Kevin Wigton – Structural Option

Technical Report I

Simmons College School of Management, Boston, Ma

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10/5/2009

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Executive Summary: Technical Report I

The Simmons College School of Management is a newly constructed five story educational facility located in Boston, Massachusetts. The building is 65,000 SF and sits on the south east corner of a five level below grade parking garage. Accommodations have been made in the original design for a future expansion of the building which would top out at a nine story building. In the sections that follow this is one source that is noted for discrepancies between the original design and what appears in this report.

The below grade parking garage is a post tensioned concrete system with a slurry wall as the exterior foundation wall system. Interior columns are W14 shapes extend into the ground to form load bearing element foundations. At the plaza level provisions were made for the use of a crane in the construction of the above grade building. The five story building is steel with composite floors and primarily uses wide flange shapes.

In the first technical report the existing condition of the structural system was evaluated. Gravity and lateral loads acting on the building were determined first. Original design of the building was in accordance with the Massachusetts State Building Code, 6th edition. This report evaluated loads according to ASCE 7-05. As a result some differences between the original loads and those determined in this report. This is the second major cause for differences between the original design and the calculations in this report.

Seismic loads were determined to be the most critical lateral force applied to the building. In both the north-south and east-west directions base shear and overturning moment were greater under seismic loading. A primary reason for the high seismic loads is the assumed site class E soil classification. This was determined to most closely reflect the S3 designation of soils given in the Massachusetts State Building Code.

Given the building geometry some assumptions were made with regards to the area which loads would act on. In future reports this is a topic which will be further investigated to find the most accurate lateral loading which the building will experience.

The effects of torsion on the building due to lateral loading were not considered in this report. The lateral load resisting system is a combination of braced and moment frames. The effects of the combined action of these elements while resisting lateral loads will be investigated in future technical reports.

Gravity load resisting elements were evaluated in this report under the loads that were determined according to ASCE 7-05. In all cases member sizes determined for this report were smaller than those used in the original design. Differences in existing loading conditions or provisions for future load carrying requirements are discussed as reasons for the differences in member sizes.

Introduction

The Simmons College School of Management is a newly completed five story educational facility to be located on the Simmons College campus in Boston, Massachusetts. The $63 million building which was completed in December of 2008 was designed by Cannon Design.

As part of the project a five level below grade parking structure was provided to replace the parking lot that previously occupied the site. This relocation of parking allowed for the creation of a new green space quad to serve the school.

When the building was completed it achieved the LEED Gold rating by the USGBC. The project received 40 LEED points which included recognition for significant reductions in water and energy usage.

The project includes design considerations for a future building expansion to be topped out at nine stories. This design parameter was considered from the beginning of the design process including the original geotechnical evaluation of the site.

Structural Systems

Foundations

The below grade parking structure was constructed by the top down method with the installation of a slurry wall and load bearing elements (LBE) prior to excavation. Slurry wall panels have varying widths ranging from 10’-0” to 25’-0” with the typical panel width being 24’-0”. Penetration of the 10’-0” centerbite into marine sands on site ranges from 1’-0” to 43’-0” depending on the bearing capacity demands of the wall section. See figure 2 for typical slurry wall panel elevation.

Load bearing elements are constructed with W14 columns from the garage embedded in concrete shafts. Depths of the concrete shafts are divided into four categories summarized in figure 1. W14 column embedment into the concrete shafts ranges from 16’ to 27’. Typical shear studs are 4” long ¾” diameter and arranged in patterns of eight, ten, or 12 studs per foot. See figure 3 for typical LBE configuration below the slab on grade.

LBE INSTALLATION CRITERIA CATEGORIES	
CATEGORY 1	MINIMUM EMBEDMENT OF FIVE (5) FEET BELOW THE TOP OF THE GLACIAL TILL DEPOSIT
CATEGORY 2	MINIMUM EMBEDMENT OF FIFTEEN (15) FEET BELOW THE TOP OF THE GLACIAL TILL DEPOSIT OR MINIMUM EMBEDMENT OF TWO (2) FEET BELOW THE TOP OF THE BEDROCK DEPOSIT AND A MINIMUM TOTAL EMBEDMENT OF TEN (10) FEET BELOW THE TOP OF THE GLACIAL TILL/BEDROCK DEPOSITS
CATEGORY 3	MINIMUM EMBEDMENT OF FIVE (5) FEET BELOW THE TOP OF THE BEDROCK DEPOSIT AND A MINIMUM TOTAL EMBEDMENT OF FIFTEEN (15) FEET BELOW THE TOP OF THE GLACIAL TILL/BEDROCK DEPOSIT
CATEGORY 4	MINIMUM EMBEDMENT OF FIFTEEN (15) FEET BELOW THE TOP OF BEDROCK DEPOSIT

Figure 1 Typical LBE Configuration

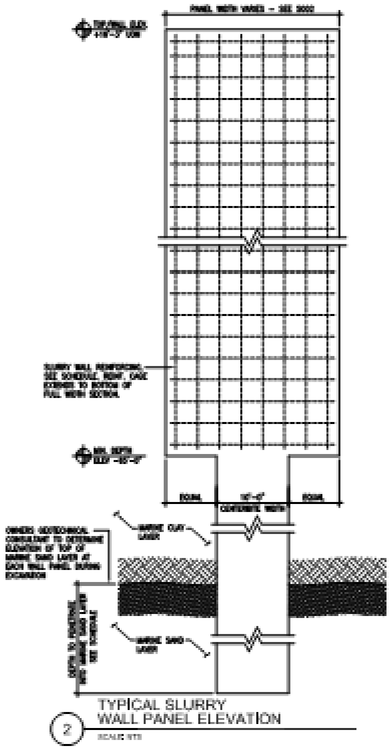


Figure 2 Slurry Wall Foundation Detail

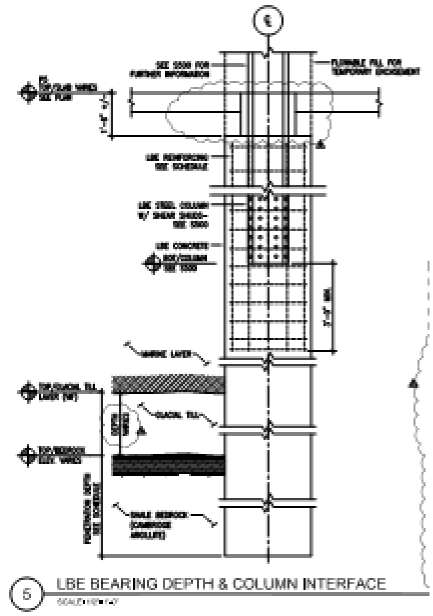


Figure 3 Load Bearing Element Foundation Detail

Beneath the area of the superstructure that is not located on top of the parking garage .365" thick, 10.75" diameter concrete filled steel pipe piles are used for foundations at column locations. Arrangements of piles include three, four, five, and eleven pile configurations.

Floor Systems

Post tensioned concrete slabs are utilized for the typical floor system in the sub grade parking garage. Slab thickness in levels P1 through P4 is 14" with 6500 psi concrete. Bay sizes in the parking garage range from 36'x32' to 42'x49'.

Banded reinforcement spans in the north south direction of the parking garage plan with the typical bottom drape in each tendon meeting the minimum concrete cover at 1.75 inches. The typical force after all losses in these tendons is 1600 kips. Distributed reinforcement is placed in the east west direction at a maximum of 48 inches on center. At the column connections various patterns of stud rail arrangements and additional mild reinforcement are provided. For the lower four parking levels steel columns are encased in concrete to form a round 2'-8" diameter round column.

At the plaza and first floor level the structural floor system changes from post tensioned concrete to steel beams with composite floor slabs. In the main quad area typical bay sizes remain the same. Typical horizontal framing in this area ranges from W24x76 beams with 52 shear studs to W 36s135 beams with

80 shear studs. Three inch deck with 9" of 3000psi concrete is typical for all horizontal surfaces at the main quad space. Plate girders are used to transfer load from superstructure columns above this level to the columns extending through the parking garage. All plate girders are 48 inches deep with weights from 330 to 849 lb/ft.

The use of steel beams with composite action is continued in the floor framing of the building above grade. See the framing the third floor in figure 4 for a typical plan and framing layout.

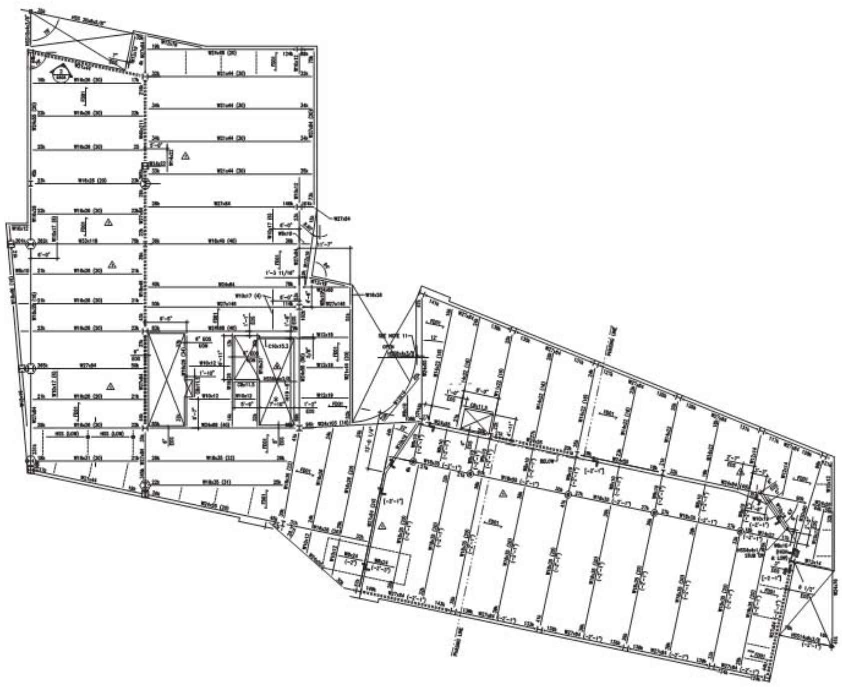


Figure 4 Second Floor Framing Layout

Columns

Typical column sections for the superstructure of the Simmons College School of Management wide flange sections with some usage of hollow structural steel (HSS) sections. Wide flange sections are all W14s with weights varying from 43 to 109 lb/ft. The most commonly used wide flange column is a W14X90. HSS sections are either HSS6x6 or HSS8x8. In addition to carrying gravity loads the majority of the columns participate in the lateral force resisting systems as part of either the moment frames or braced frames.

Once the building column loads have been transferred by the plate girders W14 column sections continue to carry the load through the parking garage. Weights vary from 159 to 398 lbs/ft. In two different locations W14x398 with side plates or W14x500 columns are used. Here all columns below the first parking garage level are encased in concrete to form a 2'-8" diameter round column.

