Science Center Research Park
3711 Market St.
Philadelphia, PA

The Pennsylvania State University Department of Architectural Engineering
Senior Thesis 2009-2010

Prepared by:
Zachary Yarnall

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[TECHNICAL REPORT 1]

Structural Concepts and Existing Conditions Report

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EXECUTIVE SUMMARY

Technical report 1is a structural concepts and existing conditions report. The purpose of this report is to describe and analyze the structural system of Science Center Research Park located in Philadelphia, PA. This report uses current standards and the governing standards to explain the design and loading.

The Science Center Research Park is a 401,032 GSF mixed-use building and is approximately 144 feet tall. It currently has the largest green roof in the city of Philadelphia. The building includes offices, wet labs, retail space, and a 500 car parking garage. The structure is made up of steel construction, and composite deck. Lateral support is provided by steel braced frames.

First, gravity loads were calculated using ASCE 7-05 and compared to the loads determined by Keast & Hood, Structural Engineers. Following ASCE 7-05 standards, the wind loads were calculated using method 2, and the seismic loads were calculated using the equivalent lateral force procedure.

Spot checks were performed using columns and beams that are typical in the building to verify which code or design approach was originally used.

At the end of this report is an appendix that contains all the calculations for the loads stated above.

INTRODUCTION

The structural system and existing conditions report contains a description of physical conditions that influence the structure of the Science Center Research Park. The design loads are calculated using current standards and codes, and then compared to the original design loads used. Drawings, specifications, and soils reports were used to provide information on the existing conditions.

CODE

CODE / REFERENCES

- ASCE 7-05 *Minimum Design Loads for Buildings and Other Structures*
- IBC 2006 *International Building Code*
- ACI 318-08 *Building Code Requirements for Structural Concrete*
- AISC 13th Edition Steel Construction Manual

STRUCTURAL SYSTEM

FOUNDATION

The foundation system is composed of cast-in-place reinforce concrete grade beams and piers. The deep foundation consists of drilled caissons that range from 3 to 5 feet in diameter, and 20 to 30 feet below elevation. These caissons can carry loads up to 1900 kips depending on the size. The general thickness of the slab on grade is either 4 or 6inches depending on indication on plans, but is also 12 inches thick in some areas. The columns are also cast-in-place in some areas of the ground floor, but transfer to steel columns. All the concrete in the building, except the caissons and steel column encasements, has a compressive strength of 4000 psi. The caissons and steel column encasements have a compressive strength of 3000 psi.

FLOOR SYSTEM

The floor system is a composite system with a typical bay size of 31'6" x 31'6". The typical composite deck is composed of 6inches of normal weight concrete and 1.5" – 18 gauge composite steel decking with ¾" studs. The floor is supported typically by W 18 x 40 beams and W 24 x 84 girders, but there are large amount of other W - shapes used. The roof consists of 1.5" – 18 gauge steel roof deck supported typically by W 16 x 26 beams and W 24 x 55 girders.

LATERAL SYSTEM

The lateral system is composed of braced frames strategically place on each floor. The braced frames can be located in the walls of the main elevator and stairwell core in the center of the building, in some exterior walls, and in the exterior walls of the penthouse. The braces are hollow structural steel members. Typical brace members are HSS 8 x 8's and HSS 6 x 6's were used, but several different sizes were used. The shear at the end of the beams is typically 10 kips, unless indicated otherwise on the plans. Also, column splices transmit compression forces in end bearing with a minimum of 15 kips of shear. Two bays of the braced frames in the center core connect into the buildings foundation transfer the shear load.

MATERIAL

CONCRETE

Slabs on grade	f'c = 4000 psi
Slab on steel deck	f'c = 4000 psi
Drilled caissons	f'c = 3000 psi
Foundation walls, piers &grade beams	f'c = 4000 psi
Steel column encasement	f'c = 3000 psi

STRUCTURAL STEEL

W – shapes	ASTM A992
Bars, rods and plates	ASTM A36 (UNO)
All other structural shapes	ASTM A36
Pipes	ASTM A53, Grade B
Cold-formed hollow structural sections (tubing)	ASTM A500, Grade B
High strength bolts	ASTM A325
Deformed bar anchors	ASTM A706 Low Carbon
Anchor rods	ASTM A36
Shear connectors (headed)	ASTM A108, Grade 1010 to 1020

GRAVITY AND DESIGN LOADS

DEAD LOADS

Concrete	150 pcf
Light Weight Concrete	115 pcf
Partitions	20 psf
M.E.P.	5 psf
Finishes and Misc.	3 psf
Roof Deck	2.6 psf
Rigid Insulation	4 psf

LIVE LOADS

Corridors, Lobbies & Exits	100 psf
Labs / Offices	100 psf
Garage	40 psf
Mechanical Equip. Rooms	150 psf
Roof	30 psf

SNOW LOADS

Snow load were calculated to determine whether the roof design is sufficient to carry the applied snow load. Table 1 contains all the design values required to calculate the flat roof snow load. When the applied snow load was compared to the existing designed snow load it was noticeable that designers had used a larger value. All the design criteria is the same used by the designers, but the designers used a more conservative value for the snow load. Snow drift was not calculated, but it vary depending on the different roof levels of the building.

Flat Roof	
p_g	30 psf
c_e	1.0 (Terrain Category B)
c_t	1
I	1.0 (Category II)
p_f	21 psf
using $pf = 0.7c_ec_tIp_g$ (Eq. 7-1)	

TABLE 1 – Snow load design criteria

LATERAL LOADS

WIND LOADS

Wind loads were determined using ASCE 7-05 Section 6.5 which describes Method 2. The detailed analysis of the wind loads can be found in appendix B. Below are tables including wind factors and wind loads calculated for north-south and east-west elevations.

