Technical Report I

Student Health Center

Penn State University

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Executive Summary

The Student Health Center (SHC) is a five story building on the Penn State campus that serves as a health care services and hospital facility. After completion in the fall of 2008, this building now houses University Health Services and Counseling and Psychological Services, two departments of Penn State's Division of Student Affairs.

The facility is 77 feet in height from the first level and is approximately 64,000 SF in area. It has a brick façade rising from the ground with large curtain wall on the south side the building. The structure is held up primarily by a steel frame. The overall structure sits on a mini-pile foundation through use of pile caps, piers, and grade beams. Composite steel with concrete slab on deck is use for the floor system throughout the SHC.

In this technical report, the existing conditions of the SHC are highlighted in detail. Materials, lateral systems, foundations, floors, codes, and other information will be provided. Several loads were calculated to see what affects dead, live, snow, wind, and seismic activity would have on the structure. Also, spot checks were performed on gravity members to check the sizes in relation to the engineer's design.

After calculation, I found that wind in the N-S wind direction caused the greatest pressures on the building. The controlling wind base shear and overturning moment were 337.93 K and 13,648 ft-K respectively in the N-S direction. Seismic loads calculated produced a base shear of 291K and an overturning moment of 45,311 ft-K. The base shear was close to the engineer's calculated base shear of 252K, with differences possibly due to differing values of total building weight and/or interpretation of the code. The spot checks of a typical beam and girder produced the same results as those shown on the drawings but my calculation of a base column differed from the drawings slightly. I calculated a need for a W14x61 and the drawings showed a W14x48 being used. This is probably due to my estimation of loads. More detailed calculations and explanations are shown further in the report.

Structural Systems

Foundation:

The foundation of the SHC is composed of grade beams and piers that are supported by mini-piles with pile caps. The mini-piles are arranged in configurations of 1-5 piles per pile cap. They are to be at a depth of 45 feet and have an 80 ton allowable capacity. The partially-restrained moment frame employed in this building is either connected directly to a pile cap or to a concrete pier. The depth of these mini-piles will counteract the moment of the partially-restrained moment frame caused by lateral loads. Locations of the piles are shown in *(Fig. 1)*.

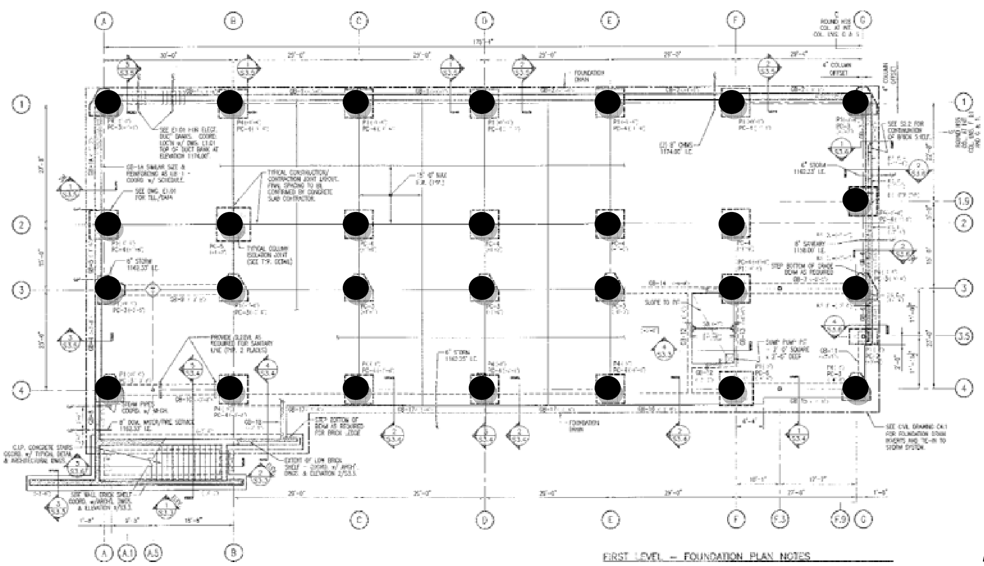


Fig. 1 – Pile Locations

Floor System / Beams:

The floor system used in the SHC is composed of 3 1/4” lightweight concrete fill on 2”-20 gage galvanized composite floor deck LOK floor for a total slab thickness of 5 1/4". Also included are 3/4ϕ x 4” long shear studs equally spaced along the entire lengths of all interior beams and girders that are not part of the partially-restrained moment frame. The shear studs are not on the moment frame because the beams on the frame cannot be too rigid so that they can deform. This composite floor deck is supported by steel W-shape beams spanning between steel columns.

Columns:

The P.R. moment frame consists of W14 steel columns running from the foundation up to the roof level. Columns that are not part of the P.R. moment frame range in size and shape. Round HSS shapes are used both with and without concrete fill, as well as square HSS shapes and W shapes to resist gravity loads.

Roof / Penthouse Level:

The roof system is composed of 5 1/4" normal weight concrete fill on 3"-20 gage galvanized composite floor deck LOK floor for a total slab thickness of 8 1/4". The main roof is at the 6th level with a screen wall around the rooftop mechanical equipment. There is also a green roof around the perimeter of the main roof level *(Fig. 2)*. On the north end of the building, at the 5th level, there is another green roof *(Fig. 3)* that is nearly 20 feet wide and runs the length of the building.

