St. Elizabeth Hospital Boardman Campus Inpatient Facility

Boardman, Ohio

Josh Behun

Structural Option

Technical Report #1

November 25, 2007

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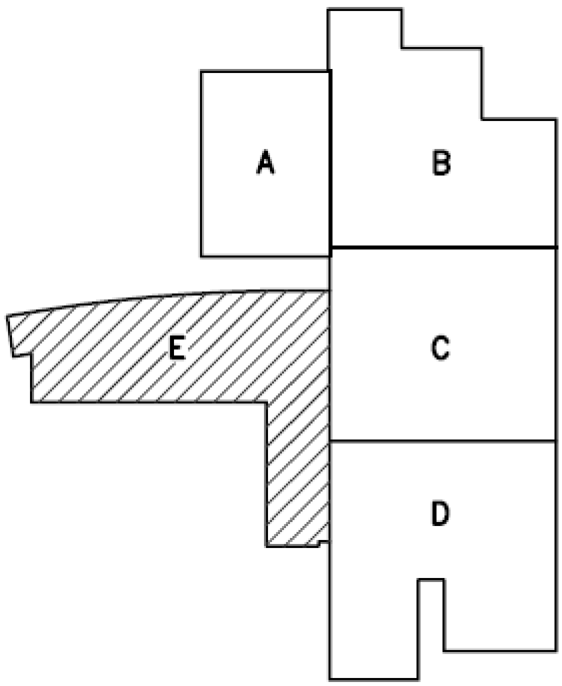
Executive Summary

The purpose of this report is to analyze the structural system used for the new Inpatient Facility addition as well as the original structure in place at the St. Elizabeth Boardman Campus Emergency and Diagnostic Center in Boardman, Ohio. The new building consists of a seven story tower addition plus an additional two story wing that was constructed adjacent to a previously two story facility. The analysis taken place in this report is comprised of the basic lateral forces that affect the building, including seismic and wind forces, as well as some of the gravity loads, such as overall building weight and snow loads, that may have a substantial influence on the lateral loading.



Plan View of Building Broken into Wings with New Tower Addition Highlighted

Conclusion

After evaluating the lateral forces that are applied to the building, it is apparent that the seismic loading will control the design of the building and its lateral bracing system. The results of the wind analysis were rather close to controlling the design, and provided a slight differential in the calculations they very well could have. However, the design engineer that did the original analysis for the building seems to have separated the building into three different parts, which yielded even larger seismic results, whereas for this calculation the building was considered as one uniform structure. Differing assumptions present differing results, though it seems that the seismic factors would most likely control the building's structural design.

Introduction to Structural System

Foundation

The foundation for the St. Elizabeth Hospital Inpatient Facility consists of 16" diameter auger cast grout injected piles with a capacity of 50 tons and an f'c of 4000 psi, including (4) #6 vertical bars for the top 20' of the piles and #3 ties spaced at 16" on center. The vertical reinforcement from each pile is to extend 18" into its corresponding pile cap or grade beam with a 90° hook of 2'-0" in length. Several of the column piers will be constructed on existing footings, subsequent reinforcement bars are to be drilled and grouted into the existing footing with Hilti epoxy adhesives, providing a minimum embedment of 8".

Super Structure

The framing for the structural system consists by in large of wide flange structural steel members. The typical column size for the building is within the range of W12x40 to W12x136, while there are a minimal number of W10 and W14 columns throughout the atypical areas of the new addition. The girders for the building are on average W30x90 where the façade is brick and W18x40 where the outer façade is the aluminum panel curtain wall system. The floor to floor height of each story two through seven is 14'-8" tall while the floor to floor height for the first floor is 15'-4" in height. The bracing system for the lateral load resistance consists of several types of bracings on each story comprised of HSS members, including chevron braces, knee braces, and cross braces.

Floor System

The floor system of the St. Elizabeth Hospital Inpatient Facility is a two-way slab system comprised of a 4" light weight concrete slab on 2" – 20 gage galvanized composite decking with 5" long ¾" diameter shear studs and a 6x6-W2.1xW2.1 welded wire fabric reinforcement system. The majority of the beams for the floor framing are 21" in depth with a typical span of 34'. On the first two floors, the new addition's floor systems are connected to the existing floor slabs as well as the masonry walls by ½" diameter Hilti adhesive anchors spaced at 24" on center, with a minimum embedment of 4½".

Roofing

The roofing system is a flat roof which consists of structural steel members similar to that of the floor system. The area where the HVAC units rest has a slab of 4½" light weight concrete on 2"- 20 gage galvanized composite decking with 6x6-W2.1xW2.1 welded wire fabric reinforcement. While the remainder of the roof area, including the penthouse roof, is constructed of 1½"-20 gage galvanized wide ribbed steel roof deck.

Codes

Building Design Codes

Ohio Building Code, 2005
International Building Code, 2003

Reinforced Concrete

American Concrete Institute

Building Code Requirements for Structural Concrete (ACI 318, Latest Edition)
Specifications for Structural Concrete (ACI 301, Latest Edition)

Masonry

American Concrete Institute

Building Code Requirements for Masonry Structures (ACI 530, Latest Edition)
Specifications for Masonry Structures (ACI 530.1, Latest, Edition)

Structural Steel

American Institute of Steel Construction (1989 Edition, As Revised)

Open Web Steel Joists

Steel Joist Institute

Standard Specifications and Load Tables for Open Web Joists, K-Series or LH-Series

Metal Decking

Steel Deck Institute

Steel Roof Deck Specifications and Load Tables (Latest Edition)

Material Strengths

Concrete

Minimum Design Compression Strength (F'c) Required at 28 Days:

- Grout for Auger Piles…..………………………………….. 4000 psi
- Foundations and Concrete Fill…………………………….. 3000 psi
- Walls………………………………………………………. 4000 psi
- Slabs on Grade and Elevated Floor Slabs…………….…… 4000 psi
- Columns, Beams, Elevated Slabs
  and Tilt-Up Wall Panels…………………………… 5000 psi
- Masonry Grout…………………………………………….. 3000 psi

Maximum Water to Cementitious Materials Ratio:

- Foundations and Concrete Fill…………………………….. 0.60
- Walls………………………………………………………. 0.45
- Slabs on Grade and Elevated Floor Slabs………………… 0.45

Reinforcement

- Deformed Bars (Grade 60)………………………………… ASTM A615
- Welded Wire Fabric……………………………………….. ASTM A185
- Headed Shear Studs……………………………………….. ASTM A108, Grade 1015 or 1020 Cold Finished Carbon Steel

Structural Steel

- Structural Shapes…………………………………………... ASTM A572, Grade 50
- Steel Tubes…………………………………………………. ASTM A500, Grade B
- Steel Pipe…………………………………………………... ASTM A53, Grade E or S
- Angles and Plates………………………………………….. ASTM A36

