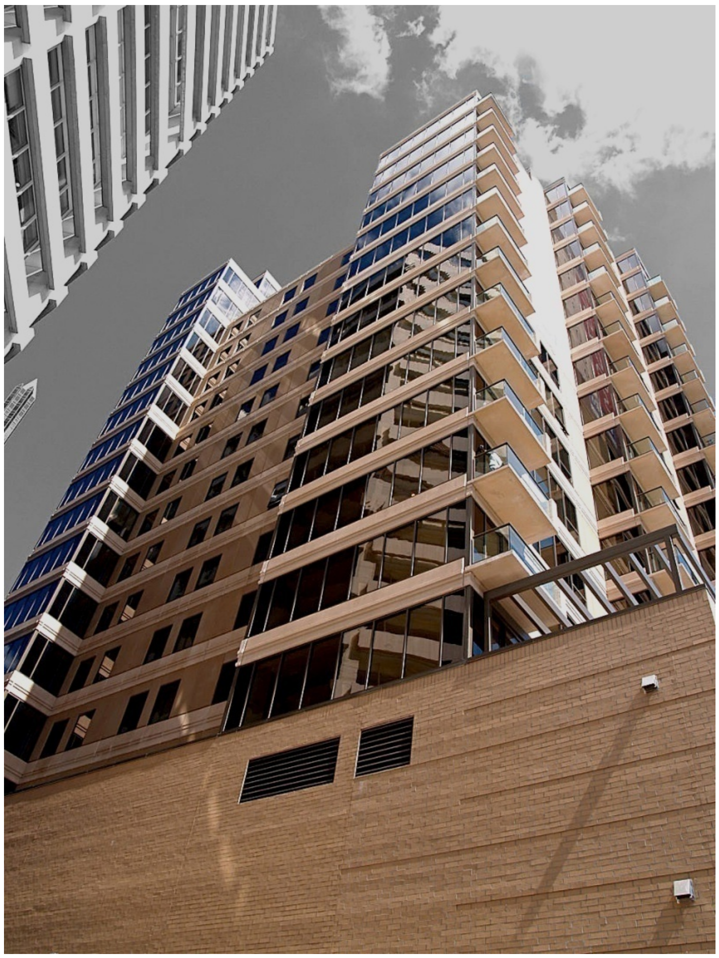
151 First Side

Technical Assignment 1

October 5th, 2007



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AE 481w – Senior Thesis

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Executive Summary

This report has been prepared as a form of documenting and understanding the as-built conditions of 151 First Side.

Building Summary:
151 First Side is an 18 condominium project located on 151 Fort Pitt Boulevard in Pittsburgh, PA. The first 3 floors consist of a parking garage with an entrance in the core of the building. The remaining floors accommodate one to four separate condominiums. The building's structure consists of a steel frame with both concrete and composite joist floor systems. Lateral stability is achieved using a combination of braced frames as well as moment connections within the condominium units.

Conclusions:
151 First Side was designed using IBC 2003 which references ASCE-7 02. For my analysis I have used IBC 2006 with ASCE-7 05. The difference in codes may account for some of the discrepancies in the values obtained. Other differences were done intentionally. An example of this is the thickening of the slab within the Hambro system by an extra ½" to help stop vibration. Also, this report takes a simplified approach at analysis and therefore may not make all of the same assumptions that the designer had made. The discrepancies do not indicate any error by the engineer.

Structural System

Foundation:

The foundation was designed based on soil reports prepared by Engineering Mechanics, Inc. and Ackenheil Engineering, Inc., dated April, 2002 and July 1, 2005 respectively. The piles are pressure injected auger cast piles, 18" in diameter. Pile tips were placed at an elevation of 674'-0". Each pile has a capacity of 120 tons. Pile caps are made of concrete with a 28 day strength of f'_c = 3000psi.

Slab on Grade:

The sub-basement and basement floors consist of slab on grade at elevations 725'-0" and 728'-0" respectively. The slabs are 5" of concrete with a 28 day strength of f'_c = 4000psi and are reinforced with 6x6 w2.1 x w2.1 welded wire fabric. Concrete was placed above 4" of AASHTO 57 well graded compacted granular stone.

Floor System:

The parking levels on the first three stories as well as the terrace level have poured concrete floors. All floors are 4" of light weight concrete atop a 2" 20ga. galvanized composite metal deck with the exception of some highly loaded areas of the ground floor in which there is a 6" slab. The 4" sections on the parking levels are reinforced with #4 rebar spaced at 12" in both the bottom and the top of the slab with the top bars continuing for ¼ of the span length past the supports. The 6" sections contain 6x6-W2.9xW2.9 welded wire fabric. The terrace level has 6x6-W1.4xW1.4 welded wire fabric for its reinforcement.

The residential and mechanical levels, as well as the roof, contain an MD200 composite floor joist system provided by Hambro. The concrete slab is 3¼" thick and is made with concrete with a 28 day strength of f'_c=4000psi. Reinforcing within the concrete is a 6x6-W2.9xW2.9 welded wire mesh. The concrete is supported by 22ga. 1½" galvanized steel deck. The joist depth is 16" unless otherwise noted. The top chord is an "S' shape piece of cold-rolled, ASTM A 1008, Grade 50, 13ga. steel which works as both a compressive member as well as a shear connector. The bottom chord is made of two steel angles. Both chords have a minimum F_y=50,000psi. The web is formed from 7/16" hot-rolled steel bars with an F_y=44,000psi.

Structural Frame:

The structural framing is made of steel I shapes. The beams range from W10 to W16 with the most common size being a W14x61. The columns are W12 shapes with weights ranging from 40 to 336 pounds per linear foot. Common column splices occur at every second floor.

Lateral System:

The lateral system is composed of both braced frames as well as special moment frames. On column grid lines 2, 3, 4, E, and F there is some braced frames in the parking levels. Above level 5 every frame is braced, or if bracing is not architecturally feasible a special moment frame is used. Diagonal braces are made from W12 shapes.

Codes

Building Code:

International Building Code (IBC), 2003 edition

Structural Concrete:

Building Code Requirements for Reinforced Concrete (ACI 318, latest edition)

Specifications for Structural Concrete (ACI 301, latest edition)

Steel Design:

Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings (AISC, 9th Edition)

Code of Standard Practice for Steel Buildings and Bridges (with exception of Section 4.2)

Building Design Loads:

ANSI/ASCE-7 2002

Design Loads

General Loads:

Floor Live Loads

Load Area	**Design Load**	**Minimum Load (ASCE 7-05)**
Common Areas	100 psf	100 psf
Corridors	100 psf	100 psf
Parking	40 psf	40 psf
Residential	40 psf	40 psf
Mechanical	150 psf	n/a

Roof Live Loads

Item	**Design Value**	**Code Reference**
Roof	20 psf	ASCE 7-05
Ground Snow Load (Pg)	30 psf	IBC Fiure 1608.2
Flat-roof Snow Load (Pf)	21 psf	IBC Section 1608.3
Snow Expsore Factor (Ce)	1	IBC Table 1608.3.1
Snow Importance Factor (I)	1	IBC Table 1604.5
Thermal Factor (Cf)	1	IBC Table 1608.3.2

Dead Loads

Item	**Design Value**
Concrete Slab	70 psf
Superimposed Dead Loads	
Mechanical , Electrical, Sprinkler	20 psf
Ceiling Finishes	5 psf
Floor Finishes	5 psf
Steel Structure	Varies
Other Dead Loads	Where Applicable

Wind Loads:

The wind pressures and resulting base shear and overturning moment were calculated based on an exposure category B. The following spreadsheets give a detailed view of the pressure applied to each height level, and the corresponding floors. See Figure 4 in the Appendix for more calculations and diagrams regarding wind.