Fig. 2 – Green Roof on Main Roof

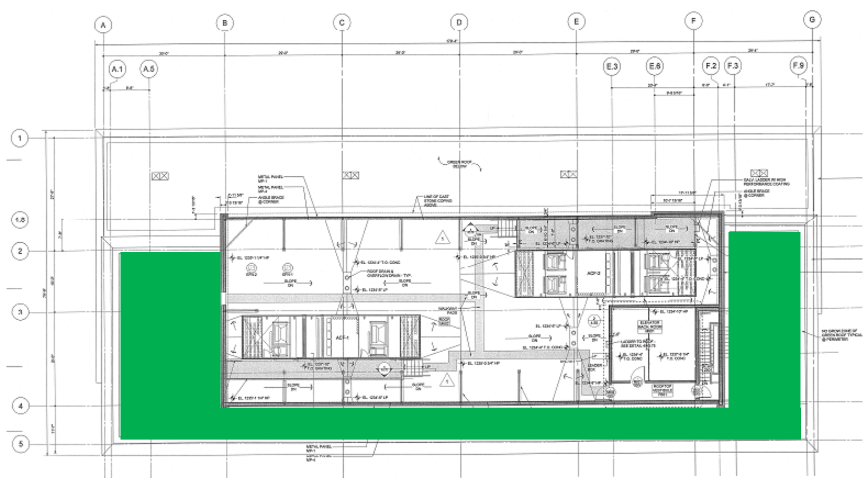


Fig 3 – Green Roof on 5th Floor

Lateral System:

A partially-restrained moment frame is used to resist lateral loads on the SHC. These frames are to have Flexible Moment Connections (FMC) designed by the steel fabricator per Part 11 of the AISC- Load & Resistance Factor Design Manual. A typical beam to column flange connection for these frames is detailed below *(Fig. 4)*. There are eight partially-restrained frames employed in this building, with seven running in the north/south direction, and one in the east/west direction *(Fig. 5)*. These frames run vertically up to the 5th Level or Main Roof Level of the building depending on the location. Frames are shown below in elevation (Fig. 6-8).

