EDENWALD NEW TOWER

Technical Report #1

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EXECUTIVE SUMMARY

The purpose of this report is to provide an understanding for the methods used to design the Edenwald New Tower, currently under construction in Baltimore, Maryland. It will also provide a summary of loads derived from industry standards and codes. The primary code that I used in my analysis is ASCE 7-05.

The Edenwald New Tower is a 12 story building, comprised of flat-plate, post-tensioned concrete slabs supported by concrete columns and shear walls. A seismic analysis was conducted in accordance with the equivalent lateral force method to determine the minimum allowable base shear. Next, a Method 2 wind analysis was performed, also to determine a base shear value. These values were then compared to determine the controlling lateral force. In both analyses, loads were distributed along the building's faces according to code.

The lateral system was then analyzed based on the assumption that the concrete shear walls received 100 percent of the controlling lateral load. Shear wall number 1 was picked as the wall to analyze, and was analyzed strictly for shear strength.

Additionally, spot checks were performed on a typical column and a one way slab. (One way slabs exists in many locations were post-tensioning was not found feasible and/or practical.) The results of these checks can be found in the conclusions of this report. Backup calculations can be found in the Appendix.

STRUCTURAL SYSTEM OVERVIEW

Foundation:
The geotechnical analysis of the sub-surface conditions prior to construction revealed great variances in soil type and depth to bedrock, ranging from 50 to 150 feet deep, making deep foundations impractical. Given two recommendations from the geotechnical engineer, it was decided by the designers to use a geopier system as opposed to an alternative of driven HP 12x74 piles. Comprised of densified "rammed" stone aggregate piers, geopiers are referred to as "intermediate foundation systems" in that they strengthen, stiffen and reinforce soil layers beneath the building. The use of this option provided the opportunity to utilize a shallow foundation system of typical spread footings. (It should be noted, however, that pre-existing utilities only discovered upon excavation in the north end of the site required the use of the HP piles, in that localized area only.) The geopiers were determined to require a 30 inch diameter, and range from 20 to 30 feet in length. The allowable bearing pressure of the strengthened soil beneath the building was then determined to be 6 ksf beneath the tower, and 4 ksf beneath the parking garage. Total settlement expected from the geopier design amounts to one inch.

All concrete used in the Edenwald New Tower is normal weight (145 pcf dry unit weight). Footings, grade beams and slabs on grade have a minimum 28-day strength of 3000 psi. Shear wall footings have a minimum 28-day strength of 4000 psi. The slab on grade is reinforced with 6x6-W2.9x2.9 WWF on a vapor barrier on 4 inches of granular subbase.

Floor System:
The typical floor system used is a 9 inch, post-tensioned concrete slab having a minimum 28-day strength of 5000 psi. In specific locations where the post tensioned system is not feasible and/or practical, reinforced one way slabs were used, ranging in thickness from 8 to 9 inches, with cast in place concrete beams, both requiring a minimum 28-day strength of 5000 psi.

Roof System:
The flat roof system is almost identical to the typical floor system. Still utilizing the post-tension reinforcement, the slab thickness reaches up to 16 inches underneath the penthouse. The penthouse is supported by a steel braced frame and is covered by 1.5 inch deep, wide rib, 20 gage galvanized metal deck. The pentouse roof is supported by a combination of steel W shapes and 12k3 joists. The columns supporting the penthouse are W8x31 shapes.

Columns:
The building is supported by rectangular concrete columns laid out in a geometric grid. The columns range in size, the most common being 22x22 and 22x36. The largest column found in the building is 22x60. Column loads range from 203 kips in the garage to 1600 kips at the base of the tower. From the ground level to the seventh floor, the columns are required to have a minimum 28-day strength of 6000 psi. From the seventh floor to the roof, that value decreases to 5000 psi.

Lateral System:
The building is laterally supported in both the N-S and E-W directions by a total of 15 simply reinforced concrete shear walls, with thickness ranging from 12 to 14 inches. These shear walls are required to have a minimum 28-day strength of 5000 psi. Located throughout the building, the shear walls are often conveniently placed around stair and elevator shafts. All but one of the 15 shear walls run the entire height of the building.

CODES

CODES EMPLOYED FOR ORIGINAL DESIGN

- "International Building Code – 2003", International Code Council
- "Minimum Design Loads for Building and Other Structures", (ANSI/ASCE 7-2002) American Society of Civil Engineers
- "Building Code Requirements for Reinforced Concrete, ACI 318", American Concrete Institute
- "ACI Manual of Concrete Practice – Parts 1 through 5"
- "Manual of Standard Practice", Concrete Reinforcing Steel Institute
- "Post Tensioning Manual", Post Tensioning Institute
- "Manual of Steel Construction – Allowable Stress Design", Ninth Edition, 1989, American Institute of Steel Construction
- "Manual of Steel Construction, Volume II Connections", ASD 9th Edition, American Institute of Steel Construction
- "Detailing for Steel Construction", American Institute of Steel Construction
- "Structural Welding Code ANSI/AWS D1.1", American Welding Society

CODES SUBSTITUTED FOR THESIS DESIGN

- "Minimum Design Loads for Building and Other Structures", (ANSI/ASCE 7-2005) American Society of Civil Engineers

Original Design Loads

Gravity: Superimposed Dead Loads

Item	Design Value
Typical Floor Areas	30 psf
Typical Parking	5 psf
Parking above occupied space	35 psf
Garage Roof	35 psf
Main Roof	30 psf

Gravity: Live Loads

Item	Design Value	Comment (Values found in Table 4-1 of ASCE 7)
Framed Floor Areas	40 psf	Code Minimum: 40 psf (residential)
Lobbies/Stairs/Exits	100 psf	Code Minimum: 100 psf
Corridors above 1st Floor	100 psf	Code Minimum: 100 psf
Parking Decks	50 psf	Code Minimum: 40 psf
Balconies	100 psf	Code Minimum: 100 psf
5th Floor Terrace/Roof	100 psf	Code Minimum: 100 psf (roofs used for assembly purposes)

Gravity: Roof Live Loads

Item	Design Value	Comment
Roof Live Load (snow load used when greater than 30 psf)	30 psf	Code Minimum: 20 psf (ordinary flat roof) (See Table 4-1 of ASCE 7
Roof Snow Load	P_f = 19.25 psf C_e = 1.0 I = 1.1 C_t = 1.0	Calculated Snow Load: Pf = 19.25 psf (See Appendix, calculated according to chapter 7 of ASCE 7)

Original Design Loads

Lateral Loads: Seismic

Seismic Use Group: II
Sesimic Importance Factor: Ie = 1.25
Mapped Spectral Response Coefficients: SDS = 0.210 g SD1 = 0.070 g
Site Class: D
Spectral Response Coefficients: SDS = 0.224 g SDS = 0.112 g
Sesimic Design Category: B
Design Base Shear: V = 997 kips
Seismic Response Coefficient: Cs = 0.022
Response Modification Factor: R = 5.0
Analysis Procedure: Equivalent Lateral Force Procedure

