213.3	1,188.6	25,599.0
10	3,423.5	135.0	2,439.1	8,350,167	0.131	257.0	931.7	34,688.5
11	5,154.2	150.0	2,883.9	14,864,371	0.234	457.4	474.2	68,611.1
Roof	3861.7611	184.0	3,990.8	15,411,530	0.243	474.2		87,261.0
		$\sum w_i h_i^k =$	63,545,182	$\sum F_x = V =$	1,955.4		$\sum M =$	261,134.7

Seismic Design Loads

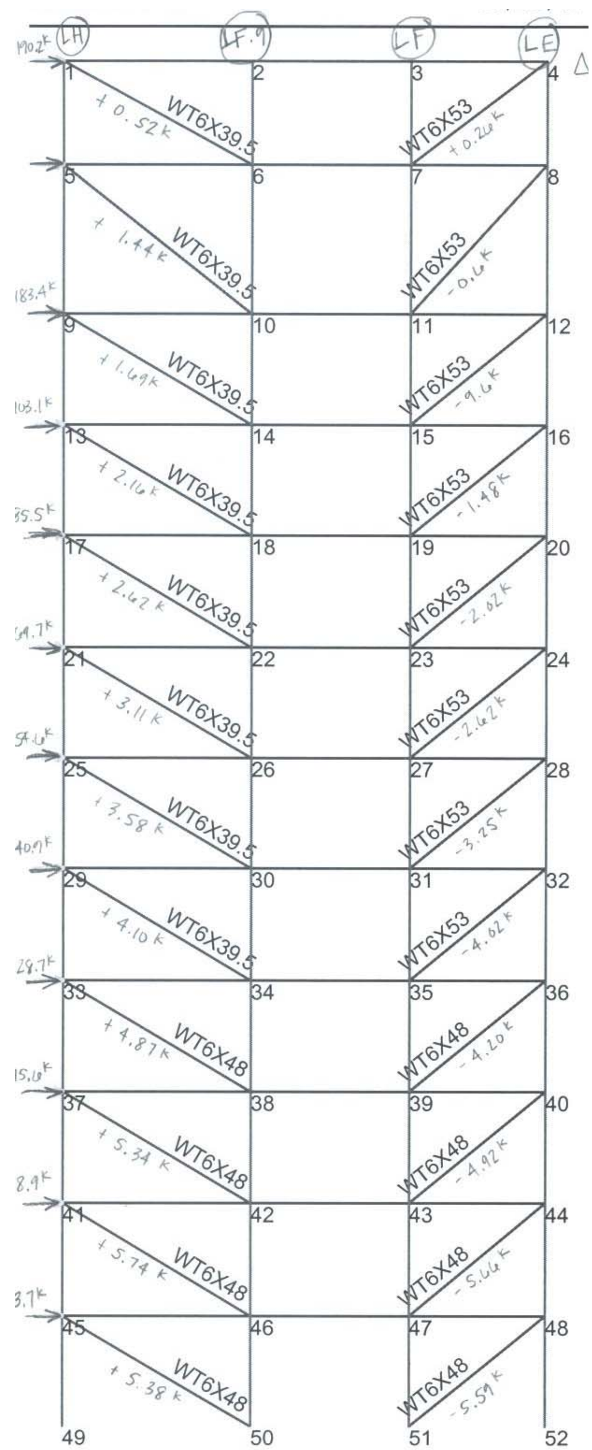


C) Simplified Lateral Analysis:

BF7 (North/South)



BF9 (North/South)



C) Simplified Lateral Analysis (con)

Lateral Distribution of Loads
North/South Direction

Percentage of Load Distributed to Frame, by Floor

Frame	1/Defl	2	3	4	5	6	7	8	9	10	11-M	11
BF7	12.99	59.9	59.9	59.9	59.9	59.9	59.9	59.9	59.9	59.9	59.9	59.9
BF9	8.7	40.1	40.1	40.1	40.1	40.1	40.1	40.1	40.1	40.1	40.1	40.1
(total)	21.69	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