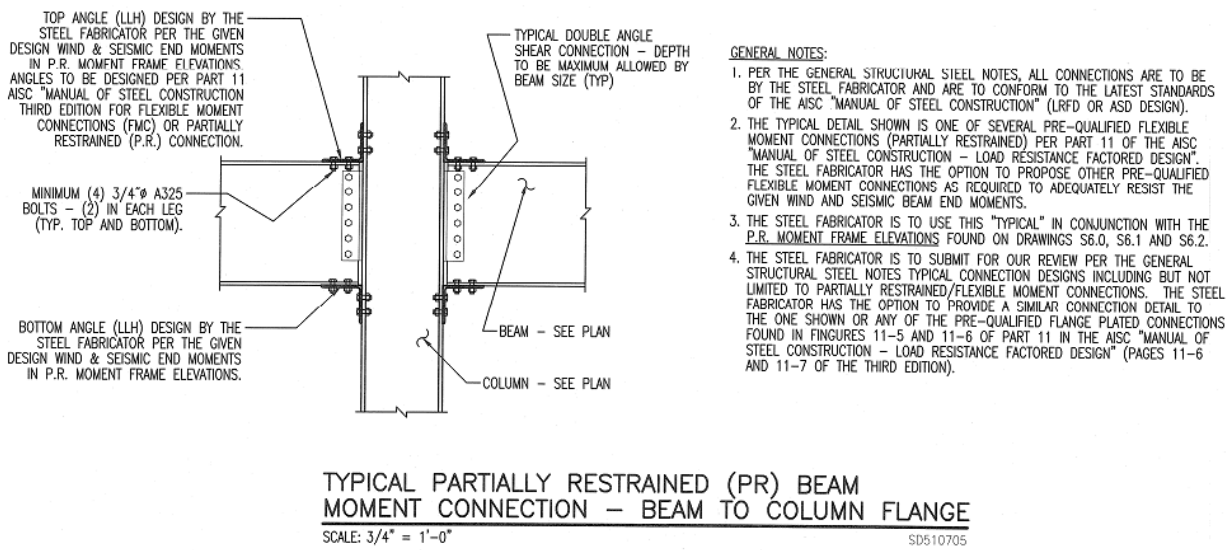


Fig. 4

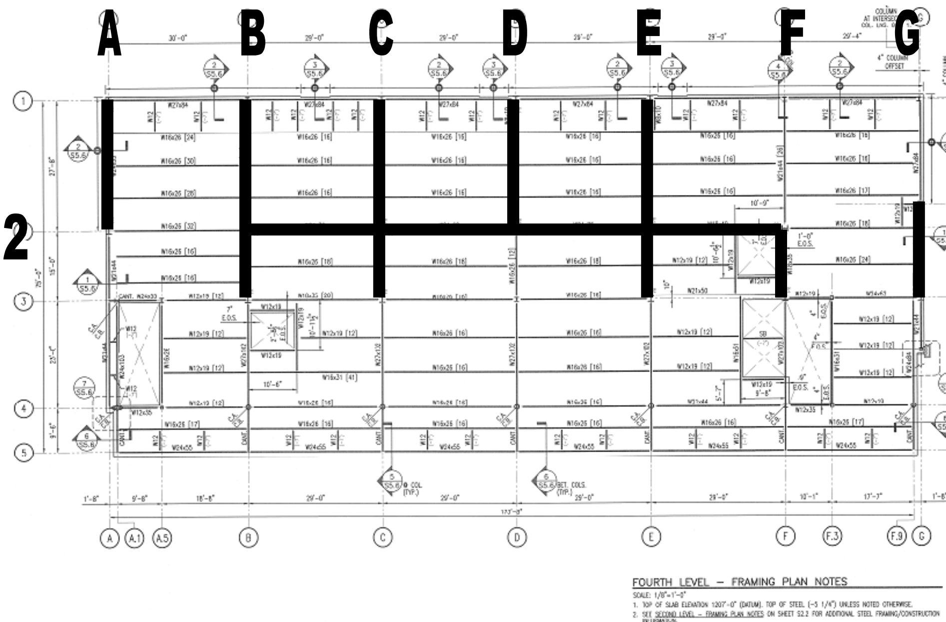


Fig. 5 – Partially-restrained Frame Locations

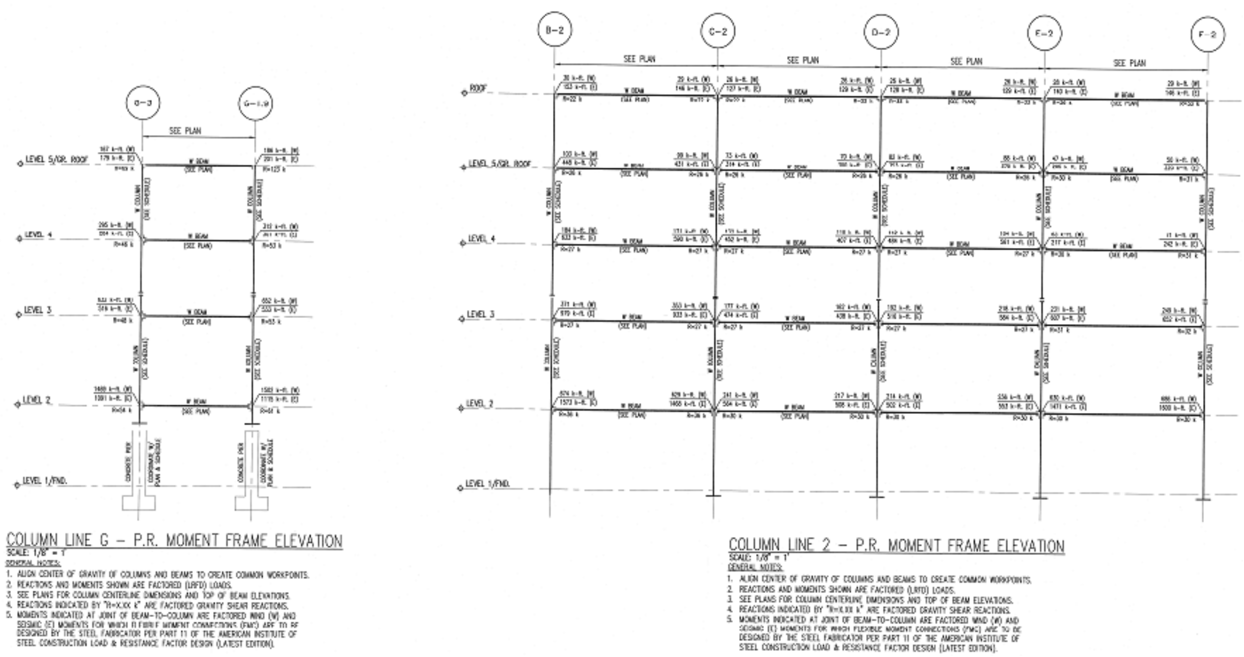


Fig. 6 – P.R. Moment Frame Elevations (G and 2)

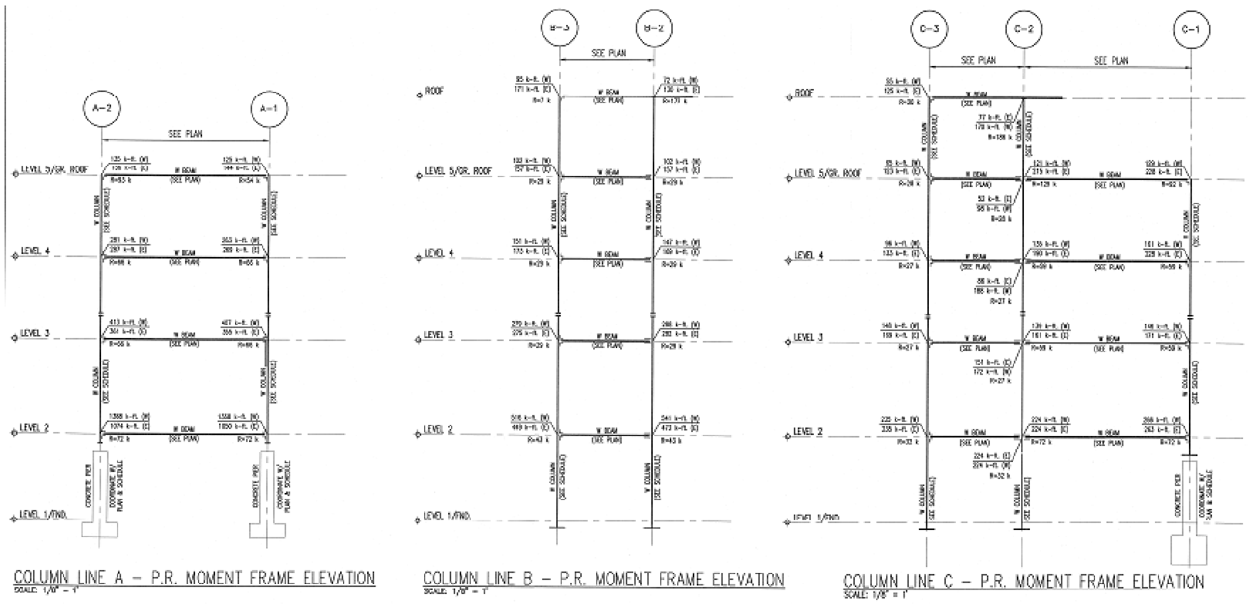


Fig. 7 – P.R. Moment Frame Elevations (A, B, and C)