Lateral Systems

Two structural systems are used in the Simmons College School of Management to resist lateral forces applied to the building. In the north south direction of the building steel braced frames carry lateral loads. The lateral force resisting system in the east west direction is a combination of steel braced frames and steel moment frames. Locations of steel braced frames can be seen in figure 5 and steel moment frames are noted in figure6. The number of steel braced frames used is reduced in the upper floors of the building. In some cases areas moment frames are used where braced frames are present on lower floors.

At all levels the concrete floor deck forms a ridged diaphragm which transfers lateral load to either the braced or moment frames. The amount of force that each lateral load resisting element receives is dependent on that element's relative stiffness in the system.

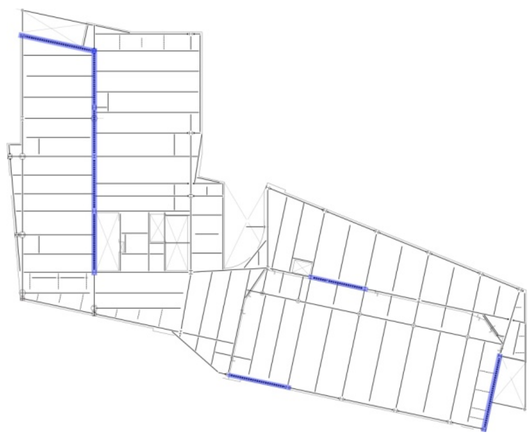


Figure 5 Braced Frame Locations

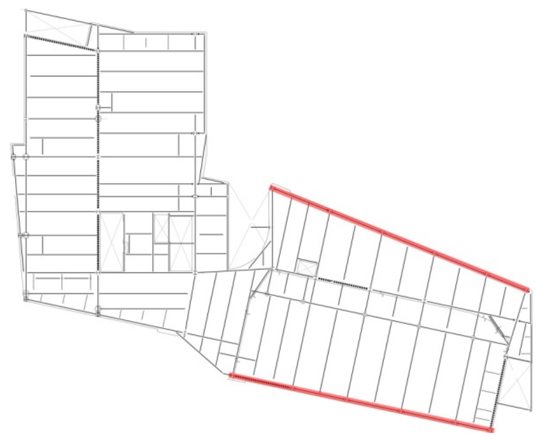


Figure 6 Moment Frame Locations

In the parking garage levels of the building, soil pressures generate lateral forces that need to be counteracted. Here the post tensioned floors provide the lateral bracing for the slurry walls. To ensure lateral stability during construction the parking garage was constructed in a top down method. Slurry walls and load bearing element columns were installed first with excavation and installation of the area beginning with the top slab.

Supplementary Structural Systems

Two supplementary structural systems are used in the building in addition to the main load carrying elements. At the roof a braced frame screen is used to hide the penthouse and mechanical equipment. HSS sections are used for vertical and horizontal members while angles form the diagonal bracing.

In the parking garage reinforced concrete members are used to form the ramp access to all parking levels. Edge beams span the length of the length of the ramp with a 12 inch slab bridging the 21'-2" for the driving surface. Girders are 2'-7" deep and span below the slab at columns locations.

Code Requirements

Design Codes

Building Code, Design Loads:	Massachusetts State Building Code CMR 780 6th Addition
Reinforced Concrete:	American Concrete Institute (ACI) 318
Structural Steel:	American Institute of Steel Construction (AISC)

Substitute Codes for Thesis

Building Code:	International Building Code (IBC) 2006
Building Loads:	American Society of Civil Engineers (ASCE) 7-05
Structural Steel:	American Institute of Steel Construction (AISC) 13th Edition 2005
Reinforced Concrete:	American Concrete Institute (ACI) 318-08

Materials

Concrete	
Footings	3,000 psi
Foundation Walls	4,000 psi
Grade Beams, Pile Caps	4,000 psi
Concrete in pipe piles	4,500 psi
Slab on Grade	3,500 psi
Slab on metal deck	3,000 psi (Normal and Light Weight)
All other concrete	4,000 psi
Columns at P/T slab	4,000 psi
Post Tensioned Concrete	6,500 psi
Slurry Walls	4,000 psi
Reinforcing	
Mild Reinforcing Bars	ASTM A-615, Grade 60
Welded Bars	ASTM A-706, Grade 60
Welded Wire Fabric	ASTM A-185
Steel Fibers	ASTM A-820 Type 1
Masonry	
Hollow Concrete Masonry Units	ASTM C90 Grade N, Type 1 F'm = 1900psi
Grout	ASTM C476 3,000 psi min.
Mortar	Type S - ASTM C270
Structural Steel	
Wide Flange Shapes, WT's	ASTM A-992
Channels & Angles	ASTM A-36
Pipe	ASTM A-53 Grade B
Pipe Piles	ASTM A252 Grade 3
Tubular Shapes (Rect.)	ASTM A-500 Grade B

Base Plaets	ASTM A572 Grade B
All Other Steel Members	ASTM A-36 (Unless Otherwise noted)
High Strength bolts	ASTM A-325, or A-490
Nuts and washers	Min. ¾" Diameter
Anchor rods	ASTM F1554
Welding Electrode	E70XX
Metal Deck Welding Electrode	E60XX min.
Metal Deck	ASTM A653 Fy=33,000psi

Building Loads

Dead Loads

FD01	43.2
FD02	42.7
FD03	69.0
FD04	96.8
PT floor slab	175
Structural Steel	Per AISC Manual
Green Roof	100
Superimposed Dead loads:	
MEP	10
Partitions	20
Finishes/Misc.	5
Curtain Wall	10

Live Loads

Space:	Design Value	ASCE 7-05
Parking Floors	50	40
Plaza	100	100
	300 Construction	
Exit Corridors	100	100
Stairs	100	100
Lobbies	100	100
Typical Floor	50	50 (office load)
Corridors above 1st Floor	80	80
Roof Garden	100	100
Flat Roof	-	20
Mechanical Areas	150	

Snow Loads

The snow loading for the building was determined according to ASCE 7-05. The flat roof snow load was found to be pf=27.7 psf which closely reflects what was determined for design, pf=30psf. For this report two snow drift conditions were evaluated. One that would develop at the roof of the building and one that the plaza level structural elements would experience. These are the snow drift load diagrams that were developed.

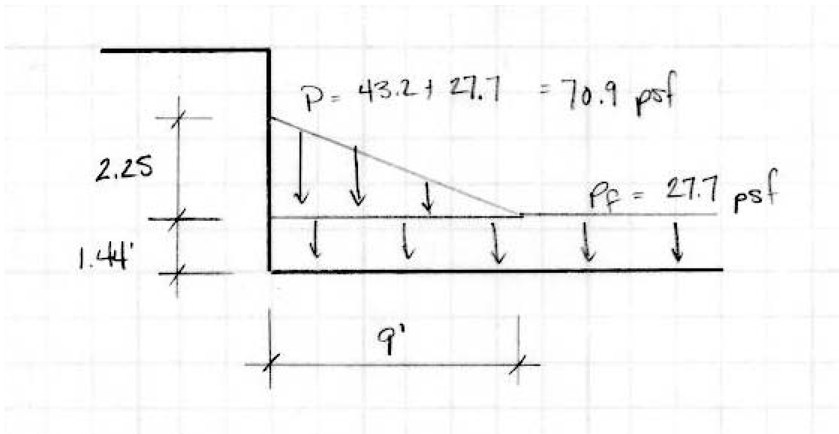


Figure 7 Snow Drift, Roof

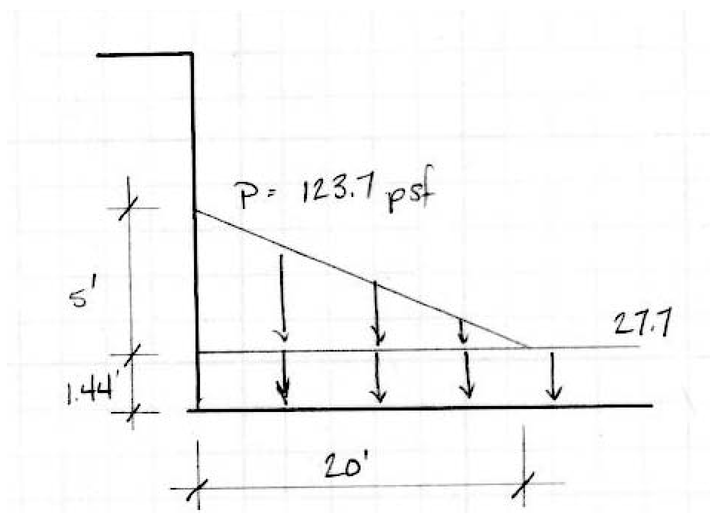


Figure 8 Snow Drift, Plaza

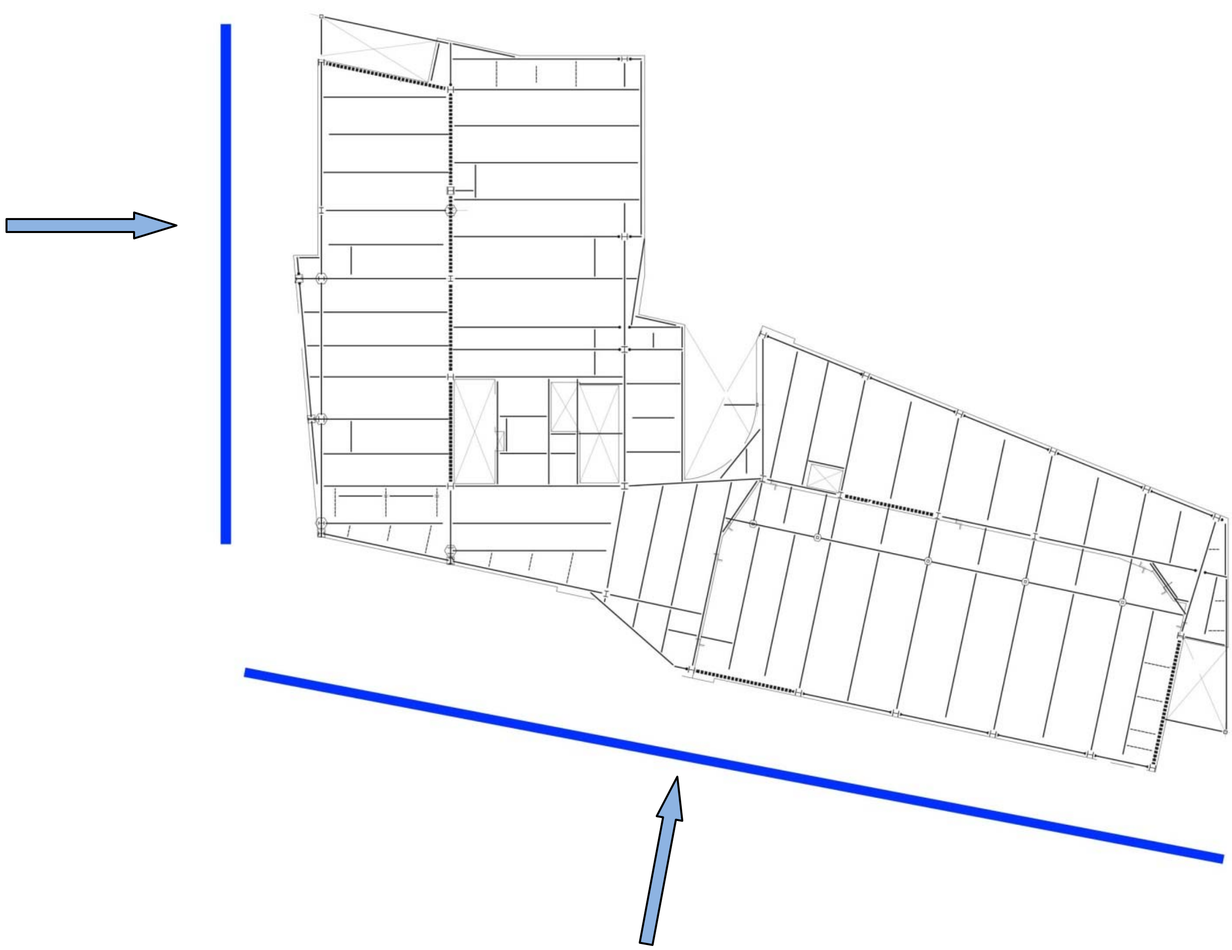
Lateral Loads

Lateral loads acting on the structure were determined according to ASCE 7-05. The original loading for the building was in accordance with the sixth addition of the Massachusetts State Building Code. This is one source of variance that is observed between design loads that those calculated in this report. Seismic loads were the controlling lateral force on the building. Both base shear and overturning moment values for seismic design were higher than the values for wind design.

