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Structural Option
Technical Assignment #1
Thesis Advisor – Thomas E. Boothby

Sallie Mae Headquarters
Reston, VA

Executive Summary

The Sallie Mae HQ is located in Reston, VA. Sallie Mae features a 5 level parking garage, and a 9 story office building. The parking garage enables spread footings to be used since the bottom of the parking garage consist of soil classified as rock. Spans reach 28' and normal column sizes are 30" x 30" concrete columns. The first exposed floor is the third level (P-3) of the garage, but ground level (P-1) is the first level exposed on the east side of the building which is the main entrance. The garage is made from a two-way slab system that features drop panels. The ground level of the parking garage primarily consists of a fire-lane, athletic field and heavy planters, with about 20 parking spots total. The load that can be caused by the moving vehicles, and heavy planters and caused post-tensioned beams to be used on the P-1 level.

The office's structure is a steel framing system that features braced concentric frames in the E-W direction, and mixed eccentric and concentric frames in the N-S direction. Since lateral loads in the E-W direction cause the most load, the strong axis of most columns are lined up in that direction. According to calculations the wind loads are much larger than any seismic loads. Normal floor have a 2"- 18 gage deck with 3.25" light weight concrete. However, the penthouse floor contains the elevator room and heavy mechanical equipment, so the slab depth and beam strength must be increased for that floor. Due to architectural requirements the spans of some of the steel beams reach up to 55'. Camber must be used to counteract deflection.

In analyzing any possible ways to redesign this building while maintaining the architectural spans, it becomes clear that either heavily reinforced concrete, a different steel framing method, or post-tensioned concrete are viable considerations.

Building and Site

Sallie Mae HQ is located at the southeast quadrant of the intersection of Town Center Parkway and Bluemont Way, along the south side of Bluemont Way in Reston, Fairfax County, Virginia. Sallie Mae consists of a steel tower office building and a concrete parking garage. The office building is nine stories above grade and has a ground print of approximately 25,000sf. It is located in the west portion of the site, at the southeast quadrant of the intersection of Town Center Parkway and Bluemont Way, along the south side of Bluemont Way. The office building's lowest floor (P-1) is located at an elevation of 403.5'. The parking garage is five stories and extends downward from P-1 to P-5, which is located at an elevation of 364'. The garage plan is approximately 75,000sf, and is located on the east portion of the site, along the south side of Bluemont Way.

Codes Used

- BOCA 96*
- VIRGINIA UNIFORM STATEWIDE BUILDING CODE – 2000*
- ASCE 7-98*
- ACI 318-99*
- ASD MANUAL – 9th EDITION

Loads

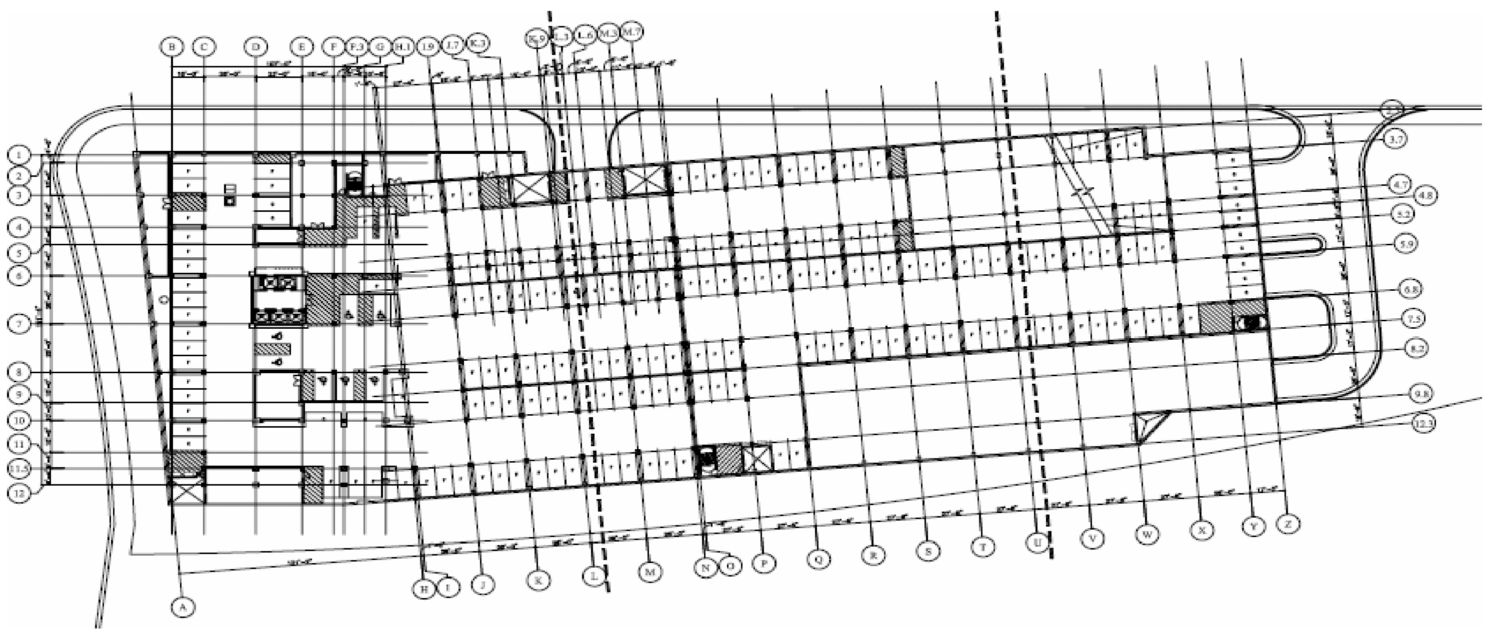
LL – 80 PSF for upper corridors in an office building/ 100 PSF for 1st floor corridors and lobbies
Minimum RL if snow doesn't control – 30 PSF
Mechanical Rooms – 150PSF
Stairs – 100 PSF
Parking Garage – 50 PSF