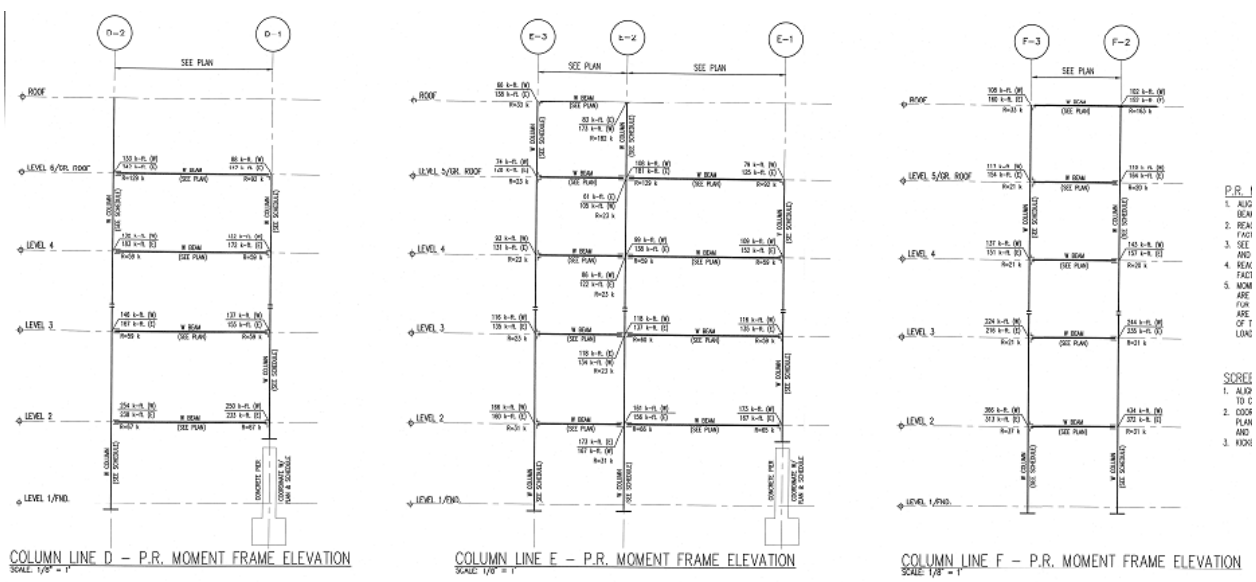


Fig. 8 – P.R. Moment Frame Elevations (D, E, and F)

Code and Design Requirements

Design Codes and References:

Codes used by Project Team:

International Building Code (IBC)/2003 with Borough Amendments
International Mechanical Code (IMC)/2003 with Borough Amendments
International Plumbing Code (IPC)/2003 with Borough Amendments
International Energy Conservation Code (IECC)/2003 with Borough Amendments
International Code Council Electrical Code (ICCEC)/2003
International Fire Code (IFC)/2003
ACI 318-05
AISC "Steel Construction Manual" (13th Edition)
ACI 530.1/ASCE 6/TMS 602 (2005)

Codes used for Thesis:

International Building Code (IBC)/2006
ACI 318-08
AISC "Steel Construction Manual" (13^{th} Edition)
ASCE 7-05

Deflection Criteria:

Maximum Floor Deflections:
- L/360 Live load
- L/240 Total load
- L/240 Roof

Maximum Lateral Deflections:
- L/600 Building Drift due to wind
- L/400 Story Drift due to wind

Material Properties

Material	A.S.T.M.	Minimum Strength
Concrete		
Foundation Walls, Pile Caps, Slab on Grade, Retaining Walls, Footings	-	3000 PSI
Exterior Slabs, Curbs	-	4000 PSI
Reinforcement	A615 (Grade 60)	60 KSI
WWF	A185, A497	70 KSI
Structural Tubing, Round	A500 (Grade B)	42 KSI
Structural Tubing, Shaped	A500 (Grade B)	46 KSI
Steel Pipe	A53 (Type E, Grade B)	35 KSI
Rolled Shapes	A992	50 KSI
Other Rolled Plates	A36	36 KSI
Connection Bolts	A325	92 KSI
Anchor Bolts	A307	-
Threaded Rods	A36	36 KSI
Non-shrink Grout	C1107	8000 PSI
CMU	C90 (lightweight)	2800 PSI

Loads

Gravity Loads:

Dead Load:

Dead Loads were obtained using typical design values, material specifications, or educated assumptions. My values were very similar to values stated by the Engineer of Record.

Component	Obtained Values
2" Steel Deck (on floors 1-5)	2 PSF
3-1/4" Concrete on Deck (on floors 1-5)	43 PSF
3" Steel Deck (on main roof level)	2 PSF
5-1/4" Concrete on Deck (on main roof level)	82 PSF
Green Roof	25 PSF
Ceiling with Mechanical/Electrical	15 PSF
Floor Finish	3 PSF

Live Load:

Live Loads were taken from ASCE 7-05 along with an assumption for the mechanical rooms. My obtained values were once again very similar to the values on the drawings.