Wind Load Analysis

Wind loads were calculated using method two, the analytical procedure from section 6.5 of ASCE 7-05. Given the configuration of the building, loads were assumed to act on projected widths of the building. Further analysis will be performed in the following technical reports for the effects of input direction of lateral loads and the creation of torsion due to building configuration.

Figure 9 Wind Loading Directions



Design Wind pressures p EAST WEST direction

Location	Height above ground	q (psf)	External Pressure qGCp (psf)	Internal Pressure qh(Gcpi) (psf)	Net Pressure p (psf)	
					+(Gcpi)	-(Gcpi)
Windward	70	32.1	21.57	5.78	27.35	15.79
	60	30.6	20.56	5.78	26.34	14.78
	50	29.2	19.62	5.78	25.40	13.84
	40	27.4	18.41	5.78	24.19	12.63
	30	25.2	16.93	5.78	22.71	11.15
	25	23.8	15.99	5.78	21.77	10.21
	20	22.3	14.99	5.78	20.77	9.21
	15	20.5	13.78	5.78	19.56	8.00
Leeward	All	32.1	-8.09	5.78	-2.31	-13.87
Side	All	32.1	-18.87	5.78	-13.09	-24.65
Roof	70.5	32.1	-24.26	5.78	-18.48	-30.04
	70.5	32.1	-13.48	5.78	-7.70	-19.26
	70.5	32.1	-8.09	5.78	-2.31	-13.87

East West

Pressure	height	width	moment arm	Shear (K)	overturning moment (ft-K)
48	1.5	95	70.75	6.84	483.93
32	1.5	95	70.75	4.56	322.62
13.8	70.5	95	35.25	92.43	3258.00
19.6	15	95	7.5	27.93	209.48
20.8	5	95	17.5	9.88	172.90
21.8	5	95	22.5	10.36	232.99
22.7	5	95	27.5	10.78	296.52
24.2	10	95	35	22.99	804.65
25.4	10	95	45	24.13	1085.85
26.3	10	95	55	24.99	1374.18
27.4	10	95	65	26.03	1691.95
				260.91	9933.06

Design Wind pressures p NORTH SOUTH direction

Location	Height above ground	q (psf)	External Pressure qGCp (psf)	Internal Pressure qh(Gcpi) (psf)	Net Pressure p (psf)	
					+(Gcpi)	-(Gcpi)
Windward	70	32.1	21.06	5.78	26.84	15.28
	60	30.6	20.07	5.78	25.85	14.29
	50	29.2	19.16	5.78	24.94	13.38
	40	27.4	17.97	5.78	23.75	12.19
	30	25.2	16.53	5.78	22.31	10.75
	25	23.8	15.61	5.78	21.39	9.83
	20	22.3	14.63	5.78	20.41	8.85
	15	20.5	13.45	5.78	19.23	7.67
Leeward	All	32.1	-13.16	5.78	-7.38	-18.94
Side	All	32.1	-18.42	5.78	-12.64	-24.20
Roof	70.5	32.1	-31.58	5.78	-25.80	-37.36
	70.5	32.1	-18.42	5.78	-12.64	-24.20
	70.5	32.1	-18.42	5.78	-12.64	-24.20

North South

Pressure	height	width	moment arm	Shear (K)	overturning moment (ft-K)
48	1.5	170	70.75	12.24	865.98
32	1.5	170	70.75	8.16	577.32
18.9	70.5	170	35.25	226.52	7984.71
19.2	15	170	7.5	48.96	367.20
20.4	5	170	17.5	17.34	303.45
21.4	5	170	22.5	18.19	409.28
22.3	5	170	27.5	18.96	521.26
23.8	10	170	35	40.46	1416.10
24.9	10	170	45	42.33	1904.85
25.9	10	170	55	44.03	2421.65
26.8	10	170	65	45.56	2961.40
				522.74	19733.19

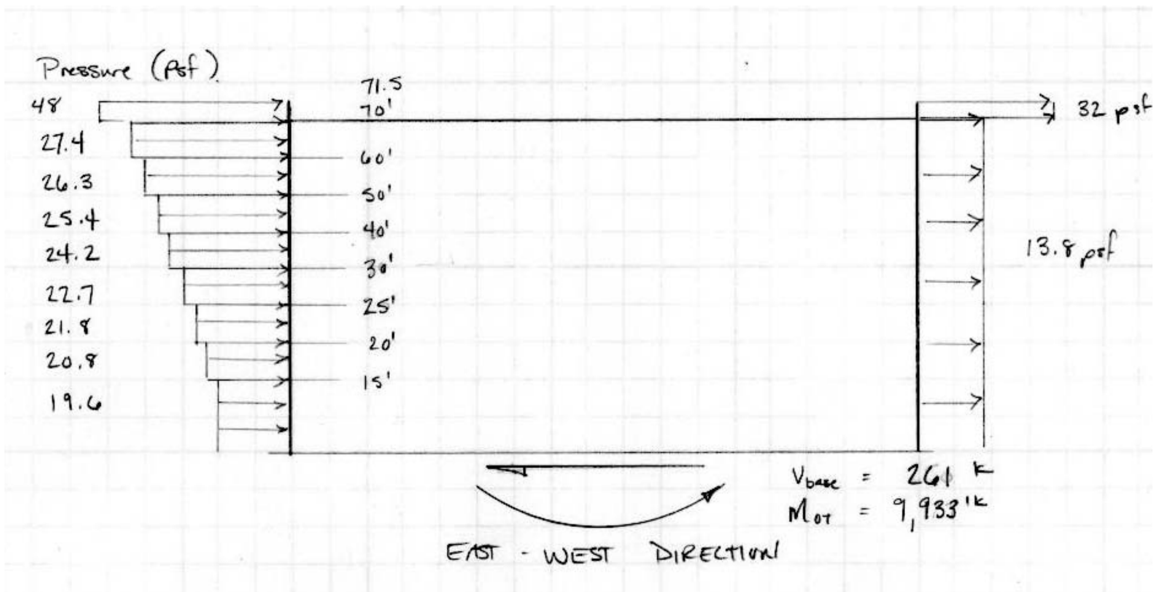


Figure 10 Wind Pressures, East-West

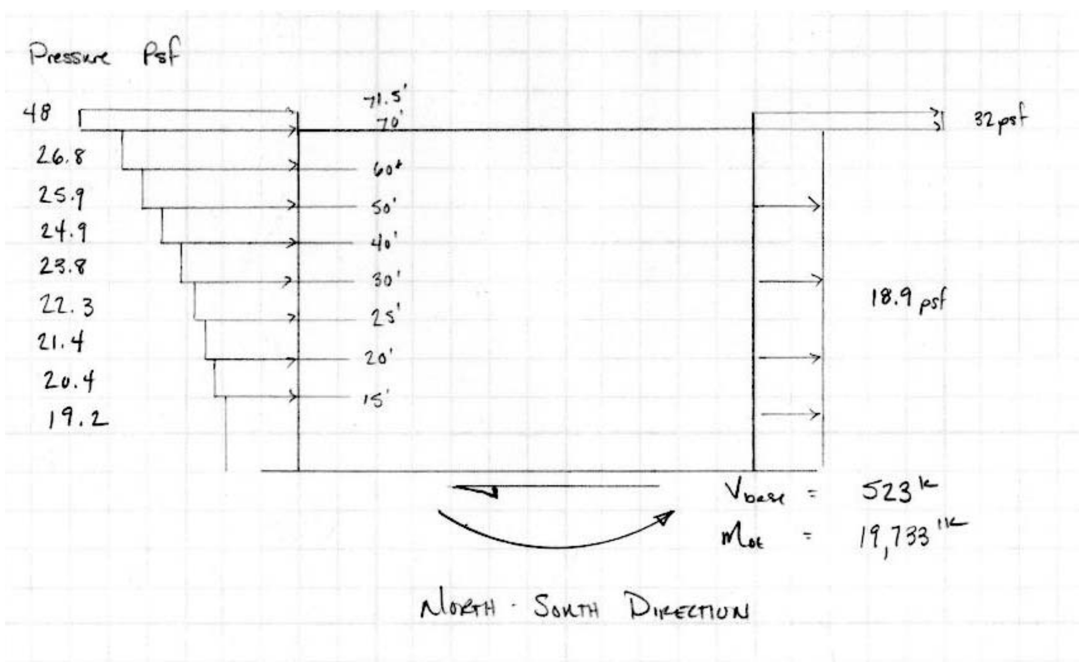


Figure 11 Wind Pressures, North-South

Seismic Load Analysis

Seismic loads, similar to the wind loads, were determined in accordance with ASCE 7-05 rather than the Massachusetts State Building Code. Site class E was used as a conservative approximation for the soil classification. This was determined to be the closest to the S3 soil classification that was used during design. The ground motion acceleration values used in this report were determined with the USGS Ground Motion Parameter Calculator. Given these differences in the design procedures and those used in this evaluation, variance between final loadings can be expected.