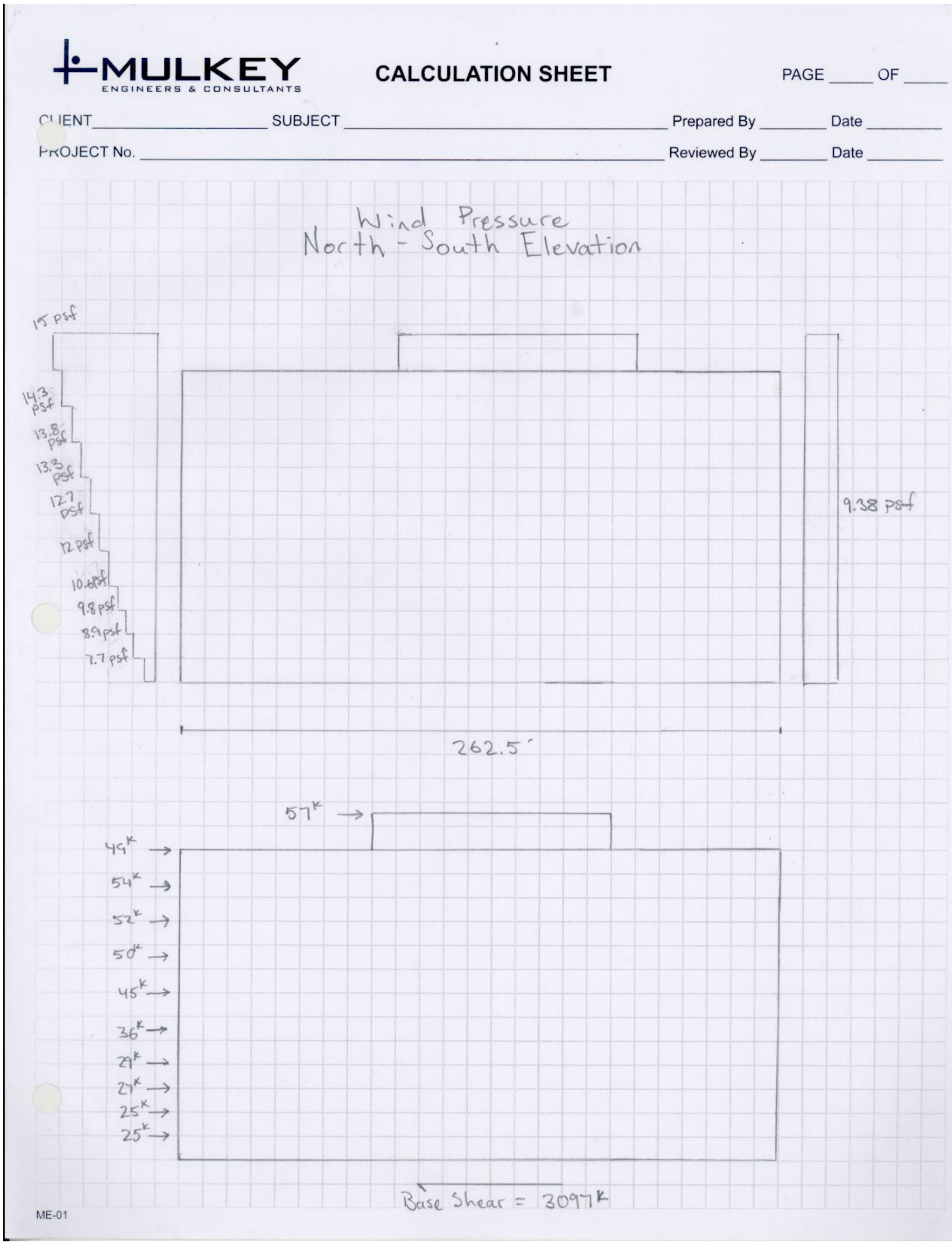
	Level	Height Above Ground (ft)	Kz	qz	Wind Pressures	
					N-S (psf)	E-W (psf)
windward	Pent House	147.5	1.10	21.41	15.00	15.19
	Roof Level	140.17	1.09	21.10	14.79	14.97
	T.O. Parapet	131.17	1.07	20.70	14.51	14.69
	10	125.5	1.05	20.44	14.33	14.51
	9	110.83	1.02	19.73	13.83	14.00
	8	96.17	0.98	18.95	13.28	13.45
	7	81.5	0.93	18.07	12.67	12.82
	6	66.83	0.88	17.08	11.97	12.12
	5	53.5	0.83	16.02	11.23	11.37
	4	43.5	0.78	15.10	10.59	10.72
	3	33.5	0.72	14.02	9.82	9.95
	2	23.5	0.65	12.67	8.88	8.99
	1	13.5	0.57	11.05	7.74	7.84
	Leeward	All	1.1	21.41	-9.38	-7.98

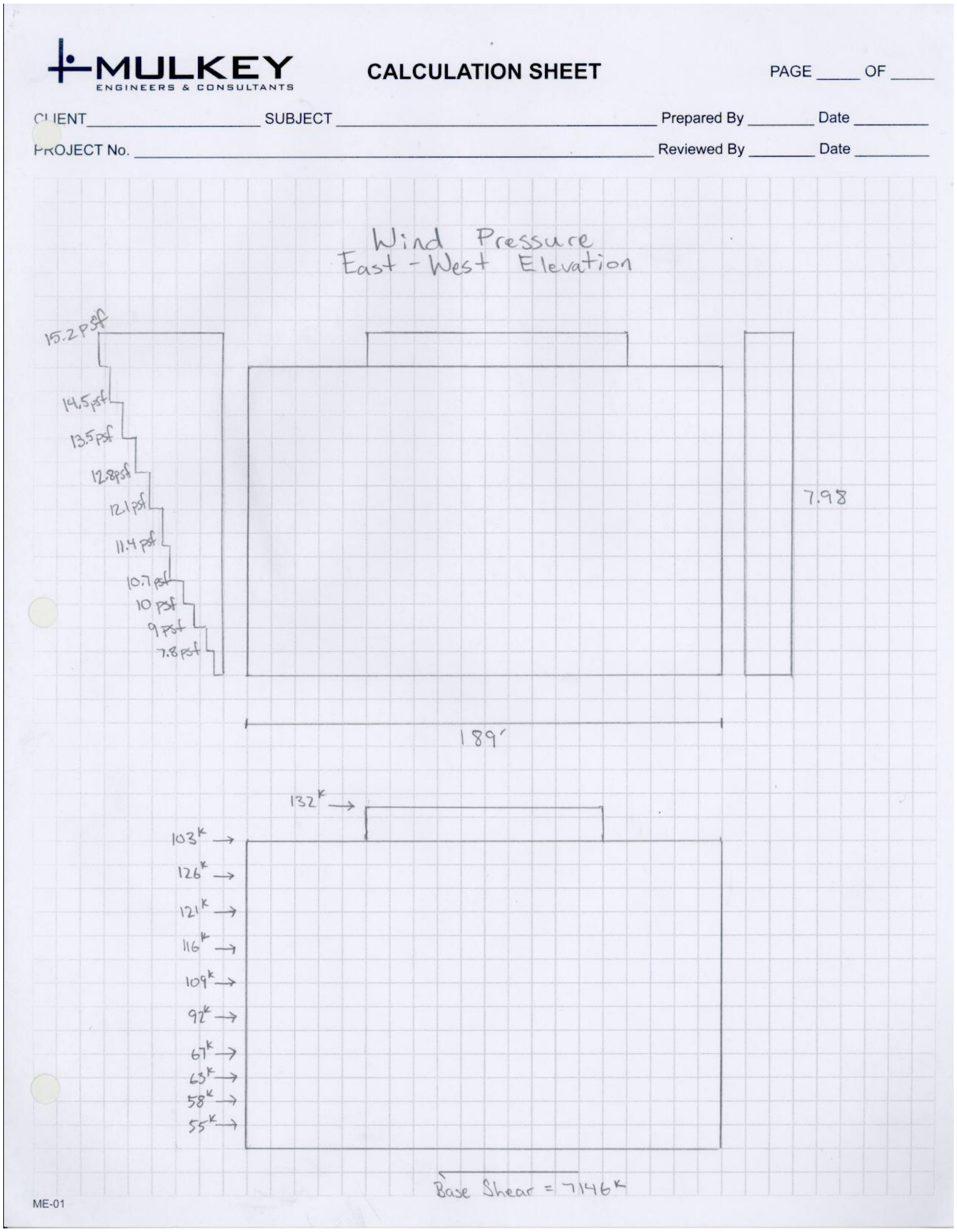
TABLE 2 – Wind Pressure at each level

Level	Height Above Ground (ft)	Floor Height (ft)	h/2 above	h/2 below	Wind Forces			
					Load (kips)		Shear (kips)	
					N-S	E-W	N-S	E-W
Pent House	147.5	0						
Roof Level	140.17	7.33	7.33	7.33	57	132	57	132
T.O. Parapet	131.17	9						
10	125.5	5.67	5.67	7.33	49	103	107	236
9	110.83	14.67	7.33	7.33	54	126	161	361
8	96.17	14.67	7.33	7.33	52	121	213	483
7	81.5	14.67	7.33	7.33	50	116	263	599
6	66.83	14.67	7.33	6.67	45	109	308	708
5	53.5	13.33	6.67	5	36	92	344	800
4	43.5	10	5	5	29	67	372	867
3	33.5	10	5	5	27	63	399	930
2	23.5	10	5	5	25	58	424	988
1	13.5	10	5	6.75	25	55	449	1043
Total	147.5						3097	7146

TABLE 3 – Wind Loads, and Shear at each level

The base shear of 7146 kips in the east-west direction controls over the north-south direction.


