Technical Report 1

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Table of Contents

EXECUTIVE SUMMARY

This technical report will examine the physical existing conditions of the Millennium Hall residence building on the Drexel Campus. Through this study, a better understanding of the structural system and the major loads used during its design will be gained.

To begin, an overview of the major structural systems are provided. Foundation, column, slab, lateral resistance, and façade support are described. Pictures and figures have been added to give a quick overview and help obtain a better idea of the building. Major building materials and their specific characteristics are also given. Column layouts and basic framing plans were placed in Appendix A for reference.

A simple overview of the major codes and standards that derived the building's loads has been given. The gravity load is looked at first. Dead load and live load values are assigned to different loading zones. Next wind and seismic lateral loads are determined. Using ASCE7-05, values for these procedures where addressed and then used to calculate these loads. Loading in the North/South direction was determined to be controlled by wind, with a total base shear of 545 kips compared to 475 kips for seismic. In the East/West direction seismic controlled with a total base shear of 416 kips, compared to 305 kips in wind. These calculations were placed in Appendix B and C for reference.

Finally spot checks for member strength were provided. These calculations of different slabs in the flat plate system were taken at a typical floor level. All calculations found the size and amount of reinforcing to be adequate for strength

INTRODUCTION

Millennium Hall is the newest residence hall on the Drexel University Campus located in Philadelphia, Pennsylvania. Built among existing residence halls made from brick and stone, its dramatic glass façade makes the building stand out from the surroundings. The building's contrast also comes from its unique shape, a slender tower appearing to spiral upwards. This was accomplished by offsetting each floor about the building's central core by 10 inches, creating a bold statement for the university. Millennium Hall symbolizes Drexel's commitment towards the future and embraces their great history of engineering and architecture.

Required to house 482 students, the building's main design came from its main constraint, the 20,000 square foot site. Originally a lot containing 3 tennis courts, Millennium Hall had to rise upward, reaching 17 stories. The ground floor takes up most of the lot and contains the main lobby, elevator bank, reception area, a small lounge, offices and storage space. Attached to this base is the tower, where all of the student living facilities are located. Each floor includes 16 two person dorm rooms, individual shower and restrooms, shared kitchen and a study space. The 17th floor is a study lounge providing unobstructed views of the campus and the city skyline of Philadelphia. All of this is achieved the tower's compact 5,000 square foot layout.

The majority of the building is clad with a combination of a glass and aluminum. These reflective surfaces catch the light and reflections off the neighboring buildings, quickly catching and drawing your eye towards it. The curtain wall allows maximum natural light to enter the dorm rooms and providing pleasing views for the students. Aluminum rain screen panels give the building a unique look and provide enough cover to the curtain wall to achieve an acceptable level of privacy to the rooms. These panels also work as solar shades, further reducing the building's cooling load.

INTRODUCTION *(Continued)*

The structural system for Millennium Hall uses two main types. A steel frame holding a slab on metal deck forms the ground floor and supports a green roof over the office and storage area. The tower is comprised of a cast in place concrete flat plate system. Two radial lines of concrete columns circle the central core and provide all of the strength for the tower. These columns extend the entirety of the building. Beams were then added between the columns to provide torsional strength for the slab. Each floor slab is then cantilevered outward 15 feet to the exterior of the building. Lateral forces are resisted by an ordinary concrete shear wall in one direction and an ordinary concrete moment frame in the other.

STRUCTURAL SYSTEM

Foundation Design

A geotechnical Report was prepared by Pennoni Associates, Inc. on March 12th, 2008. It concluded with the following:

- Spread footings and continuous wall footings shall be designed for a net allowable bearing pressure of 6 KSF.
- Drilled piers (caissons) shall be designed for a net allowable bearing pressure of 60 KSF.

With the absence of any sub grade levels, the Millennium Halls foundation consist of only spread footings for the ground floor load and fourteen caissons to support the tower's gravity load. These caissons, spaced approximately 10 feet apart, are 5 feet in diameter, giving each one a bearing strength of 1200 Kips. (See Figure 1.1) Running along the line of these caissons is a 30 inch by 60 inch grade beam, reinforced with 4 - #8 on the top and bottom and #5 stirrups spaced at 12 inches. (See Figure 1.2)