Seismic Forces in the North/South Direction							
Level	Story weight w_x (kips)	Height h_x (ft)	$w_x h_x^k$	Cvx	Lateral force Fx (kips)	Story Shear Vx (Kips)	Moment contribution (ft-K)
R	1023	69.33	70924.6	0.24	258.18	258.18	17899.62
5	1832	56	102592.0	0.34	373.46	631.64	20913.53
4	1438	43	61834.0	0.21	225.09	856.72	9678.80
3	1449	30	43470.0	0.14	158.24	1014.96	4747.19
2	1404	15.66	21986.6	0.07	80.04	1095.00	1253.36
				Total:	1095.00		54492.51

Seismic Forces in the East/West Direction							
Level	Story weight w_x (kips)	Height h_x (ft)	$w_x h_x^k$	Cvx	Lateral force Fx (kips)	Story Shear Vx (Kips)	Moment contribution (ft-K)
R	1023	69.33	70924.6	0.24	127.32	127.32	8827.21
5	1832	56	102592.0	0.34	184.17	311.49	10313.52
4	1438	43	61834.0	0.21	111.00	422.49	4773.11
3	1449	30	43470.0	0.14	78.04	500.53	2341.08
2	1404	15.66	21986.6	0.07	39.47	540.00	618.10
				Total:	540.00		26873.02

Spot Checks

Design checks were performed on main structural elements that support gravity loads. Only elements which primarily carry gravity loads and do not participate in the resistance of lateral loads were examined.

The beam that was checked was one of the commonly used interior floor framing members, a W21x50[30] c=3/4". First a design process was followed to confirm the member size which was used in the original design of the floor system. The design member that was chosen was a W16x40[44] c=3/4". Due to construction loads it was necessary to provide a ¾" camber for the member prior to composite beam action. Due to the discrepancy in member sizes, the capacity of the original member was investigated to find the load carrying capability of the beam. After these member checks it was determined that the members were able to carry higher loads that what they were being checked for in this report. One probable cause for this difference is the ability for the building to go through a future expansion. It is likely that allowances were made for future increased loading at this time.

The next element that was investigated was an interior column at column line ZB-Z6. The original design is for a W14x82 to extend up to the third floor where a W14x61 column then extended to the roof. Again the member sizes that were chosen for this design check were smaller than those that were used in the original design. Future expansion which includes additional levels would directly affect the member sizes that are used in the building. Additionally in all column checks it was assumed that k=1 due to braced frames being present in both directions. However lateral loads are resisted by both braced and moment frames and it is likely that k=1 would yield results that are not conservative. The effect of the lateral system on the columns in the building is something that will be investigated in future technical reports.

Conclusion

In conclusion, the Simmons College School of Management was determined to have a structural system that was adequate to carry the loads applied. Loads determined in this report in many cases were different than those likely used for the initial design of the building. Future considerations will be given for more accurate evaluation of structural elements in the following technical reports. This is to include the evaluation of lateral loads and the effects that they have on all structural components of the building. This report did not address the technical evaluation of the post tensioned floor slabs in the parking garage. Given the considerations for constructability and long term performance this will be one of the components of the structure that will be examined further.

Appendix A: Typical Layout

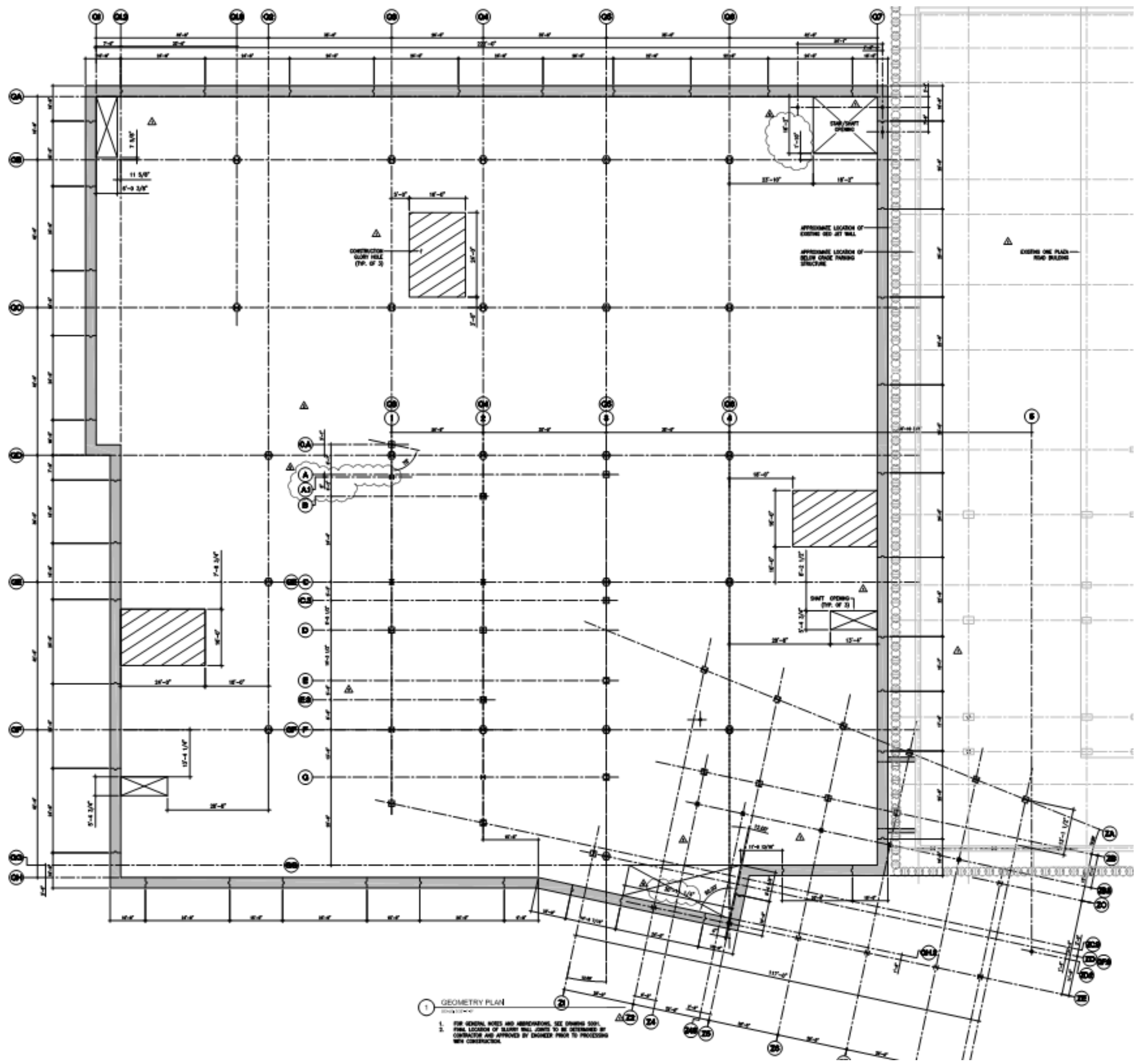


Figure 12 Sub Grade Parking Garage Layout

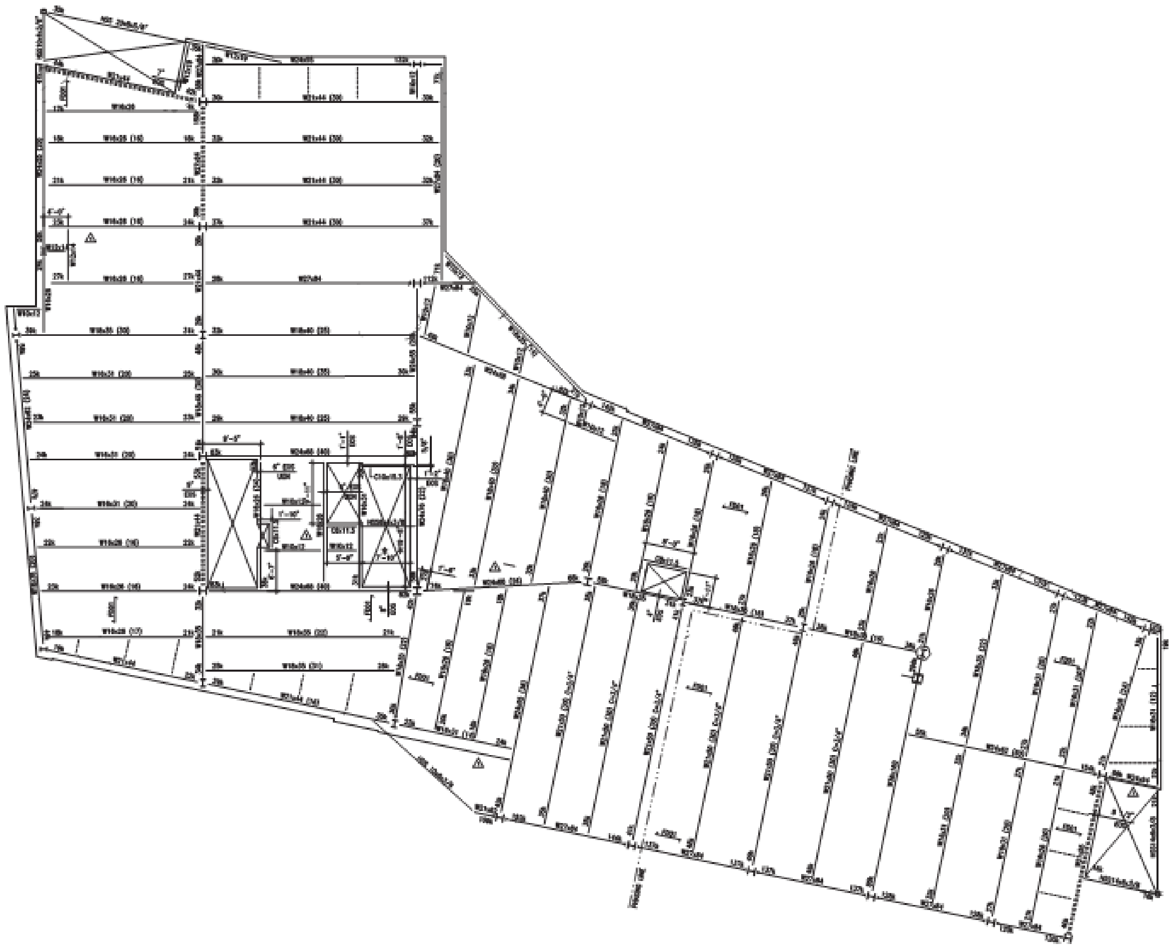


Figure 13 Typical Above Grade Building Framing

Appendix B: Gravity Loads

1

Simmons College - SoM
Design Loads
9/26/09 KTW

Dead Loads
Composite Floor
FD01 — Concrete = $[(5.25' - 2") + \frac{1}{2}(2")](\frac{1}{12}")(115\ pcf)$
= 40.7 psf
Floor
Steel Deck 2", Gage 18 = 2.5 psf ← Conservative (USD Manual)
Total = 40.7 + 2.5 = $\boxed{43.2\ psf}$

FD02 - Concrete (Same as above) = 40.7 psf
Steel Deck 2", Gage 20 (Estimate) = + 2.0 psf
Total $\boxed{42.7\ psf}$

FD03 - Concrete :
$w = [(6.5 - 2) + \frac{1}{2}(2")](\frac{1}{12}")(145\ pcf)$
$w = 66.5$ psf
Steel Deck 2", Gage 18 (Assumed value from above)
$w = 2.5$ psf
Total: $\boxed{69.0\ psf}$