Pressure				
Wind from the North/South				
Windward		Leeward		Total
h (ft)	P (psf)	h (ft)	P (psf)	
0-15	6.72	0-15	-9.43	16.15
20	7.31	20	-9.43	16.74
25	7.78	25	-9.43	17.21
30	8.25	30	-9.43	17.68
40	8.96	40	-9.43	18.39
50	9.55	50	-9.43	18.98
60	10.02	60	-9.43	19.45
70	10.49	70	-9.43	19.92
80	10.96	80	-9.43	20.39
90	11.32	90	-9.43	20.75
100	11.67	100	-9.43	21.10
120	12.26	120	-9.43	21.69
140	12.85	140	-9.43	22.28
160	13.32	160	-9.43	22.75
180	13.79	180	-9.43	23.22
200	14.15	200	-9.43	23.58
250	15.09	250	-9.43	24.52

Pressure				
Wind from the East/West				
Windward		Leeward		Total
h (ft)	P (psf)	h (ft)	P (psf)	
0-15	6.68	0-15	-9.26	15.94
20	7.26	20	-9.26	16.53
25	7.73	25	-9.26	16.99
30	8.20	30	-9.26	17.46
40	8.91	40	-9.26	18.17
50	9.49	50	-9.26	18.75
60	9.96	60	-9.26	19.22
70	10.43	70	-9.26	19.69
80	10.90	80	-9.26	20.16
90	11.25	90	-9.26	20.51
100	11.60	100	-9.26	20.86
120	12.19	120	-9.26	21.45
140	12.77	140	-9.26	22.03
160	13.24	160	-9.26	22.50
180	13.71	180	-9.26	22.97
200	14.06	200	-9.26	23.32
250	15.00	250	-9.26	24.26

Wind from the North/South							
Floor	Height (Ft.)	Story Height (Ft.)	Trib. Area (Sf.)	P-total (psf)	Story Force (Kip)	Total Shear (Kip)	Overturning Moment (Ft.-Kip)
1 (ground)	0	0	0	16.15	0.00	**473.61**	**556969.93**
2	13.33	13.33	1242.50	16.15	20.07	473.61	6314.85
3	23.33	10.00	1215.88	17.21	20.93	453.55	10582.79
4	192.83	12.83	1251.38	18.39	23.01	432.62	83424.05
5	180.00	10.67	1136.00	18.98	21.56	409.61	73729.99
6	169.33	10.67	1136.00	19.45	22.10	388.05	65710.08
7	158.67	10.67	1136.00	19.92	22.63	365.96	58065.11
8	148.00	10.67	1136.00	20.39	23.17	343.33	50812.23
9	137.33	10.67	1136.00	20.75	23.57	320.16	43968.57
10	126.67	10.67	1136.00	21.69	24.64	296.59	37568.25
11	116.00	10.67	1171.50	21.69	25.41	271.95	31546.44
12	105.33	11.33	1171.50	22.28	26.10	246.54	25969.16
14	94.00	10.67	1136.00	22.28	25.31	220.44	20721.62
15	83.33	10.67	1136.00	22.75	25.84	195.13	16261.16
16	72.67	10.67	1153.75	22.75	26.25	169.29	12301.69
17	62.00	11.00	1171.50	23.22	27.20	143.04	8868.53
18	51.00	11.00	1171.50	23.22	27.20	115.84	5907.65
Penthouse	40.00	11.00	1544.25	23.58	36.41	88.63	3545.26
Mech. Level	29.00	18.00	1544.25	24.52	37.86	52.22	1514.52
Roof	11.00	11.00	585.75	24.52	14.36	14.36	157.98

North/South Direction:

Base Shear: 473.61 Kip

Overturning Moment: 556969.93 Ft.-Kip

Wind from the East/West							
Floor	Height (Ft.)	Story Height (Ft.)	Trib. Area (Sf.)	P-total (psf)	Story Force (Kip)	Total Shear (Kip)	Overturning Moment (Ft.-Kip)
1 (ground)	0	0	0	15.94	0.00	**468.27**	**550854.54**
2	13.33	13.33	1242.50	15.94	19.81	468.27	6243.61
3	23.33	10.00	1215.88	16.99	20.66	448.47	10464.19
4	192.83	12.83	1251.38	18.17	22.73	427.80	82494.47
5	180.00	10.67	1136.00	18.75	21.30	405.07	72912.39
6	169.33	10.67	1136.00	19.22	21.84	383.77	64984.40
7	158.67	10.67	1136.00	19.69	22.37	361.93	57426.38
8	148.00	10.67	1136.00	20.16	22.90	339.56	50255.38
9	137.33	10.67	1136.00	20.51	23.30	316.66	43488.44
10	126.67	10.67	1136.00	21.45	24.36	293.36	37159.44
11	116.00	10.67	1171.50	21.45	25.13	269.00	31203.98
12	105.33	11.33	1171.50	22.03	25.81	243.87	25688.08
14	94.00	10.67	1136.00	22.03	25.03	218.06	20497.85
15	83.33	10.67	1136.00	22.50	25.56	193.03	16086.03
16	72.67	10.67	1153.75	22.50	25.96	167.47	12169.50
17	62.00	11.00	1171.50	22.97	26.91	141.51	8773.53
18	51.00	11.00	1171.50	22.97	26.91	114.60	5844.52
Penthouse	40.00	11.00	1544.25	23.32	36.02	87.69	3507.53
Mech. Level	29.00	18.00	1544.25	24.26	37.46	51.67	1498.52
Roof	11.00	11.00	585.75	24.26	14.21	14.21	156.31

East/West Direction:

Base Shear: 468.27 Kip

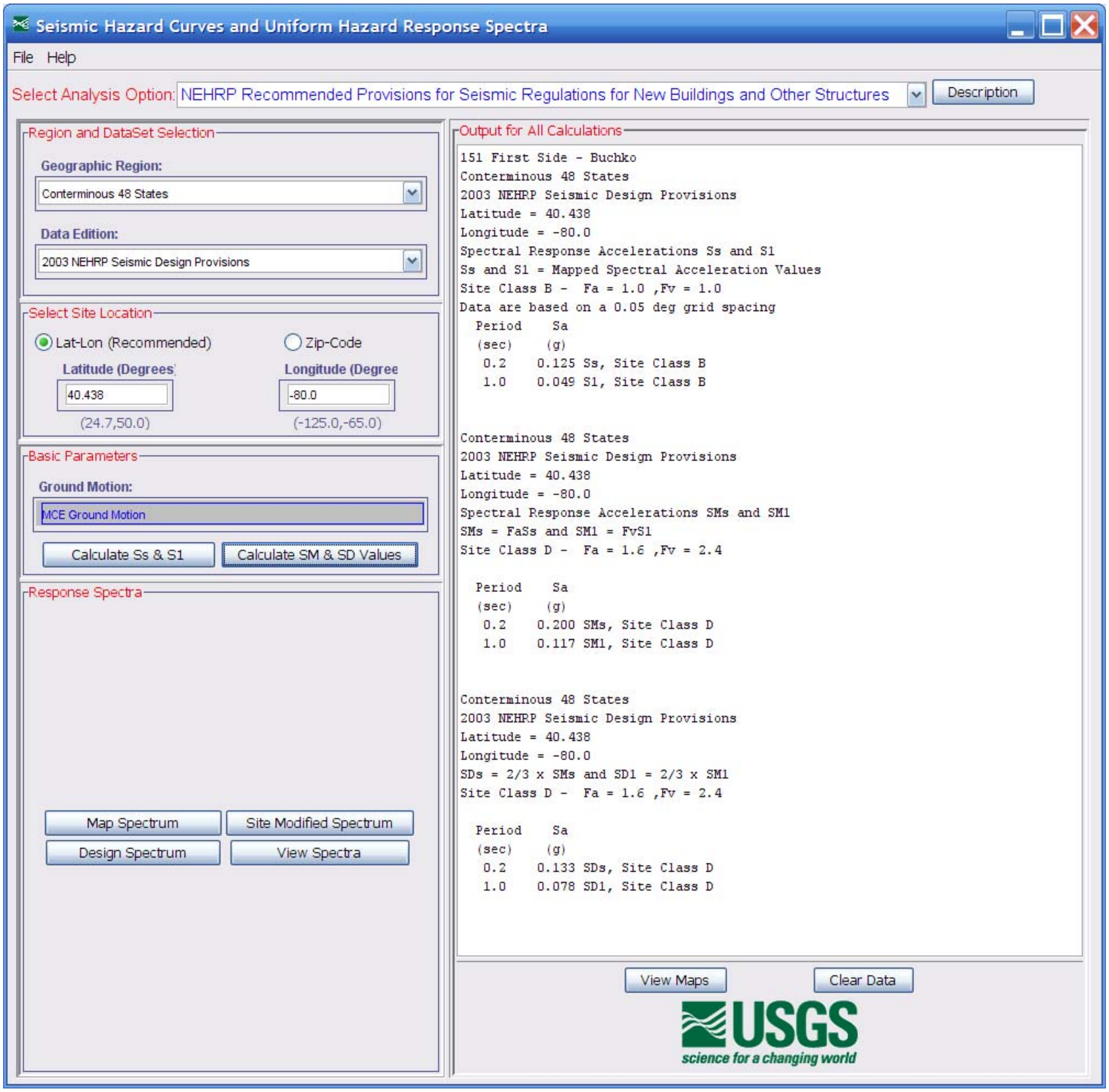
Overturning Moment: 550854.54 Ft.-Kip

Seismic Loads:

Even though Pittsburgh is not known for its seismic activity, a simplified check has been done to ensure that wind loading is indeed the controlling case. The building has been analyzed as a seismic design category B with braced framing as its main seismic force resisting system. I have used software from the USGS website as an aid in calculating the required data. I have also preformed a vertical distribution of the seismic load. A sketch of the resultant loads can be found in Figure 5 within the Appendix.

When I checked my value for the design base shear with that of the designer I noticed that mine was almost 1% off. When I investigated this further I found that the designer and I had started with different values for spectral response acceleration (S_1 and S_s). This can be accounted for based on the method of obtaining these values. I determined these values based on the output of the USGS software after inputting the longitude and latitude. It seems that the designer had used the then-current generic values for south eastern Pennsylvania. This discrepancy does not affect the overall design as both values are still less than the wind loads.

The following pages include a print out of the USGS website displaying the values that I have used for my analysis in addition to a spreadsheet showing the vertical distribution of the seismic load and final base shear.


Seismic Hazard Curves and Uniform Hazard Response Spectra
File Help
Select Analysis Option: NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures
Description
Region and DataSet Selection
Geographic Region:
Conterminous 48 States
Data Edition:
2003 NEHRP Seismic Design Provisions
Select Site Location
Lat-Lon (Recommended)
Zip-Code
Latitude (Degrees
40.438
(24.7,50.0)
Longitude (Degree
-80.0
(-125.0,-65.0)
Basic Parameters
Ground Motion:
MCE Ground Motion
Calculate Ss & S1
Calculate SM & SD Values
Response Spectra
Map Spectrum
Site Modified Spectrum
Design Spectrum
View Spectra
Output for All Calculations
151 First Side - Buchko
Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Latitude = 40.438
Longitude = -80.0
Spectral Response Accelerations Ss and S1
Ss and S1 = Mapped Spectral Acceleration Values
Site Class B - Fa = 1.0 ,Fv = 1.0
Data are based on a 0.05 deg grid spacing
Period Sa
(sec) (g)
0.2 0.125 Ss, Site Class B
1.0 0.049 S1, Site Class B
Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Latitude = 40.438
Longitude = -80.0
Spectral Response Accelerations SMs and SM1
SMs = FaSs and SM1 = FvS1
Site Class D - Fa = 1.6 ,Fv = 2.4
Period Sa
(sec) (g)
0.2 0.200 SMs, Site Class D
1.0 0.117 SM1, Site Class D
Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Latitude = 40.438
Longitude = -80.0
SDs = 2/3 x SMs and SD1 = 2/3 x SM1
Site Class D - Fa = 1.6 ,Fv = 2.4
Period Sa
(sec) (g)
0.2 0.133 SDs, Site Class D
1.0 0.078 SD1, Site Class D
View Maps
Clear Data
USGS
science for a changing world

Vertical Distribution of Seismic Load					
		K=1.67	Vb=304.7		
Level	wx (Kip)	hx (Ft.)	wxhx^1.67	Cvx	Fx (Kip)
Roof	1304.04	216.17	10336846.93	0.1342	40.88
Mech. Level	1304.04	205.17	9473474.13	0.1230	37.47
Penthouse	1304.04	187.17	8126668.00	0.1055	32.14
18	1304.04	176.17	7344860.53	0.0953	29.05
17	1304.04	165.17	6595099.13	0.0856	26.08
16	1304.04	154.17	5878073.59	0.0763	23.25
15	1304.04	143.50	5214751.14	0.0677	20.62
14	1304.04	132.83	4583674.00	0.0595	18.13
12	1304.04	122.17	3985675.73	0.0517	15.76
11	1358.64	110.83	3529424.99	0.0458	13.96
10	1358.64	100.17	2980658.20	0.0387	11.79
9	1358.64	89.50	2469726.52	0.0321	9.77
8	1358.64	78.83	1998066.39	0.0259	7.90
7	1358.64	68.17	1567363.51	0.0203	6.20
6	1358.64	57.50	1179640.56	0.0153	4.67
5	1358.64	46.83	837396.93	0.0109	3.31
4	1358.64	36.17	543850.54	0.0071	2.15
3	1473.20	23.33	283650.10	0.0037	1.12
2	1473.20	13.33	111406.21	0.0014	0.44
1 (ground)	1473.20	0.00	0.00	0.0000	0.00
Totals	27025.08			1.00	304.70

Seismic Loading:

Base Shear: 304.7 Kip

Spot Checks

Floor System:

I checked the Hambro composite joist system as well as the beams for the typical interior bay (Figure 3). Using the Hambro design guide I found that a smaller joist as well as a smaller slab could have been used. I spoke with a representative of Hambro who mentioned that the slab was thickened by ½" for serviceability reasons; namely vibration. There are many reasons why I obtained a smaller joist than what was used. In addition to any miscellaneous loads I may not have accounted for, it may have been more economical and feasible to use the larger joists. It may have been more cost effective in both material and labor to order all 16" joists as opposed to smaller joists for the bay I checked and larger joists for other bays. Also, the larger joist makes it easier to frame into the beams as well as leave a clearance for mechanical systems. More detailed information can be found in Figure 6 in the Appendix.