Foundations and Soil

Sallie Mae's geotechnical work was performed by Engineering Consulting Services, LTD. A total of 20 borings were performed, with the addition of two pressuremeter tests. Shallow spread footings were recommended due to the deep depth needed for the parking garage, and the soil at 364' El. was considered rock. The footings were recommended to be designed using an allowable bearing pressure of 30,000psf. The majority of the columns were supported on square isolated spread footings that ranged from 4' to 10' wide. The foundation walls that extend around the perimeter of the building range from 1' -1'-4" thick.

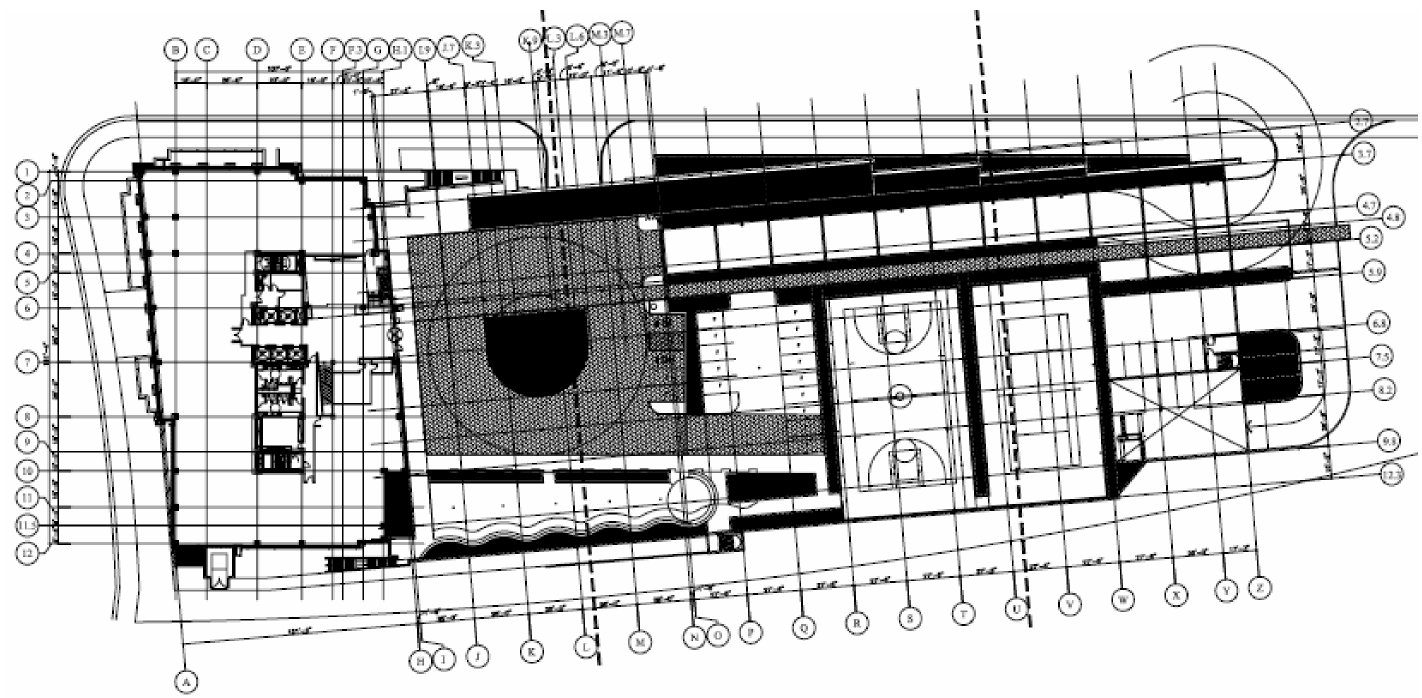
Structural Overview for Parking Garage Levels