Lateral Loads: Wind

Basic wind speed (3-sec gust) = 90 mph
Importance Factor: 1.15
Exposure Category: B
Internal Pressure Coefficient: Gcpi = ±0.18

CONCLUSIONS

SEISMIC: Though Baltimore, Maryland is not a high risk seismic zone, seismic forces must still be considered due to code requirements. The resulting shear force that I derived through the equivalent lateral force method is 760 kips. This difference of 230 kips from the designed shear value of 990 kips was most likely caused by weight calculation differences. It can be proven through a simple back calculation that the original seismic weight estimated by the engineer was around 45,000 kips. This is 10,000 kips, or 10 million pounds, lighter than my estimate. After re-examining my estimate, I believe this weight difference to be a result of several things. There is no evidence in the load summary provided on the drawings that the engineer included a partition load. However, according to ASCE 07-05 12.7.2, this must be included in the seismic weight. (Section 4.2.2 states that, where minimum required live loads exceed 80 psf, partition loads may be disregarded. However, the live loads are not included in the seismic weight, and as such it seemed appropriate to include the partition load in my calculations.) This additional weight from partitions accounts for 5,000 kips alone – half of the excess. Though I believe my column and shear wall estimates to be rather accurate, the additional weight may have resulted from a poor estimate of the veneer.

Regardless, a larger weight would suggest a larger base shear. My base shear was lower because I arrived at a significantly lower value for C_s. While the designers found C_s to be 0.022, I found it to be 0.014. ASCE 07-05 provides a number of different ways in calculating the period, and by trying some different equations, I arrived at a C_s of 0.023, which is much closer. However, I decided to use the lowest allowable value by code for obvious reasons. As such, my shear value is much closer to the design value than it would have been had I used the design C_s value.

WIND: Using Method 2 from chapter 6 of ASCE 07-05, I examined the Edenwald New Tower's main wind-force resisting system. For simplification purposes, I normalized the shape of my building to a rectangular footprint and continued the shape from first floor to roof. Because the projected area of surface receiving wind-load does not change, this is a reasonable assumption for the sake of this report.

Because of the weight of the heavy concrete roof slab, the penthouse and mechanical equipment, uplift forces on this building will be negligible and not necessary to consider as a part of this report. The main forces acting on the building are windward and leeward forces, both needing to be considered simultaneously to compute the overall base shear. Because the tower is partially blocked in the east and west direction from the adjoining structure, the total base shear would have to be adjusted to reflect that half the leeward pressure (or windward pressure, depending on the direction of the wind) not be considered.

WIND VS. SEISMIC: To compare wind and seismic results, I examined the building's base shear from wind moving in the N-S direction because that is the direction which controls, due to the building's dimensions. The resulting base shear of 658 kips is lower than the seismic base shear of 770 kips (and the design base shear of 990 kips). I believe that this result is due to the fact that building simply is not high enough for wind to control. Additionally, the weight of this building must be considered when compared to what it could have been had the designers chosen a steel frame. In that case, wind may have controlled since a lower weight would have yielded a lower base shear. A possible thesis proposal would be to design the building as a steel frame for the sake of reducing lateral forces.

SHEAR WALL ANALYSIS: The Edenwald New Tower has a total of 15 shear walls to resist lateral loads. Because the east wing is angled 13.5 degrees clockwise, I first made the assumption that taking the projected length of shear walls in the N-S direction would provide more accurate stiffness values in that direction. My calculations concluded that the shear wall number 1, as designed, could resist a base shear of 600 kips. However, according to the distribution of the base shear with respect to relative stiffness, it only receives 173 kips in shear. The main cause of this difference is that my analysis examined the wall in shear only and did not account for axial loading of the shear wall. The interaction of both compression, bending and shear would merit a larger wall in which greater shear strength would be available.

COLUMN SPOT CHECK: Using the spreadsheet found in the appendix, I calculated the effective axial force on Column G3 at its base. I assumed that the shear wall system would carry 100 percent of the lateral seismic load, and as such determined that the column would only need to be designed to withstand axial forces. The force I found, 1670 kips, was significantly higher than the working load of 1225 kips listed in the column schedule. Because of the thoroughness of my calculations, and the inclusion of all possible live load reductions, I am left with only one possible reason as for the difference in value. Column G3 is located 12'-5" from a shear wall, and as such, it is possible the shear wall will take more than half the tributary area between the two. I made the assumption that each element would take half, but that may have been overly conservative. If this is the case, a more thorough analysis will be needed to evaluate how the post-tensioning system distributes the loads to the vertical members.

To maintain alignment with the drawings, I chose to design the column based on the listed load of 1225 kips at the base. The result was a 22x14 square inch column reinforced with (8) #10 bars. This compares to the listed dimensions of a 22x36 square inch column reinforced with (8) #10 bars. The large cross sectional area may be an indicator that the building was designed as a Dual System, where the shear walls are not designed to carry the entire lateral load. In my design, I only considered axial loading based on the above mentioned assumption. However, in this case, the columns would need to be larger to carry the moments distributed to them.

ONE WAY SLAB SPOT CHECK: Using the method of treating the slab as a series of 1 foot wide rectangular beams, I determined a particular slab to be 4 inches in depth and be reinforced with No. 4 bars spaced at 12 inches on center for both positive and negative reinforcement. This is compared to the same slab designed for 9 inch thickness and No. 5 bars spaced at 12 inches on center. The need for a thicker slab could be to maintain the same depth as the adjacent, 9 inch post-tensioned slab for ease of constructability.

SUMMARY: It is clear from the wide range of differences between my results and those of the designers that further research into their design methods is required, as well as more accurate analysis and design methods. Because of the irregularity and complexity of the building's shape, a computer model will be necessary to properly understand the distribution of lateral loads to the building's frame. It is my goal to address these issues in subsequent technical reports.

Seismic Analysis: Equivalent Lateral Force Method

In the following analysis, several assumptions were made. First, the new tower was treated as an independent structure. In reality, it is connected to an existing tower with an expansion joint. This assumption that the building will respond to seismic forces independently is conservative, because the actual stiffness of the building would be influenced by the existing tower. Secondly, computing the weight of the building (the calculations of which can be found in the appendix) required a number of assumptions in estimating material weights. Those assumptions are clear when a detailed review of the calculations is made.