Column:

I chose a typical interior column on the 7^{th} floor for my spot check (Figure 3). Using live load reduction where applicable, I was able to use a W12x170 as opposed to the W12x210 which was used. These members are fairly close and I attribute the difference to my simplified gravity load analysis where as they have used a computer model to find the worst case scenario with lateral and gravity loading. Figure 7 in the Appendix contains more calculations.

Lateral System:

The scope of this report does not include the complex computer modeling needed to analyze and check the dual-system lateral bracing. However, an analysis and model will be included within a future report to show the relative stiffness of both the braced frames and moment connections.

Appendix

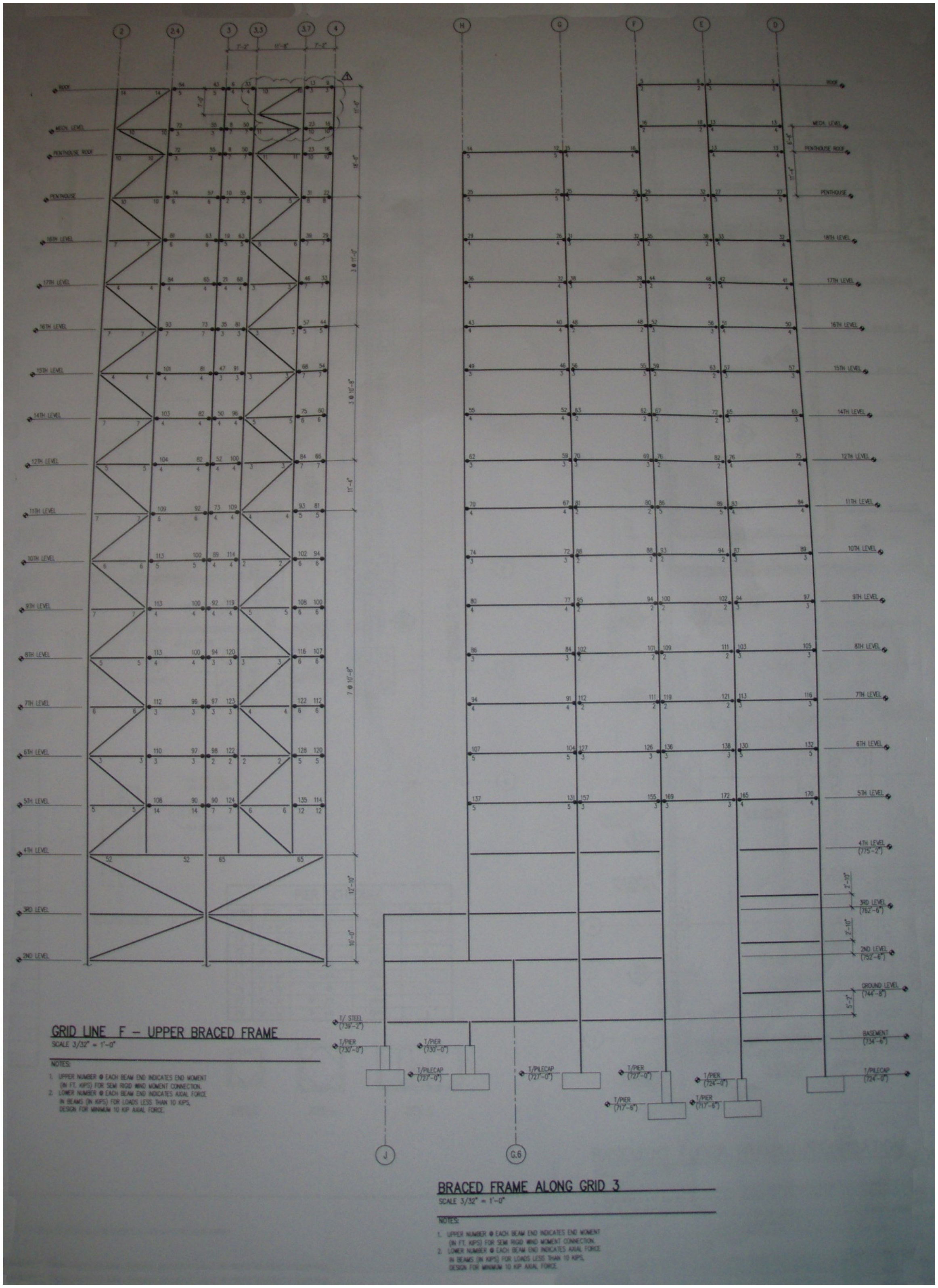


Figure 1

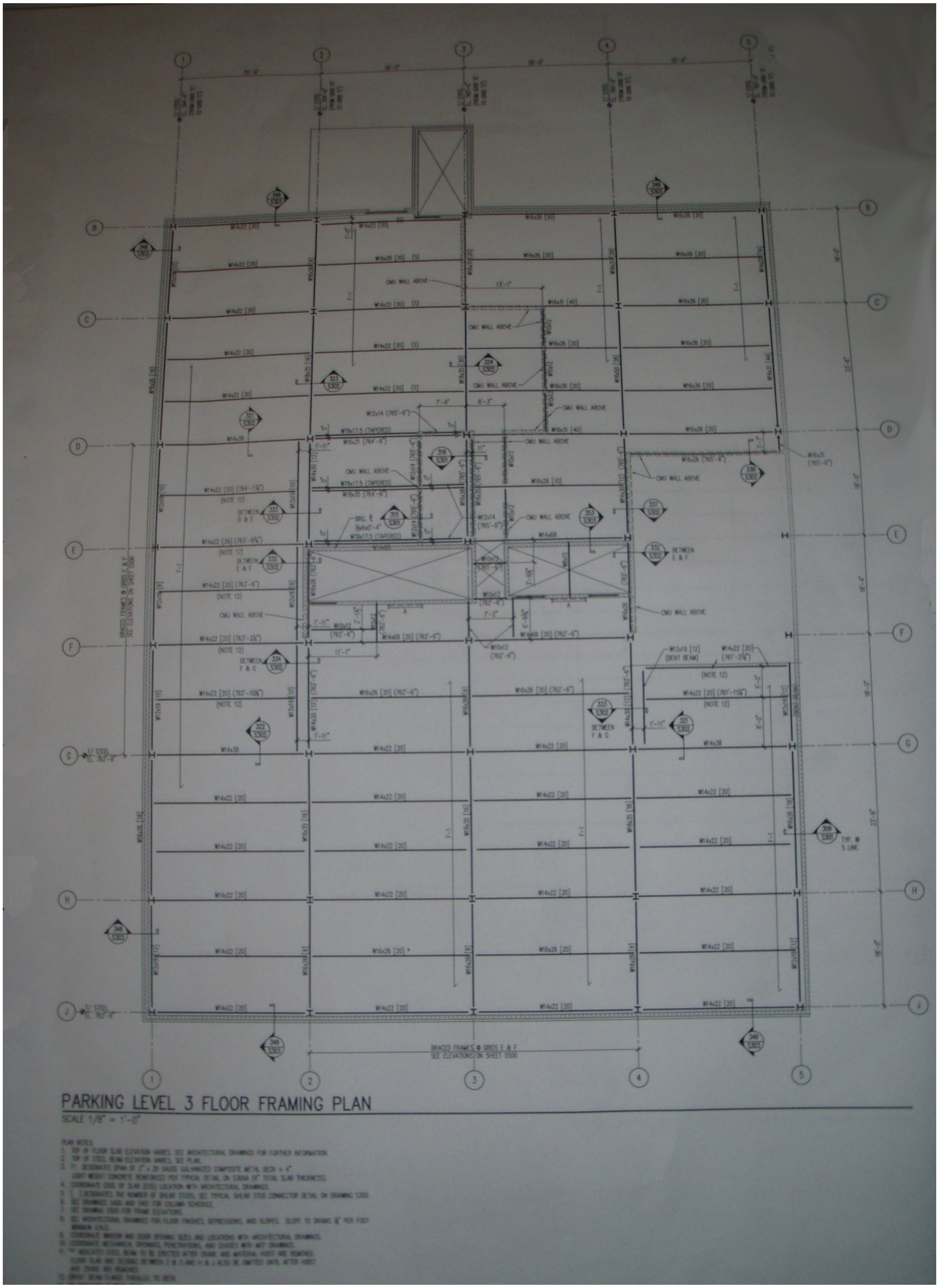


Figure 2

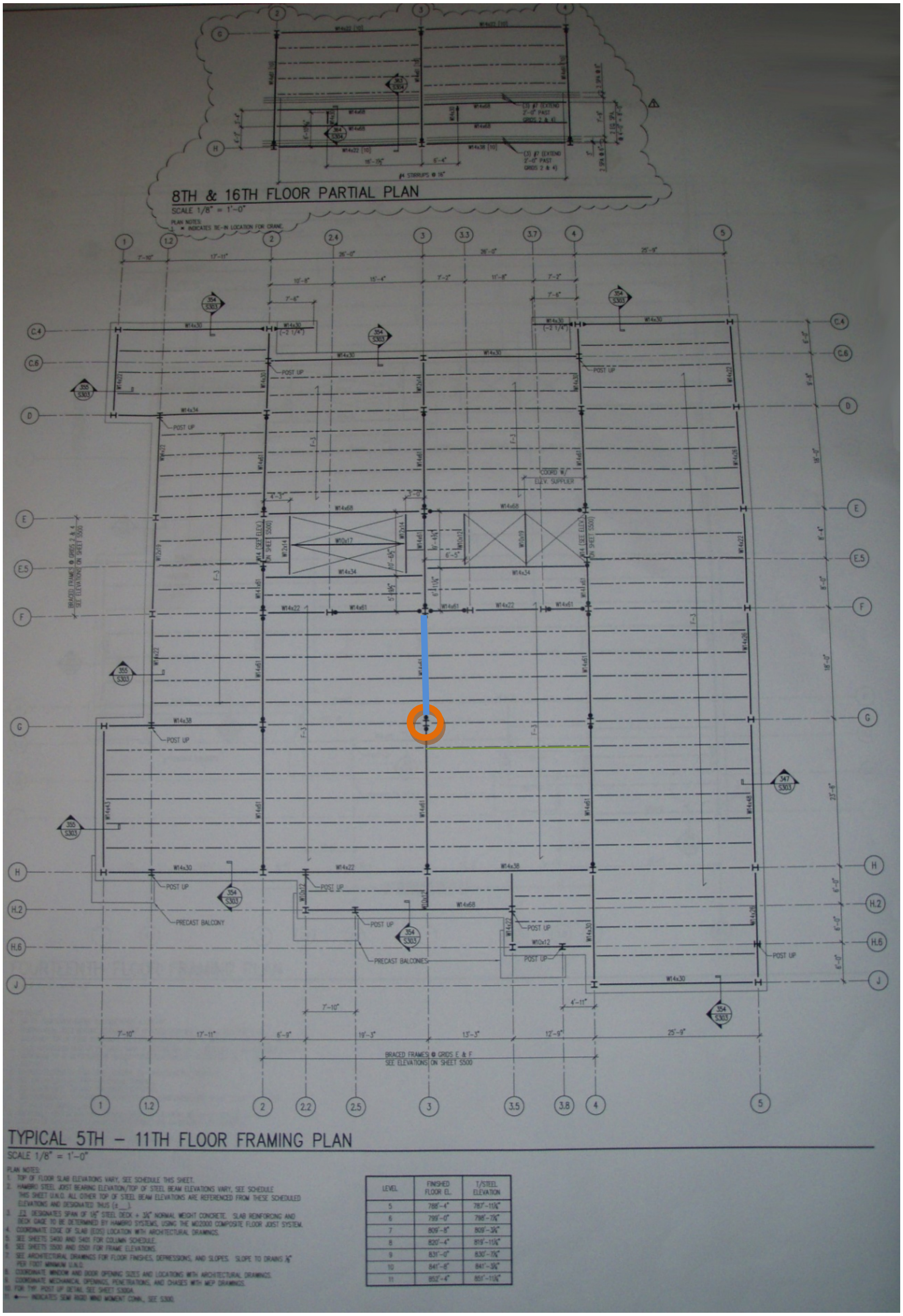
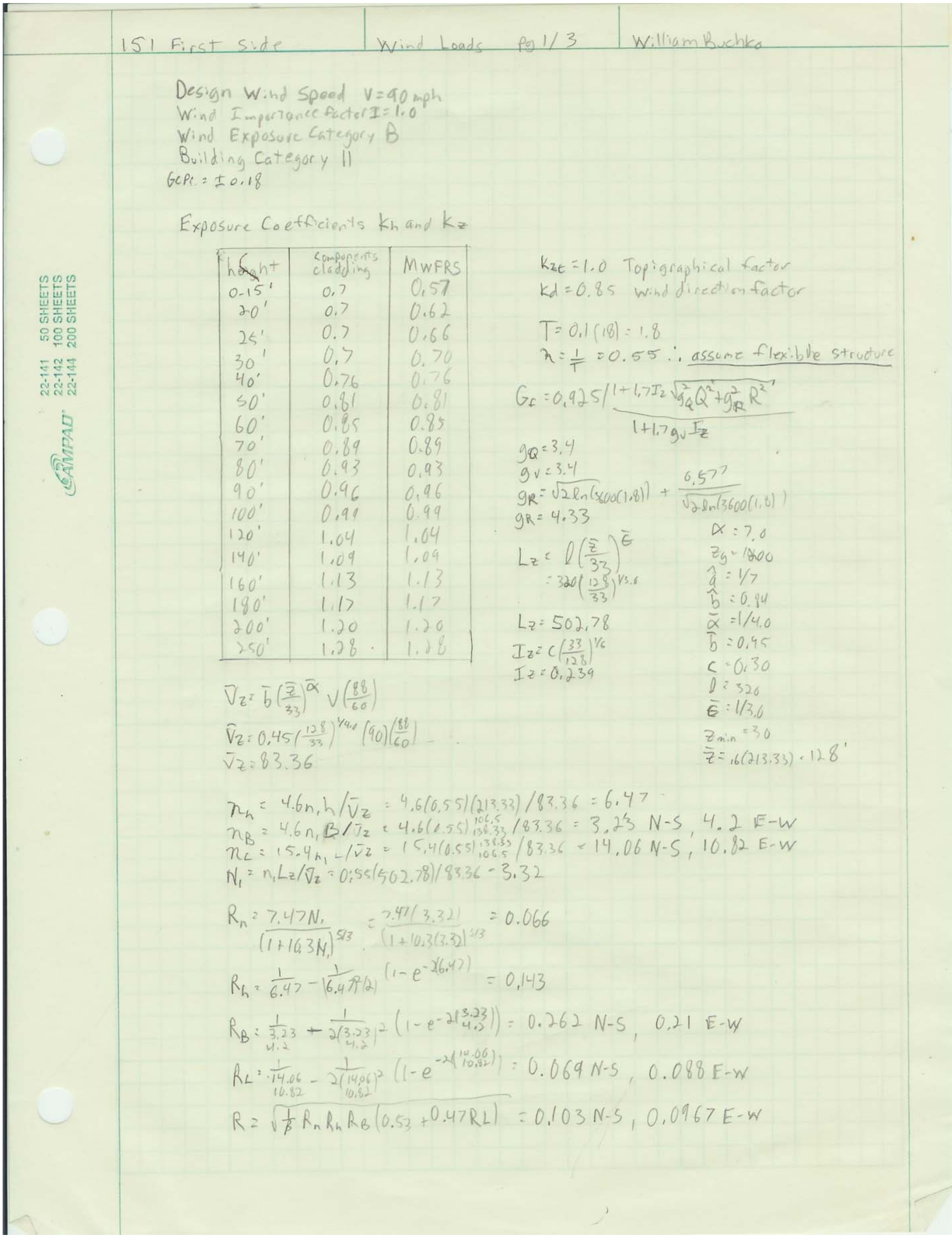