Gravity:



Picture (P-5)

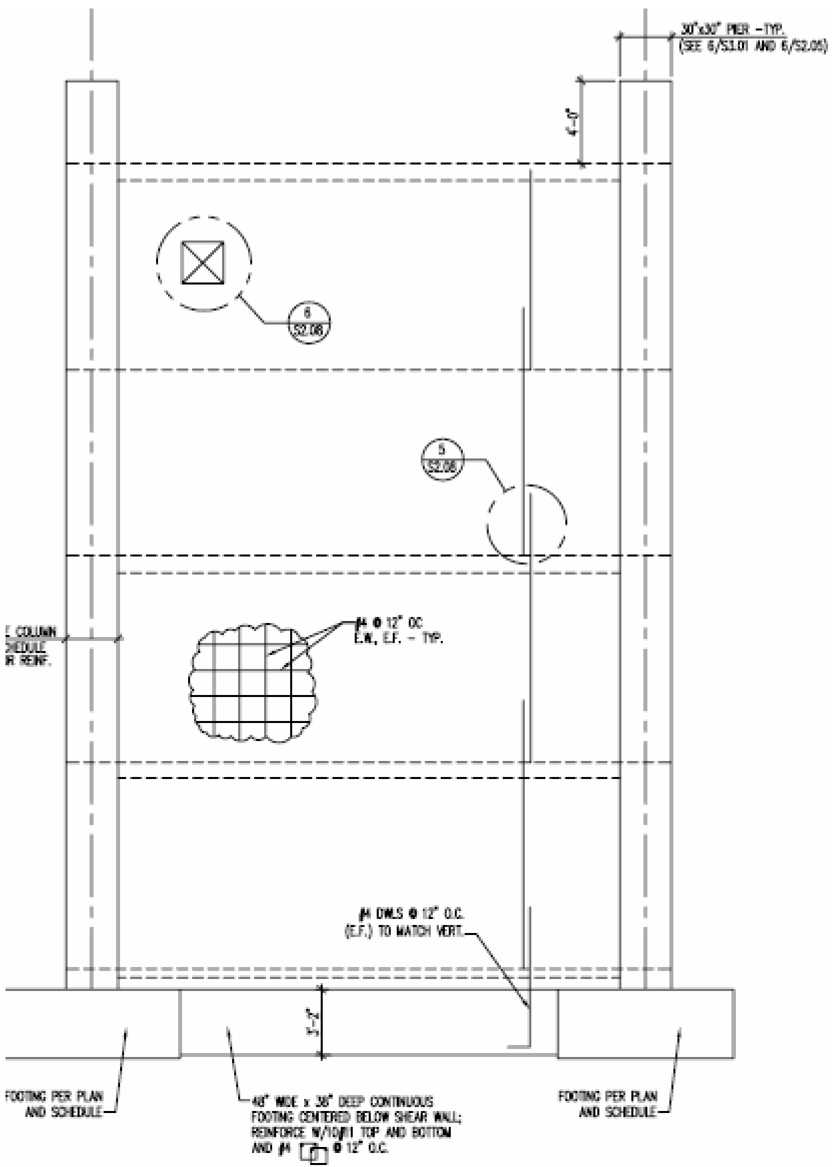
The parking structure of Sallie Mae consists of 5 levels. The picture shows the lowest level of the parking structure which is located at an elevation of 364’ and consists of a 5” slab on grade that has 3500 psi in strength. The level above, P-4, consist of a 9” two-way slab with drop panels that has a strength of 5000 psi. These spans get up to 28’ in the parking garage, and bottom reinforcing for levels P-3 and above is #5 @ 12. The reason for the increase in reinforcement compared to #4 @ 12 (P-4 and P-5) is because on P-3 there is a portion of the parking garage that is a cafeteria. And at some locations on this level there is a 14” slab with a 5” topping for architectural purposes. P-2 has the same reinforcing as P-3. P-1 is ground level from the east side main entrance; this floor is the transition floor between office building and parking garage. Where originally the parking garage was on the lower floors, is now a fire lane which consist of a large circle loop, athletic courts, and planters that extend around the building. Due to the larger loads that these three new items impose, post-tensioning was needed in some beams in the 1st floor.