Galvanized Structural Steel

- Structural Shapes and Rods……………………………….. ASTM A123
- Bolts, Fasteners, and Hardware…………………………… ASTM A153

Design Criteria

Dead Loads

Partitions…………………………………………………… 20 psf

Live Loads

Roof…………………………………………………... 30 psf
Public Areas……………………………………………….. 100 psf
Lobbies…………………………………………………….. 100 psf
First Floor Corridors………………………………………. 100 psf
Corridors above First Floor………………………………. 80 psf
Patient Rooms……………………………………………... 60 psf
Light Storage………………………………………………. 125 psf
Catwalks…………………………………………………… 25 psf
Mechanical………………………………………………… 175 psf
Stairs………………………………………………………. 100 psf

Seismic Analysis

The seismic analysis for the hospital was determined using the base shear calculations derived from the equivalent lateral force procedure from ASCE-05. The original seismic calculations had been done breaking the building into 3 separate entities, while the calculation done for this report considered the entire building as a whole. The results of the calculation for the building as a whole fell within an average range of the initial separated results.

For other seismic calculation considerations the hospital is located at:

Latitude: 40° 59' 35"
Longitude: -80° 39' 35"

Design Properties

Velocity – Related Acceleration (SS)…………………….. 0.1518
Peak Acceleration (S1)……………………………………. 0.0558
Seismic Hazard Exposure Group………………………….. III
Seismic Performance Category……………………………. C
Basic Structural System…………………………………… Steel Frame
Seismic Importance Factor (IE)…………………………… 1.5
Response Modification Factor (R)………………………… 5
Deflection Amplification Factor (CD)…………………….. 4.5
Analysis Procedure………………………………………… Equivalent Lateral Force
SDS…………………………………………………………. 0.152
SD1…………………………………………………………. 0.056
Site Class…………………………………………………... D
Basic Seismic Force Resisting System……………………. Concentric Steel Braced Frames & Existing Masonry Shear Walls
Design Base Shear…………………………………………. Per Area
Patient Towers…………………………………….. 810 kips
Surgical Wing……………………………………... 385 kips
Diagnostic Wing and Addition……………………. 635 kips

Wind Analysis

The wind loading for the hospital addition as well its existing structure was determined using Method 2 from ASCE-05. The majority of the calculations are based upon the building properties listed below, plus numerous tables and charts included within the ASCE manual. In order to ease the calculations involved, the shape of the seven story tower addition was normalized from its original form to a standard rectangular shape, disregarding the curvilinear figure of the northern wall and all indentations on the western side of the patient tower. Being that the building is constructed using steel framing, the analysis performed was done so considering the building to be a flexible frame. The connections between the tower addition and the existing building contain expansion joints that include Teflon slide bearings, allowing the buildings to react to lateral loading as separate identities. In this analysis, since the tower addition will absorb the largest amount of lateral wind forces, it was the main area of focus.

Design Properties

Velocity………………………………………………… 90 mph
Wind Importance Factor (IW)…………………………….. 1.15
Exposure Category………………………………………… C
Enclosure Classification………………………………..…….. Enclosed
Building Classification……………………………………. 1-2
Internal Pressure Coefficient (GCPI)……………………… ± 0.25
Wind Design Pressure – P (Windward)…………………… 25 psf
Wind Design Pressure – P (Leeward)……………………... 30 psf

Flexible Building Properties for Exposure C

Table 6-2 ASCE-07

Exposure	α	z_g (ft)	$\hat{a}$	$\hat{b}$	$\bar{\alpha}$	$\bar{b}$	c	ℓ (ft)	$\bar{\epsilon}$	z_{min} (ft)*
B	7.0	1200	1/7	0.84	1/4.0	0.45	0.30	320	1/3.0	30
C	9.5	900	1/9.5	1.00	1/6.5	0.65	0.20	500	1/5.0	15
D	11.5	700	1/11.5	1.07	1/9.0	0.80	0.15	650	1/8.0	7

Design Properties for Flexible Building Frames

Flexible Building Properties		
	N-S	E-W
B	87’	318’
L	318’	87’
n1	0.8197	0.8197
h	104’	104’
h_{min} = 0.6h	62.4	62.4’
g_R	4.142	4.142
g_Q & g_v	3.40	3.40
R_n	0.0513	0.0513
I_z	0.147	0.147
V_z	94.60	94.60
R_h	4.15	4.15
ηh	4.15	4.15
β	5%	5%

	N-S Direction	E-W Direction
ηL	11.6	42.43
R_L	0.0825	0.0234
ηB	12.67	3.47
R_B	0.0758	0.267
Q	0.81	0.871
R	0.43	0.784
Gf	0.91	1.05

Lateral Analysis Conclusions

Based on the analysis performed for the lateral forces on the hospital, the seismic loading seems to govern the design of the building and its lateral bracing system. The base shear and the overturning moments were the main sources of concern with the lateral loadings, in each situation the seismic analysis yielded a larger result. However, the resulting wind pressures in the North-South direction provided a base shear that was quite close to the total seismic base shear, which with a slight variation to the calculations may produce a wind loading that could overtake the control of the building’s lateral bracing design.

While calculating the seismic forces the design engineer seemed to have separated the building into three distinct segments, whereas with these calculations the building was considered as a whole entity. Using smaller segments of the building would likely lead to a larger seismic loading. While different analysis methods were utilized, and thus different results were obtained, the seismic loading remained the controlling factor in the building’s design.