MULKEY
ENGINEERS & CONSULTANTS
CALCULATION SHEET
PAGE ___ OF ___
CLIENT ___ SUBJECT ___ Prepared By ___ Date ___
PROJECT No. ___ Reviewed By ___ Date ___
Wind Pressure
North - South Elevation
15 psf
14.3 psf
13.8 psf
13.3 psf
12.7 psf
12 psf
10.6 psf
9.8 psf
8.9 psf
7.7 psf
9.38 psf
262.5'
57k
49k
54k
52k
50k
45k
36k
29k
27k
25k
25k
Base Shear = 309k
ME-01


MULKEY
ENGINEERS & CONSULTANTS
CALCULATION SHEET
PAGE ____ OF ____
CLIENT ____ SUBJECT ____ Prepared By ____ Date ____
PROJECT No. ____ Reviewed By ____ Date ____
Wind Pressure
East - West Elevation
15.2 psf
14.5 psf
13.5 psf
12.8 psf
12.1 psf
11.4 psf
10.7 psf
10 psf
9 psf
7.8 psf
7.98
189'
132k
103k
126k
121k
116k
109k
92k
67k
63k
58k
55k
Base Shear = 7146k
ME-01

SEISMIC LOADS

Seismic loads were determined using ASCE 7-05 chapters 11 and 12. The detailed analysis of the seismic loads can be found in appendix C. Building weight was calculated for each floor of the typical steel constructed building. The building weight includes the dead loads that are listed in the tables below.

Floor	Floor Area (ft^2)	Floor Dead Load (t_{slab}*150 pcf)	Floor Weight (lbs)	h/2 above (ft)	h/2 below (ft)	Column weight/length total (plf)	Column weight= height* weight/length (lbs)
Ground						11445	
1st	33,833	93.75	3171843	5	6.75	11498	134743.75
2nd	50,705	93.75	4753593	5	5	11566	115320.00
3rd	50,705	93.75	4753593	5	5	7385	94755.00
4th	50,705	93.75	4753593	5	5	7385	73850.00
5th	40,433	93.75	3790593	6.67	5	7205	84958.33
6th	34,439	93.75	3228656	7.33	6.67	4797	83211.33
7th	34,439	93.75	3228656	7.33	7.33	4797	70356.00
8th	30,439	93.75	2853656	7.33	7.33	2960	56884.67
9th	30,439	93.75	2853656	7.33	7.33	2960	43413.33
Pent house	6,437	93.75	603468	7.33	7.33	728	27045.33
Roof pent house level	21,509	93.75	2016468		7.33	2960	21706.67
Roof	6,437	93.75	603468		7.33	728	5338.67
Total			36007781				806244

TABLE 4 – Building Weight Tabulation

Floor	Approx. Beam weight (lbs)	Curtainwall (estimated length along perimeter) (ft)	Curtainwall height (ft)	Curtainwall weight (height*length* 15 psf)	Braced frame weight (lbs)
Ground					
1st	257591.00	913.5	10	137025	11064
2nd	257591.00	913.5	10	137025	7772
3rd	257591.00	913.5	10	137025	7239
4th	257591.00	913.5	10	137025	8639
5th	240266.00	913.5	13.33	182654.325	8639
6th	178765.00	913.5	14.67	201015.675	7772
7th	141120	850.5	14.67	187152.525	7041
8th	141120	850.5	14.67	187152.525	6773
9th	141120	819	14.67	180220.95	6439
Pent house	30240.00	378	14.67	83178.9	13207
Roof pent house level	7180.00				
Roof	92547.00				
Total	1910175			1569474.9	84585
Total Building Weight		40378.26 kips			

TABLE 4 – Building Weight Tabulation (continued)

Level	Story Weight w_x (Kips)	Height h_x (ft)	$w_x h_x^k$	Lateral Force F_x (Kips)	Story Shear V_x (Kips)	Moment M_x (ft-k)
1st	3712.27	13.5	50115.61125	6.05	286.42	81.68
2nd	5271.30	23.5	123875.5911	14.95	280.37	351.44
3rd	5250.20	33.5	175881.8256	21.23	265.41	711.31
4th	5230.70	43.5	227535.3956	27.47	244.18	1194.90
5th	4307.11	53.5	230430.4603	27.82	216.71	1488.28
6th	3699.42	66.83	247232.2559	29.85	188.89	1994.66
7th	3634.33	81.5	296197.5507	35.76	159.05	2914.28
8th	3245.59	96.17	312128.0481	37.68	123.29	3623.80
9th	3224.85	110.83	357410.0738	43.15	85.61	4782.07
Penthouse	757.14	125.5	95021.06791	11.47	11.47	1439.65
$Roof_{penthouse\ level}$	2045.36	125.5	256692.1048	30.99	30.99	3889.09
Roof	701.35	140.17	98308.84858	11.87	0	1663.56
Total					1892	

TABLE 5 – Seismic Load Tabulation

The base shear of the seismic load was found not to control over the wind base shear.

SPOT CHECKS

Typical columns and a typical bay on the sixth floor was analyzed and checked with the tabulated dead loads on the building. The detailed analysis can be found in appendix D.

The first spot check was an analysis of a typical composite beam on the sixth floor. The beam is a W 18 x 40 A992 steel member. The bending moment calculated from the loads was significantly less than the bending moment of the beam. This is due to the compensation for the moment caused by lateral forces.

Dead and live loads were calculated for each floor to check the load capacity of the columns located on floor 6 to the penthouse on the roof. ASCE 7-05 was used to apply the live and dead loads. Live load reduction was included in this tabulation. The tabulation of these loads can be found in appendix D in table 5. The columns' compressive strengths were compared to the calculated compressive load. Values were taken from table 4-1 from the AISC Steel Manual after the hand calculated method was compared and proven to be accurate with table 4-1.

After this analysis of the designed building members, it was proven that the designed capacity of the structure will carry the applied dead and live loads.

CONCLUSION

In conclusion, technical report 1 analyzes the existing conditions of the Science Center Research Park's structure. This report has a purpose of achieving a better understanding of the design decisions used to design the structure of this building. Also, this report includes a detailed description of the structural system.

In order to confirm the designer's choice of member sizes, many calculations on gravity and lateral loads were done. Typical bays were chosen to tabulate spot checks on typical beams and the typical composite deck. Also, spot checks were done on typical columns. After moment capacities and compressive strengths were compared, it was obvious that the columns were over-sized to compensate for moments created from the lateral forces. Deflections were also calculated and taken into consideration. The designed structure checked out to be sufficient for the applied loads. All checks were done in accordance with the AISC Steel Manual.

After wind and seismic base shears were calculated and compared it was noted that the wind base shear in the east-west direction controls. Therefore, the lateral wind loads determined the design of the lateral bracing system within the building. The steel lateral bracing system is consisted of HSS shapes mainly around the main elevator and stair core located in the center of the building.

All design values and calculation procedures were used in accordance with all the appropriate, applicable codes. All the calculations and values can be found in the appendices located at the end of this report.