Picture (P-1)

Lateral:

Typical Shearwall



The lateral elements in the parking garage consist of 8 shearwalls; 4 span in each direction. These shearwalls continue up to the first floor level, and then connect to braced frames that extend all the way up the building.

- Spot check

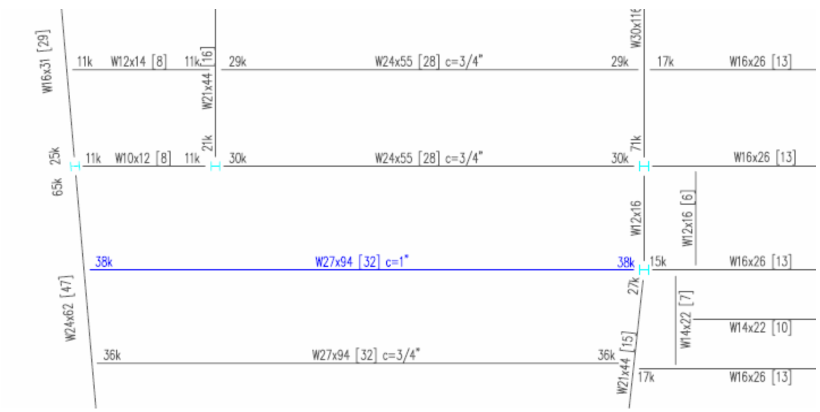
Using tributary area of 55'/194' and using that ratio for the 233k force from the E-W earthquake load. The largest force going into a shearwall is 66k. There is 23' of length so approximately 3k needs to be transferred per foot of length. #4's @ 12 are ok.

Structural Overview for Office Floors

Gravity:

The office part of Sallie Mae mostly consists of floors that resemble the typical floor on the next page. There are four line of columns along the long direction (N-S), and girders also span between those columns in the N-S direction. The beams span E-W, and reach spans up to 55 ft. These longer spans require camber due to deflection. The typical floors are made up of 3.25" lightweight concrete over 2" Deep x 18 gage galvanized composite metal deck. The connections consist of primarily simple shear connections with the exception of very few moment connections. The Penthouse houses all the heavy mechanical equipment including the elevator room. Slabs range from composite metal deck to12" reinforced concrete at the Penthouse level.