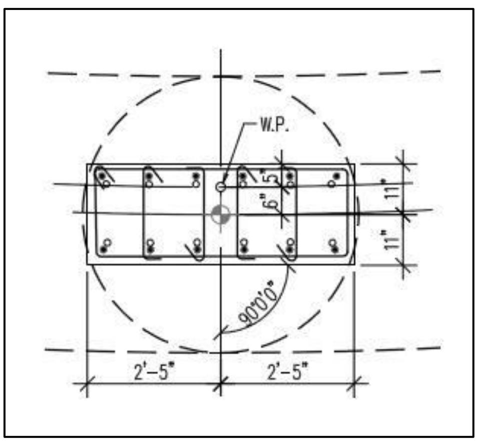


Figure 1.1

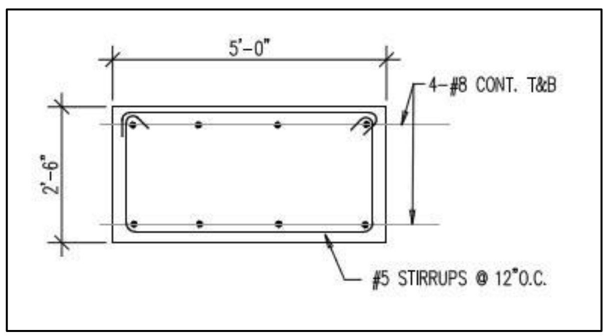


Figure 1.2

The towers twenty reinforced concrete columns then sit directly on top of the perimeter caissons and are all tied together with a grade beam using#5 stirrups at 12 inches on center. (See Figure 1.3 on Page 7)

STRUCTURAL SYSTEM *(Continued)*

This grade beam continues through the building as spread footings to provide strength for the first floors steel structure. The remaining four caissons located towards the center of the building at the elevator core are used to secure the reinforced concrete shear walls. These shear walls are connected to the ground beam and caisson in a way similar to the column connections. (See Figure 1.3)

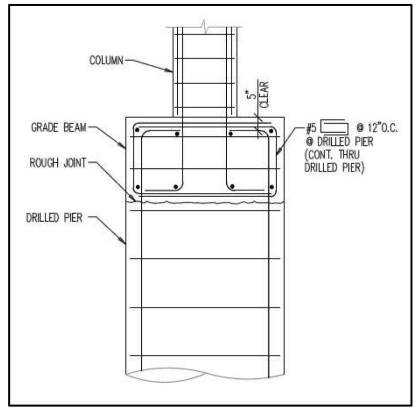


Figure 1.3

Foundation Layout

Below is a layout of the basic foundation elements for the tower. The drilled caissons are represented as red circles and the outline of the grade beams are shown in blue.

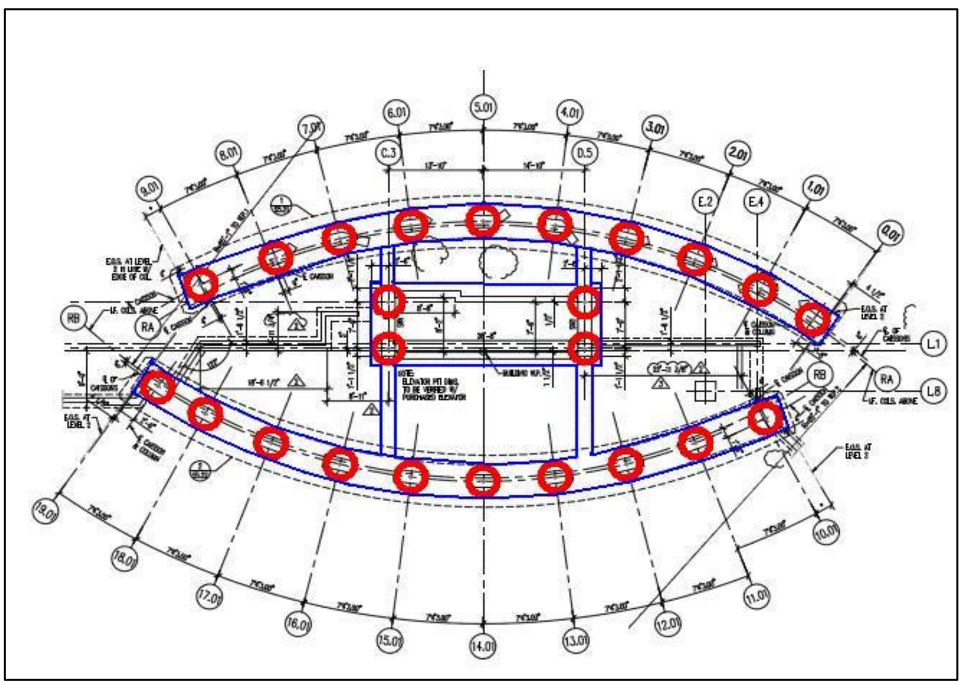


Figure 1.4

STRUCTURAL SYSTEM *(Continued)*

Column Design

The millennium Hall's tower is supported by ten concrete columns which circle around the core. This typical beam is 22 inches by 60 inches and extends the entire height of the tower. Each column is directly supported by one of the foundations caissons.
(See Appendix A for typical floor column layout)

Floor Framing System

Each floor slab is cantilevered outward from the column line 15 feet. This 12 inch slab is reinforced in the east west direction with #4 bar spaced at 18 inches top and bottom. #6 bars spaced at 20 inches on the bottom and #7 bars at the top spaced 7.5 inches reinforce the north/south direction. A typical slab connection to both the column and beams between columns can be seen below. (See Figure 2.1/2.2) The slab located between the column lines is 14 inches thick and is reinforced with #4 bar spaced at 15 inches top and bottom in the east/west direction and #4 bar at 18 inches on the bottom in the north/south direction. (See Appendix A for typical floor framing)