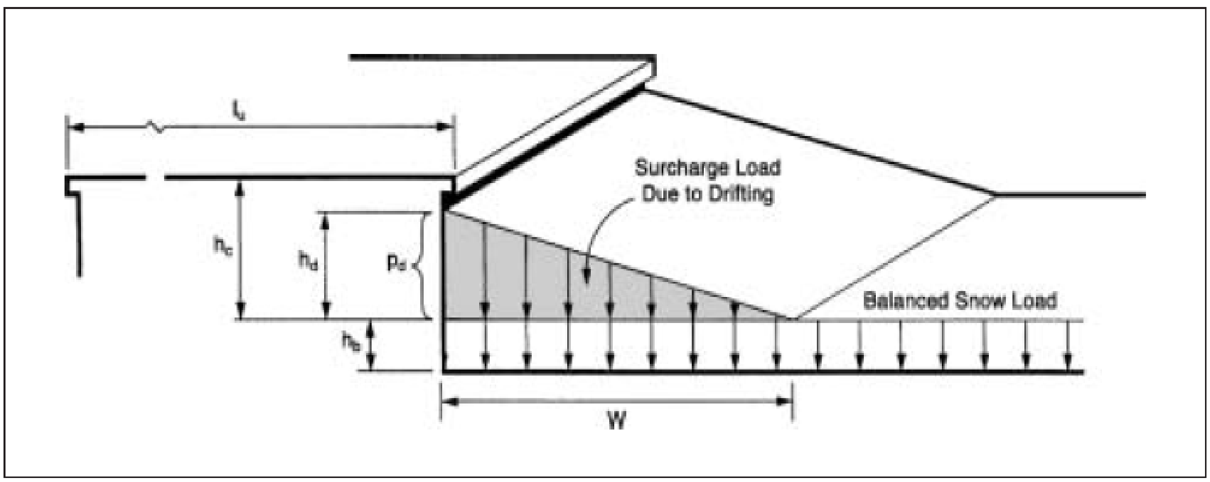
Snow Loads

The snow load for the hospital was determined using the procedures and chats supplied by the ASCE-05 manual. All of the roofs of the hospital are flat, thus aside from the actual weight of snow accumulation, there is also an amount of snow buildup due to wind blown snow drifts which add significant snow mass. The two instances that need to be accounted for are windward snow drifts that are blown up against the wall of a taller portion of the building (such as pictured below) and leeward snow drifts that are blown off of the roof of a taller section of the building.

Design Properties

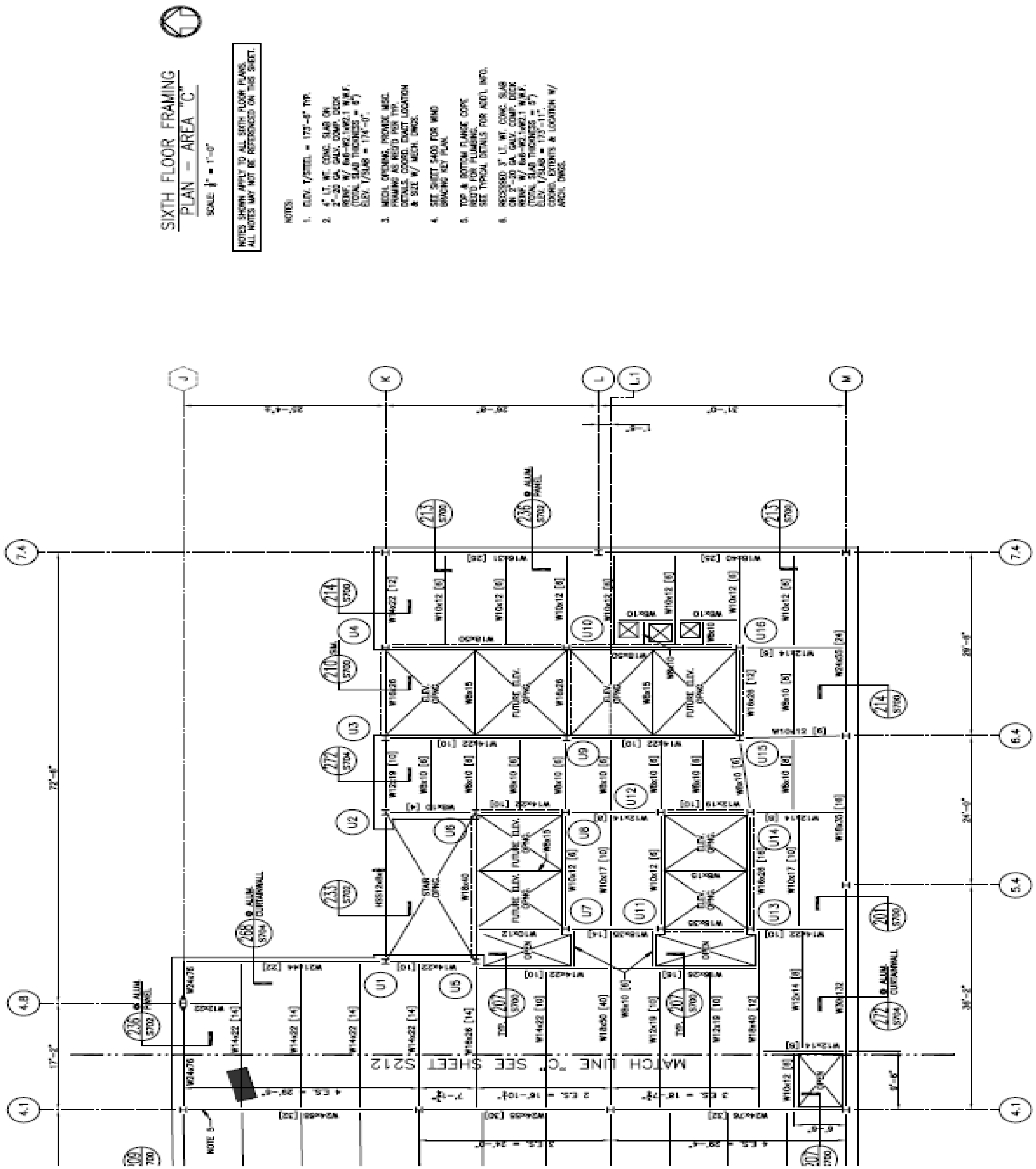
Base Ground Snow Load (Pg)…………………………….. 30 psf
Flat Roof Snow Load (Pf)…………………………………. 21 psf
Snow Drift Load Per Code
Snow Exposure Factor (Ce)……………………………….. 1.0
Snow Load Importance Factor (Is)………………………… 1.2
Thermal Factor (Ct)……………………………………….. 1.0