FD05 - Concrete:
$w = [(9.0 - 3) + \frac{1}{2}(3")](\frac{1}{12}")(150)$
$w = 93.8$ psf
Steel Deck 3", Gage 18 (Assumed, based on USD Manual)
$w = 3.0$ psf
Total = $\boxed{96.8\ psf}$

Summary
Composite Floor - Dead Loads by Type

FD01	43.2 psf
FD02	42.7 psf
FD03	69.0 psf
FD04	96.8 psf

2

Dead Loads

Simmons College - SoM
Design Loads
9/26/09

Post - Tensioned Parking Garage Floors

$w = (14")(\frac{1}{12})(150) = 175$ psf

Assumed Superimposed Loads

MEP	10 psf
Partitions	20 psf
Finishes/Misc	5 psf
Exterior Wall	10 psf
Green Roof (Additional)	100 psf

Simmons College - SoM
Snow Loads
9/27/09 KTW

Boston, Ma - Building Roof

$P_f = 0.7\, C_e C_t I\, P_g$

Terrain Category ASCE 7-05 § 6.5.6 : Exposure B

$C_e = 0.9$

$C_t = 1.0$

Occupancy Category : III

$I = 1.1$

$P_g = 40$ psf

$P_f = 0.7(0.9)(1.0)(1.1)(40)$

$P_f = 27.7$ psf

Drift loads - upper roofs

$h_b = P_f/\gamma$

$\gamma = 0.13\, p_g + 14 \leq 30$ pcf

$\gamma = 0.13(40) + 14 = 19.2$ pcf < 30 pcf ∴ ok

$h_b = 27.7/19.2 = 1.44'$

$h_c = 13'$

$h_c/h_b = 9 > 0.2$ ∴ drifting

Leeward Drift

$l_u = 35'$

$\Rightarrow h_d = 2.0'$

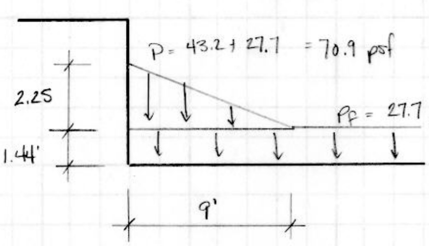
Windward Drift

$l_u^* = 75'$

$h_d = (3/4)(3) = \underline{2.25'} > 2.0'$ ∴ $\underline{2.25'}$ Controls

$w = 4h_d = 4(2.25) = 9'$

$P_d = \gamma h_d = 19.2(2.25) = 43.2$ psf



2

Simmons College Soril
Snow Loads
9/27/09
KTW

Parking Garage Roof (Plaza)

Terrain Category : Exposure B
Partially exposed

$C_e = 1.0$

$C_t = 1.1$

$I = 1.1$

$p_g = 40$ psf

$p_f = 0.7\, C_e C_t I p_g = 0.7(1.0)(1.1)(1.1)(40)$

$p_f = 33.9$ psf

Drifting on Quad —

$\gamma = 19.2$ pcf

$h_b = 33.9/19.2 = 1.77'$

$h_c = 70'$

$h_c/h_b = 70/1.77 = 39.5 > 0.2$ ∴ consider drifting

Leeward Drift

$l_u = 200'$ ← approximated building width

$h_d = 5.0'$ ← Controls

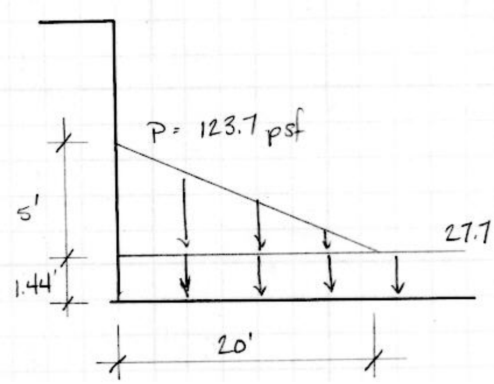
Windward Drift

$l_u = 200'$

$h_d = 3/4(5) = 3.75'$

$w = 4h_d = 4(5) = 20'$

$p_d = \gamma h_d = 19.2(5) = 96$ psf



Appendix C: Wind Loads

Wind Loads - ASCE 7-05
Mean h = 70.5' > 60 ∴ Cannot Apply MFWRS
Method 2 - Analytical Procedure § 6.5

Simmons College - SOM
Design Loads
9/28/09 KTW

Location: Boston, Ma., Exposure B

Basic Wind Speed	$V = 120$ mph	Figure 6-1
Wind Directionality Factor	$K_d = 0.85$	Table 6-4
Occupancy Category: III		Table 1-1
Importance Factor I:	$I = 1.15$	Table 6-1
Not Located on Hill, Ridge, Escarpment	$K_{zt} = 1.0$	§ 6.5.7.1
Exposure Coefficient	$K_{z,h} = 0.89$	Table 6-3

$q_p = 0.00256\, K_z K_{zt} K_d V^2 I$
$= 0.00256(0.89)(1.0)(0.85)(120)^2(1.15)$
$q_p = 32$ psf

$q_p = 32$ psf Eq. 6-15

Parapet Net Pressure Coefficient WW $GC_{pn} = 1.5$ § 6.5.12.2.4
LW $GC_{pn} = -1.0$ "

Design Pressure on Parapet
Windward: $p_p = q_p GC_{pn} = (32)(1.5)$
$p_p = 48$ psf

$p_p = 48$ psf (WW)

Leeward: $p_p = q_p GC_{pn}$
$= 32(-1.0)$

$p_p = -32$ psf (LW)

Natural Frequency
EW - Steel Moment Resisting Frame

$n_1 = 22.2/H^{0.8} = 22.2/(70.5)^{0.8} = 0.74$

NS/EW - Steel braced frame
Use (C6-17 (average value)

$n_1 = 100/H = 100/70.5 = 1.42 \geq 1$

Check Lower bound

$n_1 = 75/70.5 = 100/70.5 = 1.06 \geq 1$

Since the average value and Lower bound are both greater than 1:

ASSUME STRUCTURE IS RIGID

$g_Q = g_v = 3.4$

$\bar{z} = 0.6h = 0.6(70.5)$
$\bar{z} = 42.3$ ft $> z_{min} = 30'$

$\bar{z} = 42.3$ ft.

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.3\left(\frac{33}{42.3}\right)^{1/6}$
$= 0.29$

$I_{\bar{z}} = 0.29$

WIND LOADS 2

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{42.3}{33}\right)^{(1/3)}$$

$$L_{\bar{z}} = 347.6'$$

$L_{\bar{z}} = 347.6'$

$$Q_{E-W} = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$$

$$Q_{E-W} = \sqrt{\frac{1}{1 + 0.63\left(\frac{95 + 70.5}{347.6}\right)^{0.63}}} =$$

$Q_{E-W} = 0.85$

$$Q_{N-S} = \sqrt{\frac{1}{1 + 0.63\left(\frac{170 + 70.5}{347.6}\right)^{0.63}}} = 0.82$$

$Q_{N-S} = 0.82$

$$G = 0.925\left[\frac{1 + 1.7 g_Q I_{\bar{z}} Q}{1 + 1.7 g_v I_{\bar{z}}}\right]$$

$$G_{EW} = 0.925\left[\frac{1 + 1.7(3.4)(0.29)(0.85)}{1 + 1.7(3.4)(0.29)}\right] = 0.84$$

$G_{EW} = 0.84$

$$G_{N-S} = 0.925\left[\frac{1 + 1.7(3.4)(0.29)(0.82)}{1 + 1.7(3.4)(0.29)}\right] = 0.82$$

$G_{NS} = 0.82$

Velocity Pressure Coefficients

Height Above Ground	K_z
70	0.89
60	0.85
50	0.81
40	0.76
30	0.70
25	0.66
20	0.62
< 15	0.57

WIND LOADS 8

$q_z = 0.00256\, K_z K_{zt} K_d V^2 I$
$= 0.00256\, K_z (1.0)(0.85)(120^2)(1.15)$
$= 36.03\, K_z$

Height Above Ground	K_z	q_z (psf)
70	0.89	32.1
60	0.85	30.6
50	0.81	29.2
40	0.76	27.4
30	0.70	25.2
25	0.66	23.8
20	0.62	22.3
<15	0.57	20.5

Pressure Coefficient (E-W)
Windward Wall: $C_p = 0.8$ w/ q_z
Leeward Wall
$(L/B) = (170/95) = 1.8 \Rightarrow 2$ $C_p = -0.3$ w/ q_h
Side Walls $C_p = -0.7$ w/ q_h
Roof Pressures
$h/L = 70.5/170 = 0.41 < 0.5$
from WW edge to $h = 70.5$ $C_p = -0.9, -0.18$ w/ q_h
from 70.5 to $2h = 141$ $C_p = -0.5, -0.18$ w/ q_h
from 141 to 170 $C_p = -0.3, -0.18$ w/ q_h

Pressure Coefficients (N-S)
Windward Wall $C_p = 0.8$ w/ q_z
Leeward Wall
$(L/B) = (95/170) = 0.56$ $C_p = -0.5$ w/ q_h
Side Walls $C_p = -0.7$ w/ q_h
Roof Pressures
$h/L = 70.5/95 = 0.74$
$(h/2)(w) = 6000 \Rightarrow$ Reduction factor $= 0.9$
from WW edge to $h/2 = 35.25$ $C_p = -1.2, -0.18$ w/ q_h
from 35.25 to 95 $C_p = -0.7, -0.18$ w/ q_h

Windward walls, side walls, leeward walls, roofs
$q_i = q_h = 32.1$ psf

Internal Pressure Coefficient $GC_{pi} = \pm 0.18$

EAST - WEST
Windward Walls
$p_z = q_z G C_p - q_h GC_{pi}$
$= q_z(0.84)(0.8) - 32.1(\pm 0.18) = 0.672\, q_z \pm 5.78 = p_z$

Leeward Side walls & Roof
$p_h = q_h G C_p - q_h (GC_{pi})$
$= C_p(32.1)(0.84) - (32.1)(\pm 0.18) = 26.96\, C_p \pm 5.78 = p_h$

WIND LOADS 4

NORTH SOUTH

Windward Walls

$p_z = q_z (0.82)(0.8) - 32.1(\pm 0.18)$

$p_z = 0.656\, q_z \pm 5.78$

Leeward, Side walls & Roof

$p_h = C_p (32.1)(0.82) \pm 5.78$

$p_h = 26.32\, C_p \pm 5.78$

Design Wind pressures p in the EAST WEST direction

Location	Height above ground	q (psf)	External Pressure qGCp (psf)	Internal Pressure qh(Gcpi) (psf)	Net Pressure p (psf)	
					+(Gcpi)	-(Gcpi)
Windward	70	32.1	21.57	5.78	27.35	15.79
	60	30.6	20.56	5.78	26.34	14.78
	50	29.2	19.62	5.78	25.40	13.84
	40	27.4	18.41	5.78	24.19	12.63
	30	25.2	16.93	5.78	22.71	11.15
	25	23.8	15.99	5.78	21.77	10.21
	20	22.3	14.99	5.78	20.77	9.21
	15	20.5	13.78	5.78	19.56	8.00
Leeward	All	32.1	-8.09	5.78	-2.31	-13.87
Side	All	32.1	-18.87	5.78	-13.09	-24.65
Roof	70.5	32.1	-24.26	5.78	-18.48	-30.04
	70.5	32.1	-13.48	5.78	-7.70	-19.26
	70.5	32.1	-8.09	5.78	-2.31	-13.87