Weight of Building = 54171 kips (see appendix)
Occupancy Category: III
Site Class: D
S_S = 0.178 g
S_1 = 0.052 g

> Note: These values taken from following website:
> http://earthquake.usgs.gov/research/hazmaps/design/
> However, in an effort to keep results as close as possible to those of designers, I will use S_S and S_1 values provided in structural drawings:
>
> **S_S = 0.210 g**
> **S_1 = 0.070 g**

F_a = 1.6
F_v = 2.4

S_{MS} = (0.210*1.6) = 0.336 g　　S_{DS} = 2/3(0.336) = 0.224 g
S_{M1} = (0.070*2.4) = 0.168 g　　S_{D1} = 2/3(.168) = 0.112 g

Seismic Design Category: B

R = 5 (table 12.2-1, ordinary reinforced concrete shear walls)
Importance Factor: I = 1.25

$Ta = C_T h_n^{\,x} = 0.016(121.4)^{0.9} = 1.202$ s
T = Cu*Ta = 1.7(1.202) = 2.04 s
T_L = 6 s > T

$C_S = S_{DS}/(R/I) = 0.224/(5/1.25) = 0.056$
$C_S = S_{D1}/[T(R/I)] = 0.112/[2.04(5/1.25)] = 0.014$ controls

$V = C_S * W = 0.014*54171k = 760$ kips

Seismic Analysis: Equivalent Lateral Force Method

Vertical Distribution of Seismic Forces:
k = 1.77 (interpolated from values given in section 12.8.3)

Level	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x	M_x
*Roof	3637	121.40	9808910265	0.1854	141	17108
12	3989	107.33	9286650625	0.1755	133	14320
11	3935	98.00	7717471989	0.1459	111	10866
10	3935	88.67	6465017158	0.1222	93	8236
9	3935	79.33	5308956929	0.1004	76	6051
8	3935	70.00	4254303203	0.0804	61	4278
7	3935	60.67	3302698118	0.0624	47	2879
6	3759	50.00	2163197794	0.0409	31	1554
5	5290	39.33	2588975037	0.0489	37	1463
4	4739	28.00	1168155227	0.0221	17	470
3	5396	18.67	717365758	0.0136	10	192
2	3911	9.33	118841574	0.0022	2	16
			Overturning Moment (ft-k) =			67431

*Includes weight of Penthouse

SEISMIC LOAD DISTRIBUTION

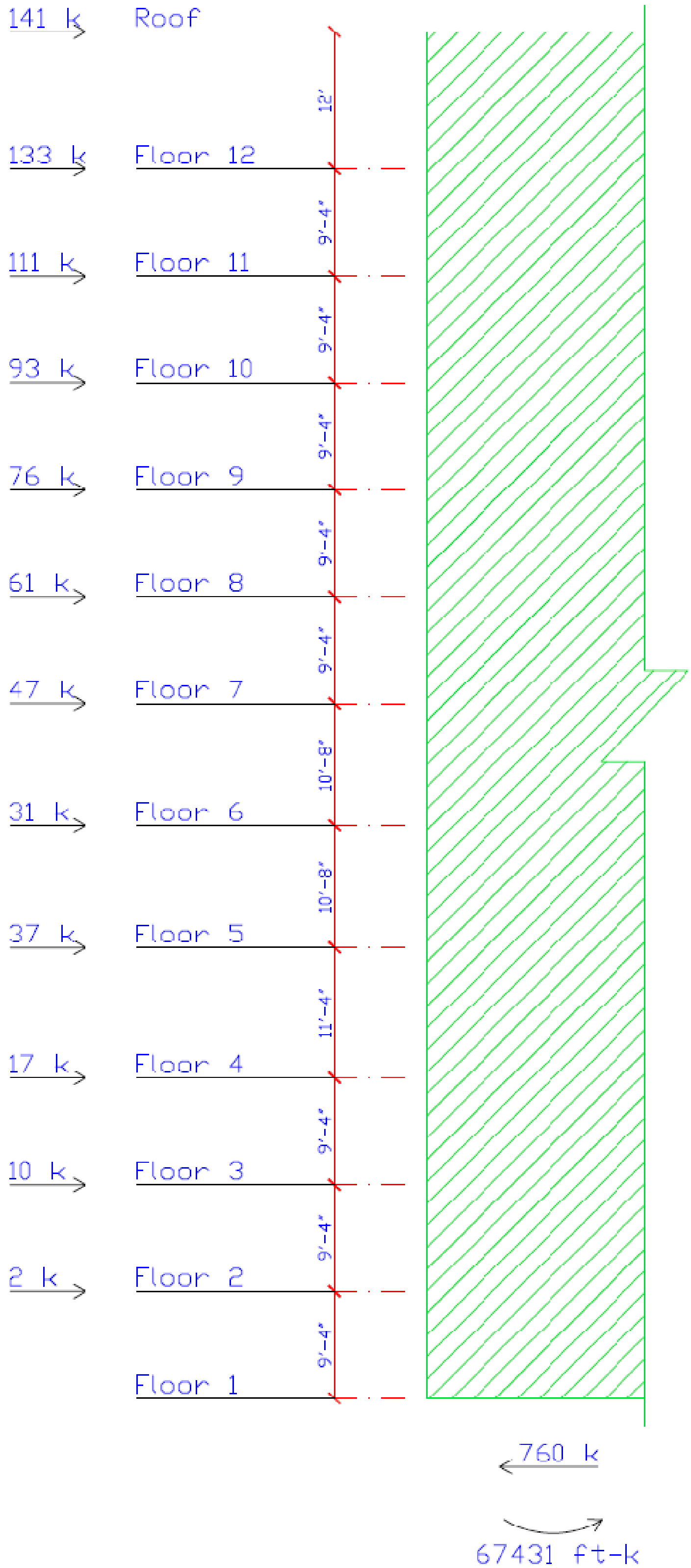


Figure 1: Seismic Load Distribution

Wind Analysis

Below are the assumptions and main calculations for the derivation of wind loads on the Edenwald New Tower. Please see the appendix for back up calculations. For the wind analysis, all assumptions and results matched those of the designers provided in the above tables. Though the building is partially blocked in the E-W direction, half of the building still receives wind forces from both directions, which is why the diagram does not account for the blockage.

Leeward Wall (EW)(psf)	Max (psf)
P = -8.06 ± 3.8	-11.86

Leeward Wall (NS)(psf)	Max (psf)
P = -8.75 ± 3.8	-12.55

Basic Wind Speed	90 mph
Occupancy Category	III
Importance Factor	1.15
Exposure	B
Topographic Factor (K_{zt})	1.0
Wind Directionality Factor (K_d)	0.85
Gust Factor (both directions)	0.83
Internal Pressure Coefficient	± 0.18

Windward Wall				
Height (ft)	Pressure (psf)			Max (psf)
0-15	11.55	±	3.8	15.35
20	12.57	±	3.8	16.37
25	13.38	±	3.8	17.18
30	14.19	±	3.8	17.99
40	15.40	±	3.8	19.20
50	16.42	±	3.8	20.22
60	17.23	±	3.8	21.03
70	18.04	±	3.8	21.84
80	18.85	±	3.8	22.65
90	19.46	±	3.8	23.26
100	20.07	±	3.8	23.87
120	21.08	±	3.8	24.88
140	22.09	±	3.8	25.89

N-S Wind Force Summary				
Height (ft)	Windward Pressure (psf)	Resultant force (lbs)	Leeward Pressure (psf)	Resultant force (lbs)
0-15	11.55	30848.06	12.6	33642
20	12.57	11184.68	12.6	11214
25	13.38	11906.27	12.6	11214
30	14.19	25255.72	12.6	22428
40	15.40	27420.50	12.6	22428
50	16.42	29224.48	12.6	22428
60	17.23	30667.66	12.6	22428
70	18.04	32110.85	12.6	22428
80	18.85	33554.03	12.6	22428
90	19.46	34636.42	12.6	22428
100	20.07	35718.81	12.6	22428
120	21.08	75045.57	12.6	44856
	Total Kips:	377.57	Total Kips:	280.35
Resultant Total Base Shear: V = 657.92 Kips				