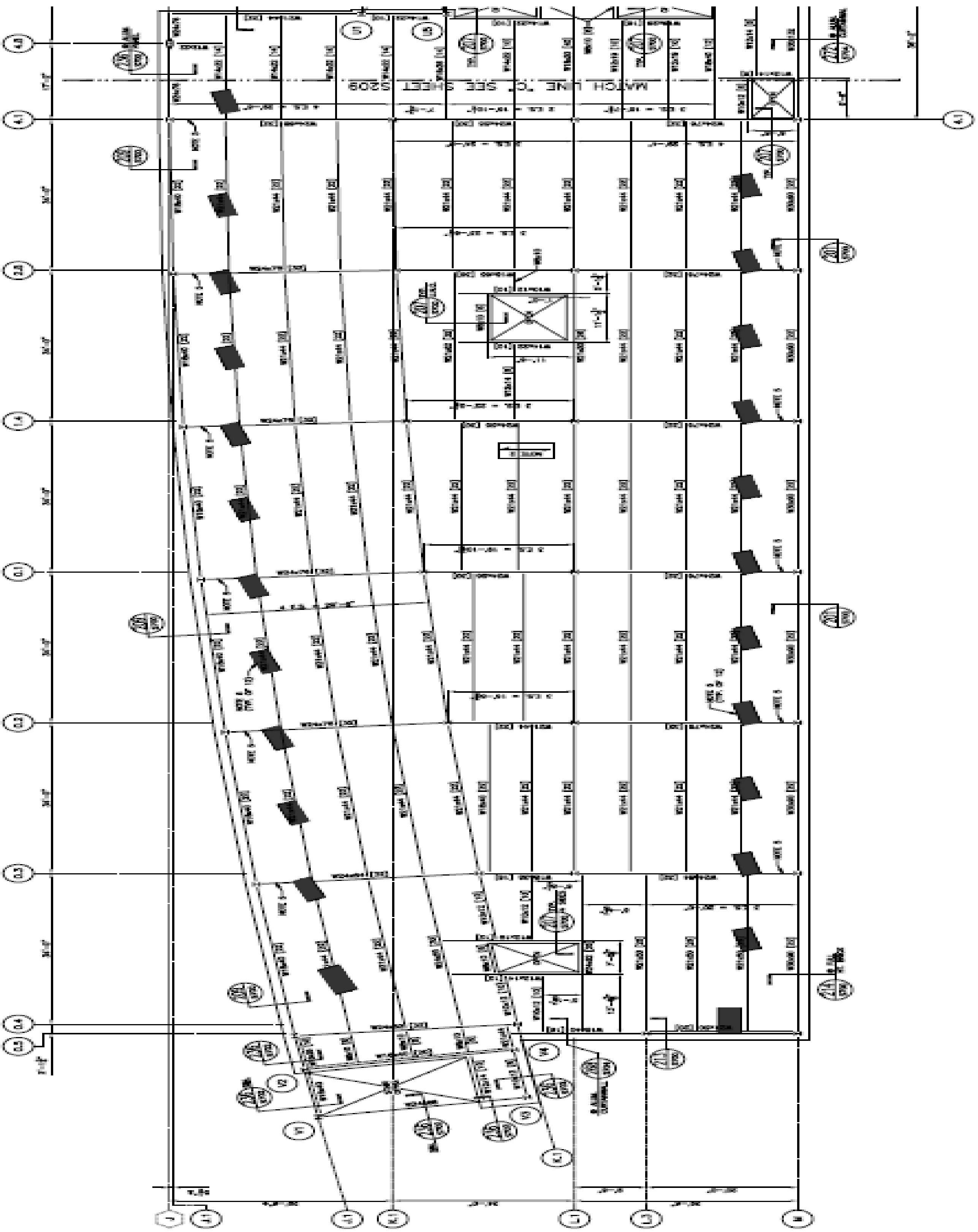
ASCE-07
Figure 7-8 Configuration of Snow Drifts on Lower Roofs



Appendix

Appendix A - Typical Tower Addition Framing Plans





Appendix B – Building Weight

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Floor Load

- Concrete slabs

  $\frac{5}{12}(110 \text{ lb/ft}^3) = 46 \text{ psf} + 2 \text{ psf deck} = 48 \text{ psf}$

- Steel Framing (typical bay)

  $$\frac{90 \text{ plf}(34') + 4(44 \text{ plf})(34') + 2(78 \text{ plf})(29.33')}{34'(29.33')}$$

  $= 13.7 \text{ psf} \rightarrow$ use 14 psf to account for connections

- Steel Columns (per typical bay)

  $(4)\ 40 \text{ lb/ft}\ (14.67) = \frac{2350}{34(29.33)} = 2.5 \text{ psf}$

- Partitions = 20 psf

- MEP/collateral = 20 psf (assumed)

1st Floor

$= 104.5 \text{ psf}\ (119,700 \text{ ft}^2) = 12,510^K$

2nd Floor

$= 104.5 \text{ psf}\ (62,625 \text{ ft}^2) = 6,545^K$

3rd Floor

$= 104.5 \text{ psf}\ (32,870 \text{ ft}^2) = 3,435^K$

4th – 7th Floors

$= 4(104.5 \text{ psf})(23,525 \text{ ft}^2) = 9,835^K$

Total Building Load

= Floors + Roof + Walls

$= 36000^K$

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Roof Load

- Concrete slab

$\frac{5}{12}'(110\text{ pcf})\left[(180)(41) + \frac{1}{2}(140)(12)\right] = 378^K$

- Decking = 2 psf
- Rubber Membrane = 1 psf

$(3\text{ psf})(23{,}525\text{ ft}^2) = 71^K$

- Steel Framing (typical roof bay under Mech Equipment)

$\frac{68\text{ plf}(34') + 4(44\text{ plf})(34') + 2(94\text{ plf})(29.33)}{34'(29.33')}$

$= 13.8 \rightarrow 14\text{ psf}\left[(180)(41) + \frac{1}{2}(140)(12)\right] = 115^K$

- Steel Framing (typical roof bay under decking)

$\frac{26\text{ plf}(34) + 2(50\text{ plf})29.33 + 5(11\text{ plf})34'}{34(29.33)}$ ← open web joists K-series 24 K

$= 5.7 \rightarrow 6\text{ psf}(23{,}525 - 8220) = 92^K$

Roof total

$378 + 71 + 115 + 92 = 656^K$

Wall Load

- Windows = $10\text{ psf}\left[6'(225')6 + 6'(210')6 + 4'(9')80 + (12(32) + 32(64) + 34(24) + 18(8))\right] = 220^K$
- Alluminum = $15\text{ psf}\left[(255(105) - 6'(210')6) + (105(50) - 16(6)) + (105)(10) + 72(21)\right] = 405^K$
- Brick = $39\text{ psf}\left[(105(216) - 6(225)6) + 70(80) + 46(110) + (34)(106) + (68)(56) + (32)(75) + (102)(13) + 40(105) + 60(105) + 32(60) + 32(135)\right] = 2400^K$

Exterior Wall total

$220 + 405 + 2400 = 3025^K$

Appendix C– Seismic Analysis

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$S_S = 0.1518$
$S_1 = 0.0558$