APPENDIX A-TYPICAL PLANS AND DESIGN VALUES

DESIGN LOADS AND FACTORS

DESIGN CODE: INTERNATIONAL BUILDING CODE 2003 ED

LIVE LOAD DATA

FLOOR OR ROOF AREA	LOAD (psf)
LABS / OFFICES	100
CORRIDORS, LOBBIES & EXITS	100
GARAGE	40
MECHANICAL EQUIP ROOMS	150
ROOF	30

LIVE LOAD REDUCTION APPLIED TO:

- ☒ COLUMNS
- ☒ GIRDERS
- ☐ BEAMS
- ☐ 2-WAY SLABS

SPECIAL LOADING:

SNOW LOAD DATA

ROOF AREA	LOAD (psf)
GROUND SNOW LOAD (P_g)	30
FLAT ROOF SNOW LOAD (P_r)	23
DRIFT	VARIES
FACTOR	VALUE
SNOW EXPOSURE (C_e)	1.0
SNOW LOAD IMPORTANCE (I_s)	1.0
THERMAL FACTOR (C_t)	1.0

SPECIAL SNOW CONSIDERATIONS:

☒ GOVERNS ROOF DESIGN

WIND LOAD DATA

FACTOR	VALUE
BASIC WIND SPEED (V_{3S}) (MPH)	90
WIND IMPORTANCE (I_w)	1.0
OCCUPANCY CATEGORY	II
WIND EXPOSURE	B
INTERNAL PRESSURE COEFFICIENT	±0.18
COMPONENTS AND CLADDING WIND PRESSURE (PSF)	*VARIES

* CALCULATED PRESSURES TO BE DETERMINED BY COMPONENT AND CLADDING PROVIDER.

SPECIAL WIND CONSIDERATIONS:

☐ GOVERNS LATERAL DESIGN

EARTHQUAKE DESIGN DATA

FACTOR	VALUE
SEISMIC IMPORTANCE FACTOR (I_E)	1.0
SEISMIC USE GROUP	I
SPECTRAL RESPONSE ACCELERATION 0.2 SEC (S_S)	0.33
SPECTRAL RESPONSE ACCELERATION 1.0 SEC (S_1)	0.082
SITE CLASS	C
DESIGN SPECTRAL RESPONSE COEFFICIENT (S_{DS})	0.27
DESIGN SPECTRAL RESPONSE COEFFICIENT (S_{D1})	0.09
SEISMIC DESIGN CATEGORY	B

ANALYSIS PROCEDURE – EQUIVALENT LATERAL FORCE

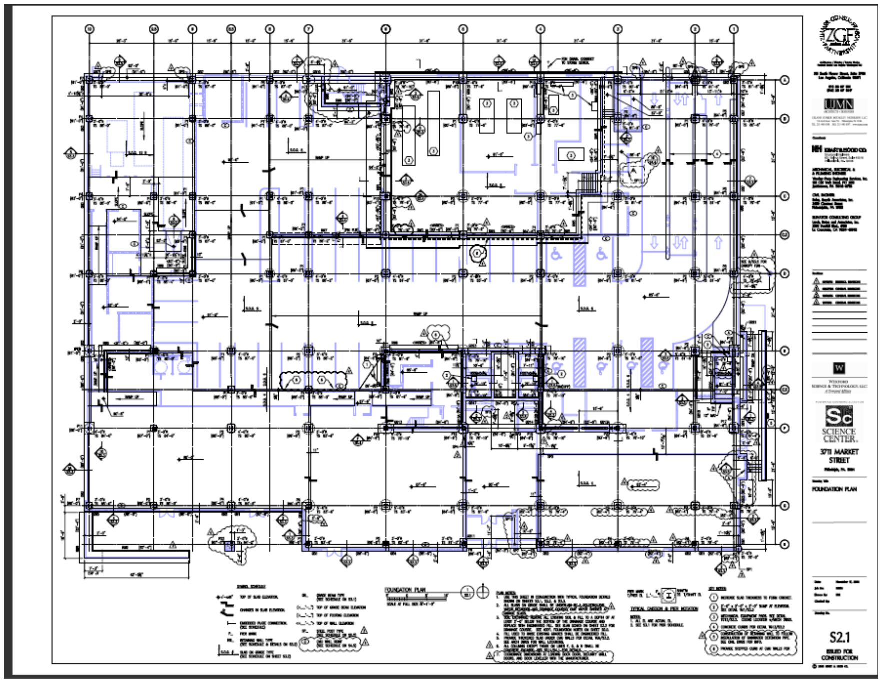
BASIC SEISMIC-FORCE-RESISTING SYSTEM	ORDINARY STEEL CONCENTRIC BRACED FRAMES
	Cs=0.021 R=3
DESIGN BASE SHEAR (kips)	

SPECIAL SEISMIC CONSIDERATIONS:

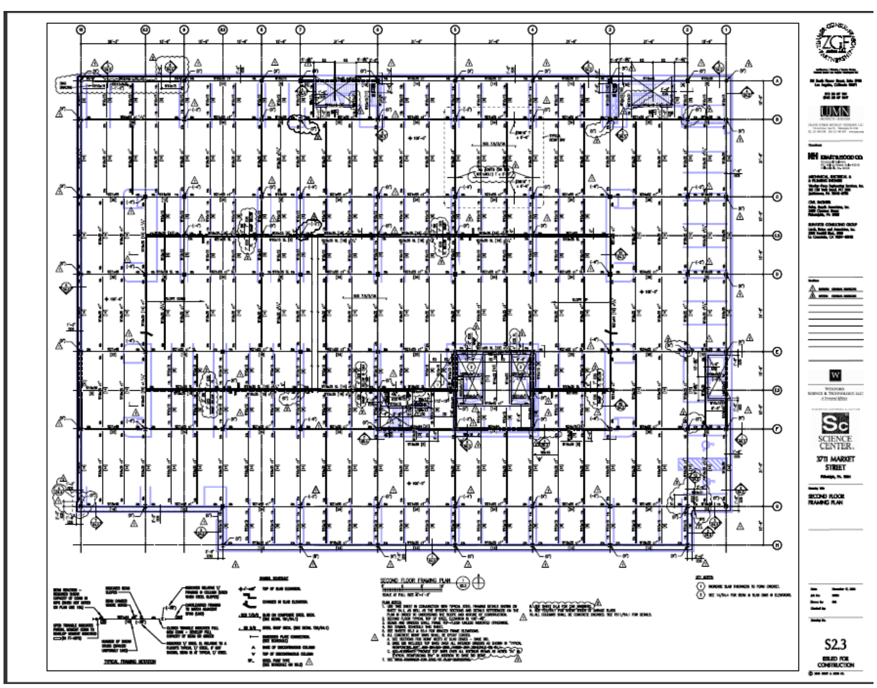
☒ GOVERNS LATERAL DESIGN

2

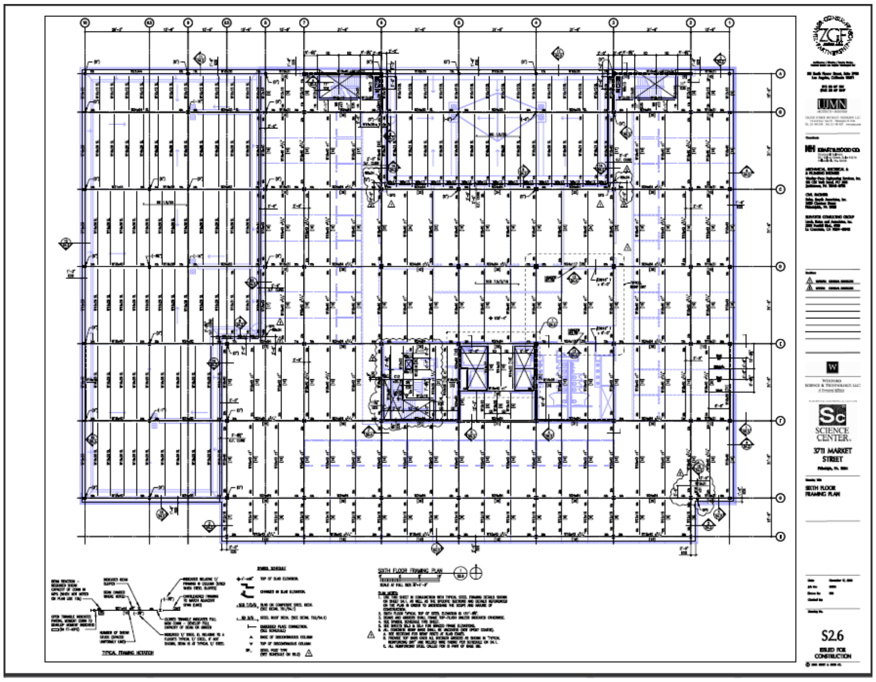
Foundation Plan



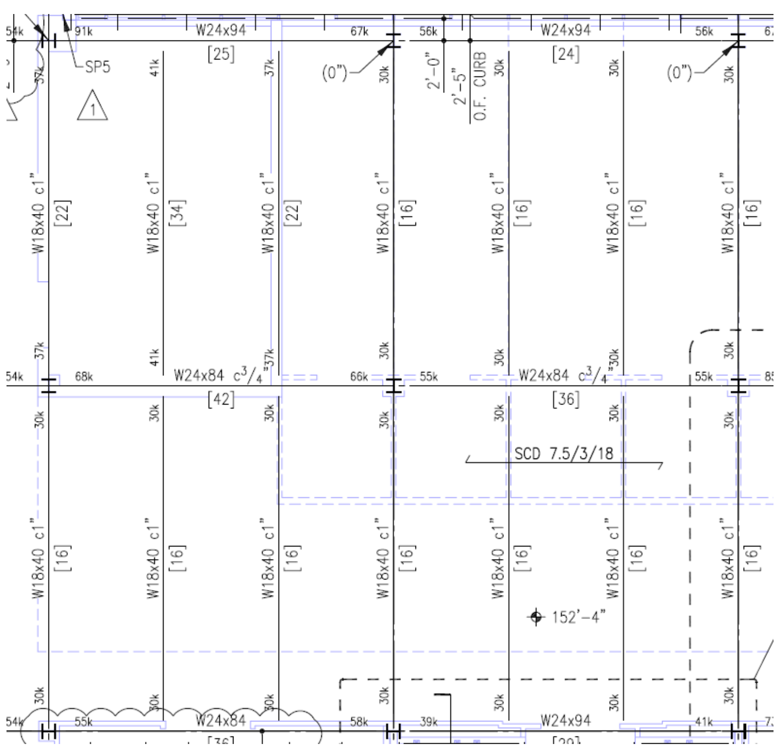
2nd Floor Plan



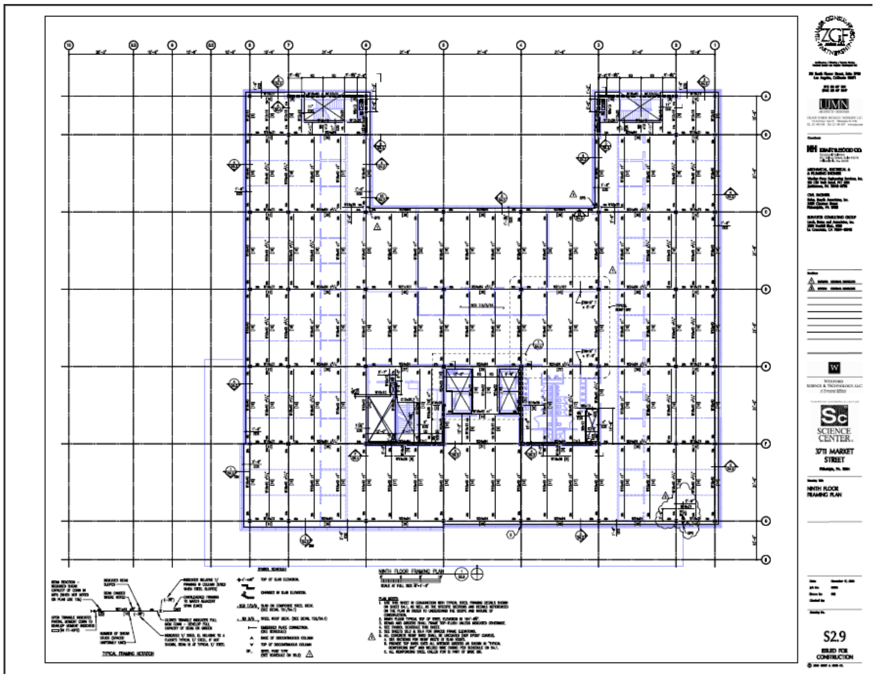
6th Floor Plan



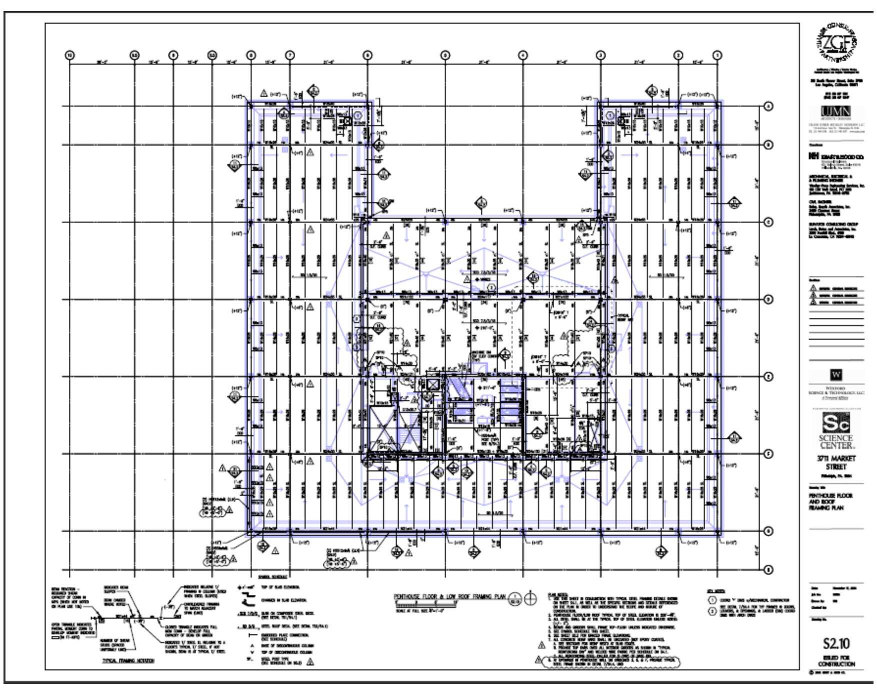
Typical Bay (6th Floor)