WIND ANALYSIS

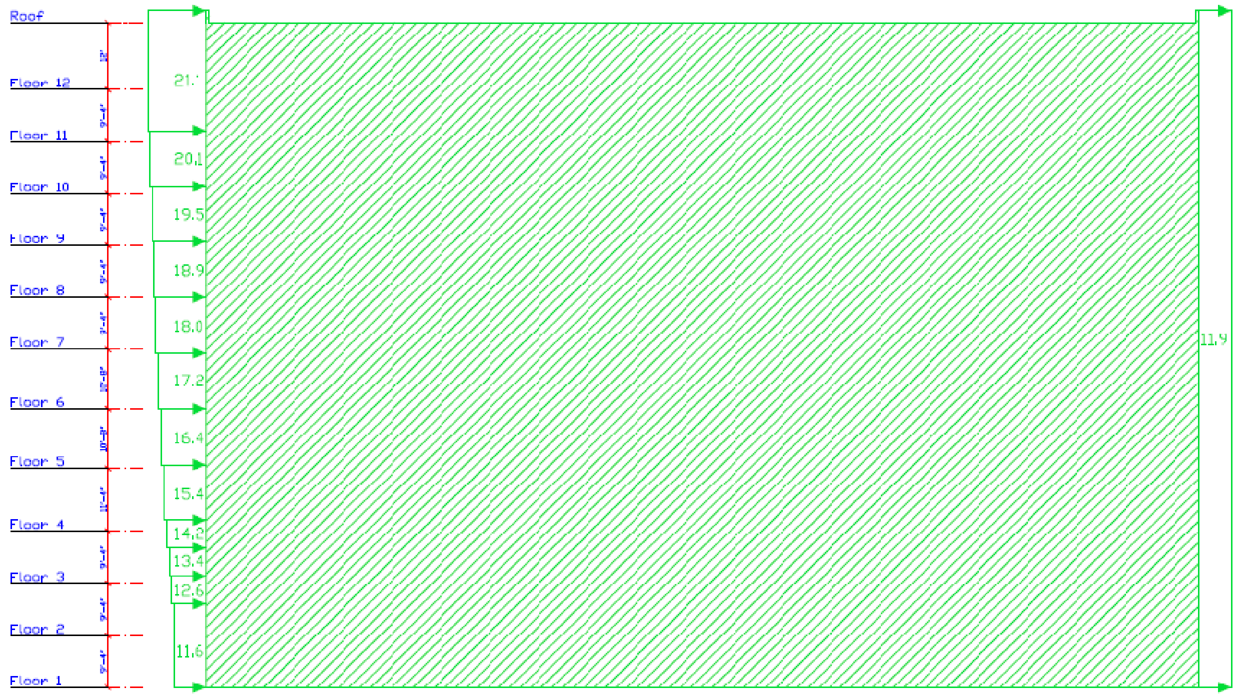


Figure : Longitudinal Section with East-West Wind Loads (psf)

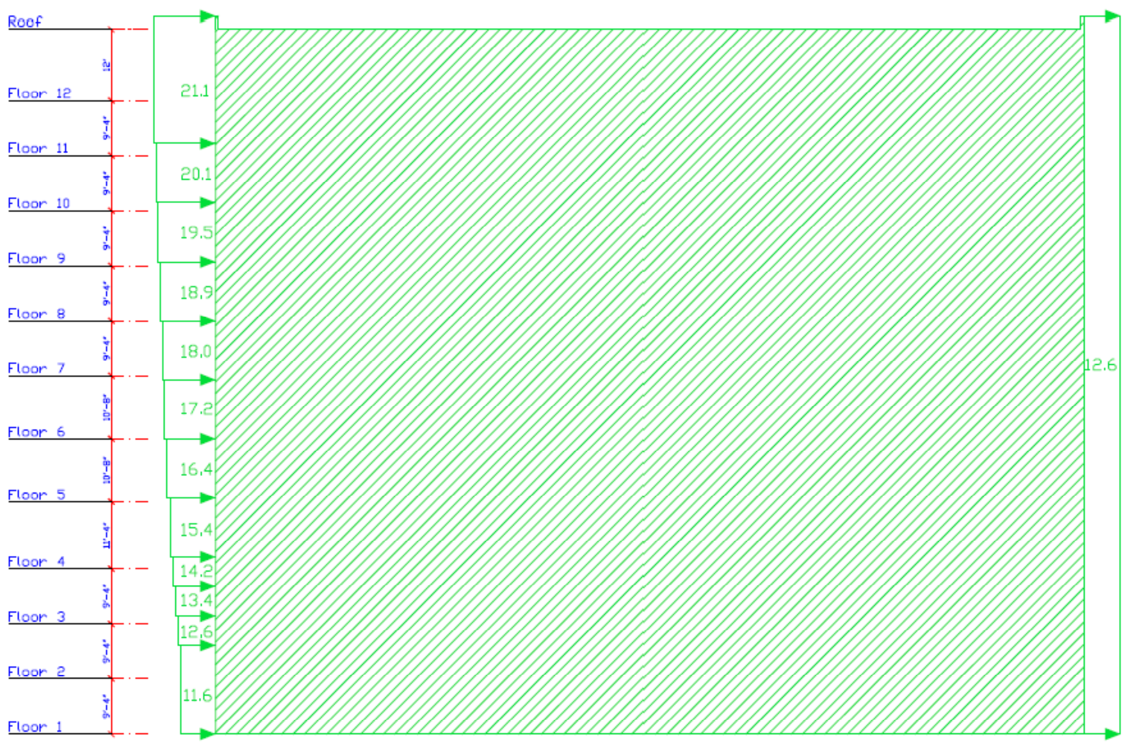
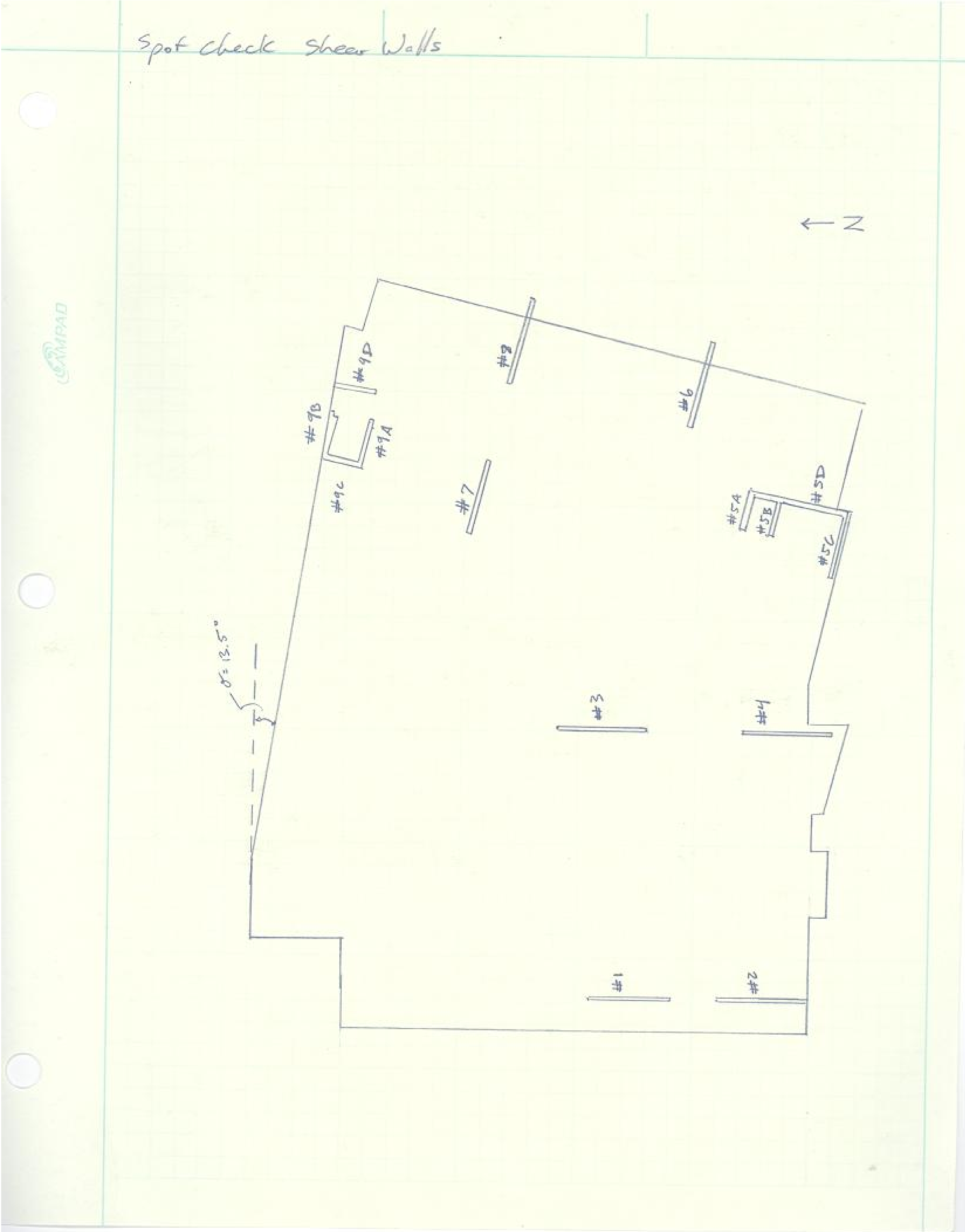