East West

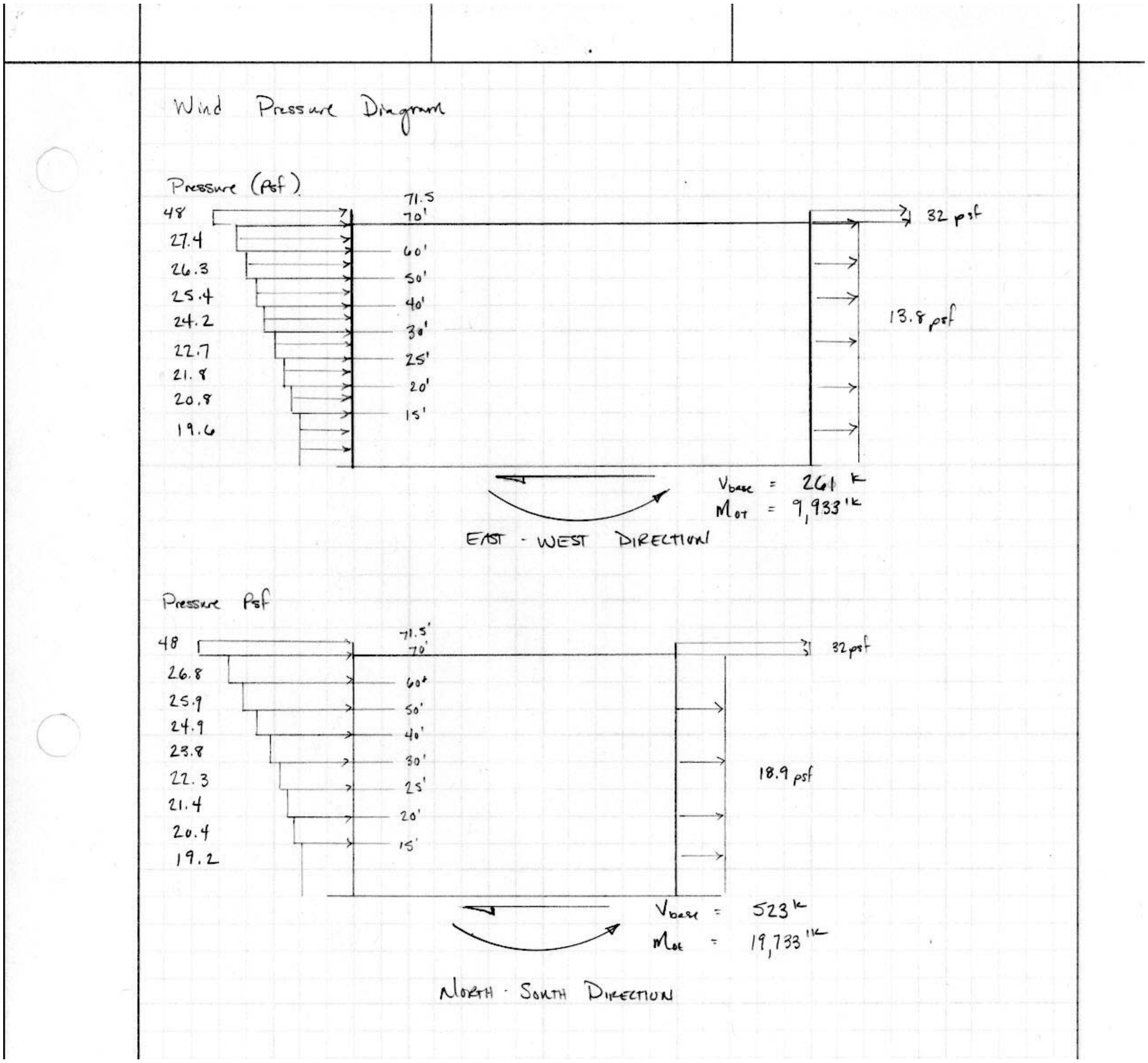
Pressure	height	width	moment arm	Shear	overturning moment
48	1.5	95	70.75	6.84	483.93
32	1.5	95	70.75	4.56	322.62
13.8	70.5	95	35.25	92.43	3258.00
19.6	15	95	7.5	27.93	209.48
20.8	5	95	17.5	9.88	172.90
21.8	5	95	22.5	10.36	232.99
22.7	5	95	27.5	10.78	296.52
24.2	10	95	35	22.99	804.65
25.4	10	95	45	24.13	1085.85
26.3	10	95	55	24.99	1374.18
27.4	10	95	65	26.03	1691.95
				260.91	9933.06

Design Wind pressures p in the NORTH SOUTH direction

Location	Height above ground	q (psf)	External Pressure qGCp (psf)	Internal Pressure qh(Gcpi) (psf)	Net Pressure p (psf)	
					+(Gcpi)	-(Gcpi)
Windward	70	32.1	21.06	5.78	26.84	15.28
	60	30.6	20.07	5.78	25.85	14.29
	50	29.2	19.16	5.78	24.94	13.38
	40	27.4	17.97	5.78	23.75	12.19
	30	25.2	16.53	5.78	22.31	10.75
	25	23.8	15.61	5.78	21.39	9.83
	20	22.3	14.63	5.78	20.41	8.85
	15	20.5	13.45	5.78	19.23	7.67
Leeward	All	32.1	-13.16	5.78	-7.38	-18.94
Side	All	32.1	-18.42	5.78	-12.64	-24.20
Roof	70.5	32.1	-31.58	5.78	-25.80	-37.36
	70.5	32.1	-18.42	5.78	-12.64	-24.20
	70.5	32.1	-18.42	5.78	-12.64	-24.20

North South

Pressure	height	width	moment arm	Shear	overturning moment
48	1.5	170	70.75	12.24	865.98
32	1.5	170	70.75	8.16	577.32
18.9	70.5	170	35.25	226.52	7984.71
19.2	15	170	7.5	48.96	367.20
20.4	5	170	17.5	17.34	303.45
21.4	5	170	22.5	18.19	409.28
22.3	5	170	27.5	18.96	521.26
23.8	10	170	35	40.46	1416.10
24.9	10	170	45	42.33	1904.85
25.9	10	170	55	44.03	2421.65
26.8	10	170	65	45.56	2961.40
				522.74	19733.19



Wind Pressure Diagram
Pressure (Psf)
48
27.4
26.3
25.4
24.2
22.7
21.8
20.8
19.6
71.5
70'
60'
50'
40'
30'
25'
20'
15'
32 psf
13.8 psf
Vbase = 261 k
MOT = 9,933 'k
EAST - WEST DIRECTION
Pressure Psf
48
26.8
25.9
24.9
23.8
22.3
21.4
20.4
19.2
71.5'
70'
60'
50'
40'
30'
25'
20'
15'
32 psf
18.9 psf
Vbase = 523 k
MOT = 19,733 'k
NORTH - SOUTH DIRECTION

Appendix D: Seismic Loads

SEISMIC LOADS 1

Building Data
Location: Boston, Ma (Latitude 42.35; Longitude -71.1°)
Soil Classification: S_3 Mass. State Bldg. Code, Site Class E Assumed as a conservative approximation.
Occupancy: III
Material: Structural Steel
Structural System
N-S: Ordinary Concentric Braced Frames
E-W: Dual System, Ordinary Moment Resisting frames with Ordinary Concentric Braced Frames.

Seismic Ground Motion Values
Mapped Accelerations: USGS Ground Motion Parameter Calculator $S_S = 0.277$
$S_1 = 0.068$

Soil Modified Accelerations
Site Class E, $S_S = 0.277$ Table 11.4-1 interpolation $F_a = 2.0$
Site Class E, $S_1 = 0.068 < 0.1$ Table 11.4-2 $F_v = 3.5$
$S_{MS} = F_a S_S = 2.0(0.277) = 0.55$ $S_{MS} = 0.55$
$S_{M1} = F_v S_1 = 3.5(0.068) = 0.24$ $S_{M1} = 0.24$

Design Accelerations
$S_{DS} = 2/3\, S_{MS} = 2/3\,(0.55) = 0.37$ $S_{DS} = 0.37$
$S_{D1} = 2/3\, S_{M1} = 2/3\,(0.24) = 0.16$ $S_{D1} = 0.16$

Determine SDC
Check if $T_a < 0.8\,T_S$
N-S Direction
$T_a = C_t h_n^x = 0.02(69.25)^{0.75} = 0.48$ sec
E-W Direction
$T_a = C_t h_n^x = 0.02(69.25)^{0.75} = 0.48$ sec $T_a = 0.48$ sec
$T_S = S_{D1}/S_{DS} = 0.16/0.37 = 0.43$ $T_S = 0.43$
T_a is not less than $0.8\,T_S$ ∴ use 11.6-1 & 11.6-2
Table 11.6-1 SDS = C
Table 11.6-2 SDS = C SDC = C

Determine Analytical Process
Check if $T < 3.5\,T_S$
$3.5\,T_S = 3.5(0.43) = 1.5 > T$

Determine if the Structure is Regular
Determine Response Modification Coefficient
E-W Direction (E1) No limit. $R_{EW} = 6$
N-S Direction (B4) No Limit $R_{N\text{-}S} = 3.25$
Importance Factor
$I = 1.25$ $I = 1.25$
Long term period
$T_L = 6$ sec $T_L = 6$ sec

Seismic Response Coefficient

E-W Direction

$C_s = \frac{S_{DS}}{R/I} = \frac{0.37}{(6/1.25)} = 0.077$

$\leq C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.16}{(0.48)(6/1.25)} = \underline{0.069}$

$\geq C_s = 0.01$

E-W: $C_s = 0.069$

N-S Direction

$C_s = \frac{S_{DS}}{R/I} = \frac{0.37}{3.25/1.25} = \underline{0.14}$

$\leq C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.37}{(0.48)(3.25/1.25)} = 0.28$

$\geq C_s = 0.01$

N-S $C_s = 0.14$

Effective Seismic Weight

See Spread Sheet

$W = 7{,}820^k$

Base Shear

N-S Direction

$V = C_s W = 0.14(7{,}820) = 1095^k$

$V_{N\text{-}S} = 1095^k$

E-W Direction

$V = C_s W = 0.069(7{,}820) = 540^k$

$V_{EW} = 540^k$

Exponent for Structural Period

$T_{EW} = T_{N\text{-}S} < 0.5 \quad \therefore k = 1.0$

Seismic Forces in the North/South Direction							
Level	Story weight w_x (kips)	Height h_x (ft)	$w_x h_x^k$	Cvx	Lateral force Fx (kips)	Story Shear Vx (Kips)	Moment contribution (ft-K)
R	1023	69.33	70924.6	0.24	258.18	258.18	17899.62
5	1832	56	102592.0	0.34	373.46	631.64	20913.53
4	1438	43	61834.0	0.21	225.09	856.72	9678.80
3	1449	30	43470.0	0.14	158.24	1014.96	4747.19
2	1404	15.66	21986.6	0.07	80.04	1095.00	1253.36
				Total:	1095.00		54492.51