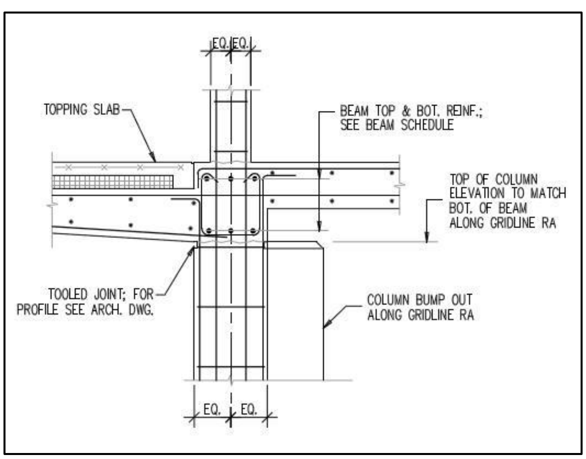


Figure 2.1

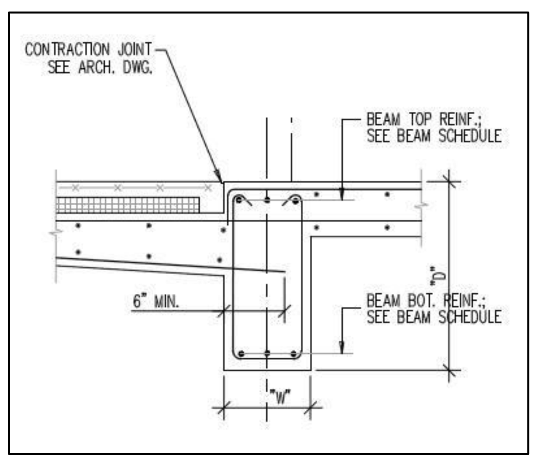


Figure 2.2

STRUCTURAL SYSTEM *(Continued)*

Lateral System

Lateral forces in the tower are resisted using an interactive system with ordinary reinforced concrete shear walls and ordinary reinforced concrete moment frame. The shear walls are located at the center of the building in the elevator core, reinforced by splicing a series of #11 steel rebar. (See Figure 3.2) All shear walls from the ground floor to the fourth floor support a compressive strength of 7000 psi. From the fourth floor upward 5000 psi concrete is used. On the ground floor's steel frame, a steel moment connection is used. (See Figure 3.1)

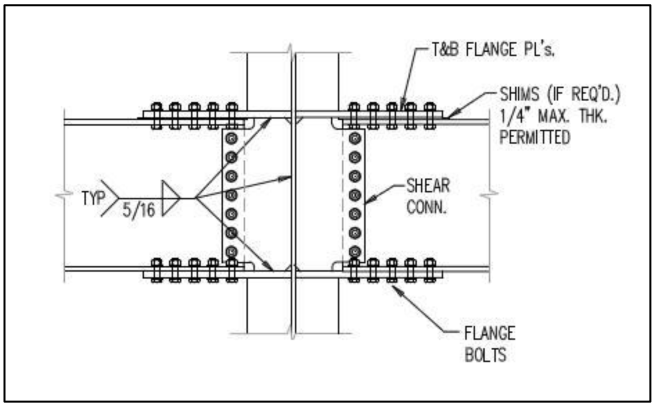


Figure 3.1

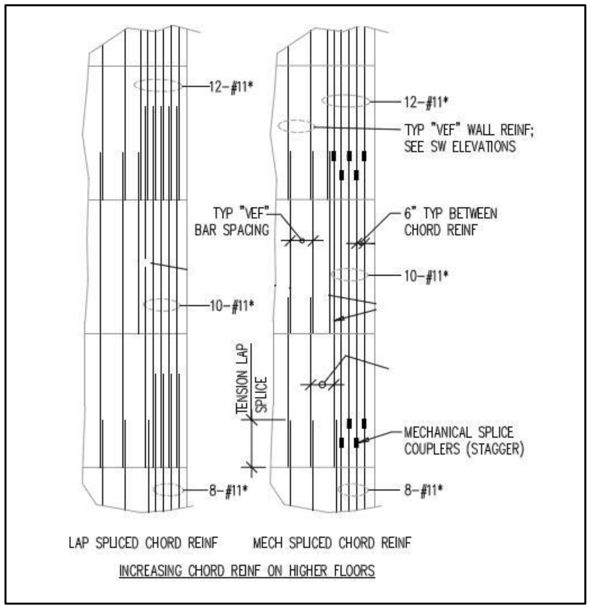


Figure 3.2

Envelope Support

The building's curtain wall and aluminum rain screen panels are connected in two main ways to the building structure. On the first level the wall hangs from the slab on metal deck assembly that is supported by the ground levels steel frame. This is accomplished with 10 gauge metal plate that has been bent.

STRUCTURAL SYSTEM *(Continued)*

The plate which runs continuously along the wall is secured to the slab on metal deck using shear bolts and is reinforced with 4 foot #4 spaced at 12 inches and fastened to the curtain wall with screws placed at 12 inches. (See Figure 4.1) This bent plate runs continuously along the face of the slab.