Building Location	Drawing Values	Obtained Values
Corridors (first floor)	100 PSF	100 PSF
Corridors (above first floor)	80 PSF	80 PSF
Procedure/Exam Rooms	50 PSF + 20 PSF partition	40 PSF + 15 PSF partition
Lobbies	100 PSF	100 PSF
Stairs	125 PSF	100 PSF
Mechanical Rooms	75 PSF	150 PSF
Offices	50 PSF + 20 PSF partition	50 PSF + 15 PSF partition
Light Storage	125 PSF	125 PSF
Heavy Storage	250 PSF	250 PSF

Snow Load:

Snow loads were determined using IBC 2006 and Centre Region Code.

$p_f = 0.7 \times C_e \times C_t \times I \times p_g = 30.8$ psf
$p_g = 40$ psf
$C_e = 1.0$
$C = 1.0$
$I = 1.1$

Lateral Loads:

Wind Load:

Wind loads were calculated using ASCE 7-05, Section 6.5. “Method 2 - Analytical Procedure” was used to determine wind loads in the N-S and E-W directions. The façade in each direction was assumed to be rectangular to simplify calculations.

The controlling base shear and overturning moment for wind loading were due to the wind in the N-S direction. These values were 337.93 K and 13,648 ft-K respectively. Wind Pressure Diagrams are shown in *(Fig. 9)*. Detailed calculations are shown in Appendix A.

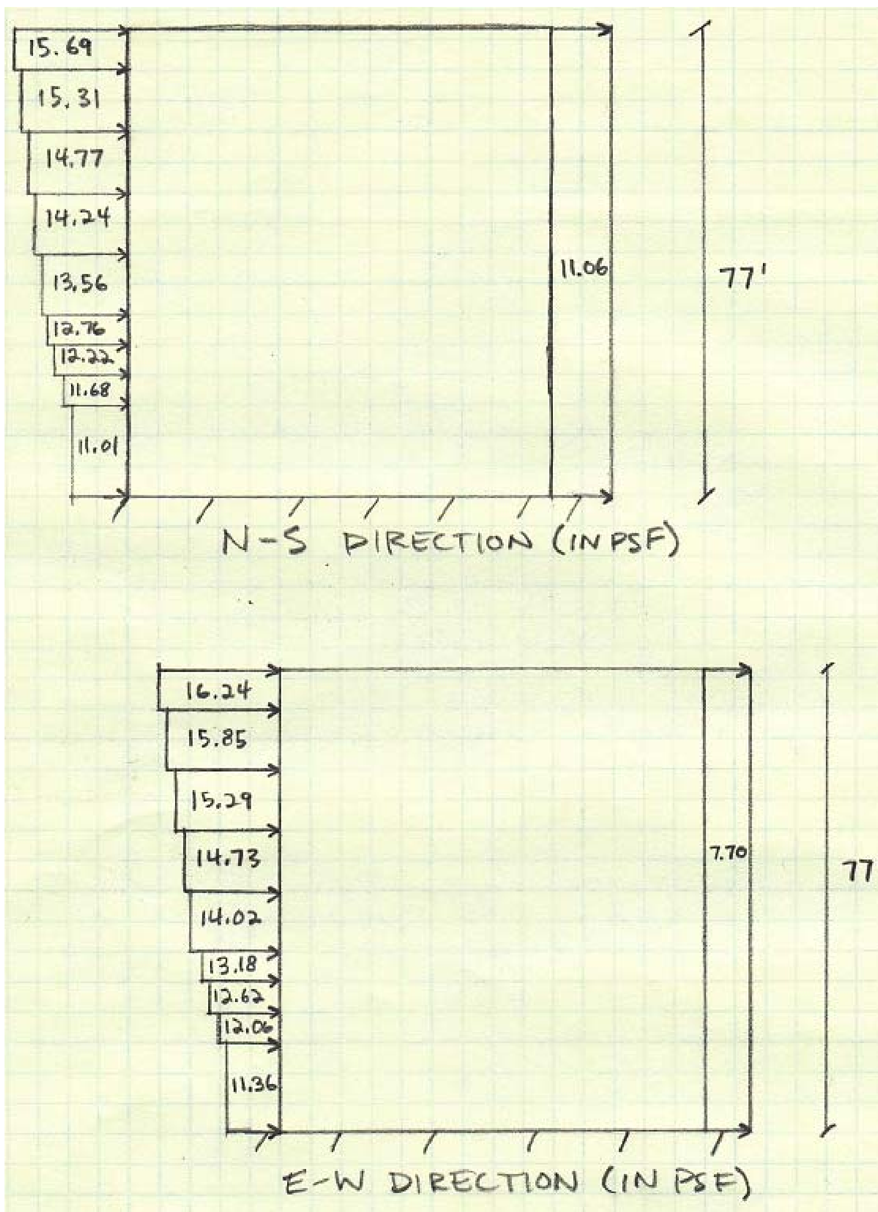


Fig 9 – Wind Diagrams

Seismic Load:

Using ASCE 7-05, Chapters 12, seismic loads were calculated. Information particular to the SHC was taken from the geotechnical report, the Centre Region Code, and the drawings. For details of these calculations, refer to Appendix B.