Seismic Forces in the East/West Direction							
Level	Story weight w_x (kips)	Height h_x (ft)	$w_x h_x^k$	Cvx	Lateral force Fx (kips)	Story Shear Vx (Kips)	Moment contribution (ft-K)
R	1023	69.33	70924.6	0.24	127.32	127.32	8827.21
5	1832	56	102592.0	0.34	184.17	311.49	10313.52
4	1438	43	61834.0	0.21	111.00	422.49	4773.11
3	1449	30	43470.0	0.14	78.04	500.53	2341.08
2	1404	15.66	21986.6	0.07	39.47	540.00	618.10
				Total:	540.00		26873.02

Total Building Weight	7816 K

Roof Weight				
Area	10103			
Floor to Floor Height	0			
Exterior Walls			**Slab**	
Perimeter	493		Unit Weight	43.2
Unit Weight	10		Total	436449.6
Total	0			
Superimposed			**Special**	Mech.
Partition	20		Area	1067.5
MEP	10		Unit Weight	150
Finishes	5		Total	160125
Total	353605			
Columns				
Shape	Weight	Quantity	Length	Total
Beams				
Shape	Quantity	Weight	Length(avg.)	Total
W16X26	21	26	20	10920
W18X35	19	35	25	16625
W12X16	7	16	12	1344
W16X31	3	31	35	3255
W24X55	10	55	20	11000
W14x22	11	22	20	4840
W27X84	6	84	15	7560
W10X12	17	12	8	1632
W21X44	7	44	20	6160
W27x129	2	129	35	9030
Total				72366
		Kips		psf
Roof Weight		1023		101

5th Floor Weight				
Area	15620			
Floor to Floor Height	13.25			
Exterior Walls			**Slab**	
Perimeter	493		Unit Weight	43.2
Unit Weight	10		Total	674784
Total	65322.5			
Superimposed			**Special**	Green Roof
Partition	20		Area	3740
MEP	10		Unit Weight	100
Finishes	5		Total	374000
Total	546700			
Columns				
Shape	Quantity	Weight	Length	Total
W14x48	4	48	13.25	2544
W14x90	13	90	13.25	15502.5
W14x68	2	68	13.25	1802
w14x61	3	61	13.25	2424.75
W14x53	1	53	13.25	702.25
W14X74	1	74	13.25	980.5
W14X43	1	43	13.25	569.75
HSS6X6X1/4	2	19	13.25	503.5
HSS6X6X1/2	1	35	13.25	463.75
HSS8X8X3/8	1	38	13.25	503.5
Total				25996.5
Beams				
Shape	Quantity	Weight	Length(avg.)	Total
W16X26	25	26	25	16250
W18X35	13	35	30	13650
w10x12	19	12	10	2280
W16X31	12	31	25	9300
W21X50	3	50	35	5250
W24X55	7	55	35	13475
W27X84	14	84	20	23520
W33X141	3	141	40	16920
W21X44	10	44	25	11000
W30X116	4	116	30	13920
w24x62	9	62	35	19530
Total				145095
		Kips		psf
5th Floor Weight		1832		117

4th Floor Weight				
Area	15620			
Floor to Floor Height	13			
Exterior Walls			Slab	
Perimeter	493		Unit Weight	43.2
Unit Weight	10		Total	674784
Total	64090			
Superimposed			Special	-
Partition	20		Area	0
MEP	10		Unit Weight	0
Finishes	5		Total	0
Total	546700			
Columns				
Shape	Quantity	Weight	Length	Total
W14x48	5	48	13	3120
W14x90	14	90	13	16380
W14x68	3	68	13	2652
w14x61	6	61	13	4758
W14x109	2	109	13	2834
W14X74	1	74	13	962
W14X43	2	43	13	1118
HSS6X6X1/4	0	19	13	0
HSS6X6X1/2	0	35	13	0
HSS8X8X3/8	2	38	13	988
Total				32812
Beams				
Shape	Quantity	Weight	Length(avg.)	Total
W16X26	23	26	25	14950
W18X35	14	35	25	12250
W18X40	6	40	35	8400
W16X31	8	31	20	4960
W21X50	6	50	40	12000
W24X55	4	55	30	6600
W27X84	14	84	20	23520
W21X44	8	44	35	12320
W24X62	6	116	20	13920
W24X68	5	68	30	10200
Total				119120
		Kips		psf
4th Floor Weight		1438		92

3rdFloor Weight				
Area	15620			
Floor to Floor Height	13			
Exterior Walls			**Slab**	
Perimeter	493		Unit Weight	43.2
Unit Weight	10		Total	674784
Total	64090			
Superimposed			**Special**	-
Partition	20		Area	0
MEP	10		Unit Weight	0
Finishes	5		Total	0
Total	546700			
Columns				
Shape	Quantity	Weight	Length	Total
W14x48	5	48	13	3120
W14x90	14	90	13	16380
W14x68	3	68	13	2652
w14x61	7	61	13	5551
W14x109	2	109	13	2834
W14X74	1	74	13	962
W14X43	1	43	13	559
HSS8X8X3/8	2	38	13	988
Total				33046
Beams				
Shape	Quantity	Weight	Length(avg.)	Total
W16X26	22	26	25	14300
W18X35	11	35	20	7700
W18X40	6	40	35	8400
W16X31	10	31	25	7750
W21X50	6	50	35	10500
W24X55	5	55	35	9625
W27X84	13	84	25	27300
W36X160	1	160	35	5600
W21X44	9	44	30	11880
W24X62	2	116	30	6960
W24X76	7	76	20	10640
W24X68	4	68	35	9520
Total				130175
		Kips		psf
3rdFloor Weight		1449		93

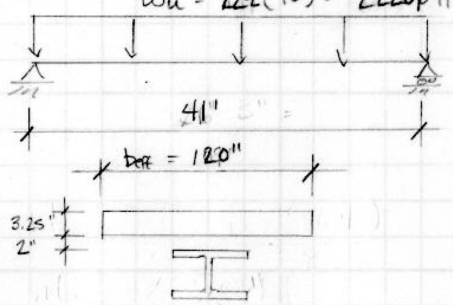
2nd Floor Weight				
Area	15000			
Floor to Floor Height	14.25			
Exterior Walls			Slab	
Perimeter	493		Unit Weight	43.2
Unit Weight	10		Total	648000
Total	70252.5			
Superimposed			Special	-
Partition	20		Area	0
MEP	10		Unit Weight	0
Finishes	5		Total	0
Total	525000			
Columns				
Shape	Quantity	Weight	Length	Total
W14x48	3	48	14.25	2052
W14x90	15	90	14.25	19237.5
W14x68	1	68	14.25	969
w14x61	1	61	14.25	869.25
W14x109	4	109	14.25	6213
W14X74	1	74	14.25	1054.5
W14X82	5	82	14.25	5842.5
W14X99	1	99	14.25	1410.75
W14X120	1	120	14.25	1710
W14X145	1	145	14.25	2066.25
HSS8X8X3/8	2	38	14.25	1083
Total				42507.75
Beams				
Shape	Quantity	Weight	Length(avg.)	Total
W16X26	15	26	25	9750
W18X35	18	35	25	15750
W18X40	3	40	35	4200
W16X31	18	31	25	13950
W24X55	8	55	25	11000
W27X84	21	84	25	44100
W40X211	1	211	25	5275
W21X44	6	44	35	9240
W24X68	2	68	35	4760
Total				118025
		Kips		psf
2nd Floor Weight		1404		94

1st Floor Weight				
excludes slab weight				
Area	15620			
Floor to Floor Height	15.66			
Exterior Walls			**Slab**	
Perimeter	493		Unit Weight	0
Unit Weight	10		Total	0
Total	77203.8			
Superimposed			**Special**	-
Partition	20		Area	0
MEP	10		Unit Weight	0
Finishes	5		Total	0
Total	546700			
Columns				
Shape	Quantity	Weight	Length	Total
W14x90	19	90	15.66	26778.6
W14x68	1	68	15.66	1064.88
w14x61	1	61	15.66	955.26
W14x109	3	109	15.66	5120.82
W14X74	1	74	15.66	1158.84
W14X82	4	82	15.66	5136.48
W14X99	1	99	15.66	1550.34
W14X120	1	120	15.66	1879.2
W14X145	1	145	15.66	2270.7
HSS6X6X3/8	1	0	15.66	
HSS6X6X5/16	4	0	13	0
HSS8X8X3/8	2	38	15.66	1190.16
Total				47105.28
Beams				
Shape	Quantity	Weight	Length(avg.)	Total
Total				0
		Kips		psf
1st Floor Weight		671		43

Appendix E: Spot Checks

SPOT CHECK: W21x50(30) C=3/4" @ 3RD FL

$w_u = 222(10) = 2220\text{ plf} = 2.22^{k/ft}$



Spacing = 10'
f'_c = 3 Ksi
f_y = 50 Ksi
w_{LL} 80 psf ← use corridor LL
w_{DL} 78.2 psf
2" Deck
No LL Reductions taken

$w_u = 1.2(78.2) + 1.6(80)$
$w_u = 222$ psf

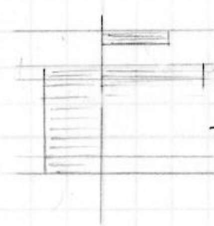
$M_u = \frac{w\ell^2}{8} = \frac{2.22(41^2)}{8}$
$M_u = 466^{1k}$

$b_{eff} \le \begin{cases} 10(12) = 120" \leftarrow \text{controls} \\ (41/4)(12) = 123" \end{cases}$

Start by assuming a = 1"
$\therefore Y_2 = 5.25 - 1/2 = 4.75"$

W16x40 PNA = 3 $\Sigma Q_n = 378$

$a = \frac{378}{0.85(3)(120)} = 1.23$



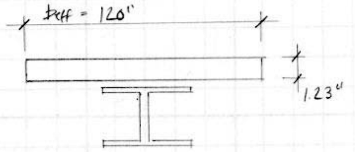
$C_c = 0.85(4)(1.23)(120) = 376^k$
$C_s = 2(50)(7)(0.306) = 214^k$
$T_s = 11.8(50) = 590^k$

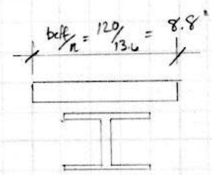
$M_n = 376(5.25 - 1.23/2) + 214(0.306/2) + 590(16/2) = 6430^{ik}$
$M_n = 535.8^{1k}$
$\phi M_n = 0.9(535.8) = 482^{1k} > 466^{1k}$ ∴ ok