The second type of connection is found on the tower where shear bolts embedded into the concrete slab supports the curtain wall as well as the aluminum rain screen panels. These bolts connect directly to tabs specified by the curtain wall manufacturer. The slab edge is reinforced with #3 hoops spaced at a minimum of 3 inches that extend 2 feet into the slab. (See Figure 4.2)

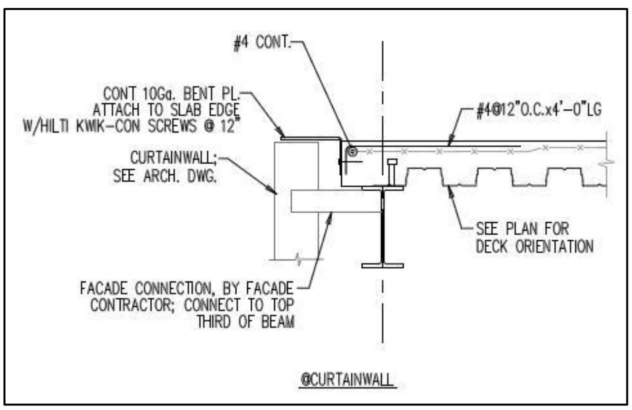


Figure 4.1

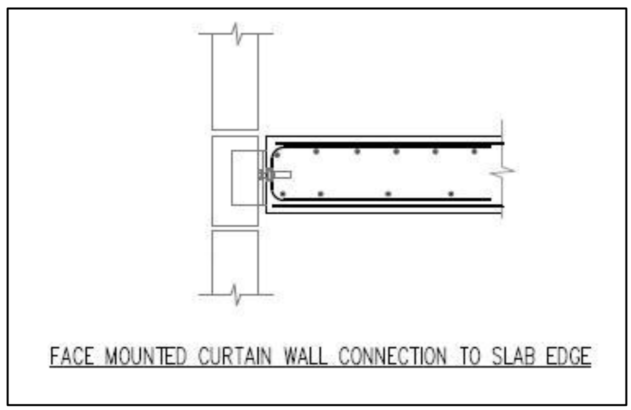


Figure 4.2

CODES AND DESIGN STANDARDS

The following codes and standards were applied to the design, construction, quality control, and safety of all work performed on the Millennium Hall project.

Building Code:

2007 Philadelphia Building Construction and Occupancy Code
(IBC 2006)

Minimum Design Loads for Buildings:

(ASCE 7 - 05)

Building Code Required for Structural Concrete:

(ACI 318 - 05)

Steel Construction Manual:

American Institute for Steel Construction

(AISC 13^{th} Edition, 2005)

Detailing for Steel Construction:

(AISC 13^{th} Edition - 2005)

Structural Welding Code - Steel:

(AWS D1.1 - 2004)

Design Manual for Floor Deck and Roof Deck:

Steel Deck Institute

Building Code Requirements for Masonry Structures:

(ACI 530 – 05/ ASCE 5 – 05)

MATERIALS

Concrete

Location	F'_c @ 28 Days (PSI)	Unit Wt. (PCF)
Spread & Wall Footings	3000	145
Drilled Piers	3000	145
Grade Beams	3000	145
Topping Slab	3000	145
NW Slabs on Metal Deck	3500	145
Slabs-on-Grade	4000	145
Walls (Other than Shear)	4000	145
Framed Slabs & Beams	5000	145
Columns Above Level 4	5000	145
Columns Below Level 4	7000	145
Shear Walls Above Level 4	5000	145
Shear Walls Below Level 4	7000	145

Reinforcement Steel

Type	Design Standard
Deformed Reinforcing Bar	ASTM A615, Grade 60
Weldable Deformed Reinf. Bars	ASTM A706
Welded Wire Reinf. (W.W.R.)	ASTM A185
Epoxy Coated Reinf. Bars	ASTM A775

MATERIALS *(Continued)*

Structural Steel

Type	Design Standard	Strength F_y (KSI)
W Shapes	ASTM A992	50
Channels, Angels, Plates & Bars	ASTM A36	36
Round Pipe	ASTM A53, Grade B	35
Square and Rectangular HSS's	ASTM A500, Grade B	46
High Strength Bolts	ASTM A325 (TYP.)/ A490	-
Anchor Bolts	ASTM F1554, Grade 55	-
Round & Threaded Rod	ASTM A36	-
Headed Shear Studs	ASTM A108	-
Welding Electrodes	AWS A5.1 or A5.5, E70XX	-
Galvanized Metal Floor Deck	ASTM A653, Grade 40	40
Galvanized Metal Roof Deck	ASTM A653, Grade 33	33
Light Gage Metal Studs	ASTM A446 (Galvanized)	-

GRAVITY LOAD

Gravity Loads		
Load Type	**Description**	**Design Load**
Material Self Weight	Reinforced Concrete	145 pcf
	Structural Steel	490 pcf
Superimposed D.L.	Partitions	20 psf
	Suspended MEP	15 psf
	Topping Slab	50 psf
	Soil/Landscaping Green Roof	35 psf
	Roofing/ Insulation	10 psf
	Façade	150 plf
Live Load	Lobbies	100 psf
	Public Area	100 psf
	Residence Room	40 psf
	Roof	20 psf
	Green Roof	100 psf
	Elevator Machine Room	250 psf