8TH & 16TH FLOOR PARTIAL PLAN
SCALE 1/8" = 1'-0"
TYPICAL 5TH – 11TH FLOOR FRAMING PLAN
SCALE 1/8" = 1'-0"

Figure 3



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Design Wind Speed $V = 90$ mph
Wind Importance Factor $I = 1.0$
Wind Exposure Category B
Building Category II
$GCP_i = \pm 0.18$

Exposure Coefficients K_h and K_z

height	components cladding	MWFRS
0-15'	0.7	0.57
20'	0.7	0.62
25'	0.7	0.66
30'	0.7	0.70
40'	0.76	0.76
50'	0.81	0.81
60'	0.85	0.85
70'	0.89	0.89
80'	0.93	0.93
90'	0.96	0.96
100'	0.99	0.99
120'	1.04	1.04
140'	1.09	1.09
160'	1.13	1.13
180'	1.17	1.17
200'	1.20	1.20
250'	1.28	1.28

$K_{zt} = 1.0$ Topographical factor
$K_d = 0.85$ Wind direction factor

$T = 0.1(18) = 1.8$
$n = \frac{1}{T} = 0.55$ ∴ assume flexible structure

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

$g_Q = 3.4$
$g_v = 3.4$
$g_R = \sqrt{2\ln(3600(1.8))} + \frac{0.577}{\sqrt{2\ln(3600(1.8))}}$
$g_R = 4.33$

$L_z = l\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{128}{33}\right)^{1/3.0}$
$L_z = 502.78$
$I_z = c\left(\frac{33}{128}\right)^{1/6}$
$I_z = 0.239$

$\alpha = 7.0$
$z_g = 1200$
$\hat{a} = 1/7$
$\hat{b} = 0.84$
$\bar{\alpha} = 1/4.0$
$\bar{b} = 0.45$
$c = 0.30$
$l = 320$
$\bar{\epsilon} = 1/3.0$
$z_{min} = 30$
$\bar{z} = .6(213.33) = 128'$

$\bar{V}_z = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right)$
$\bar{V}_z = 0.45\left(\frac{128}{33}\right)^{1/4.0}(90)\left(\frac{88}{60}\right)$
$\bar{V}_z = 83.36$

$n_h = 4.6 n_1 h / \bar{V}_z = 4.6(0.55)(213.33)/83.36 = 6.47$
$n_B = 4.6 n_1 B / \bar{V}_z = 4.6(0.55)\frac{106.5}{138.33}/83.36 = 3.23$ N-S, 4.2 E-W
$n_L = 15.4 n_1 L / \bar{V}_z = 15.4(0.55)\frac{138.33}{106.5}/83.36 = 14.06$ N-S, 10.82 E-W
$N_1 = n_1 L_z / \bar{V}_z = 0.55(502.78)/83.36 = 3.32$

$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47(3.32)}{(1 + 10.3(3.32))^{5/3}} = 0.066$

$R_h = \frac{1}{6.47} - \frac{1}{(6.47)^2(2)}\left(1 - e^{-2(6.47)}\right) = 0.143$

$R_B = \frac{1}{\frac{3.23}{4.2}} - \frac{1}{2\left(\frac{3.23}{4.2}\right)^2}\left(1 - e^{-2\left(\frac{3.23}{4.2}\right)}\right) = 0.262$ N-S, 0.21 E-W

$R_L = \frac{1}{\frac{14.06}{10.82}} - \frac{1}{2\left(\frac{14.06}{10.82}\right)^2}\left(1 - e^{-2\left(\frac{14.06}{10.82}\right)}\right) = 0.069$ N-S, 0.088 E-W

$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)} = 0.103$ N-S, 0.0967 E-W

Figure 4

151 First Side | Wind Loads Pg 2/3 | William Buchko

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_z}\right)^{0.63}}} = 0.824 \text{ N-S},\ 0.816 \text{ E-W}$$

$$G_f = 0.925\left(\frac{1+1.7(0.239)\sqrt{3.4^2(Q^2)+4.33^2(R^2)}}{1+1.7(3.4)(0.239)}\right) = 0.836 \text{ N-S}$$
$$= 0.831 \text{ E-W}$$

Building is Enclosed
$GC_{pi} = \pm 0.18$

C_p windward $= 0.8$, use with q_z
C_p leeward $= -0.5$ N-S use with q_h
-0.494 E-W from interpolation

$q_z = 0.00256\, k_z (1.0)(0.85)(90^2)(1.0) = 17.6256(k_z)$

See spreadsheet for results

$q_h = 0.00256(1.28)(1.0)(0.85)(90^2)(1.0) = 22.56$

$p = qGC_p$ ignore internal pressure

for N-S use q from spreadsheet, $G_f = 0.836$, $C_{pww} = 0.8$, $C_{pLw} = -0.5$
for E-W use q from spreadsheet, $G_f = 0.831$, $C_{pww} = 0.8$, $C_{pLw} = -0.494$

Figure 4 (Cont'd)