Distribution of Seismic Load on BF7 and BF9
North/South Direction

Approximate Load on Each Frame Story, kips

Frame	1/Defl	2	3	4	5	6	7	8	9	10	11-M	11	Total Load
BF7	12.99	5.5	13.2	23.4	42.9	61.0	81.5	104.0	127.8	153.9	274.0	284.0	1171.3
BF9	8.7	3.7	8.9	15.6	28.7	40.9	54.6	69.7	85.5	103.1	183.4	190.2	784.2
(total)	21.69	9.2	22.1	39	71.6	101.9	136.1	173.7	213.3	257	457.4	474.2	1955.5

D) Spot Check: Gravity Column

Reference: AISC LRFD Steel Manual

Floor	Tributary Area (ft2)	Dead Load (psf)	Live Load (psf)	Influence Area (ft2)	Reduction Factor ≥ 0.4	Live Load (k)	Dead Load (k)	Load Combo	Total Load per Floor (k)	Total Accumulated Load (k)
11	806	140	100	3,224	1.000 = 1.000	80.6	112.8	$1.2D + 0.5L_r$	175.7	175.7
10	806	180	120	3,224	0.437 = 0.437	42.2	145.1	1.2D + 1.6L	241.7	417.3
9	806	119	100	3,224	0.403 = 0.403	32.4	95.9	1.2D + 1.6L	167.0	584.3
8	806	120	100	3,224	0.382 = 0.400	32.2	96.7	1.2D + 1.6L	167.6	751.9
7	806	120	100	3,224	0.368 = 0.400	32.2	96.7	1.2D + 1.6L	167.6	919.6
6	806	121	100	3,224	0.358 = 0.400	32.2	97.5	1.2D + 1.6L	168.6	1088.2
5	806	121	100	3,224	0.350 = 0.400	32.2	97.5	1.2D + 1.6L	168.6	1256.7
4	806	121	100	3,224	0.343 = 0.400	32.2	97.5	1.2D + 1.6L	168.6	1425.3
3	806	123	(1/2) 50 (1/2) 100	3,224	0.338 = 0.400	24.2	99.1	1.2D + 1.6L	157.6	1583.0
2	806	122	100	3,224	0.334 = 0.400	32.2	98.3	1.2D + 1.6L	169.6	1752.5
1	806	150	(1/2) 60 (1/2) 100	3,224	0.330 = 0.400	25.8	120.9	1.2D + 1.6L	186.3	1938.8
SC1-M	806	121	50	3,224	0.326 = 0.400	16.1	97.5	1.2D + 1.6L	142.8	2081.6
SC1	806	121	50	3,224	0.323 = 0.400	16.1	97.5	1.2D + 1.6L	142.8	2224.4
SC2-M	806	121	50	3,224	0.321 = 0.400	16.1	97.5	1.2D + 1.6L	142.8	2367.2
SC2	806	121	50	3,224	0.318 = 0.400	16.1	97.5	1.2D + 1.6L	142.8	2510.0

D) Spot Check: Gravity Column (con)

1/5

SPOT CHECK : GRAVITY COLUMN

Check Column LC/L6.
See spreadsheet for loadings.

FLOOR 11 : $P_u = 175.7^k$

W 14 × 61 , h = 34'

$A_g = 17.9\ in^2$

$I_x = 640\ in^4$ $\quad I_y = 107\ in^4$

$r_x = 5.98\ in$ $\quad r_y = 2.45\ in$

$\frac{KL}{r_x} = \frac{34'(12)}{5.98} = 68.2 \quad || \quad \frac{KL}{r_y} = \frac{34'(12)}{2.45} = 166.5$ ← CONTROLS.

$\frac{KL}{r} \leq 4.71\sqrt{E/F_y}$

$166.5 \leq 4.71\sqrt{29000/50} = 113.4$ No, ∴ elastic behavior.

$F_{CR} = 0.877F_e = 0.877\left[\frac{\pi^2 \cdot 29000}{166.5^2}\right] = 9.05\ ksi$

$P_n = F_{CR}A_g = 9.05(17.9) = 162^k$

$\phi P_n = 0.9(162) = 145.9^k < P_u = 175.7^k$ No good.

Note: Using Table 4-1 (AISC Steel Manual, 13th Ed) gives the following results :

$P_u = 175.7^k$

$K_L = 34'$

From Table 4-1, choose W14 × 74

$\phi P_n = 182^k > P_u = 175.7^k$ ✓ okay

∴ W 14 × 61 will not work; select W 14 × 74.

For all other floors, I will use Table 4-1 to check column designs since it is quicker and is based on the same methods as the calculations above.