WIND LOAD

Wind loading has been determined using ASCE7-05 Method II for wind analysis. For this analysis, I will use the projected width and length of the building to estimate the loads. (See Figure 6.1) This assumption will keep wind loads conservative. A simple story elevation has also been provided. (See Figure 6.2)

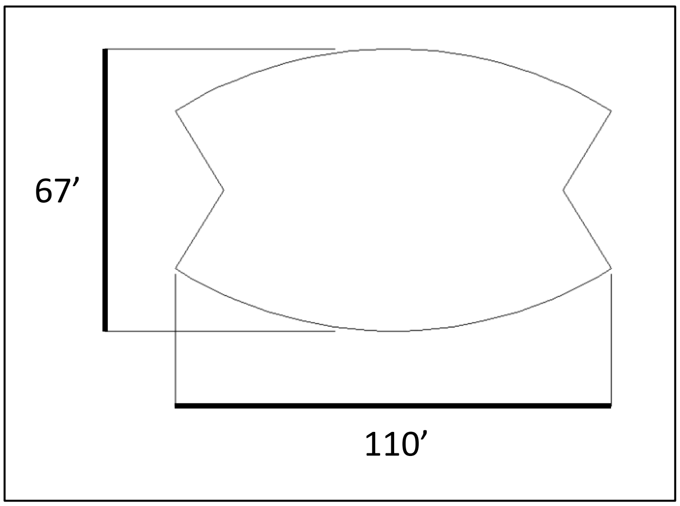


Figure 6.1

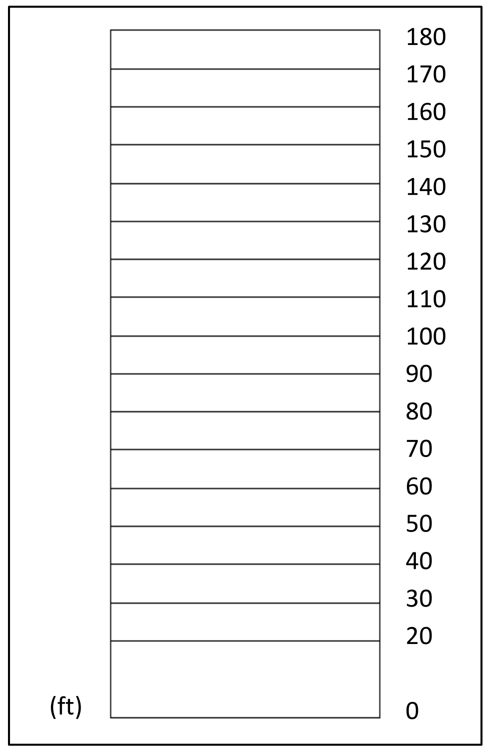


Figure 6.2

The following basic parameters were used for the wind calculations. ASCE references to find each value have been provided.

Wind Speed	V = 90mph	(ASCE7-05 Figure 6-1)
Occupancy Category	II	(ASCE7-05 Table 1-1)
Importance Factor	I = 1.00	(ASCE7-05 Table 6-1)
Exposure	B	(ASCE7-05 § 6.5.6.3)

WIND LOAD *(Continued)*

Velocity Pressure Exposure	K_z = Varies	(ASCE7-05 Table 6-3)
Topographic Factor	K_{zt} = 1.0	(ASCE7-05 Table 6-4)
Wind Directionality Factor	K_d = 0.85	(ASCE7-05 Table 6-6)
Gust Effect Factor	G =0.85	(ASCE7-05 Table 6-6)
Internal Pressure Coefficient	GC_{pi} = ±0.18	(ASCE7-05 Table 6-5)
External Pressure Coefficient	C_p (wind) = 0.8	(ASCE7-05 Table 6-6)
External Pressure Coefficient, N/S	C_p (lee,) = -0.5	(ASCE7-05 Table 6-6)
External Pressure Coefficient, E/W	C_p (lee,) = -0.372	(ASCE7-05 Table 6-6)