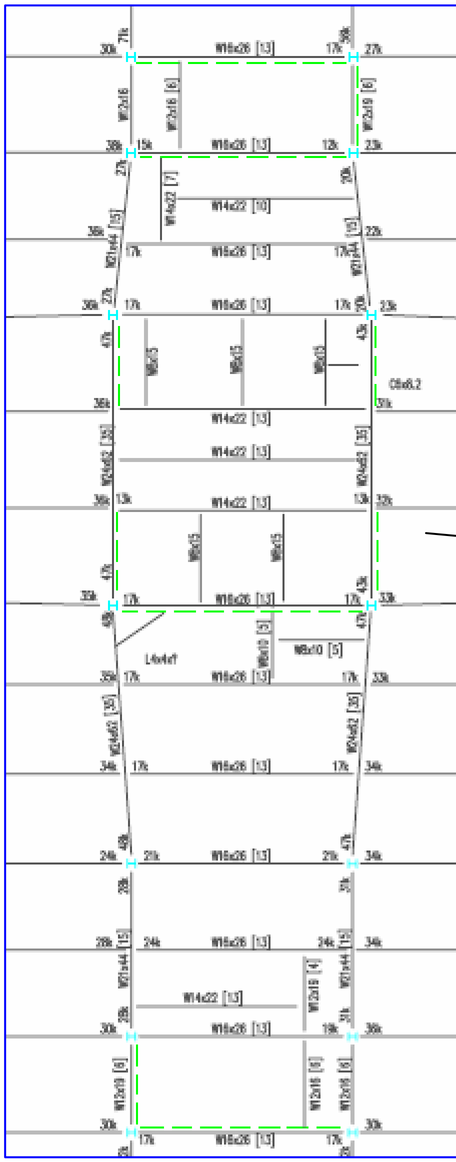
- Spot Check



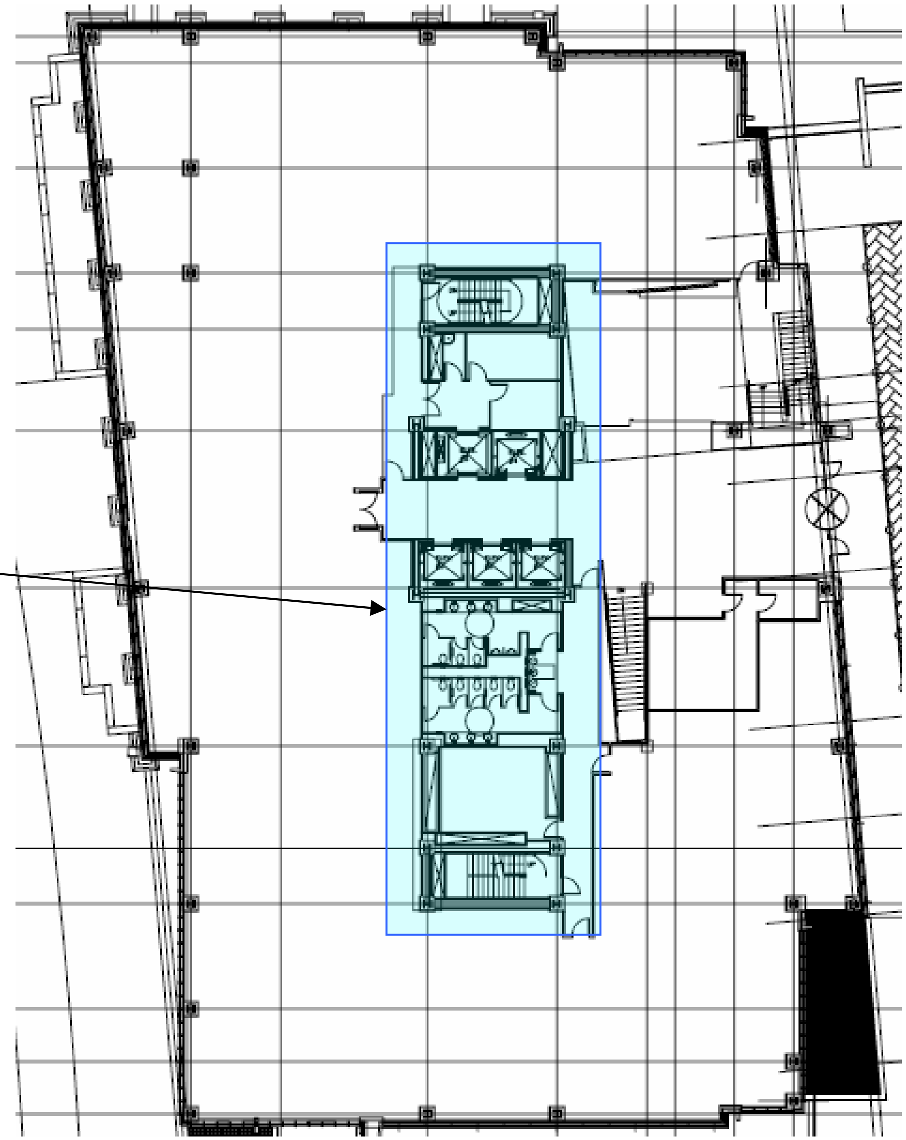
I checked this W27 x 94 due to the fact that it's the longest spanning member in the building. Due to the rough hand analysis that would be required from the beam being connected to a girder on one end and a beam on the other end, I used Ram Steel to analyze the beam, and used ASD method so I could directly compare if something was off. Using ¾ in diameter studs, I came up with 25% full composite actions with the 2" steel deck and light weight concrete. The deflection failed at first because I added no camber onto it. Trying the 1" camber that they originally designed the beam with the total load on the beam, the deflection came out to be -1.27" which provides an L/D of 519 > then the required 400.

Lateral:

Lateral Resisting System Plan



Typical



Sallie Mae consists of concentrically braced frames in the east-west direction, and in the north-south direction is eccentrically braced and concentrically braced frames. These frames are shown in the drawing above as green, and are placed in the two stairwell walls and in the elevator shaft. Due the possibility of heavy wind loads there are four sets of concentric frames in the E-W direction that extend from ground level all the way to the Penthouse. The N-S direction does not need as much bracing because the stiffness in that direction is already high due to the longer set of spans. The eccentric frames in the N-S direction also allow for higher energy dissipation in the case of a seismic event.