Figure : Longitudinal Section with North-South Wind Loads (psf)

SHEAR WALL ANALYSIS



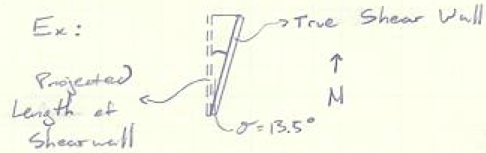
SHEAR WALL ANALYSIS

Spot check: Shearwalls · Hart

Step 1: Find Relative Stiffnesses of shear walls resisting N-S lateral load

- Will examine shear wall #1 ⇒ N-S Direction
- For shear wall influence of angled East wing, take projected length of shear wall

Ex:



- Assumptions:
  Shearwalls 5A-C, 6, 7, 8, 9A-B will not have significant influence in resisting N-S lateral loads (see shear wall layout)
- Find Projected length of shear walls 5D, 9C, and 9D

5D: length, $l = 26.42'$
Projected length, $l_p = 26.42' \cos(13.5°)$
$= 25.7'$

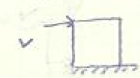
9C: $l = 9.9'$
$l_p = 9.9' \cos(13.5°)$
$= 9.62'$

9D: $l = 11'$
$l_p = 11' \cos(13.5°)$
$= 10.7'$

- List properties of shear walls resisting N-S lateral load

Wall 1: $l = 21.8' = 261.6''$
$t = 12''$
$A = 3139.2 \text{ in}^2$
$I = 1.79 \times 10^7 \text{ in}^4$
$f'_c = 4000 \text{ psi}$ }
$E_m = 3.6 \times 10^6 \text{ psi}$ } same for all walls
$E_v = 1.44 \times 10^6 \text{ psi}$ }
$H = 119.33' = 1431.96''$
$\Delta = 15 + 0.38 = 15.38$
$k = 0.065$

Note: - Assume cantilever behavior
- Assume base shear applied at roof



$V = 997^k$
$= 997000 \text{ lb}$

Equations used:

$A = t \times l$

$I = \frac{t l^3}{12}$

$E_m = 5700 \sqrt{f'c}$

$E_v = 0.4 E_m$

$\Delta = \frac{VH^3}{3E_m I} + \frac{1.2VH}{E_v A}$

$k = \frac{1}{\Delta}$

SHEAR WALL ANALYSIS

Shear Walls Cont'd — Hart

List Properties Cont'd

Wall 2: $l = 22.4' = 268.5"$
$t = 12"$
$A = 3222\ in^2$
$I = 1.94 \times 10^7\ in^4$
$f'_c = 4000\,psi$
E_m = same
E_v = same
$H = 1431.96"$
$\Delta = 13.97 + 0.37 = 14.34$
$K = 0.07$

Wall 3: $l = 242.5"$
$t = 14"$
$A = 3395$
$I = 1.66 \times 10^7$
$H = 1431.96"$
$\Delta = 16.33 + 0.35 = 16.68$
$k = 0.06$

Wall 4: same as wall 3
$k = 0.06$

Wall 5D: $l = 25.7' = 308.4"$
$t = 12"$
$A = 3701\ in^2$
$I = 2.93 \times 10^7$
$H = 1431.96$
$\Delta = 9.25 + 0.32 = 9.57$
$k = 0.104$

Wall 9C: $l = 9.9' = 119"$
$t = 12"$
$A = 1428$
$I = 1.69 \times 10^6$
$H = 1431.96"$
$\Delta = 160.4$
$K = 0.006$

Wall 9D: $l = 10.7'$
$t = 12"$
$K \approx 0.006$

Relative stiffness Wall #1:

$$k = \frac{.065}{.065 + .07 + .06 + .06 + .006 + .104 + .006} = \underline{0.173}$$

Step 2: Determine Shear Received by Wall 1

$$V_u = 0.173(997) = 172.5^k$$

Step 3: Determine Nominal Shear Capacity, V_n of Wall, compare to V_u

$$V_n = A_{cv}\left[\alpha_c \sqrt{f'c} + \rho_T f_y\right]$$
$$= 3139\left[2\sqrt{4000} + 0.0032(60000)\right]/1000$$
$$= 1000^k$$