North/South Direction L = 67 ft B = 110 ft

Level	Story Ht. (ft)	Ht - z (ft)	Kz	qz	Wind Pressure (psf)		Total Pressure (psf)	Total Force (k)	Total Story Shear (K)	Total Moment (ft-k)
					Windward	Leeward				
17	10	180	1.17	20.622	17.735	-12.476	30.211	33.232	33.232	5815.652
16	10	170	1.150	20.269	17.495	-12.476	29.971	32.969	66.201	5439.822
15	10	160	1.130	19.917	17.255	-12.476	29.732	32.705	98.906	5069.265
14	10	150	1.110	19.564	17.016	-12.476	29.492	32.441	131.347	4703.982
13	10	140	1.090	19.212	16.776	-12.476	29.252	32.178	163.525	4343.973
12	10	130	1.065	18.771	16.476	-12.476	28.953	31.848	195.373	3980.998
11	10	120	1.040	18.331	16.177	-12.476	28.653	31.518	226.891	3624.614
10	10	110	1.015	17.890	15.877	-12.476	28.353	31.189	258.080	3274.822
9	10	100	0.990	17.449	15.578	-12.476	28.054	30.859	288.939	2931.622
8	10	90	0.960	16.921	15.218	-12.476	27.694	30.464	319.403	2589.412
7	10	80	0.930	16.392	14.858	-12.476	27.335	30.068	349.471	2255.111
6	10	70	0.890	15.687	14.379	-12.476	26.855	29.541	379.012	1920.151
5	10	60	0.850	14.982	13.900	-12.476	26.376	29.013	408.025	1595.739
4	10	50	0.810	14.277	13.420	-12.476	25.896	28.486	436.511	1281.873
3	10	40	0.760	13.395	12.821	-12.476	25.297	27.827	464.338	973.941
2	10	30	0.700	12.338	12.102	-12.476	24.578	27.036	491.374	675.896
1	10	20	0.700	12.338	12.102	-12.476	24.578	54.072	545.446	811.075
								Total	545.446	51287.947

WIND LOAD *(Continued)*

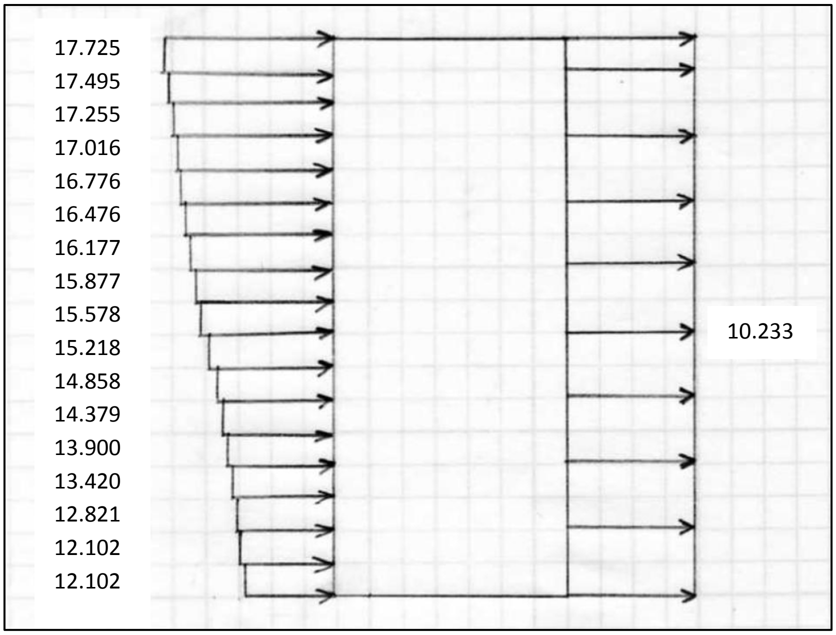
East/West Direction L = 110 ft B = 67 ft

Level	Story Ht. (ft)	Ht - z (ft)	Kz	qz	Wind Pressure (psf) Windward	Wind Pressure (psf) Leeward	Total Pressure (psf)	Total Force (k)	Total Story Shear (K)	Total Moment (ft-k)
17	10	180	1.17	20.622	17.735	-10.233	27.968	18.738	18.738	3279.190
16	10	170	1.150	20.269	17.495	-10.233	27.728	18.578	37.316	3065.308
15	10	160	1.130	19.917	17.255	-10.233	27.488	18.417	55.733	2854.638
14	10	150	1.110	19.564	17.016	-10.233	27.248	18.256	73.989	2647.181
13	10	140	1.090	19.212	16.776	-10.233	27.009	18.096	92.085	2442.935
12	10	130	1.065	18.771	16.476	-10.233	26.709	17.895	109.980	2236.882
11	10	120	1.040	18.331	16.177	-10.233	26.409	17.694	127.674	2034.845
10	10	110	1.015	17.890	15.877	-10.233	26.110	17.494	145.168	1836.822
9	10	100	0.990	17.449	15.578	-10.233	25.810	17.293	162.461	1642.815
8	10	90	0.960	16.921	15.218	-10.233	25.451	17.052	179.513	1449.410
7	10	80	0.930	16.392	14.858	-10.233	25.091	16.811	196.324	1260.823
6	10	70	0.890	15.687	14.379	-10.233	24.612	16.490	212.813	1071.835
5	10	60	0.850	14.982	13.900	-10.233	24.132	16.169	228.982	889.271
4	10	50	0.810	14.277	13.420	-10.233	23.653	15.847	244.829	713.131
3	10	40	0.760	13.395	12.821	-10.233	23.053	15.446	260.275	540.604
2	10	30	0.700	12.338	12.102	-10.233	22.334	14.964	275.239	374.101
1	10	20	0.700	12.338	12.102	-10.233	22.334	29.928	305.167	448.921
								Total	305.167	28788.712