Check Construction Loads
weight of conc. + deck = 43.2 psf
$w_{DL} = 43.2(10) + 40 = 472$ #/ft
Assume LL $w_u = 20\text{ psf}(10) = 200$ plf

$w_u = 1.2(472) + 1.6(200) = 886$ #/ft
$M_u = \frac{0.886(41^2)}{8} = 186.2^{1k} < \phi M_p = 274^{1k}$

SPOT CHECK: W21 x 50 (36) C = 3/4" @3RD FL.





$$n = \frac{29000}{115^{1.5}\sqrt{3}} = 13.6$$

$$I_{st} = 612$$

$$I_T = \frac{8.8\,(1.23^3)}{12} = 1.36\ in^4$$

$$A_T = 8.8(1.23) = 10.82\ in^2$$

$$A_s = 11.8\ in^2$$

$$\bar{y} = \frac{11.8\,(16.0/2) + 10.82\,(16 + 5.25 - 1.23/2)}{(11.8) + 10.82} = 10.5"$$

$$I_{tr} = 612 + 11.8\,(8 - 10.5)^2 + 1.36 + 10.82\,(16 + 5.25 - 1.23/2 - 10.5)^2$$

$$I_{tr} = 1731\ in^4$$

$$\Delta_{LL} = \frac{5}{384}\,\frac{w\ell^4}{EI} = \frac{5}{384}\,\frac{(0.782)(41^4)(1728)}{29000\,(1731)}$$

$$\Delta_{LL} = 0.99"$$

Limit: $\Delta_{max} = L/360 = 41(12)/360 = 1.37"$

$\Delta_{LL} < \Delta_{max}$ $\therefore$ OK

SPOT CHECK | W21x50 (30) $C = 3/4"$ @ 3RD FL | 2

Check Construction Deflection

$$\Delta = \frac{5}{384}\frac{(0.472)(41)^4(1728)}{29000\,(612)} = 1.69$$

Construction Deflection Criteria

$$\left| \begin{array}{l} L/360 = \frac{41(12)}{360} = 1.37" \\ 1" \longleftarrow \text{controls} \end{array} \right.$$

min

$\therefore$ Camber $3/4"$

$$\Delta = 1.69" - 0.75 = 0.94" < 1" \quad \therefore \underline{OK}$$

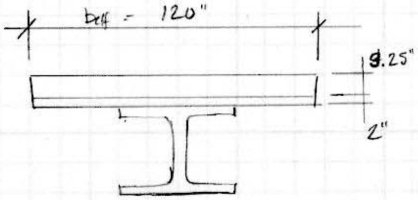
Number of Shear Studs

$$\Sigma Q_n = 378^k$$

$$\#\ \text{Studs} = 2\,(378/17.2) = 44$$

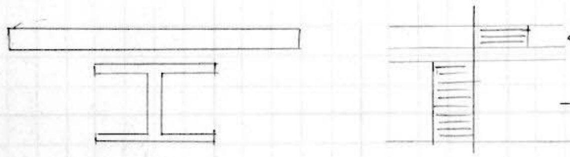
W16x40 [44] $C = 3/4"$

Member Capacity Check | W21x50 [30] c = 3/4" | @3RD fl.



$l = 41'$
$\Sigma Q_n = 258^k$
$f'_c = 3$ ksi
$f_y = 50$ ksi

$b_{eff} \le \begin{cases} S = 10(12) = 120" \leftarrow \text{controls} \\ l/4 = 41(12)/4 = 123" \end{cases}$

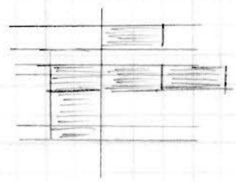


$\leftarrow C_c = 0.85 f'_c A_c = 994.5$

$\rightarrow T_s = A_s f_y = 735^k$

$C_c > \Sigma Q_n \quad \therefore \quad C_c = \Sigma Q_n = 258^k$

$a = \dfrac{258}{0.85(3)(120)} = 0.843"$



$A_{sc} = \dfrac{735 - 235}{2(50)} = 5 \text{ in}^2$

$A_f = 6.53(0.535) = 3.50 \text{ in}^2$

$A_{wc} = \dfrac{5 - 3.5}{0.38} = 3.95"$

$M_n = 238(5.25 - 0.843/2) + 735(20.8/2) - 2(50)(3.5)(0.535/2) - 2(50)(1.5)(0.535 + (3.95/2))$

$M_n = 8323^{\text{in-k}} \Rightarrow 693^{\text{'k}}$

$\phi M_n = 0.9(693) = 624^{\text{'k}}$

Allowable Load

$M = \dfrac{w l^2}{8}$

$w = 8M_u / l^2 = 8(624)/(41^2) = 2.97$ K/ft

$w_u = (2.97)(1000)/(10') = 297$ psf

Member Capacity Check | W21 x 50 [30] c = 3/4" @ 3RD FL

Construction Loads

$$\Delta_{max} = \begin{cases} L/360 = 41(12)/360 = 1.37" \\ 1" \leftarrow \text{Controls} \end{cases}_{min}$$

Camber = 3/4"

$\therefore \Delta_{max_{memb.}} = 1" + 3/4" = 1\,3/4"$

$$\Delta = \frac{5}{384}\frac{w\,\ell^4}{E\,I}$$

$$w_{cL} = \frac{384\Delta EI}{5\ell^4} = \frac{384(1.75)(29000)(984)}{5(41^4)(1728)}$$

$w_{cL} = \underline{0.741}$ K/ft

$w_{cL} = 74.1$ psf

Live Load Deflection

$n = \frac{29{,}000}{115^{1.5}\sqrt{3}} = 13.6$

$I_{st} = 984\ in^4$

$I_T = \frac{(120/13.6)(0.843^3)}{12} = 0.44$

$A_s = 14.7$

$A_T = 8.8(0.843) = 7.42\ in^2$

$$\bar{y} = \frac{14.7(20.8/2) + 7.42(20.8 + 5.25 - 0.843/2)}{(14.7 + 7.42)} = 15.5$$

$I_{tr} = 984 + 14.7(15.5-10.4)^2 + 0.44 + 7.42(20.8+5.25-0.843/2 - 15.5)^2$

$I_{tr} = 2128\ in^4$

$\Delta_{max} = L/360 = \frac{41(12)}{360} = 1.37"$

$1.37" = \frac{5}{384}\frac{w\,L^4}{EI}$

$$w_{max_{LL}} = \frac{1.37(384)EI}{5L^4} = \frac{1.37(384)(29{,}000)(2128)}{5(41^4)(1728)}$$

$w_{LL} = 1.33$ K/ft $\Rightarrow$ 133 psf.

SPOT CHECK. | Column ZB-Z6 W14x6

Loads (No Live Load Reduction)

Tributary Area

$A_T = (41/2)\,20 + (\tfrac{1}{2})(\tfrac{22}{2} + \tfrac{19}{2})(20) = 615\ \text{sf}$

Dead Loads

$P = (78.2)(615) \Rightarrow 48{,}093^{\text{lbs}} \Rightarrow 48.1^{k}$

Live Loads

$L_r = 20\,(615) = 12{,}300^{\text{lbs}} \Rightarrow 12.3^{k}$

$L = 80\,(615) = 49{,}200^{\text{lbs}} \Rightarrow 49.2^{k}$

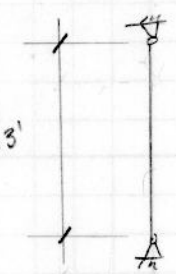
Snow

$S = 27.7\,(615) = 17{,}035 \Rightarrow 17^{k}$

Column Below Roof

$P_u = 1.2D + 1.6S$

$= 1.2(48.1) + 1.6(17) = 85^{k}$



13.3'

$KL_x = KL_y = 13.3'$

Check Capacity of W14x61

Weak Axis will Control

$\left(\frac{KL}{r}\right)_y = \frac{(13.3)(12)}{2.45} = 65.1$

A992 Steel

$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000}{50}} = 113$

$KL/r = 65.1 < 113$

$F_{cr} = \left(0.658^{(F_y/F_e)}\right)F_y$

$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2(29000)}{(65.1)^2} = 67.5\ \text{ksi}$

$F_{cr} = \left(0.658^{(50/67.5)}\right)50 = 36.7$

$P_n = 36.7\,(17.9) = 657^{k}$

$\phi P_n = 0.9\,(657) = 591^{k} > 85^{k}\ \therefore\ \underline{ok}$

Find the least weight W14 to carry the Load.

W14x43 still higher capacity than Demand

$\phi P_n = -\left(\frac{13.3 - 13}{14 - 13}\right)(345 - 319) + 345 = 337^{k} > 85^{k}\ \therefore\ \underline{ok}$

	SPOT CHECK	Column Z13-Z6	2

Column Below 5th Floor

$P_u = 1.2D + 1.6L + 0.5S$

$P_u = 1.2(48.1)(2) + 1.6(49.2) + 0.5(17)$

$P_u = 203^k$

$KL_y = KL_x = 13'$

Capacity W14x61

$\phi P_n = 599^k > 203^k \therefore$ ok

Least Weight W14 $\rightarrow$ W14x43

$\phi P_n = 345^k > 203^k \therefore$ ok

Column Below 4th Floor

$P_u = 1.2D + 1.6L + 0.5S$

$P_u = 1.2(48.1 \times 3) + 1.6(49.2 \times 2) + 0.5(17)$

$P_u = 339.1^k$

$KL_x = KL_y = 13'$

Capacity W14x61

$\phi P_n = 599 > 339.1^k \therefore$ ok

Least Weight W14 $\rightarrow$ W14x43

$\phi P_n = 345 > 339.1^k \therefore$ ok

Column Below 3RD Floor

$P_u = 1.2D + 1.6L + 0.5S$

$P_u = 1.2(48.1(4)) + 1.6(49.2 \times 3) + 0.5(17)$

$P_u = 476^k$

$KL_x = KL_y = 14.25'$

Capacity W14x82

$\phi P_n = 774 - \left(\frac{14.25 - 14}{15 - 14}\right)(774 - 736)$

$\phi P_n = 764.5^k > 476^k \therefore$ ok

Least Weight W14 $\rightarrow$ W14x61

$\phi P_n = 572 - \left(\frac{14.25 - 14}{15 - 14}\right)(572 - 543)$

$\phi P_n = 565^k > 476^k \therefore$ ok

Column Below 2nd Floor

$P_u = 1.2D + 1.6L + 0.5S$

$P_u = 612^k$

$KL_x = KL_y = 15.6'$

Capacity W14x82

$\phi P_n = 736 - \left(\frac{0.25}{1}\right)(736 - 698)$

$\phi P_n = 726.5^k > 612^k \therefore$ ok

Least Weight W14 $\rightarrow$ W14x74

$\phi P_n = 667 - \left(\frac{0.25}{1}\right)(667 - 632)$

$\phi P_n = 658 > 612 \therefore$ ok