Site Class C

$F_a = 1.2 \quad F_v = 1.65$

$S_{MS} = F_a(S_S) = 1.2(0.1518) = 0.18216$

$S_{M1} = F_v(S_1) = 1.65(0.0558) = 0.09207$

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.18216) = 0.12144$

$S_{D1} = 2/3\ S_{M1} = 2/3\ (0.09207) = 0.06138$

from table 12.8-1 $\quad S_{D1} \leq 0.1 \quad C_u = 1.7$

Concentrically Braced Frame & Existing Masonry Shear Wall

from table 12.8-2

$C_t = 0.02 \quad x = 0.75$

$T_a = C_t(\text{height})^x$

$= 0.02(118')^{0.75} = 0.716$

period $= T = C_u T_a = 1.7(0.716) = 1.22$

frequency $= \frac{1}{1.22} < 1 \quad \therefore$ Flexible

$R = 5$
$I = 1.5$

from table 22.15
$T_L = 12\ s$

option I

$\frac{S_{DS}}{(R/I)} = \frac{0.12144}{(5/1.5)} = 0.03643$

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option 2

$$\frac{S_{D1}}{\left(T\frac{(R)}{I}\right)} = \frac{0.06138}{\left(\frac{1.22(5)}{1.5}\right)} = 0.01509 \quad \leftarrow \text{governs}$$

option 3

$$\frac{S_{D1}(T_L)}{T^2(R/I)} = \frac{0.06138(12)}{1.22^2(5/1.5)} = 0.14846$$

base shear

$$V = C_s W$$

$$= 0.01509(36000) = 544 \text{ K}$$

Lateral Seismic Force Distribution								
Level	Weight (kips)	Story Height h (ft)	Exponent k	Wx*hx^k (kips)	Cvx	Story Force Fx (kips)	Vx (kips)	Mx (ft-kips)
1	12,510	15.33	1.36	512383	0.0896	48.7	48.7	747
2	6,545	30	1.36	668030	0.1168	63.5	112.2	1905
3	3,435	44.67	1.36	602488	0.1053	57.3	169.5	2560
4	2,460	59.33	1.36	634730	0.1109	60.3	229.8	3580
5	2,460	74	1.36	857215	0.1498	81.5	311.3	6030
6	2,460	88.67	1.36	1096255	0.1916	104.3	415.6	9250
7	2,460	103.33	1.36	1349842	0.2359	128.3	543.9	13257
Sum	32330	104		5720943	1.0	**V = 544 K**		**M = 37330 Ft-K**

Appendix D – Wind Analysis

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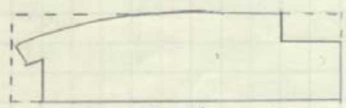
Wind Speed = 90 mph

I = 1.15

Category C

Flexible Structure



n_1 = frequency

$= \frac{1}{1.22} = 0.81967$

$$g_r = \sqrt{2\ln(3600(n_1))} + \frac{0.577}{\sqrt{2\ln(3600(n_1))}} = 4.142$$

g_Q and $g_v = 3.4$

$\bar{z} = .6h = .6(104) = 62.4 > \bar{z}_{min} = 15$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 500\left(\frac{62.4}{33}\right)^{1/5} = 568$$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}(V)\left(\frac{88}{60}\right) = 0.65\left(\frac{62.4}{33}\right)^{1/6.5}\left(\frac{88}{60}\right)90 = 94.6$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{0.81967(568)}{94.6} = 4.92$$

N-S

R_h $$n_h = \frac{4.6 n_1 h}{\bar{V}_z} = \frac{4.6(.81967)(104)}{94.6} = 4.15$$

R_B $$n_B = \frac{4.6 n_1 EB}{\bar{V}_z} = \frac{4.6(.81967)(318)}{94.6} = 12.67$$

R_L $$n_L = \frac{15.4 n_1 L}{\bar{V}_z} = \frac{15.4(.81967)(87)}{94.6} = 11.6$$

Note: equation n for R_B contains a symbol E which has been determined to be a mistake in the code. Both ASCE '02 and '05 contain typo's in this equation.

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$$R_L = \frac{1}{n} - \frac{1}{2n^2}(1 - e^{-2n})$$

$$= \frac{1}{11.6} - \frac{1}{2(11.6)^2}(1 - e^{-2(11.61)}) = 0.0825$$

$$R_B = \frac{1}{12.67} - \frac{1}{2(12.67)^2}(1 - e^{-2(12.67)}) = 0.0758$$

$$Q = \sqrt{\frac{1}{1 + .63\left(\frac{318+104}{568}\right)^{.63}}} = 0.81$$

damping ratio β is assumed to be 5%

$$R = \sqrt{\frac{1}{\beta} R_N R_h R_B (0.53 + 0.47 R_L)}$$

$$= \sqrt{\frac{1}{0.05}(0.0513)4.15(0.0758)(0.53 + 0.47(0.0825))} = 0.43$$

$$R_N = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47(4.92)}{(1 + 10.3(4.92))^{5/3}} = 0.0513$$

$$I_{\bar{z}} = c\left(\frac{10}{\bar{z}}\right)^{1/6} = 0.2\left(\frac{10}{62.4}\right)^{1/6} = 0.147$$

$$G_f = 0.925\left[\frac{1 + 1.7(I_{\bar{z}})\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right]$$

$$= 0.925\left[\frac{1 + 1.7(0.147)\sqrt{3.4^2(0.81)^2 + 4.142^2(0.43)^2}}{1 + 1.7(3.4)(0.147)}\right]$$

$$= 0.91$$

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E-W

R_B

$$n_B = \frac{4.6(0.81967)(87)}{94.6} = 3.47$$

$$R_B = \frac{1}{3.47} - \frac{1}{2(3.47)^2}\left(1 - e^{-2(3.47)}\right) = 0.267$$

R_L

$$n_L = \frac{15.4(0.81967)(318)}{94.6} = 42.43$$

$$R_L = \frac{1}{42.43} - \frac{1}{2(42.43)^2}\left(1 - e^{-2(42.43)}\right) = 0.0234$$

$$Q = \sqrt{\frac{1}{1 + .63\left(\frac{87+104}{568}\right)^{.63}}} = 0.871$$

$$R = \sqrt{\frac{1}{0.05}(0.0513)(4.15)(0.267)(0.53 + 0.47(0.0234))}$$

$$= 0.784$$

$$G_f = 0.925\left[\frac{1 + 1.7(0.147)\sqrt{3.4^2(0.871)^2 + 4.142^2(0.784)^2}}{1 + 1.7(3.4)(0.147)}\right]$$

$$= 1.05$$

pressure

$P = q G_f C_p - q_i (\pm G C_{pi})$

Enclosed Building
$GC_{pi} = \pm 0.18$
$C_p = 0.8$ windward
-0.5 leeward

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$
$= 0.00256 (K_z)(1)(0.85) 90^2 (1.15)$
$= 20.27 K_z$