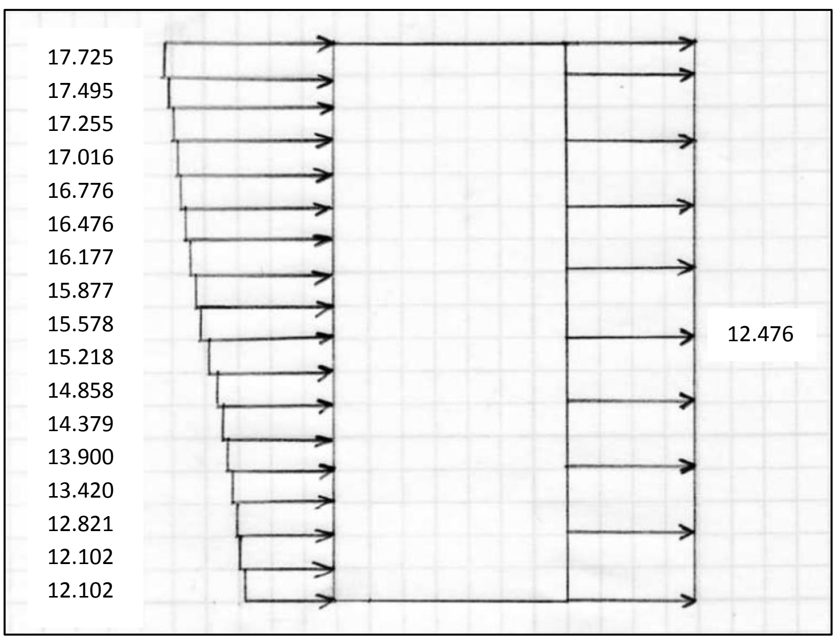
From this data, Millennium Halls wind load base shear is 545 kips in the North/South direction and 305 kips in the East/West. The winds overturning moment in both directions are 51,000 foot-kips and 29,000 foot-kips, respectively. To see sample calculations that were used to generate this data, please see Appendix B. Pressure distributions and story force and shear can be seen below.

WIND LOAD *(Continued)*

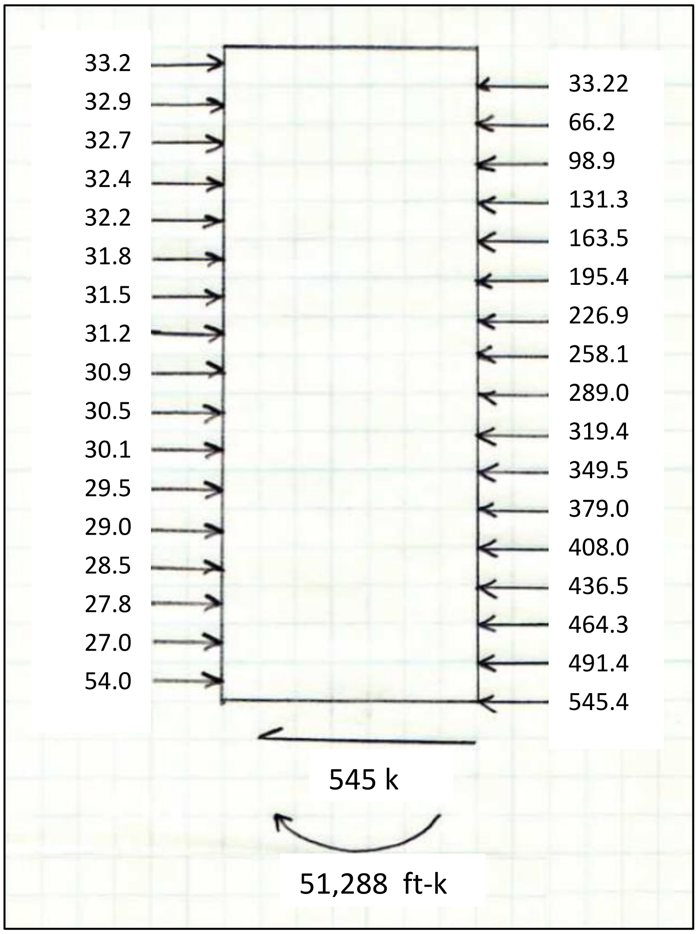
North/South
Wind Pressure
Distribution (psf)