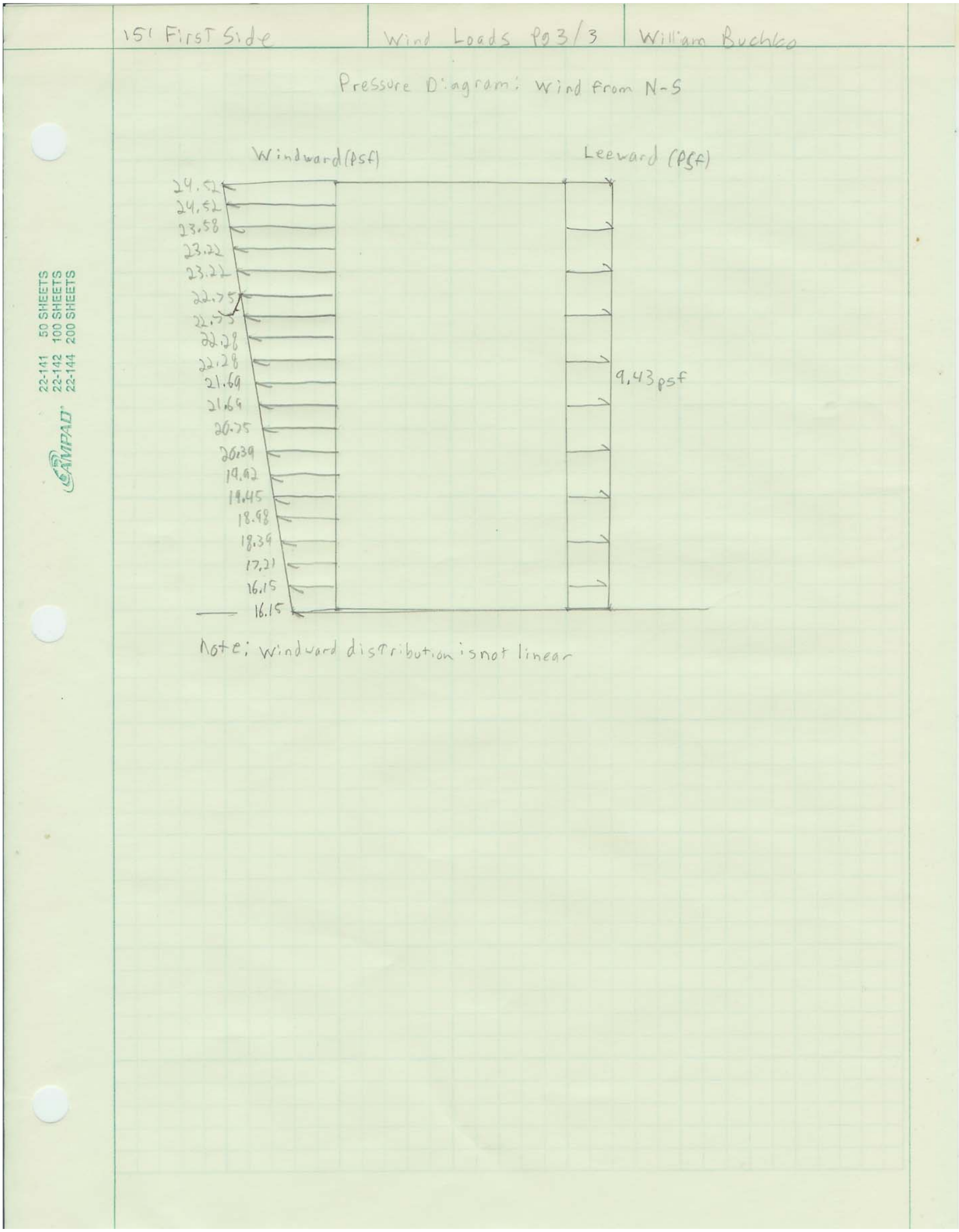
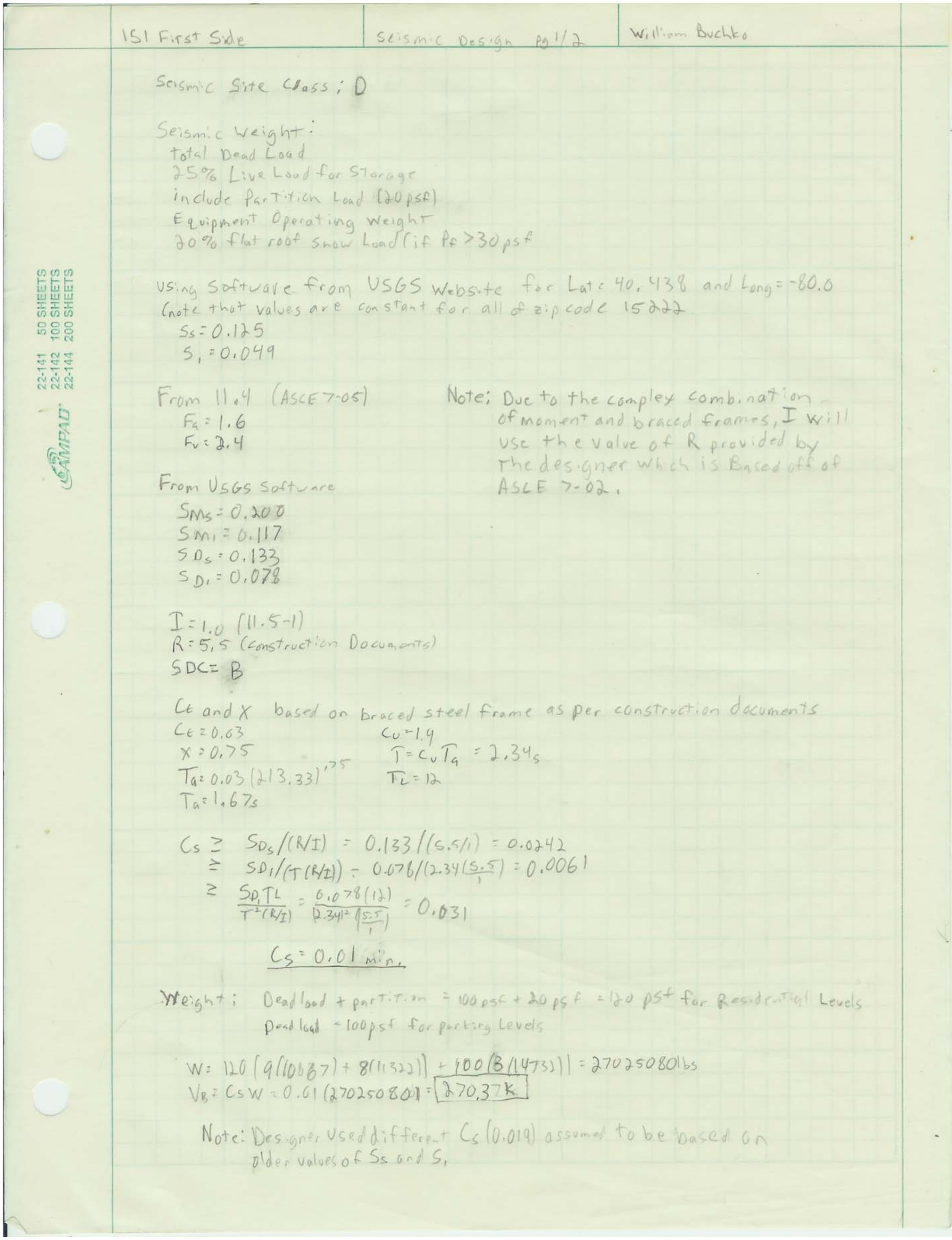


Figure 4 (Cont'd)



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Seismic Site Class ; D

Seismic Weight:
total Dead Load
25% Live Load for Storage
include Partition Load (20 psf)
Equipment Operating Weight
20% flat roof snow Load (if $P_f > 30$ psf

Using Software from USGS Website for Lat = 40.438 and Long = -80.0
(note that values are constant for all of zip code 15222

$S_s = 0.125$
$S_1 = 0.049$

From 11.4 (ASCE 7-05)
$F_a = 1.6$
$F_v = 2.4$

Note: Due to the complex combination of moment and braced frames, I will use the value of R provided by the designer which is Based off of ASCE 7-02.