Level	h_x (ft)	Story Weight (k)	$h_x^k W_x$	C_{vx}	$F_x = C_{vx}V$	V_x (k)	M_x (ft-K)
Main Roof	84	469	95567	0.120	35	35	2936
5	70	1860	304530	0.383	111	146	10242
4	56	1356	169858	0.214	62	208	11672
3	43	1501	136944	0.172	50	258	11115
2	19	1535	52555	0.066	19	278	5277
1	14	1501	35623	0.045	13	291	4070
Total	84	8222	795077	1.0	**291**		**45311**

The base shear calculated was 291 K, which is fairly close to the base shear determined by the Engineer of Record, which was 252 K. Our difference in numbers could be explained by a difference in calculated building weight, in which I made a rough estimate for simplicity, or our interpretation of the code.

Spot Checks

Spot Checks were performed in this report to check the capacity of gravity members. A typical beam, girder, and first floor column was checked. My calculations for typical beam and girder size were identical to that of the engineer while my calculated column size was a W14x61 compared to the plans which used a W14x48. The deviation in our answers is probably due to a difference in our design loads, which accumulated down to the checked column's level. A table showing the loads on column D-3 is shown in Appendix C. Details of all spots checks are also found in Appendix C.

Appendix

A: Wind Calculations:

V_{3S} (mph)	90
K_d	0.85
Occupancy Category	III
I	1.15
Exposure Category	B
K_{zt}	1
Building Height (ft)	77
$n_1 = 22.2/H^{0.8}$	0.687
$g_Q = g_v$	3.4
g_R	4.644
$\dot{z}$	46.2
c	0.3
$I_{\dot{z}} = c(33/\dot{z})^{(1/6)}$	0.284
ℓ	320
ϵ	0.333
$L_{\dot{z}} = \ell(\dot{z}/33)^{\epsilon}$	358
B (East-West) (ft)	70
B (North-South) (ft)	180
Q (E-W) (Eq. 6-6)	0.858
Q (N-S)	0.813

Ъ	0.45
α	0.25
$V_{\dot{z}}$ (Eq. 6-14)	64.613
$N_1 = n_1 L_{\dot{z}}/V_{\dot{z}}$	3.808
R_n (Eq. 6-11)	0.060
$\eta = 4.6 n_1 h/V_{\dot{z}}$	3.768
R_h (Eq. 6-13)	0.230
$\eta = 4.6 n_1 B/V\dot{z}$ (E-W)	3.425
$\eta = 4.6 n_1 B/V\dot{z}$ (N-S)	8.808
R_B (Eq. 6-13) (E-W)	0.249
R_B (Eq. 6-13) (N-S)	0.107
$\eta = 15.4 n_1 L/V\dot{z}$ (E-W)	29.487
$\eta = 15.4 n_1 L/V\dot{z}$ (N-S)	11.467
R_L (Eq. 6-13) (E-W)	0.033
R_L (Eq. 6-13) (N-S)	0.083
R (Eq. 6-10) (E-W)	0.194
R (Eq. 6-10) (N-S)	0.130
G_f (Eq. 6-8) (E-W)	0.866
G_f (Eq. 6-8) (N-S)	0.829
GC_{pi}	0.18

	C_p (E-W)	C_p (N-S)
Windward Wall	0.8	0.8
Leeward Wall	-0.27	-0.5
Side Wall	-0.7	-0.7
	L/B = 2.57	L/B = 0.39

Wind Pressures (N-S)

Height (ft)	K_z	q_z (Eq. 6-15)	p_z (Eq. 6-19) (psf)	p_h (Eq. 6-19) (psf)	Total Pressure (psf)
0-15	0.57	11.55	11.01	-11.06	22.07
20	0.62	12.57	11.68	-11.06	22.74
25	0.66	13.38	12.22	-11.06	23.28
30	0.7	14.19	12.76	-11.06	23.82
40	0.76	15.40	13.56	-11.06	24.62
50	0.81	16.42	14.24	-11.06	25.30
60	0.85	17.23	14.77	-11.06	25.83
70	0.89	18.04	15.31	-11.06	26.37
77	0.918	18.61	15.69	-11.06	26.75

Wind Pressures (E-W)

Height (ft)	K_z	q_z (Eq. 6-15)	p_z (Eq. 6-19) (psf)	p_h (Eq. 6-19) (psf)	Total Pressure (psf)
0-15	0.57	11.55	11.36	-7.70	19.06
20	0.62	12.57	12.06	-7.70	19.76
25	0.66	13.38	12.62	-7.70	20.32
30	0.7	14.19	13.18	-7.70	20.88
40	0.76	15.40	14.02	-7.70	21.73
50	0.81	16.42	14.73	-7.70	22.43
60	0.85	17.23	15.29	-7.70	22.99
70	0.89	18.04	15.85	-7.70	23.55
77	0.918	18.61	16.24	-7.70	23.95

(N-S)

Level	Height (ft)	Force (K)	Shear (K)	Moment (ftK)
Penthouse Roof	77	33.70	33.70	2595.0
Main Roof	63	66.07	99.77	4162.3
5	49	62.05	161.82	3040.4
4	36	59.99	221.81	2159.8
3	22	58.51	280.32	1287.2
2	7	57.60	337.93	403.2
1	-7	0	337.93	0
		Total	337.93	13648.0

(E-W)