N-S

leeward

$P = q_h G_f C_p - q_h (-GC_{pi})$
$= 20.23(0.91)(-0.5) - (20.23)(-0.18)$
$= -5.56$

windward

$P = q_z G_f C_p - q_i (GC_{pi})$
$= q_z (0.91)(0.8) - 20.23(0.18)$

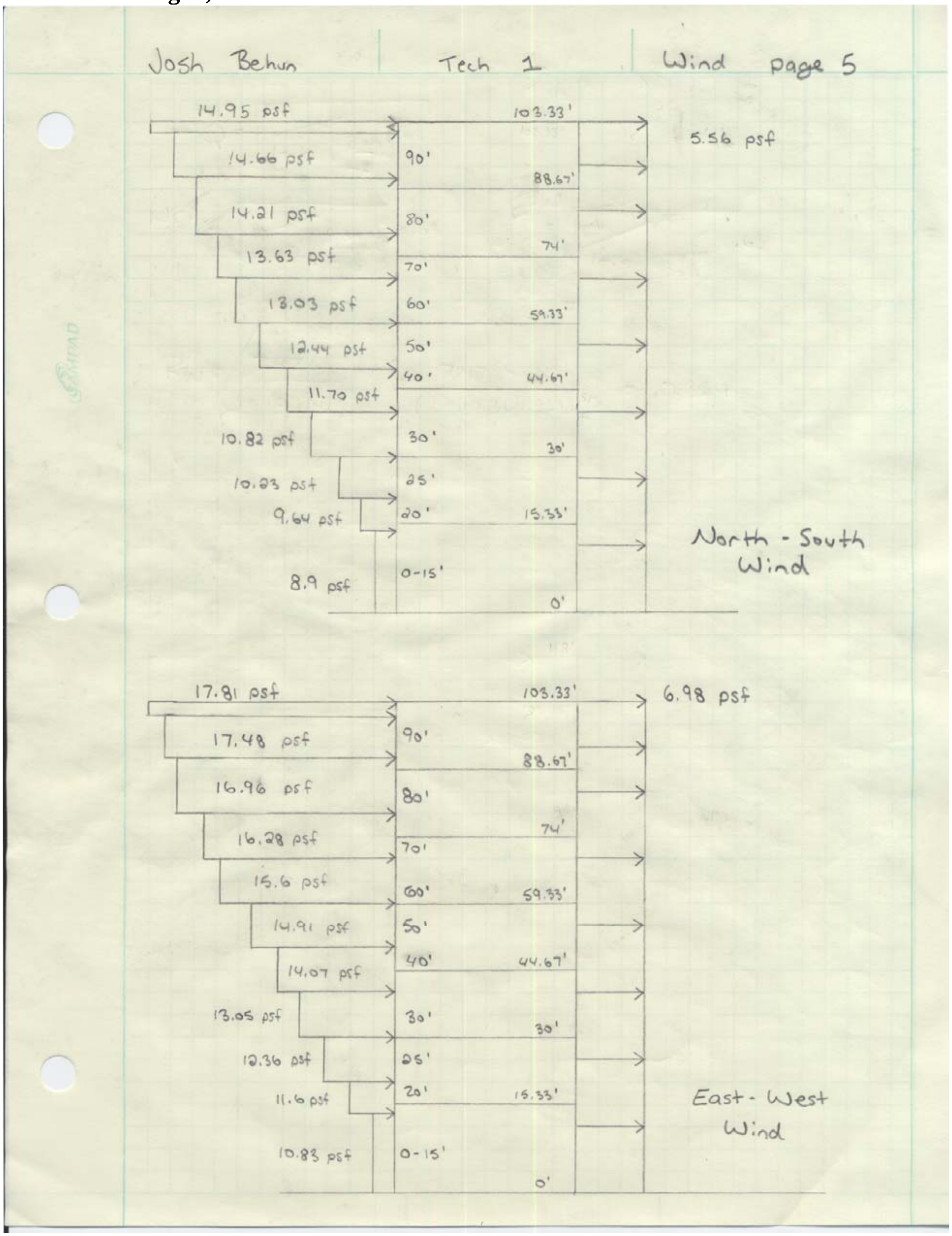
E-W

Leeward

$P = 20.23(1.05)(-0.5) - 20.23(-0.18)$
$= -6.98$

Windward

$P = q_z (1.05)(0.8) - 20.23(0.18)$


Josh Behun
Tech 1
Wind page 5
14.95 psf
103.33'
5.56 psf
14.66 psf
90'
88.67'
14.21 psf
80'
74'
13.63 psf
70'
13.03 psf
60'
59.33'
12.44 psf
50'
40'
44.67'
11.70 psf
10.82 psf
30'
30'
10.23 psf
25'
9.64 psf
20'
15.33'
North - South Wind
8.9 psf
0-15'
0'
17.81 psf
103.33'
6.98 psf
17.48 psf
90'
88.67'
16.96 psf
80'
74'
16.28 psf
70'
15.6 psf
60'
59.33'
14.91 psf
50'
40'
44.67'
14.07 psf
13.05 psf
30'
30'
12.36 psf
25'
11.6 psf
20'
15.33'
East - West Wind
10.83 psf
0-15'
0'

Wind Pressure

North – South Wind Pressures					
Height (ft)	Kz	qz	Windward Pressure (psf)	Leeward Pressure (psf)	Total
0-15	0.85	17.23	8.90	-5.56	14.46
20	0.9	18.24	9.64	-5.56	15.20
25	0.94	19.05	10.23	-5.56	15.79
30	0.98	19.87	10.82	-5.56	16.38
40	1.04	21.08	11.70	-5.56	17.26
50	1.09	22.09	12.44	-5.56	18.00
60	1.13	22.91	13.03	-5.56	18.60
70	1.17	23.72	13.63	-5.56	19.19
80	1.21	24.53	14.21	-5.56	19.78
90	1.24	25.14	14.66	-5.56	20.22
110	1.26	25.54	14.95	-5.56	20.51
120	1.31	26.55	15.69	-5.56	21.25

North – South Wind Loading								
Floor	Height (ft)	Tributary Height (ft)	Windward Pressure (psf)	Leeward Pressure (psf)	Total (psf)	Story Force (k)	Total Shear (k)	Overturning Moment (ft-k)
Ground	0	0	0	0	0	0	**V = 532**	**M = 31590**
2	15.33	15	9.64	-5.56	15.20	70.7	532.1	1084
3	30	14.67	10.53	-5.56	15.79	62.1	461.4	1863
4	44.67	14.67	11.70	-5.56	17.26	82.2	399.3	3672
5	59.33	14.67	12.74	-5.56	18.30	85.4	317.1	5067
6	74	14.67	13.63	-5.56	19.19	91.1	231.7	6741
7	88.67	14.67	14.21	-5.56	19.77	93.1	140.6	8255
Roof	103.33	7.33	14.95	-5.56	20.51	47.5	47.5	4908