$A_{cv} = 3139\ in^2$

Reinf: #5 @16" Each Way, Each Face

$$\rho_T = \frac{2(.31)}{(16")(12")} = 0.0032$$

$$\frac{H}{L} = \frac{1431.96}{261.6} = 5.47 > 2$$
$\hookrightarrow \alpha_c = 2$

$$\phi V_n = 0.6(1000) = 600^k$$

$600^k > 172.5^k$ ok

ONE WAY SLAB SPOT CHECK

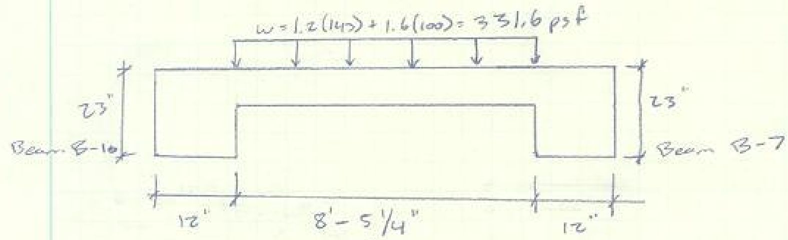
One Way Slab Spot Check | Hart | 92.7

Check Slab S-6 @ West Wing end, adjacent to existing tower, Floor 5

Space Description: Elevator lobby and Vestibule

Live Load = 100 psf

Dead Load = 113 psf + 30 psf = 143 psf



$f'_c = 5000$ psi

$f_y = 60{,}000$ psi

Simply Supported $\Rightarrow h_{min} = l/20 = 8.4375/20 = 0.42"$

13.1 $-M = \frac{1}{11} w l_n^2 = \frac{1}{11}(331.6)(8.44)^2 = 2.15$ k-ft (Exterior supports, next frame begins post tensioned slab)

$+M = \frac{1}{14} w l_n^2 = \frac{1}{14}(331.6)(8.44)^2 = 1.69$ k-ft

$\rho_{max} = 0.85\beta_1 \frac{f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + 0.004} = 0.85(.8)\frac{5}{60}\frac{.003}{.007} = 0.024$

Try $(0.25)(\rho_{max}) = 0.006$

$R = 345 = \frac{M_u}{\phi b d^2} \Rightarrow b = 1'$ (examining 1' sections)

$d^2 = \frac{2.15 \times 12 \times 1000}{0.9(345)(12)} \Rightarrow d = 2.63$ Try $h = 4"$, $d = 3"$

Assume $a = 1.0"$

$-M$: $A_s = \frac{M_u}{\phi f_y (d - a/2)} = \frac{2.15 \times 12}{0.9 \times 60 \times (3 - .5)} = 0.192$

check a

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.192(60)}{0.85(5)(12)} = 0.228$

$A_s = \frac{2.15 \times 12}{0.9 \times 60 (3 - \frac{.23}{2})} = \underline{0.166}$ in^2 $\rho = 0.0046$ $R = 261$

$a = .228\left(\frac{.166}{.192}\right) = 0.197$ use for other moment arm

$\phi M_n = 0.9(.0046)(12)(3^2)(1 - 0.59(.0046)(60/5))(60)$

$= 2.16$ 'k $> M_u = 2.15$ 'k

ONE WAY SLAB SPOT CHECK

Slab Spot check Cont'd — Hart

+M: $A_s = \dfrac{1.69 \times 12}{0.9(60)(3 - \frac{.12}{2})} = 0.13 \text{ in}^2$ $\rho = 0.0036$

$\phi M_n = 0.9(60)(.0036)(12)(3^2)(1 - 0.59(.0036)(\frac{60}{5}))$

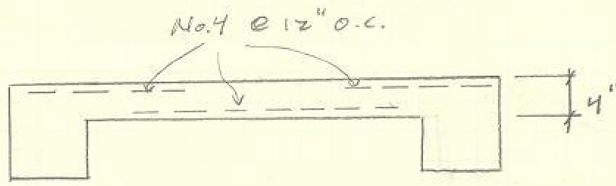
$= 1.7'k > M_u = 1.6'k$ ok

Neg Reinf use

No. 4 bars @ 12" o.c. ($A_s = 0.2 \text{ in}^2$)

Pos Reinf use

No. 4 bars @ 12" o.c. ($A_s = 0.2 \text{ in}^2$)



Designed slab

h = 9"

Bottom Bars

#5 @ 12"

Top Bars

#5 @ 12"

COLUMN SPOT CHECK

Column Spot Check

Design Column G-3 at Base

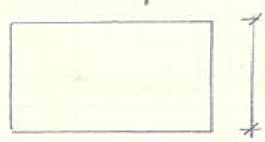
Assumption: shear walls will carry 100% of lateral forces, so column needs only be designed for axial loads.

For calculation of column loads, please see appendix

Calculated $P_u = 1670^k$

Compare with Working Load P_u from drawings: $P_u = 1225^k$

Note: Though I am confident in the thoroughness of the required load for this column which I calculated, I will design the column for 1225^k in compression.



h = 22" (given)

$f'_c = 6000$ psi

$f_y = 60{,}000$ psi

$P_u = 1225^k$, $M_u = 0'k$

$e = M_u/P_u = 0$

set target reinf ratio $\rho_g = 0.03$

$\gamma = \frac{h-2d'}{h} = \frac{22-2(2.5)}{22} = 0.77$

For $\gamma = 0.7$, use Graph A.10 *

$k_n = 1.02 \Rightarrow$ Required $A_g = \frac{P_u}{\phi f'_c k_n} = \frac{1225}{0.65(6)(1.02)} = 308 \text{ in}^2$

$b = \frac{A_g}{h} = \frac{308}{22} = 14"$

$k_n = \frac{P_u}{\phi f'_c A_g} = \frac{1225}{0.65(6)(308)} = 1.02$

From graphs A.10* and A.11* $\rho = 0.03$

$A_{s\,TOTAL} = 22(14)(.03) = 9.24$

Use 8 #10 ($A_s = 10.16 \text{ in}^2$) $\rho = 0.033$

$\phi P_n = \phi k_n f'_c A_g = (1.04)(6)(308)(0.65) = 1250 \text{ k} > 1225^k$

See conclusion for comments

Using Same equations and $P = 1670$

$A_g = 420$

$b = \frac{A_g}{h} = \frac{420}{22} = 19"$, use 20"

*

Appendix

Computation of Building Weight

Floor	Slab (psf)	Partition Load (psf)*	Superimposed Dead (psf)	Area (ft^2)**	Sub-Total Resultant (k)	Column Weight (k)	Shear Wall Weight (k)	Beam Weight (k)	Veneer Weight (k)	Total (k)
1	62.5	15	17.5	27825	2643	354	300	268	196	3761
2	113	15	30	17042	2693	354	400	268	196	3911
3	113	15	20	28232	4178	354	400	268	196	5396
4	113	15	5	26191	3483	354	400	268	234	4739
5	113	15	32.5	26191	4204	203	400	268	215	5290
6	113	15	30	16919	2673	203	400	268	215	3759
7	113	15	30	18201	2876	203	400	268	188	3935
8	113	15	30	18201	2876	203	400	268	188	3935
9	113	15	30	18201	2876	203	400	268	188	3935
10	113	15	30	18201	2876	203	400	268	188	3935
11	113	15	30	18201	2876	203	400	268	188	3935
12	113	15	30	18201	2876	203	400	268	242	3989
Roof	150	0	30	18201	3276	0	0	268	0	3544
									Penthouse	93
									***BAC Cooling Tower (on roof)	8.7
									***Dectron Energy Recovery Unit (on roof)	5.6
									Total Building Weight	54171