From USGS Software
$S_{MS} = 0.200$
$S_{M1} = 0.117$
$S_{DS} = 0.133$
$S_{D1} = 0.078$

$I = 1.0$ (11.5-1)
$R = 5.5$ (Construction Documents)
SDC = B

C_t and x based on braced steel frame as per construction documents
$C_t = 0.03$
$x = 0.75$
$T_a = 0.03(213.33)^{.75}$
$T_a = 1.67s$

$C_u = 1.4$
$T = C_u T_a = 2.34s$
$T_L = 12$

$C_s \geq S_{DS}/(R/I) = 0.133/(5.5/1) = 0.0242$
$\geq S_{D1}/(T(R/I)) = 0.078/(2.34(\frac{5.5}{1})) = 0.0061$
$\geq \frac{S_{D1}T_L}{T^2(R/I)} = \frac{0.078(12)}{(2.34)^2(\frac{5.5}{1})} = 0.031$

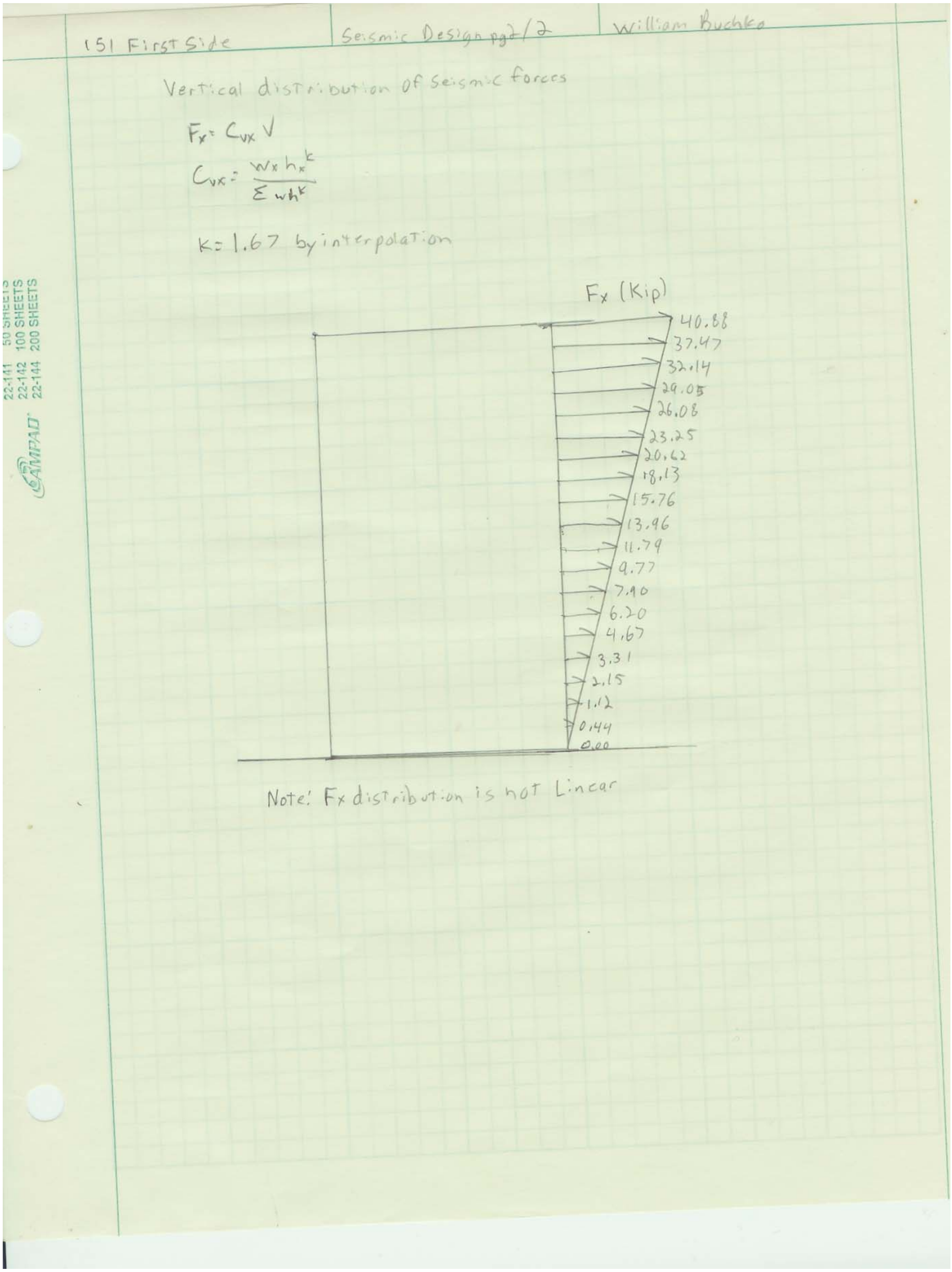
$C_s = 0.01$ min.

Weight: Dead load + partition = 100 psf + 20 psf = 120 psf for Residential Levels
Dead load = 100 psf for parking Levels

$W = 120[9(10687) + 8(11322)] + 100(3(14732)) = 27025080$ lbs
$V_B = C_s W = 0.01(27025080) = 270.37k$

Note: Designer used different C_s (0.019) assumed to be based on older values of S_s and S_1

Figure 5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Vertical distribution of seismic forces

$F_x = C_{vx} V$

$C_{vx} = \frac{w_x h_x^k}{\Sigma w h^k}$

k = 1.67 by interpolation

Note: Fx distribution is not Linear

Figure 5 (Cont'd)

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Typical internal floor joist

using Hambro Span chart:

Span: 26' LiveLoad = 40psf Superimposed Dead Load = 20psf

MD2000 with 12" Joist and 2 3/4" slab would work.

Compare to system actually used:

MD2000 with 16" Joist and 3 1/4" slab.

Typical Internal Beam check

Span = 18' Trib width = 26' LiveLoad = 40psf
Deadload = 100psf

Serviceability Loading:

Liveload = 40psf(26') = 1.04 KLf Total = 140(26') = 3.64 KLf

$$\Delta_L = \frac{18'(12")}{360} = \frac{5(1.04)(18)^4(1728)}{384(29,000)I} \quad I = 142 \text{ in}^4$$

$$\Delta_T = \frac{18'(12")}{240} = \frac{5(3.64)(18)^4(1728)}{384(29000)I} \quad I = 329.4 \text{ in}^4$$

Try W14x34

$$M = \frac{wl^2}{8} = \frac{3.64(18)^2}{8} = 147.42\text{ k-ft}$$

needs W14x38

Check Shear

$$\frac{3.64(18)}{2} = 32.76k < 87.4k \quad ok$$

choose W14x34

They chose W14x61

Figure 6

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Check 7th floor Column G-3

$K_{LL} = 4$

$A_{T_7} = 539.5\,ft^2$ DL = 100 psf LL: 40 psf

$A_{T\,6\text{-Pent}} = 10(539.5) = 5395\,ft^2$ DL = 100 psf LL = 40 psf

$A_{T\,roof} = 539.5\,ft^2$ $D_L = 100$ psf LL = 20 psf

$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) = 40\left(0.25 + \frac{15}{\sqrt{23238}}\right) = 40(.35)$ use $40(0.4) = 16$ psf

axial load = $116(539.5)(7\text{ floors}) + 116.5(539.5) + 117.5(539.5) + 119(539.5)$
$+ 123(539.5) + 120(539.5)$

$P_u = 759616$ lbs $= 759.6$ k $(1.67) = 1269$ k

from Table 4-1 in Steel manual
a W12x170 would work.

Compare to chosen W12x210

Figure 7