East – West Wind Pressures					
Height (ft)	Kz	qz	Windward Pressure (psf)	Leeward Pressure (psf)	Total
0-15	0.85	17.23	10.83	-6.98	16.39
20	0.9	18.24	11.6	-6.98	17.24
25	0.94	19.05	12.36	-6.98	17.92
30	0.98	19.87	13.05	-6.98	18.61
40	1.04	21.08	14.07	-6.98	19.63
50	1.09	22.09	14.91	-6.98	20.47
60	1.13	22.91	15.60	-6.98	21.16
70	1.17	23.72	16.28	-6.98	21.84
80	1.21	24.53	16.96	-6.98	22.52
90	1.24	25.14	17.48	-6.98	23.04
110	1.26	25.54	17.81	-6.98	23.37
120	1.31	26.55	18.66	-6.98	24.22

East – West								
Floor	Height (ft)	Tributary Height (ft)	Windward Pressure (psf)	Leeward Pressure (psf)	Total (psf)	Story Force (k)	Total Shear (k)	Overturning Moment (ft-k)
Ground	0	0	0	0	0	0	**V = 175**	**M = 10528**
2	15.33	15	11.60	-6.98	18.58	18.1	175.1	278
3	30	14.67	12.71	-6.98	19.69	25.1	157.6	753
4	44.67	14.67	14.07	-6.98	21.05	27.4	132.5	1224
5	59.33	14.67	15.26	-6.98	22.24	28.4	105.1	1685
6	74	14.67	16.28	-6.98	23.26	30.2	76.7	2235
7	88.67	14.67	16.96	-6.98	23.94	30.8	46.5	2731
Roof	103.33	7.33	17.81	-6.98	24.79	15.7	15.7	1622

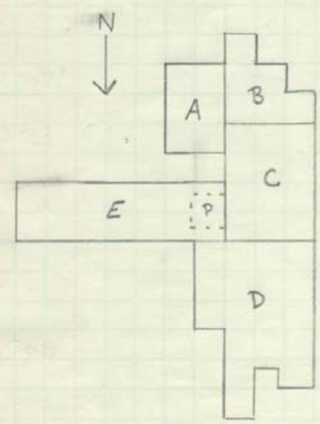
Appendix E – Snow Loads

Josh Behun | Tech 1 | Snow Loads

page 1

Flat Roof Load

$P_f = 21$ psf



Basic Footprint Elevations
A = 43'
B = 32'
C = 32'
D = 18'
E = 105'
penthouse = 125'

Drifts on Lower Roofs

$\gamma = 0.13 P_g + 14$

$= 0.13(30) + 14 = 17.9$ pcf < 30 ok

$h_b = P_f / \gamma = 21 / 17.9 = 1.17$

Drift E-C

$h_c = 93'$ $\quad h_c / h_b = \frac{93}{1.17} > .2$ ok

$l_{u\,upper} = 69'$ $\quad l_{u\,lower} = 108'$

leeward drift

$h_d = .43 (l_u)^{1/3} (P_g + 10)^{1/4} - 1.5$

$= .43 (69)^{1/3} (30 + 10)^{1/4} - 1.5$

$= 2.94$ ← governs

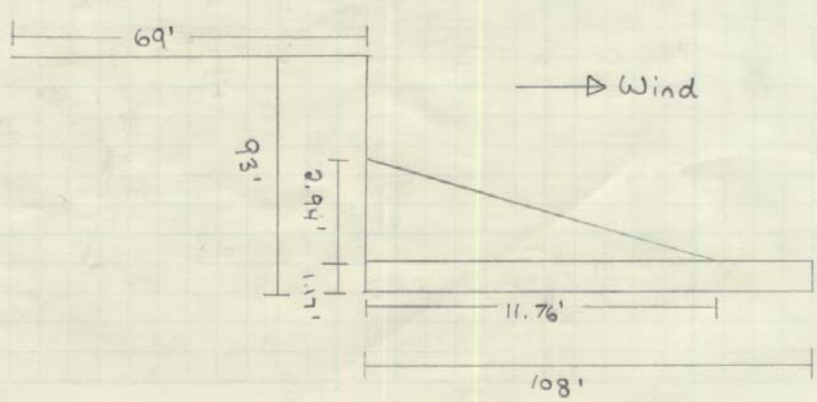
windward drift

$h_d = .75 [.43 (108)^{1/3} (30 + 10)^{1/4} - 1.5]$

$= 2.74$

$W = 4 h_d = 4(2.94) = 11.76$

$P_{max} = \gamma (h_d + h_b) = 17.9 (2.94 + 1.17) = 73.57$ psf



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Drift C-D

$h_c = 14'$ $h_c/h_b = 14/1.17 = 12 < 30$ ok

$l_{u\,upper} = 63'$

$l_{u\,lower} = 150'$

leeward drift

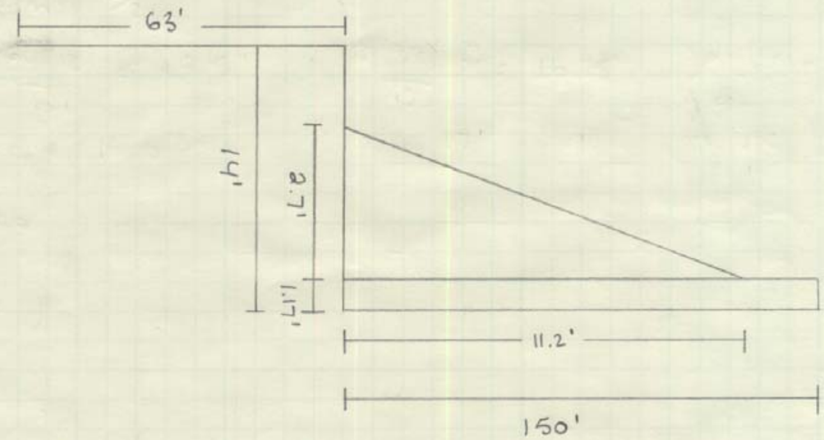
$h_d = .43\,(63)^{1/3}\,(40)^{1/4} - 1.5$
$= 2.8$ ← governs

windward drift

$h_d = .75\left[.43\,(105)^{1/3}\,(40)^{1/4} - 1.5\right]$
$= 2.7$

$w = 4h_d = 4(2.8) = 11.2$

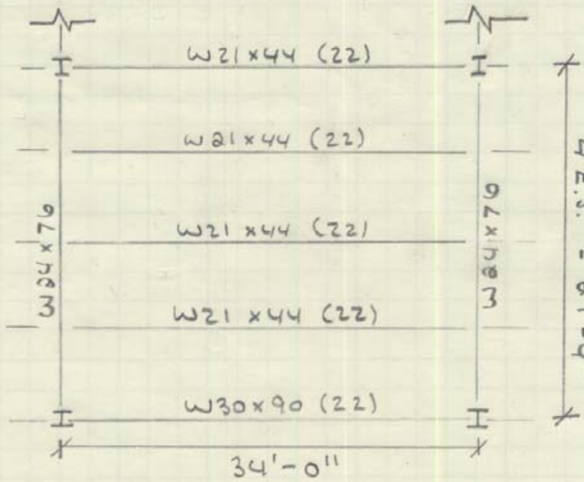
$P_{max} = \gamma\,(h_d + h_b) = 17.9\,(2.7 + 1.17) = 69.27$