* No design partition load was listed, so assumed 15 psf conservatively In accordance with ASCE 7-05 12.7.2

** Floor area expanded to cover roof of garage at level 5

*** Values taken from manufacturer's websites

Note: Verification of values used in this table can be found in Dead Load calculations in Appendix

Calculation of Axial Force on Column G3

Floor	Trib Area (ft^2)	Dead Load (psf)	Superimposed Dead Load (psf)	Total Dead Load (psf)	Type of Live Load	Partial Live Load (psf)	Total Live Load (psf)	Live Load Reduction Factor	Reduced Live Load (psf)	Factored Load (psf)	Total Factored Load (kips)
1	568	113	30	143	Public Space	100	100	0.56	56.47	261.95	148.79
2	0	0	0	0	Atrium (empty)	0	0	0.00	0.00	0.00	0.00
3	568	113	35	148	Parking	50	50	0.00	50.00	257.60	146.32
4	568	113	5	118	Parking	50	50	0.00	50.00	221.60	125.87
5	568	113	30	143	Dwelling (60%) Corridor (40%)	40 100	64	0.56	36.14	229.42	130.31
6	568	113	30	143	Dwelling (60%) Corridor (40%)	40 100	64	0.56	36.14	229.42	130.31
7	568	113	30	143	Dwelling (87%) Corridor (13%)	40 100	47.8	0.56	26.99	214.79	122.00
8	568	113	30	143	Dwelling (87%) Corridor (13%)	40 100	47.8	0.56	26.99	214.79	122.00
9	568	113	30	143	Dwelling (87%) Corridor (13%)	40 100	47.8	0.56	26.99	214.79	122.00
10	568	113	30	143	Dwelling (87%) Corridor (13%)	40 100	47.8	0.56	26.99	214.79	122.00
11	568	113	30	143	Dwelling (87%) Corridor (13%)	40 100	47.8	0.56	26.99	214.79	122.00
12	568	113	30	143	Dwelling (87%) Corridor (13%)	40 100	47.8	0.56	26.99	214.79	122.00
Roof	568	113	30	143	Roof	100	100	0.63	63.20	272.72	154.90
										Self Weight Estimate	100
										Total Load, P_u	1668.50

$$\text{Typical Live Load Reduction Factor} = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) > 0.4L$$

Roof Live Load Reduction Factor = $L_oR_1R_2$

$R_1 = 1.2 - 0.001A_t$

$R_2 = 1$

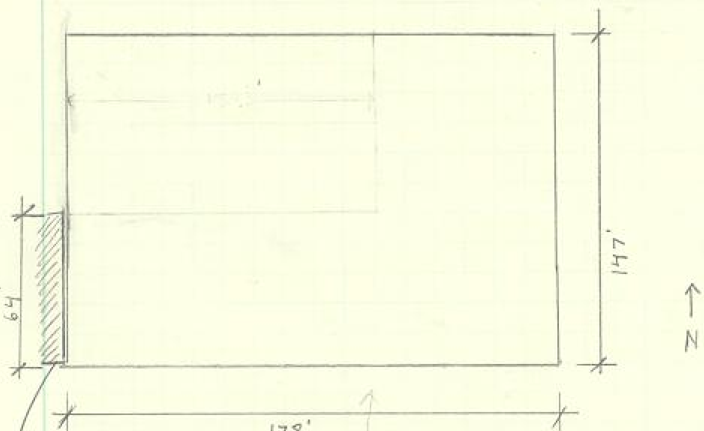
$K_{LL} = 4$ (Table 4-2 ASCE-7)

Factored Load = 1.2D + 1.6L

Appendix

Wind Analysis

For simplification, normalize bldg shape



Expansion joint connecting new tower to existing tower.

Exp B: $V = 90$ mph (3 sec gust) @ 33'
Fig 6.1
(structural notes Denote $V = 90$ mph)

For wind analysis, period will be taken as Ta (see seismic calculations)

$T_a = 1.202$ sec

$f = 1/T_a = 0.83$ hz $\Rightarrow$ flexible building, n_1

Gust Factor

From 6.5.8.2

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

$g_Q = g_v = 3.4$

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}} = \sqrt{2\ln(3600 \cdot 0.38)} + \frac{0.577}{\sqrt{2\ln(3600\ 0.83)}} = 4.145$$

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

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47(4.78)}{(1 + 10.3(4.78))^{5/3}} = 0.0522$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}}$$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right)$$

$(\bar{\alpha} = 1/4,\ \bar{b} = 0.45)$

$$= 0.45\left(\frac{72.84}{33}\right)^{0.25}(90)\left(\frac{88}{60}\right) = 72.4$$

$n_1 = 0.83$

$$N_1 = \frac{0.83(416.6)}{72.4} = 4.78$$

set $\beta = 0.05$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.3\left(\frac{33}{72.84}\right)^{1/6} = 0.26$$

$z_{min} = 30'$

$\bar{z} = 121.4'(.6) = 72.84 > 30'$ ok

$c = 0.3$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{.63}}}$$

$$L_{\bar{z}} = l\left(\frac{\bar{z}}{33}\right)^{\epsilon} = 320\left(\frac{72.84}{33}\right)^{1/3} = 416.6$$

$$Q_L = \sqrt{\frac{1}{1 + 0.63\left(\frac{178 + 121.4}{416.6}\right)^{.63}}} = 0.813$$

$B_L = 178'$ $B_S = 147'$

$h = 121.4'$

$$Q_S = \sqrt{\frac{1}{1 + .63\left(\frac{147 + 121.4}{416.6}\right)^{0.63}}} = 0.823$$