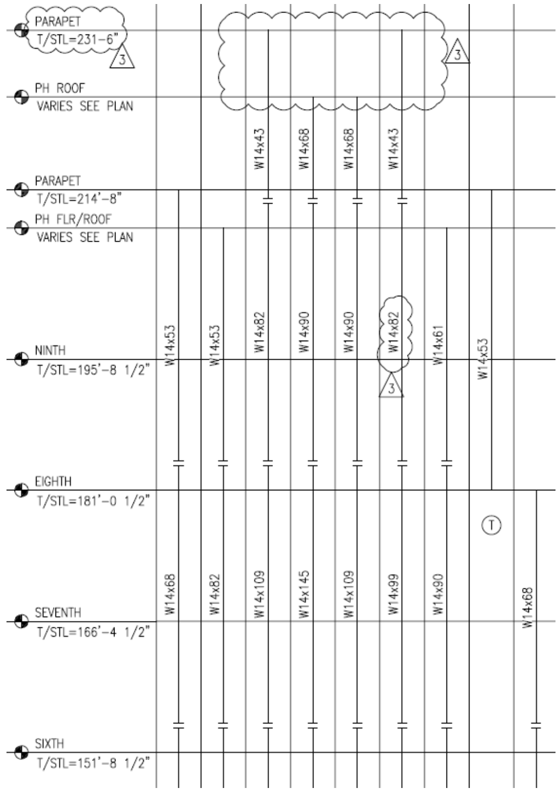
9th Floor Plan



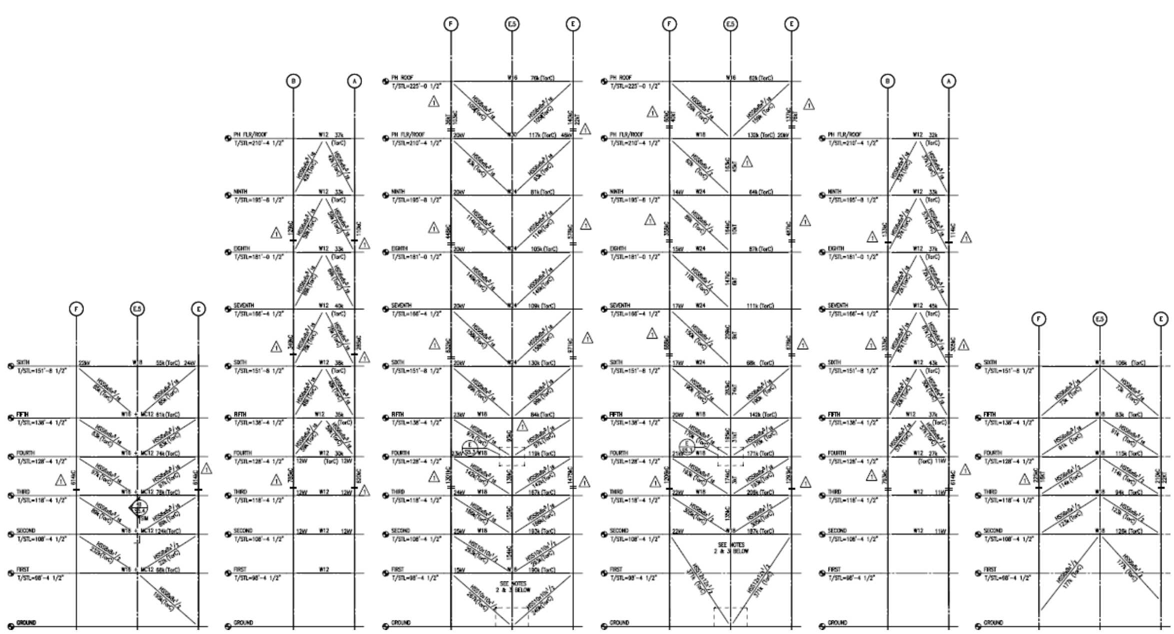
Penthouse Floor Plan



Typical Column Schedule



Typical Braced Frame Schedule



APPENDIX B-WIND LOADS

Zachary Yarnall

MULKEY ENGINEERS & CONSULTANTS CALCULATION SHEET PAGE ____ OF ____

CLIENT ____ SUBJECT Senior Thesis Prepared By ____ Date 09/29/09

PROJECT No. Method 2 - Wind Analysis Reviewed By ____ Date ____

Velocity Pressures, q_z and q_h

- From ASCE 7-05 Fig. 6-1

  Basic wind speed, V = 90 mph / 40 m/s

- From ASCE 7-05 Fig. 6-4

  Wind directionality factor, $K_d = 0.85$ (for buildings)

- From ASCE 7-05 fig. 7-4

  Importance factor, $I = 1.1$ (Category III)

- From ASCE 7-05 6.5.6

  Exposure Category B (located in urban area)

- Are all 5 conditions of 6.5.7.1 met? No

- From ASCE 7-05

  Topographic factor, $K_{zt} = 1.0$

- From ASCE 7-05 Table 6-3 and Table 6-2

  Velocity pressure exposure coefficients

  $K_z = 1.10$ ($z_g = 1200$, $\alpha = 7.0$)

  $K_h = 1.10$

  $K_z = 2.01\left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}}$ (sample Calculation)

  $= 2.01\left(\frac{147.5}{1200}\right)^{\frac{2}{7}} = 1.10$

- From ASCE 7-05 Eq. 6-15

  Velocity pressure at height z and h (refer to spread sheet)

  $q_z = 0.00256 K_z K_{zt} K_d V^2 I$ (sample calculation)

  $= 0.00256(1.10)(1.0)(0.85)(90)^2(1.10)$

  $= 21.47$

ME-01

MULKEY ENGINEERS & CONSULTANTS — CALCULATION SHEET — PAGE ____ OF ____

CLIENT ____ SUBJECT ____ Prepared By ____ Date ____

PROJECT No. Method 2 - Wind Analy ____ Reviewed By ____ Date ____

Gust Effect Factors, G and G_f

- Building natural frequency, n_1 (ASCE 7-05, C6.5.8, Eq. C6-17)

  $n_1 = 100/H = 100/147.5 = 0.68$ (average value)

- Damping ratio, B (ASCE 7-05, C6.5.8)

  $B_1 = 1.0\%$ per ISO

- Structure Dimensions

  $h = 147.5'$
  $B = 262.5'$ (N/S Elevation)
  $L = 189'$ (E/W Elevation)

  $n_1 < 1 \text{ Hz}$

- ∴ Structure is flexible

  $g_Q = g_v = 3.4$

  $$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}} \quad \text{(Eq. 6-9)}$$

  $$= \sqrt{2\ln(3600(0.68))} + \frac{0.577}{\sqrt{2\ln(3600(0.68))}} = 4.097$$

  $\bar{z} = 0.6h = 0.6(147.5) = 88.5^{ft} > 30^{ft} = z_{min}$ (from ASCE 7-05 Fig. 6-2)

  $I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.30\left(\frac{33}{88.5}\right)^{1/6} = 0.255$ (c = 0.30 from ASCE 7-05 Fig. 6-2)

  $L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{88.5}{33}\right)^{1/3} = 444.6$ (ℓ = 320 from ASCE 7-05 Fig. 6-2)

  $$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} \quad \text{(Eq. 6-6)}$$

  $$Q_{N/S} = \sqrt{\frac{1}{1 + 0.63\left(\frac{262.5+147.5}{444.6}\right)^{0.63}}} = 0.791$$

ME-01

MULKEY ENGINEERS & CONSULTANTS

CALCULATION SHEET PAGE ____ OF ____

CLIENT ____________ SUBJECT ____________ Prepared By ________ Date ________

PROJECT No. ____________ Reviewed By ________ Date ________

Gust Effect Factors, G and G_f - continued

$$Q_{E/W} = \sqrt{\frac{1}{1+0.63\left(\frac{189+147.5}{444.6}\right)^{0.63}}} = 0.809$$

- Basic wind speed, V

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) \quad \text{(Eq. 6-14)} \quad (\bar{b} = 0.45,\ \bar{\alpha} = \tfrac{1}{4} \text{ from ASCE 7-05 Fig. 6-2})$$

$$= (0.45)\left(\frac{88.5}{33}\right)^{\frac{1}{4}}(90)\left(\frac{88}{60}\right) = 76.01$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{0.68(444.6)}{76.01} = 3.98 \quad \text{(Eq. 6-12)}$$

$$R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}} = \frac{7.47(3.98)}{(1+10.3(3.98))^{5/3}} = 0.059$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right) = \frac{1}{6.07} - \frac{1}{2(6.07)^2}\left(1-e^{-2(6.07)}\right) = 0.151$$

$$\eta = 4.6 n_1 \frac{h}{\bar{V}_{\bar{z}}} = 4.6(0.68)\frac{147.5}{76.01} = 6.07$$

$$R_{B_1} = \frac{1}{10.8} - \frac{1}{2(10.8)^2}\left(1-e^{-2(10.8)}\right) = 0.088 \quad \text{(N/S)}$$