D) Spot Check: Gravity Column (con)

SPOT CHECK : GRAVITY COLUMN 2/5

FLOOR 9
W 14 × 120

$P_u = 584.3^k$
$KL = 15'$

From Table 4-1, choose W 14 × 68

$\phi P_n = 608^k > P_u = 584.3^k$ ✓

For W 14 × 120,
$\phi P_n = 1340^k \gg P_u = 584.3^k$ ✓

∴ W14 × 120 design is satisfactory, but a smaller size could be used based on these calculations.

FLOOR 8
W 14 × 120

$P_u = 751.9^k$
$KL = 15'$

From Table 4-1, choose W 14 × 90

$\phi P_n = 1000^k > P_u = 751.9^k$ ✓

For W 14 × 20,
$\phi P_n = 1340^k \gg P_u = 751.9^k$ ✓

∴ W 14 × 120 design is satisfactory, but a smaller size could be used based on these calculations.

FLOOR 7
W 14 × 145

$P_u = 919.6^k$
$KL = 15'$

From Table 4-1, choose W 14 × 90

$\phi P_n = 1000^k > P_u = 919.6^k$ ✓

For W14×145,
$\phi P_n = 1650^k \gg P_u = 919.6^k$ ✓

∴ W 14 × 145 design is satisfactory, but a smaller size could be used based on these calculations.

FLOOR 6
W 14 × 159

$P_u = 1088.2^k$
$KL = 15'$

From Table 4-1, choose W 14 × 99
$\phi P_n = 1100^k > P_u = 1088.2^k$ ✓

For W 14 × 159, $\phi P_n = 1810 \gg P_u = 1088.2^k$ ✓

∴ W 14 × 159 is satisfactory, but a smaller size could be used based on gravity analysis.

D) Spot Check: Gravity Column (con)

SPOT CHECK : GRAVITY COLUMN

3/5

FLOOR 5
W 14 × 193

$P_u = 1256.7^k$
$KL = 15'$

From Table 4-1, chose W 14 × 120
$\phi P_n = 1340^k > P_u = 1256.7^k$ ✓

For W 14 × 193, $\phi P_n = 2210^k \gg P_u = 1256.7^k$ ✓

∴ W 14 × 193 design is satisfactory, but a smaller size could be used based on gravity analysis.

FLOOR 4
W 14 × 211

$P_u = 1425.3^k$
$KL = 15'$

From Table 4-1, choose W 14 × 132
$\phi P_n = 1480^k > P_u = 1425^k$ ✓

For W 14 × 211, $\phi P_n = 2420^k \gg P_u = 1425^k$

∴ W 14 × 211 design is satisfactory, but a smaller size could be used based on gravity analysis.

FLOOR 3
W 14 × 233

$P_u = 1583.0^k$
$KL = 15'$

From Table 4-1, choose W14 × 145
$\phi P_n = 1650^k > P_u = 1583^k$ ✓

For W 14 × 233, $\phi P_n = 2680^k \gg P_u = 1583^k$ ✓

∴ W 14 × 233 design is satisfactory, but a smaller size could be used based on gravity analysis.

FLOOR 2
W 14 × 257

$P_u = 1752.5^k$
$KL = 15'$

From Table 4-1, choose W14 × 159
$\phi P_n = 1810^k > P_u = 1753^k$ ✓

For W 14 × 257, $\phi P_n = 2960^k \gg P_u = 1753^k$ ✓

∴ W 14 × 257 is satisfactory, but a smaller size could be used based on gravity analysis.

D) Spot Check: Gravity Column (con)

4/5

SPOT CHECK : GRAVITY COLUMN

FLOOR 1
W 14 × 283

$P_u = 1938.8^k$
$KL = 15'$

FROM Table 4-1, choose W 14 × 176
$\phi P_n = 2010^k > P_u = 1938.8^k$ ✓

For W 14×283, $\phi P_n = 3270^k \gg P_u = 1938.8^k$ ✓

∴ W 14 × 283 design is satisfactory, but a smaller size could be used based on gravity analysis.

SC1 - M
W14 × 311

$P_u = 2081.6^k$
$KL = 11'$

FROM Table 4-1, choose W 14 × 176
$\phi P_n = 2150^k > P_u = 2081.6^k$ ✓

FOR W14 ×311, $\phi P_n = 3830^k \gg P_u = 2081.6^k$ ✓

∴ W 14 × 311 design is satisfactory, but a smaller size could be used based on gravity analysis.