LOAD CALCULATIONS

Dead Load Calculation per floor disregarding special considerations

- Assume 8 PSF for dead weight of steel beams
- Steel decking with lightweight concrete is 115 PCF x 3.25/12 = 32 PSF + 2 PSF for Deck = 34 PSF
- Miscellaneous Equipment = 10 PSF
- Total Dead = **52 PSF**

Snow Load Calculations assuming flat roof

- pg = 30 PSF
- Ct =1.0
- Cs =1.0
- I =1.1
- pf = 30 PSF x 1.1 = **33 PSF**

Wind Load Calculation (E-W)

Total Building Height	127.01	ft.	OK
Design Wind Speed	90	mph	AISC Figure 6.1
L	136	ft	Horizontal dim. of bldg, parallel to wind
B	194	ft	Horizontal dim. of bldg, normal to wind

Exposure Category	B	ASCE7 - 6.5.6.3	Kd	0.85	ASCE7 - Table 6-4
Importance Factor	1.15	ASCE7 - Table 6-1	Kzt	1	ASCE7 - Figure 6-4
GCpi	-0.18	ASCE7 - Table 6-7			

Cpw, windward wall	0.8	ASCE7 - figure 6-6	use with qz
Cpl, leeward wall	-0.5	ASCE7 - figure 6-6	use with qh
Cps, side wall	-0.7	ASCE7 - figure 6-6	use with qh

ASCE - Table 6-2

zg (ft)	1200
α	7
zmin (ft)	30
c	0.3
l (ft)	320
ε	1/3

6.5.8 Gust Effect Factor - For Rigid Structures

zbar	76.206
Lzbar	423
Q	0.81
Izbar	0.26
G	0.82

Leeward wall pressure

qh	21.4
pl (+GCpi)	-12.6
pl (-GCpi)	-4.9

Side wall pressure

qh	21.4
pl (+GCpi)	-16.1
pl (-GCpi)	-8.4

Windward wall pressure

Floor #	Height (ft.)	z (ft.)	*Ae (ft^2)	**Kz	qz (psf)	pw (psf)	p (psf) (+GCpi)	p (psf) (-Gcpi)	p (psf) (Total)	***F (kip) max(p)
1	14									
2	14	14	2716	0.57	11.6	7.6	3.8	11.5	16.41	44.56
3	14	28	2716	0.69	13.9	9.1	5.3	13.0	17.89	48.60
4	14	42	2716	0.77	15.6	10.2	6.4	14.1	19.01	51.64
5	14	56	2716	0.84	17.0	11.1	7.3	15.0	19.89	54.03
6	14	70	2716	0.89	18.1	11.8	8.0	15.7	20.62	56.01
7	14.67	84	2780.99	0.94	19.1	12.5	8.6	16.3	21.26	59.12
8	15.67	98.67	2942.98	0.98	20.0	13.1	9.2	16.9	21.84	64.29
PH	12.67	114.34	2748.98	1.03	20.8	13.6	9.8	17.5	22.41	61.59
Roof		127.01	1228.99	1.06	21.4	14.0	10.2	17.9	22.82	28.05

Wind Load Calculation (N-S)

Total Building Height	127.01	ft.	OK			
Design Wind Speed	90	mph	AISC Figure 6.1			
L	194	ft	Horizontal dim. of bldg, parallel to wind			
B	136	ft	Horizontal dim. of bldg, normal to wind			
Exposure Category	B	ASCE7 - 6.5.6.3		Kd	0.85	ASCE7 - Table 6-4
Importance Factor	1.15	ASCE7 - Table 6-1		Kzt	1	ASCE7 - Figure 6-4
GCpi	-0.18	ASCE7 - Table 6-7				
Cpw, windward wall	0.8	ASCE7 - figure 6-6	use with qz			
Cpl, leeward wall	-0.3	ASCE7 - figure 6-6	use with qh			
Cps, side wall	-0.7	ASCE7 - figure 6-6	use with qh			