Appendix F – Spot Checks
Composite Beam Check

Josh Behun | Tech 1 | Beam Spot Check page 1

Typical Southern Wall Bay - 5th Floor



4" LWC slab + 2" composite steel decking
$f'_c = 4000$ psi
$f_y = 50{,}000$ psi
(N) indicates number of 3/4" shear studs

Dead Load

$\frac{5}{12}'(110 \frac{lbs}{ft^3}) = 46$ psf + 2 psf deck (7.33') = 352 plf

self weight = 44(4)+90/5 = 53 plf

partition allowance = 20 psf (7.33') = 147 plf

Superimposed / collateral (assumed) = 20 psf (7.33') = 147 plf

700 plf

Live Load

public areas = 100 psf (7.33') = 733 plf

$$w_u = 1.2(700) + 1.6(733) = 2015 \text{ plf} = 2.015 \text{ klf}$$

$$M_u = \frac{w_u L^2}{8} = \frac{2.015(34)^2}{8} = 292 \text{ ft}\cdot\text{k}$$

$$V_u = \frac{w_u L}{2} = \frac{2.015(34)}{2} = 34.25 \text{ k}$$

$$b_{eff} = \begin{cases} 7.33(12) = 88'' \leftarrow \text{governs} \\ 34/4\,(12) = 102'' \end{cases}$$

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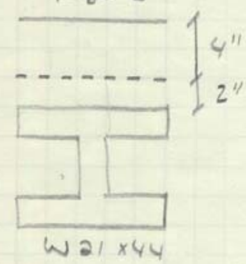

$\Sigma Q_n = F_y A_s = 50(13) = 650$ k

$a_{req'd} = \dfrac{\Sigma Q_n}{.85 f'_c b} = \dfrac{650}{88(4)(.85)} = 2.17$

$y_2 = 6 - \dfrac{2.17}{2} = 4.92$

for PNA @ TFL

$\Sigma Q_n = 649 \approx 650$ ok

$M_{allow} = 746 > \dfrac{M_u}{\phi} = \dfrac{292}{.9} = 325$ ok

$I_B = 2370$ in^4 @ TFL $y_2 = 5''$

$\Delta_{max} = \dfrac{5(2.015)(34)^4(1728)}{384(29000)(2370)} = .8815$

$\dfrac{l}{450} = \dfrac{34(12)}{450} = .907$ $\Delta_{max} < \dfrac{l}{450}$

$.8815 < .907$ ok

use W21x44 composit beam

Column Check

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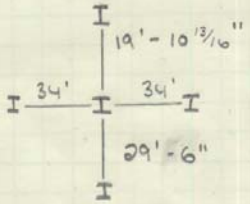
Column @ L.1 - 0.1
Bottom Column



Tributary Area = 34' (19.9 + 29.5) / 2
= 840 ft²

Loadings

Roof = 30 psf live
63 psf dead

Floors = 80 psf live
104.5 psf dead

Using 1.2D + 1.6L

Roof = 1.2(63) + 1.6(30)
= 124 psf

floors = 1.2(104.5) + 1.6(80)
= 253 psf

Live Load Reduction

$L = L_o\left(.25 + \frac{15}{\sqrt{A_T K_{LL}}}\right)$

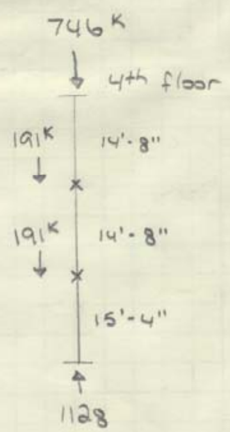
interior column
$K_{LL} = 4$

$= 80\left(.25 + \frac{15}{\sqrt{840(4)}}\right) = 40.7$

$.4L = .4 \cdot 80 = 32$
$40.7 > 32$ ok

floors = 1.2(104.5) + 1.6(40.7) = 191 psf

Load = 840 (124) + 840 (191) (4 floors)
= 746 K



K = 0.8
$KL_y = 0.8(15.33) = 12.3'$

Can use
W12 x 106
@ 13' $\phi P_n = 1170 > 1128$ ok

Design engineer use W12 x 136.
My design load is probably slightly lower since effects of wind and or seismic loading have not been factored in.

Lateral Bracing Check

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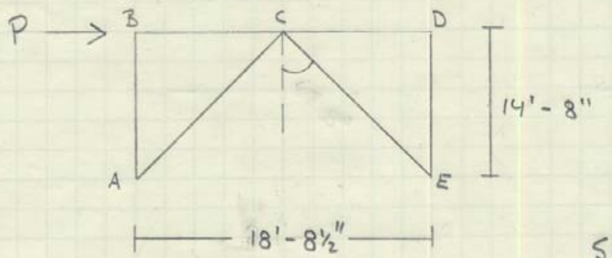
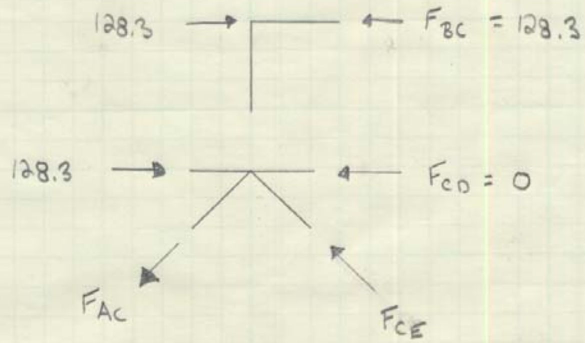
Top Bay in N-S Direction



Wind Load
P = 47.5 K
Seismic Load
P = 128.3 K

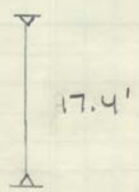
Seismic Loading Controls



$\Sigma F_y: F_{AC} = F_{CE}$

$\Sigma F_x: 128.3 = 2 F_{CE} \sin 77.7$

$F_{CE} = 65.5^K$
T and C



K = 1
KL = 17.4'

Using LRFD $\phi = .9$

$\therefore \frac{65.5}{.9} = 72.8$

Use:
HSS 6×6×1/4
or 5×5×1/2

Design engineer was able to use HSS 5×5×3/8 claiming that the Tensile/Compressive force was 75 k.