SC 1
W 14 × 311

$P_u = 2224.4^k$
$KL = 13'$

From Table 4-1, choose W 14 × 193
$\phi P_n = 2290^k > P_u = 2224.4^k$ ✓

FOR W 14×311, $\phi P_n = 3720^k \gg P_u = 2224.4^k$ ✓

∴ W 14 × 311 design is satisfactory, but a smaller size could be used based on gravity analysis.

SC2 - M
W 14 × 342

$P_u = 2367.2^k$
$KL = 11'$

From Table 4-1, choose W 14 × 211
$\phi P_n = 2580^k > P_u = 2367.2^k$ ✓

FOR W 14 ×342, $\phi P_n = 4230^k \gg P_u = 2367.2^k$ ✓

∴ W 14 × 342 design is satisfactory, but a smaller size could be used based on gravity analysis.

D) Spot Check: Gravity Column (con)

5/5

SPOT CHECK: GRAVITY COLUMN

SC2
W 14 × 342

$P_u = 2510^k$
$KL = 13'$

From Table 4-1, choose W 14 × 233
$\phi P_n = 2770^k > P_u = 2510^k$ ✓

For W 14 × 342, $\phi P_n = 4120^k \gg P_u = 2510^k$ ✓

∴ W 14 × 342 design is satisfactory, but a smaller size could be used based on gravity analysis.

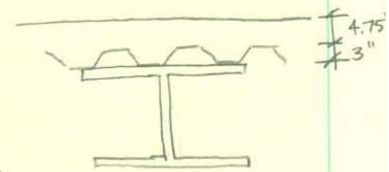
E) Spot Check: Composite Beam

Reference: AISC LRFD Steel Manual

Choose beam on a typical Lab floor (Level 5)

Location: between grid lines L6 - L7, and LC - LE

W 21 × 44 composite beam (FULL COMPOSITE ACTION)
ℓ = 33'
spaced @ 10.5' o.c.
A_s = 13 in^2
NWT conc. slab on deck → 3" deck
4 3/4" topping
f'_c = 4 ksi
3/4" dia. studs



DL (slab) = 150 pcf (4.75/12) = 59.4 psf
DL (superimposed) = 25 psf
} DL = 85 psf
LL = 100 psf

CHECK: Determine design moment and check against Mu.
Determine # shear studs.
Check deflection.

• $b_{eff} = \begin{cases} 10.5' \text{ trib width} \\ 1/4(33') = 8.25' \end{cases} = 99"$

• Determine controlling compression force:

$V'_c = 0.85 f'_c b_{eff} t = 0.85(4)(99)(4.75) = 1599^K$

$V'_s = A_s F_y = (13)(50) = 650^K = V'_q = \Sigma Q_n$

Since $V'_s < V'_c$, steel controls. PNA is at or above the top of flange.

• Determine depth of concrete to balance V'_s.

$$a = \frac{650}{0.85(4)(99)} = 1.93"$$

• Determine moment arm of compressive force from top of steel:

$Y_2 = 4.75 - \frac{1.93}{2} = 3.8$ in. → 3.5 in to be conservative

• ϕM_n →
From Table 3-19 (AISC Steel Manual)
Y_2 = 3.5", ΣQ_n = 650

$\phi M_n = 673^{1K}$

E) Spot Check: Composite Beam (con)

• Determine Mu, compare to ϕMn.

$W_{DL} = 85\ psf\ (10.5') = 0.9\ ^{k}/ft$

$W_{LL} = 100\ psf\ (10.5') = 1.05\ ^{k}/ft$

$W_u = 1.2(0.9) + 1.6(1.05) = 2.76\ ^{k}/ft$

$M_u = \frac{wL^2}{8} = \frac{(2.76)(33)^2}{8} = 375.7\ ^{\prime k}$

$\phi M_n = 673\ ^{\prime k} > M_u = 375.7\ ^{\prime k}$ ✓ Design is okay.

• Determine # shear studs required.

$Q_n = 26.1^{k}$ per stud (Table 3-21)

$\Sigma Q_n = 650^{k}$

$\#\ studs = 650^{k}/26.1 = 24.9$ studs → 25 studs required (each side) = 50 total

(26) required by design ✓ okay