$$\eta = 4.6 n_1 \frac{B}{\bar{V}_{\bar{z}}} = 4.6(0.68)\frac{262.5}{76.01} = 10.80$$

$$R_{B_2} = \frac{1}{7.78} - \frac{1}{2(7.78)^2}\left(1-e^{-2(7.78)}\right) = 0.12 \quad \text{(E/W)}$$

$$\eta = 4.6 n_1 \frac{B}{\bar{V}_{\bar{z}}} = 4.6(0.68)\frac{189}{76.01} = 7.78$$

$$R_{L_1} = \frac{1}{36.17} - \frac{1}{2(36.17)^2}\left(1-e^{-2(36.17)}\right) = 0.027 \quad \text{(N/S)}$$

$$\eta = 15.4 n_1 \frac{L}{\bar{V}_{\bar{z}}} = 15.4(0.68)\frac{262.5}{76.01} = 36.17$$

$$R_{L_2} = \frac{1}{26.04} - \frac{1}{2(26.04)^2}\left(1-e^{-2(26.04)}\right) = 0.038 \quad \text{(E/W)}$$

$$\eta = 15.4 n_1 \frac{L}{\bar{V}_{\bar{z}}} = 15.4(0.68)\frac{189}{76.01} = 26.04$$

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Gust Effect Factors, G and Gf - continued

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)} \qquad \text{(Eq. 6-10)}$$

$$R_1 = \sqrt{\frac{1}{1.0}(0.059)(0.151)(0.088)(0.53 + 0.47(0.027))} = 0.021 \qquad (N/S)$$

$$R_2 = \sqrt{\frac{1}{1.0}(0.059)(0.151)(0.12)(0.53 + 0.47(0.038))} = 0.024 \qquad (E/W)$$

$$G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right) \qquad \text{(Eq. 6-8)}$$

$$G_{f_1} = 0.925\left(\frac{1 + 1.7(0.255)\sqrt{(3.4)^2(0.791)^2 + (4.097)^2(0.021)^2}}{1 + 1.7(3.4)(0.255)}\right) \qquad (N/S)$$

$$= 0.876$$

$$G_{f_2} = 0.925\left(\frac{1 + 1.7(0.255)\sqrt{(3.4)^2(0.809)^2 + (4.097)^2(0.024)^2}}{1 + 1.7(3.4)(0.255)}\right) \qquad (E/W)$$

$$= 0.887$$

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Building, Main Wind-force Resisting Systems

- The building is enclosed
- The building has a parapet
- Velocity Pressure $q_p = 21.41$ mph
- Combined net pressure coefficient, GC_{pn}

  $GC_{pn} = +1.5$ windward

  $GC_{pn} = -1.0$ leeward
- Combined net design pressure on the parapet

  $P_p = q_p GC_{pn}$ (Eq. 6-20)

  $= (21.41)(1.5) = 32.12$ (windward)

  $= (21.41)(-1.0) = -21.41$ (leeward)
- The building is not rigid
- Determine velocity pressure q_z for windward walls along the height of the building and q_h for leeward walls, side walls; and roof
- Pressure coefficient, C_p for the walls and roof (Fig. 6-6 or 6-8)

  $\frac{L}{B} = \frac{262.5}{189} = 1.39$ (N/S) $\Rightarrow C_p = -0.5$ ($L/B = 0-1$)

  $= \frac{189}{262.5} = 0.72$ (E/W) $\Rightarrow C_p = -0.42$ (interpolated)

  $\left\{ \begin{matrix} L/B = 0-1, & L/B = 2 \\ C_p = -0.5, & C_p = -0.3 \end{matrix} \right\}$

C_p	(N/S)	(E/W)
Windward wall	0.8	0.8
Leeward wall	-0.5	-0.42
Side wall	-0.7	-0.7

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Building, Main Wind-force Resisting Systems – Continued

- Determine design wind pressures, p_z

$p_z = q_z G_z C_p$ (Eq. 6-19)

Windward sample calculation (N/S)

$p_z = 21.41(0.876)(0.8) = 15.0$ psf

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APPENDIX C-SEISMIC LOADS

Zachary Yarnall

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Consideration of Seismic Design Requirements

- Not a detached one- or two- family dwelling.
- Not an agricultural storage structure intend for incidental human occupancy
- Does the structure require special consideration with respect to response characteristics and evironment that are not addressed in Chapter 15 and for which other regulations provide criteria?

No

∴ Seismic requirements of ASCE 7-05 must be considered

Seismic Ground Motion Values

- Determine S_s and S_1 from Figs. 22-1 through 22-14
  Site Classification C
  Seismic Design Category B (from Structural Plans)

  Occupancy Category II => I = 1.0

  $S_s = 0.3$ (from Fig. 22-1) > 0.15
  ∴ No
  $S_1 = 0.06$ (from Fig. 22-2) > 0.04

- Is the structure seismically isolated or does it have damping systems on site with $S_1 \geq 0.6$? No
- Determine S_{MS} and S_{M1} by Eqs. 11.4-1 and 11.4-2

  $S_{MS} = F_a S_s = (1.2)(0.3) = 0.36$ $F_a = 1.2$ (from Fig. 11.4-1)

  $S_{M1} = F_v S_1 = (1.7)(0.06) = 0.102$ $F_v = 1.7$ (from Fig 11.4-2)

- Determine S_{DS} and S_{D1} by Eqs. 11.4-3 and 11.4-4

  $S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.36) = 0.24$

  $S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.102) = 0.068$

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Permitted Analytical Procedures

- Equivalent lateral force procedure

response modification coefficient, $R = 3$

importance factor, $I = 1.0$

approximate fundamental period of the structure, T_a

$T_a = C_t h_n^x = 0.03(144.5)^{0.75} = 1.27$

$C_t = 0.03$
$x = 0.75$ (from Table 12.8-2)

$T_L = 6$ (from Fig. 22-15) $> T_a$

Determine C_s by Eqs. 12.8-3 and 12.8-2

$C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} \le \frac{S_{DS}}{\left(\frac{R}{I}\right)}$ ✓

$\frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.068}{1.27\left(\frac{3}{1}\right)} = 0.0178$

$\frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{0.24}{3} = 0.08$

Determine effective seismic weight W in accordance with 12.7.2

$W = 40{,}378$ kips (calculation on excel sheet)

$V = C_s W = 0.0178(40{,}378) = 719$

$T < 0.5$ sec $\therefore$ $K = 1$ (12.8.3)

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- Equivalent lateral force procedure – continued

Determine lateral seismic force F_x at level x by Eqs. 12.8-11 and 12.8-12

$$F_x = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} V$$ (Calculations in Excel Sheet)

$$\sum_{i=1}^{n} w_i h_i^k = (40{,}378)(147.5) = 5{,}955{,}755$$

Determine seismic design story shear V_x by Eq. 12.8-13

$$V_x = \sum_{i=x}^{n} F$$

Determine inherent torsional moment, M_t

Determine accidental torsional moment, M_{ta}

Determine the deflection δ_x at levels x by Eq. 12.8-15

$$\delta_x = \frac{C_d \delta_{xe}}{I}$$

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APPENDIX D-SPOT CHECKS

Dead and live loads were combined and totaled from the roof to the sixth floor. The calculated loads were used to compare with designed member's load capacities on those floors.