Level	Height (ft)	Force (K)	Shear (K)	Moment (ftK)
Penthouse Roof	77	11.73	11.73	903.5
Main Roof	63	22.92	34.66	1444.2
5	49	21.43	56.09	1050.1
4	36	20.60	76.68	741.4
3	22	19.85	96.53	436.7
2	7	19.34	115.88	135.4
1	-7	0	115.88	0.0
		Total	115.88	4711.3

B: Seismic Calculations:

Main Roof					
Dead Load w/o stl framing (psf)	Steel Framing Load (lbs)	Snow Load (psf)	Perimeter Wall Load (psf)	Perimeter Length (ft)	Level Ht (ft)
0	0	0	100	335	14
Level Weight (k)	469				

5th					
Dead Load w/o stl framing (psf)	Steel Framing Load (lbs)	Snow Load (psf)	Perimeter Wall Load (psf)	Perimeter Length (ft)	Level Ht (ft)
124	25000	30.8	10	330	14
99			100	335	
Level Weight (k)	1860				

4th					
Dead Load w/o stl framing (psf)	Steel Framing Load (lbs)	Snow Load (psf)	Perimeter Wall Load (psf)	Perimeter Length (ft)	Level Ht (ft)
63	127000	30.8	10	210	13
88			100	310	
Level Weight (k)	1356				

3rd					
Dead Load w/o stl framing (psf)	Steel Framing Load (lbs)	Snow Load (psf)	Perimeter Wall Load (psf)	Perimeter Length (ft)	Level Ht (ft)
63	151000	0	10	210	14
			100	310	
Level Weight (k)	1501				

2nd					
Dead Load w/o stl framing (psf)	Steel Framing Load (lbs)	Snow Load (psf)	Perimeter Wall Load (psf)	Perimeter Length (ft)	Level Ht (ft)
63	151000	0	10	210	15
			100	310	
Level Weight (k)	1535				

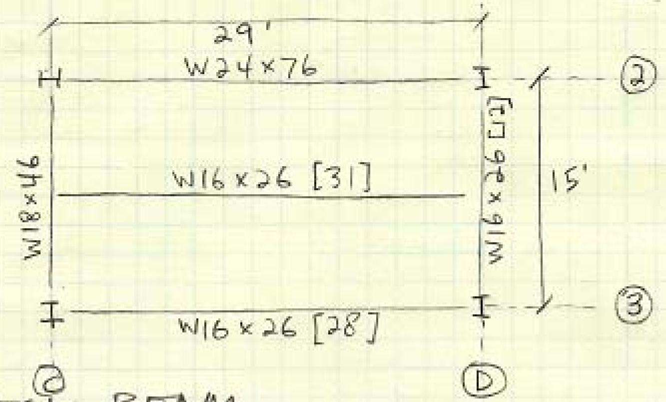
1st					
Dead Load w/o stl framing (psf)	Steel Framing Load (lbs)	Snow Load (psf)	Perimeter Wall Load (psf)	Perimeter Length (ft)	Level Ht (ft)
63	151000	0	10	210	14
			100	310	
Level Weight (k)	1501				

W, Total Building Weight (k)	8222

S_s (from Centre Region Code)	0.17
S_1 (from Centre Region Code)	0.06
Site Class (from Geotech Report)	D
F_a	1.6
F_v	2.4
$S_{MS} = F_aS_s$	0.272
$S_{M1} = F_vS_1$	0.144
$SD_S = 2S_{MS}/3$	0.181
$SD_1 = 2S_{M1}/3$	0.096
Seismic Design Category	B
R (ordinary steel moment frame)	3.5
C_d	3
I	1.25
Ct (Table 12.8-2)	0.028
x (Table 12.8-2)	0.8
$T_a = C_th_n^x$	0.970
T_L (Fig. 22-15)	6
C_s (Eq. 12.8-3)	0.035
W (k)	8222
V = C_sW (k)	**291**
k	1.2

C: Spot Checks:

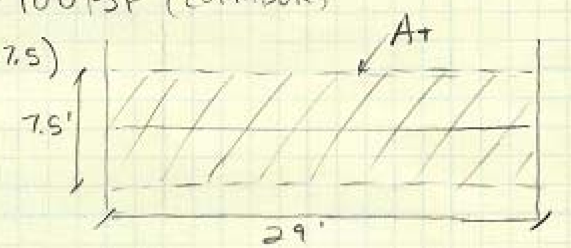
INTERIOR BAY ON SECOND LEVEL



CHECK BEAM

DL = 63 PSF   LL = 100 PSF (CORRIDOR)

$w_u = 1.2(\overset{0.026\,(SW)+}{0.063 \times 7.5'}) + 1.6(0.1 \times 7.5)$



$w_u = 1.798$ k/ft

$M_u = \frac{w_u l^2}{8} = \frac{(1.798)(29)^2}{8}$

$M_u = 189.0$ ft·k

$b_{eff} = \begin{cases} 7.5' \times 12 = 90'' \\ \frac{29(12)}{4} = 87'' \leftarrow \text{CONTROLS} \end{cases}$

$\Delta_{CONSTRUCTION} = \frac{5 w_c l^4}{384 E I}$

$\Delta_{ALLOWABLE} = l/360 = 29(12)/360 = 0.967''$

$I_{req'd} = \frac{5 w_c l^4}{384 \Delta_{const} E} = \frac{5 \times (0.063 \times 7.5)(29)^4 (1728)}{384 (0.967)(29{,}000)} = 268 \text{ in}^4$