APPENDIX

Gust Factor Cont'd

R_h: $\eta = 4.6 n_1 h / \bar{V}_z = 4.6(0.83)(121.4)/72.4$

$= 6.4$

$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta})$

$= \frac{1}{6.4} - \frac{1}{2(6.4^2)}(1 - e^{-2(6.4)}) = 0.144$

R_{B_L}: $\eta = 4.6 n_1 \cdot B / \bar{V}_z = 4.6(0.83)(178)/72.4$

$= 9.39$

$R_{B_L} = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta})$

$= \frac{1}{9.39} - \frac{1}{2(9.39)^2}(1 - e^{-2(9.39)}) = 0.101$

R_{B_S}: $\eta = 4.6 n_1 \cdot B / \bar{V}_z = 4.6(0.83)(147)/72.4 = 7.75$

$R_{B_S} = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta})$

$= \frac{1}{7.75} - \frac{1}{2(7.75)^2}(1 - e^{-2(7.75)}) = 0.12$

R_{L_L}: $\eta = 15.4 n_1 L / \bar{V}_z = 15.4(.83)(147)/72.4 = 25.95$

$R_{L_L} = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta})$

$= \frac{1}{25.95} - \frac{1}{2(25.95^2)}(1 - e^{-2(25.95)}) =$

$= 0.0378$

R_{L_S}: $\eta = 15.4 n_1 L / \bar{V}_z = 15.4(.83)(178)/72.4 = 31.4$

$R_{L_S} = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta})$

$= \frac{1}{31.4} - \frac{1}{2(31.4^2)}(1 - e^{-2(31.4^2)})$

$= 0.0313$

APPENDIX

Gust Factor Cont'd

Long Direction

$R = \sqrt{\frac{1}{0.05}(0.0522)(0.144)(0.101)(.53 + .47(0.378))}$

$= 0.091$

$G_F = 0.925\left(\frac{1 + 1.7(0.26)\sqrt{3.4^2(0.813)^2 + 4.145^2(0.091)^2}}{1 + 1.7(3.4)(0.26)}\right)$

$\boxed{= 0.825}$

Short Direction

$R = \sqrt{\frac{1}{0.05}(0.0522)(0.144)(0.12)(0.53 + 0.47(0.0313))}$

$= 0.099$

$G_F = 0.925\left(\frac{1 + 1.7(0.26)\sqrt{3.4^2(.823)^2 + (4.145)^2(.099)^2}}{1 + 1.7(3.4)(0.26)}\right)$

$\boxed{= 0.8322}$

E-W

Windward

$p = qG_fC_p - q_i(GC_{pi})$

(6.5.12.2.3)

$C_p = 0.8$

(use w/ q_z)

$GC_{pi} = \pm 0.18$

Leeward

$\frac{1.2-1}{2-1} = \frac{C_p - (-.5)}{-.3-(-.5)}$

$C_p = -0.46$

(N-S, $C_p = -0.5$)

$q_i = q_{h=121.4'} = 0.00256$

$= 0.00256(1.04)(90)^2(1.15)(.85)$

$\boxed{= 21.1}$

	K_h, K_z
@ $z = 72.84$	0.91
@ $h = 121.4$	1.04

$(q_i)(GC_{pi}) = \pm 3.8$

Leeward (N-S)

$p = 21.1(0.83)(-0.50) \pm 3.8$

Leeward (E-W)

$p = 21.1(.83)(-.46) \pm 3.8$

APPENDIX

Computation of Dead Load for Bldg WT

A) Floor 1

5" slab ⇒ 150 pcf $\left(\frac{5}{12}\right)$ = 62.5 psf
Superimposed (typical) = 30 psf (.5)
Superimposed (typical parking) = 5 psf (.5)
} 80 psf × 27825 ft^2 = 2226^k

Floor 2, 6–12

9" slab ⇒ 150 pcf $\left(\frac{9}{12}\right)$ = 113 psf
Superimposed (typical) = 30 psf
} 143 psf < × 17042 ft^2 = 2437^k (level 2); × 18201 ft^2 = 2603^k (level 6-12)

Roof slab: Varying thickness, use 12" across entire area (conservative) 150 $\left(\frac{12}{12}\right)$ = 150 psf

Floor 3

9" slab ⇒ = 113 psf
Superimposed (parking above occupied space) = 35 psf (0.5) = 17.5 psf
Superimposed (typical parking) = 5 psf (0.5) = 2.5 psf
} 133 psf × 28232 = 3755^k

Floor 4

9" slab ⇒ = 113 psf
Superimposed (typical parking) = 5 psf
} 118 psf × 26191 = 3091^k

Floor 5

9" slab ⇒ = 113 psf
Superimposed (garage roof) = 35 (0.5) = 17.5 psf
Superimposed (typical) = 30 (0.5) = 15 psf
} 145.5 psf × 26191 = 3811^k

Note: Some floors are both parking and residential, thus superimposed dead loads have adjustment factors

B) Column Weight

Full Height (121.4')

Avg Cross Section = $\frac{5(12\times24) + 17(22\times22) + 10(22\times36) + 1(12\times78) + 1(22\times60)}{34}$

= 584 in^2 = 4.06 ft^2

Approx WT = 145 pcf (121.4 × 4.06) = 71.5^k × 34 col = 2431^k

Partial Height (39.33') (Only applied to floors 1–4)

Avg Cross Section = [9(24×36) + 13(24×24)]/22
= 693 in^2 = 4.82 ft^2

Approx WT = 145 pcf (39.33 × 4.82) = 27.5^k × 22 col = 605^k

⇒ WT/Floor:

Floors 1–4:
$\frac{605}{4} + \frac{2431}{12}$ = 354^k/floor

Floors 5–12:
$\frac{2431}{12}$ = 203^k/floor

APPENDIX

Computation of Dead Load for Bldg WT Cont'd

c) Shear Wall WT

Shear Wall 1:

$28'(1' \times 121.4') \times 145\,pcf = 493^k$

Shear Wall 2:

$21'(1' \times 121.4') \times 145\,pcf = 370^k$

Shear Wall 3:

$23'(1.17' \times 121.4') \times 145\,pcf = 474^k$

Shear Wall 4:

Same as 3 (approx)

Avg Shear Wall WT: $400^k(12) = 5400^k$

↳ combine 5A + 5B
and 9A + 9B
and 9C + 9D

$5400^k / 12$ floors $= 400^k$/floor

D) Beams

Rough estimate per floor

$\left(\frac{22 \times 23}{144}\right)(550') \times 145\,pcf = 268^k$

E) Veneer

Use 40 psf. Since I am not considering WT of concrete balconies, I will take the WT of the brick over 75% of entire wall area. (Conservative)

Levels 1–4

Perimeter ≈ 700'

Level 1-3: $(9.33')(700')(40\,psf) = 261^k \times .75 = 196^k$

Level 4: $(11.33')(700')(40\,psf) = 317^k \times .75 = 234^k$

Levels 5–12

Perimeter ≈ 670'

Level 5,6: $(10.67')(670')(40\,psf) = 286^k \times .75 = 215^k$

Level 7-11: $(9.33')(670')(40\,psf) = 250^k \times .75 = 188^k$

Level 12: $(12')(670')(40\,psf) = 322^k \times .75 = 242$

F) Penthouse

15 psf steel framing + metal deck

50 psf conc

$(18.5' \times 77.5') \times (50 + 15) = 93^k$

APPENDIX

Snow Load Calculation

FLAT Roof (ASCE 7-05)

$p_f = 0.7 C_e C_t I p_g$

$= 0.7(1.1)(25)$

$= 19.25$ psf

$C_e = 1.0$ (Partially exposed Roof) Table 7-2

$I = 1.1$ Table 7-4

$C_t = 1.0$ Table 7-3

$p_g = 25$ Figure 7-1

Same as Design value