ASCE - Table 6-2

zg (ft)	1200
α	7
zmin (ft)	30
c	0.3
l (ft)	320
ε	1/3

6.5.8 Gust Effect Factor - For Rigid Structures

zbar	76.206
Lzbar	423
Q	0.83
Izbar	0.26
G	0.83

Leeward wall pressure

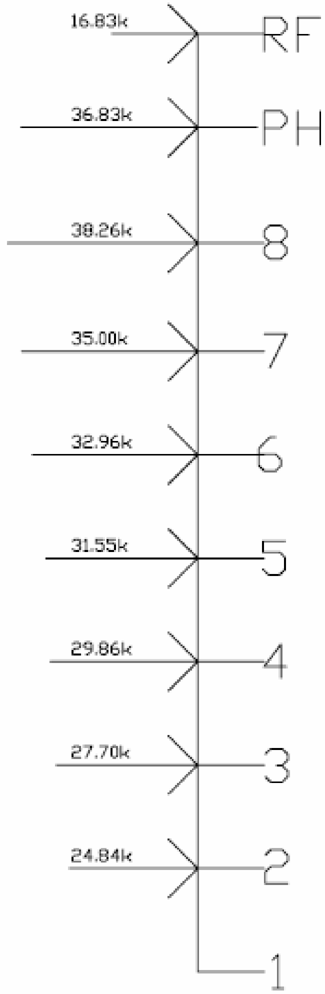
qh	21.4
pl (+GCpi)	-9.2
pl (-GCpi)	-1.5

Side wall pressure

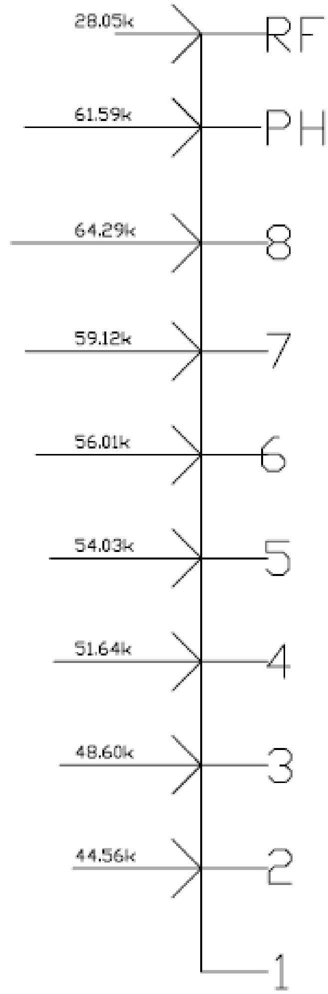
qh	21.4
pl (+GCpi)	-16.3
pl (-GCpi)	-8.6

Windward wall pressure

							p (psf)			***F (kip)
Floor #	Height (ft.)	z (ft.)	*Ae (ft^2)	**Kz	qz (psf)	pw (psf)	(+GCpi)	(-Gcpi)	(Total)	max(p)
1	14									
2	14	14	1904	0.57	11.6	7.7	3.9	11.6	13.04	24.84
3	14	28	1904	0.69	13.9	9.2	5.4	13.1	14.55	27.70
4	14	42	1904	0.77	15.6	10.4	6.5	14.2	15.68	29.86
5	14	56	1904	0.84	17.0	11.2	7.4	15.1	16.57	31.55
6	14	70	1904	0.89	18.1	12.0	8.1	15.8	17.31	32.96
7	14.67	84	1949.56	0.94	19.1	12.6	8.8	16.5	17.95	35.00
8	15.67	98.67	2063.12	0.98	20.0	13.2	9.4	17.1	18.55	38.26
PH	12.67	114.34	1927.12	1.03	20.8	13.8	9.9	17.6	19.11	36.83
Roof		127.01	861.56	1.06	21.4	14.2	10.3	18.1	19.53	16.83



Wind (E-W)



Wind (E-W)

Seismic Load Calculation:

Assumptions

Roof area is same as floor area

Enter Building Properties

	L	B
N-S	136	194
E-W	194	136

Max height 127.01

Average Floor to Floor Height 15.000 ft

Floor Area 26384 ft^2

Loads

Decking/Slab DL	25	psf	x	26384		=	660	kips				
Framing DL	8	psf	x	26384		=	211	kips				
Partition DL	10	psf	x	26384		=	264	kips				
Fascade DL	15	psf	x	15	ft	x	660	ft	=	149	kips	

Total DL	1283	kips					
LL included in seismic	15	psf	x	26384.000	=	395.760	kips
Total Load/floor	1678.772	kips					