$I_{W16\times26} = 301 \text{ in}^4 > 268 \text{ in}^4 \therefore OK$

Check bending during construction

$w_u = 1.2(0.063 \times 7.5) + 1.6(0.02 \times 7.5) = 0.807$

$M_u = \frac{w_u l^2}{8} = \frac{0.807(29)^2}{8} = 84.8$ ft·k

$\phi M_{n\,W16\times26} = 166 \text{ ft·k} > 84.8 \text{ ft·k} \therefore ok$

ASSUMING $\Sigma Q_n = 145^k$

$a = \frac{\Sigma Q_n}{0.85(3)(87)} = 0.654''$

$Y2 = 5.25'' - \frac{0.654}{2} = 4.92'' \Rightarrow 4.5''$

FROM TABLE 3-19: @ PNA = 6

$\phi M_n = 269$ ft·k $> 189 = M_u$ ∴ OK

FROM TABLE 3-21:

DECK PERP.
3/4"ϕ STUDS ; 1 STUD/RIB
$f'_c = 3000$ ksi

$\left.\right\}$ $Q_n = 17.2^k$

$\frac{\Sigma Q_n}{Q_n}(\times 2) = \frac{145}{17.2}(\times 2) = 16.9 \rightarrow 17$ STUDS REQ'D

$17 < \text{ACTUAL} = 31$ ∴ OK

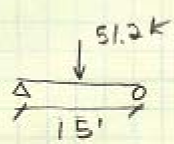
CHECK DEFLECTION:

FROM TABLE 3-20: $I_{LB} = 622 \text{ in}^4$

$\Delta = \frac{5 w l^4}{384 E I_{LB}} = \frac{5(100 \times 7.5)(29)^4(1728)}{384(29{,}000)(622)} = 0.66''$

$0.66 < \Delta_{allow} = l/360 = 29(12)/360 = 0.967''$ ∴ OK

CHECK GIRDER



$P_u = 1.2(0.063 \times 7.5 \times 29) + 1.6(0.1 \times 7.5 \times 29) = 51.2^k$

$M_{max} = Pl/4 = \frac{51.2(15)}{4} = 192.2$ ft·k

ASSUMING $\Sigma Q_n = 145$

$b_{eff} = \begin{cases} 29(12) = 348" \\ 15(12)/4 = 45" \leftarrow \text{CONTROLS} \end{cases}$

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{145}{0.85(3)(45)} = 1.26$

$Y_2 = 5.25" - 1.26/2 = 4.62" \Rightarrow 4.5"$

FROM TABLE 3-19 : @ PNA = 6

$\phi M_n = 269 \text{ ft·k} > 192.2 \text{ ft·k} \therefore OK$

FROM TABLE 3-21

DECK PARALLEL
$w/h = 3.0$
$3/4"\phi$ STUDS
LT. WT. CONC.
$\Big\} 17.1 = Q_n$

$\frac{\Sigma Q_n}{Q_n}(\times 2) = \frac{145}{17.1}(\times 2) = 16.96 \rightarrow 17$ STUDS REQUIRED

$17 > 12$ PROVIDED $\therefore \boxed{\text{NO GOOD}}$

TRY $\Sigma Q_n = 96$ @ PNA 7

$a = \frac{96}{0.85(3)(45)} = 0.837"$

$Y_2 = 5.25 - 0.837/2 = 4.83" \rightarrow 4.5"$

FROM TABLE 3-19 @ PNA = 7

$\phi M_n = 241 \text{ ft·k} > 192.2 \text{ ft·k} \therefore$ STILL OK

$\frac{\Sigma Q_n}{Q_n}(\times 2) = \frac{96}{17.1}(\times 2) = 11.2 \rightarrow 12$ STUDS REQ'D $\therefore$ OK

CHECK DEFLECTION: $I_{LB} = 535 \text{ in}^4$

$\Delta = \frac{P l^3}{48 E I_{LB}} = \frac{(0.1 \times 7.5 \times 29)(15)^3(12)^3}{48(29{,}000)(535)} = 0.17"$

$\Delta_{ALLOW} = 15(12)/360 = 0.5" > 0.17" \therefore OK$

CHECK COLUMN (D3 ON FIRST LEVEL)

W14x48 $h = 14'$ $A_g = 14.1 \text{ in}^2$

$I_x = 484 \text{ in}^4$ $I_y = 51.4 \text{ in}^4$

$r_x = 5.85 \text{ in.}$ $r_y = 1.91 \text{ in.}$

$\frac{KL}{r_x} = \frac{14(12)}{5.85} = 28.7$ $\frac{KL}{r_y} = \frac{14(12)}{1.91} = 87.96$

CONTROLS

$\frac{KL}{r} \leq 4.71\sqrt{E/F_y} = 4.71\sqrt{\frac{29,000}{50}} = 113.4 > 87.96$ ∴ INELASTIC BEHAVIOR

$F_E = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2(29,000)}{(87.96)^2} = 37.0 \text{ ksi}$

$F_{CR} = \left[0.658^{F_y/F_E}\right] F_y = \left[0.658^{50/37}\right] 50 = 28.4 \text{ ksi}$

$\phi P_n = \phi F_{CR} A_g = 0.9(28.4)(14.1) = \underline{360.4^k}$

FROM TABLE 4-1 -(IN STL MANUAL)

$\phi P_n = \underline{361^k}$

$361^k = \phi P_n < 442.6^k = P_u$ ∴ WILL NOT WORK

USE W14x61, $\phi P_n = 572^k < 442.6^k$ ∴ OK