Floor	Tributary Area (ft2)	Dead Load (psf)	Live Load (psf)	Influence Area (ft2)	Reduction Factor (>=0.4)	Live Load (k)	Dead Load (k)	Load Combination	Load at Floor (k)	Accumulated Load (k)
Roof	992.25	94	30	3969	0.49	14.5	93.3	1.2D + 1.6L	135.17	135.17
Penthouse	992.25	101	100	3969	0.49	48.4	100.2	1.2D + 1.6L	197.75	332.92
9	992.25	99	100	3969	0.49	48.4	98.2	1.2D + 1.6L	195.37	393.12
8	992.25	99	100	3969	0.49	48.4	98.2	1.2D + 1.6L	195.37	390.74
7	992.25	98	100	3969	0.49	48.4	97.2	1.2D + 1.6L	194.18	389.55
6	992.25	99	100	3969	0.49	48.4	98.2	1.2D + 1.6L	195.37	389.55

TABLE 5 – Calculated Loads for Column Spot Checks

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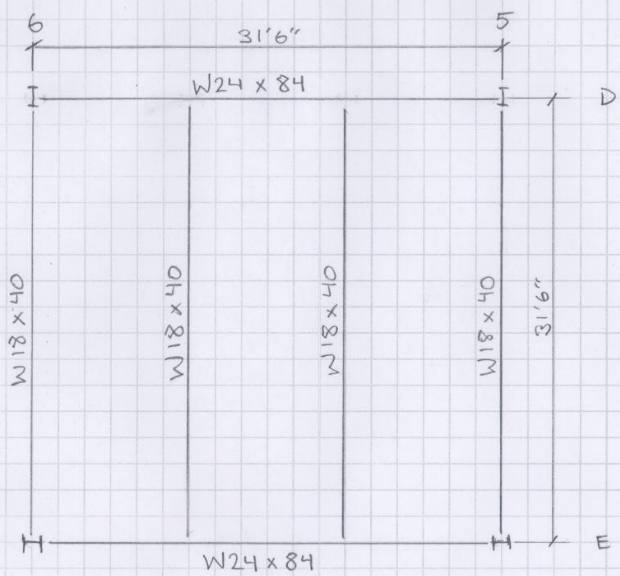
Typical Interior Bay - Sixth Floor



Loads:

Dead Load$_{(slab)}$ = (150 pcf)(6"/12) = 75 psf (Note: Assume to be 80 psf with Steel Deck)

Dead Load$_{(MEP and Finishes)}$ = 8 psf

Live Load$_{(offices/Labs)}$ = 100 psf

Floor System:

1.5" x 18 Gauge Deck

6" Normal Weight Concrete

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Beam Spot Check

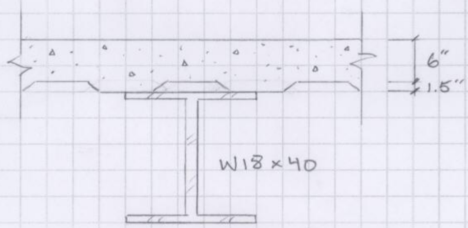
Factored Load: $1.2D + 1.6L = 1.2(80+8) + 1.6(100)$

$W_u = 266$ psf

Trib. Width = 10.5'

$W_u = 266(10.5)/1000 = 2.79$ klf

$M_u = \frac{W_u l^2}{8} = \frac{(2.79)(31.5)^2}{8} = 346$ kft



$b_{eff} = \min \begin{cases} \text{spacing} = 10.5'(12) = 126'' \\ \frac{\text{span}}{4} = \frac{31.5(12)}{4} = 94.5'' \end{cases}$ ∴ Controls

Check Deflections:

$\Delta_{Constr.} = \frac{5 W_{conc} l^4}{384 EI}$

$W_{conc} = 150\text{pcf}(6/12)(10.5) = 0.788$ klf

$\Delta_{allowable} = \frac{l}{360} = \frac{31.5(12)}{360} = 1.05''$

$I_{req} = \frac{5 W_{conc} l^4}{384 \Delta_{allow} E} = \frac{5(0.788)(31.5)^4(1728)}{384(1.05)(29{,}000)} = 573 \text{ in}^4$

$I_{W18\times40} = 612 \text{ in}^4 > 573 \text{ in}^4$ ∴ Okay

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Check Bending for construction loading:

$W_{live} = 20\,psf(10.5') = 0.21^{klf}$

$W_u = 1.2(0.788) + 1.6(0.21) = 1.28\ klf$

$M_u = \frac{w_u l^2}{8} = \frac{1.28(31.5)^2}{8} = 159^{kft}$

$\phi M_{n\,W18x40} = 215^{kft} > 159^{kft}$

From Table 3-19:

Assume $\Sigma Q_n = 147^k$

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{147}{0.85(4)(94.5)} = 0.458''$

$Y_2 = 7.5'' - \frac{a}{2} = 7.5 - \frac{0.458}{2} = 7.27''$ (round to 7")

W18x40 $Y_2 = 7''$ $\Sigma Q_n = 147^k$ @ PNA #7

$\phi M_n = 444^{kft} > M_u = 346^{kft}$ $\therefore$ Okay

Check Number of Shear Studs:

Table 3-21:

Shear stud diameter = 3/4"
Deck perpendicular
$f'_c = 4000$ psi
} $Q_n = 21.5$

$\#$ studs req'd $= \frac{\Sigma Q_n}{Q_n} \times 2 = \frac{147}{21.5}(2) = 13.7$

$\therefore$ 14 studs req'd

$\#$ of studs provided = 16 over length of beam > 14

$\therefore$ Okay

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Check Deflection:

$Y_2 = 7'' \Rightarrow I_{LB} = 1210 \text{ in}^4$

$\Delta = \frac{5 w_u l^4}{384 E I_{LB}} = \frac{5(1.28)(31.5)^4(1728)}{384(1210)(29,000)} = 0.808$

$\Delta_{allow} = \frac{l}{360} = 1.05'' > 0.808$ $\therefore$ Okay

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Column Spot Check - E5 (see excel sheet for loads)

Roof: $P_u = 135.17^K$

W14 x 68, $h = 14.67'$

$A = 20 \text{ in}^2$
$I_x = 722 \text{ in}^4$ $I_y = 121 \text{ in}^4$
$r_x = 5.98 \text{ in}$ $r_y = 2.45 \text{ in}$

$\frac{KL}{r_x} = \frac{14.67(12)}{5.98} = 29.4$ $\frac{KL}{r_y} = \frac{14.67(12)}{2.45} = 71.9$ <= controls

$\frac{KL}{r} \leq 4.71\sqrt{E/F_y} = 4.71\sqrt{29{,}000/50} = 113 > 71.9$
∴ Inelastic Behavior

$F_{CR} = \left[0.658^{F_y/F_E}\right]F_y = \left[0.658^{50/55.4}\right]50 = 34.3 \text{ ksi}$

$F_E = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2(29000)}{(71.9)^2} = 55.4 \text{ ksi}$

$\phi P_n = \phi F_{CR} A_g = 0.9(34.3)(20) = 617^K > P_u = 135.17^K$
∴ Okay

Check with Tables 4-22 & 1:

$\phi F_{CR} = 30.8 \text{ ksi}$

$\phi P_n = 624^K \approx 617^K$

∴ Method by hand Okay

* Note: Method by hand checks out, therefore Table 4-1 is used for the following column spot checks. Also, columns are large due to moment to resist lateral forces.

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Penthouse: $P_u = 333^k$

W14 X 90 , $KL = 14.5'$

Table 4-1

$\phi P_n = 1010^k > 333^k$ ∴ Okay

9th Floor: $P_u = 393^k$

W 14 x 109

Table 4-1

$\phi P_n = 1230^k > 393^k$ ∴ Okay

8th Floor: $P_u = 391^k$

W 14 x 109

Table 4-1

$\phi P_n = 1230^k > 391^k$ ∴ Okay

7th Floor: $P_u = 390^k$

W 14 x 193

Table 4-1

$\phi P_n = 2230^k > 390^k$ ∴ Okay

6th Floor: $P_u = 390^k$

W14 x 193

Table 4-1

$\phi P_n = 2230^k > 390^k$ ∴ Okay

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