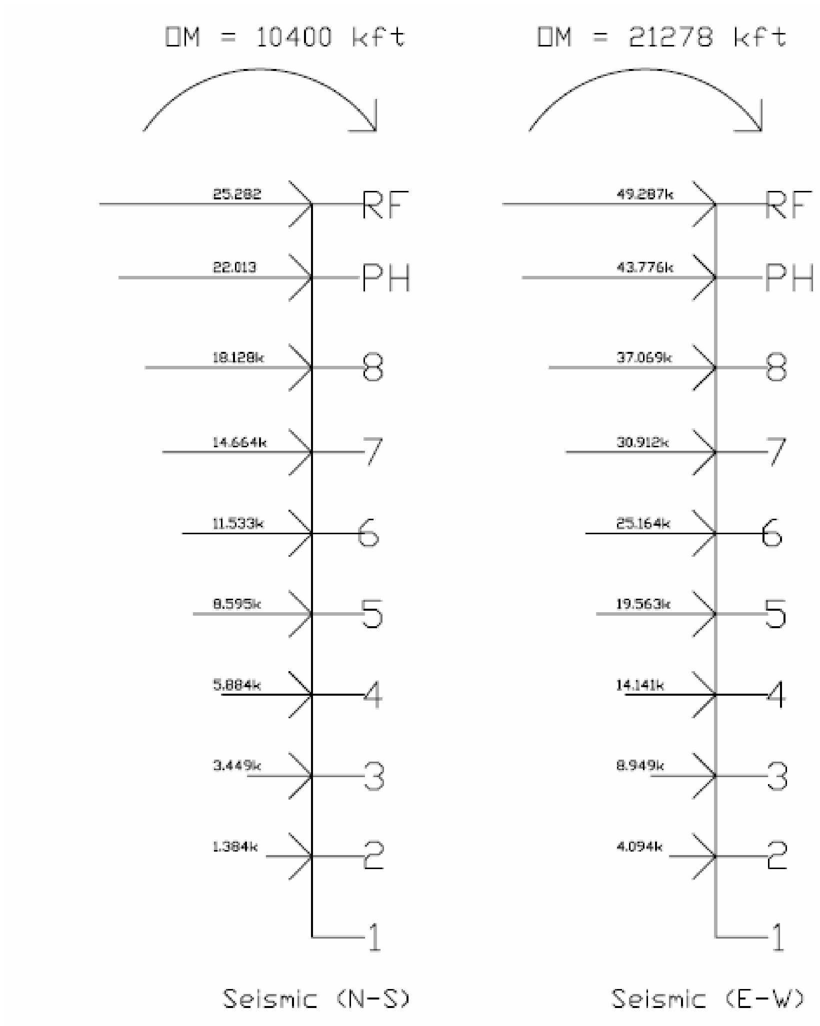
The Occupancy Category (Table 1-1)	3
Seismic Use Group (Table 9.1.3)	2
Importance Factor (9.1.4)	1.25
Site Classification (Table 9.4.1.2)	B
S_s (Figure 9.4.1.1(a))	0.195
S_1 (Figure 9.4.1.1(b))	0.070
Fa (Table 9.4.1.2.4a)	1.000
F_v (Table 9.4.1.2.4b)	1.000
$S_{ms}=F_aS_s$ (Eq. 9.4.1.2.4-1)	0.195
$S_{m1}=F_vS_1$ (Eq. 9.4.1.2.4-2)	0.070
$S_{DS}=2/3*S_{MS}$ (Eq. 9.4.1.2.4-1)	0.130

S_{D1}=2/3*S_{M1} (Eq. 9.4.1.2.4-2) 0.047

N-S direction		*E-W direction*	
R_{N-S}	7.000	R_{E-W}	5.000
T(N-S)	1.135	T(E-W)	0.757
$C_{s(N-S)}$	0.023	$C_{s(E-W)}$	0.033
$C_{s\ max(N-S)}$	0.007	$C_{s\ max(E-W)}$	0.015
$C_{s\ min}$	0.007	$C_{s\ min}$	0.007
k(N-S direction)	1.318	k(E-W direction)	1.128
V(N-S direction)	110.931	V(E-W direction)	232.955

Level	w_x	h_x	$w_xh_x^{1.318}$	$w_xh_x^{1.128}$	C_{vx}(N-S)	C_{vx}(E-W)	F_x(N-S)	F_x(E-W)	M(N-S)	M(E-W)
Roof	1678.772	127.010	992678.6952	397035.359	0.228	0.212	25.282	49.287	3211.116	6259.959
PH	1678.772	114.340	864327.1018	352640.515	0.198	0.188	22.013	43.776	2517.014	5005.354
8	1678.772	98.670	711771.5163	298609.709	0.163	0.159	18.128	37.069	1788.690	3657.576
7	1678.772	84.000	575757.107	249015.625	0.132	0.133	14.664	30.912	1231.765	2596.631
6	1678.772	70.000	452811.6788	202713.884	0.104	0.108	11.533	25.164	807.281	1761.513
5	1678.772	56.000	337472.3474	157592.801	0.077	0.084	8.595	19.563	481.322	1095.541
4	1678.772	42.000	231010.0008	113910.41	0.053	0.061	5.884	14.141	247.109	593.905
3	1678.772	28.000	135403.4302	72089.6955	0.031	0.038	3.449	8.949	96.560	250.574
2	1678.772	14.000	54327.67763	32976.9137	0.012	0.018	1.384	4.094	19.371	57.312

Total W	15108.948	Σ= 4355560	Σ= 1876585	Σ= 1	Σ= 1	Σ= 111	Σ= 233	Σ= 10400	21278



□M = 10400 kft
□M = 21278 kft
25.282
22.013
18.128k
14.664k
11.533k
8.595k
5.884k
3.449k
1.384k
49.287k
43.776k
37.069k
30.912k
25.164k
19.563k
14.141k
8.949k
4.094k
RF
PH
8
7
6
5
4
3
2
1
Seismic (N-S)
Seismic (E-W)