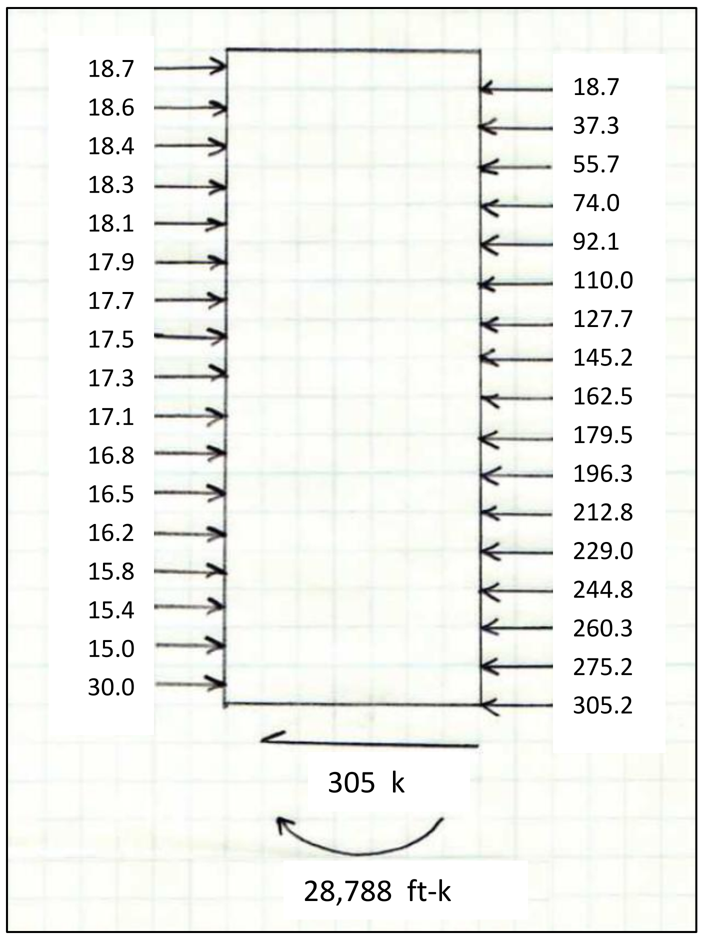
East/West
Wind Pressure
Distribution (psf)



WIND LOAD *(Continued)*



North/South
Story & Base Shear (k)
Overturning Moment (ft-k)



East/West
Story & Base Shear (k)
Overturning Moment (ft-k)

SEISMIC LOAD

Seismic Loading for the Millennium Hall building has been determined using ASCE5-07. All required variables have been listed below. See Appendix C for sample calculations of these values.

Seismic Design (North/South Direction)			
Sym.	Description	Value	ASCE Ref.
Ordinary Concrete Shear Wall			
-	Site Class	C	Table 20.3
-	Occupancy Category	II	Table 1-1
-	Importance Factor	1.00	Table 11-5
S_s	Spectral Response Acceleration, short	0.28	USGS
S_1	Spectral Response Acceleration, 1 sec.	0.06	USGS
F_a	Site Coefficient	1.20	Table 11.4-1
F_v	Site Coefficient	1.70	Table 11.4-2
S_{ms}	MCE Spectral Response Accel., short	0.336	Eq. 11.4-1
S_{m1}	MCE Spectral Response Accel., 1 sec.	0.102	Eq. 11.4-2
S_{ds}	Design Spectral Aceeleration, short	0.224	Eq. 11.4-3
S_{d1}	Design Spectral Aceeleration, 1 sec.	0.068	Eq. 11.4-4
S_{dc}	Seismic Design Category	B	Table 11.6-2
R	Response Modification Coefficent	4.50	Table 12.2-1
C_t	Approximate Period Perameter	0.02	Table 12.8-2
h_n	Building Height	180	-
x	Approximate Period Perameter	0.75	Table 12.8-2
C_u	Calculated Period Upper Limit	1.70	Table 12.8-1
T_a	Approximate Fundamental Period	0.95	Eq. 12.8-7
T	Fundamental Period	4.00	§12.8.2
T_L	Long Period Transistion Period	6.00	Figure 22-15
Cs	Seismic Response Coefficient	0.016	Eq. 12.8-6
k	Structural Period Exponent	2	§12.8.3

SEISMIC LOAD *(Continued)*

Seismic Design (East/West Direction)			
Sym.	Description	Value	ASCE Ref.
Ordinary Concrete Moment Frame			
-	Site Class	C	Table 20.3
-	Occupancy Category	II	Table 1-1
-	Importance Factor	1.00	Table 11-5
S_s	Spectral Response Acceleration, short	0.28	USGS
S_1	Spectral Response Acceleration, 1 sec.	0.06	USGS
F_a	Site Coefficient	1.20	Table 11.4-1
F_v	Site Coefficient	1.70	Table 11.4-2
S_{ms}	MCE Spectral Response Accel., short	0.336	Eq. 11.4-1
S_{m1}	MCE Spectral Response Accel., 1 sec.	0.102	Eq. 11.4-2
S_{ds}	Design Spectral Aceeleration, short	0.224	Eq. 11.4-3
S_{d1}	Design Spectral Aceeleration, 1 sec.	0.068	Eq. 11.4-4
S_{dc}	Seismic Design Category	B	Table 11.6-2
R	Response Modification Coefficent	3.00	Table 12.2-1
C_t	Approximate Period Perameter	0.016	Table 12.8-2
h_n	Building Height	180	-
x	Approximate Period Perameter	0.90	Table 12.8-2
C_u	Calculated Period Upper Limit	1.70	Table 12.8-1
T_a	Approximate Fundamental Period	1.65	Eq. 12.8-7
T	Fundamental Period	2.60	§12.8.2
T_L	Long Period Transition Period	6.00	Figure 22-15
Cs	Seismic Response Coefficient	0.014	Eq. 12.8-6
k	Structural Period Exponent	2	§12.8.3

SEISMIC LOAD *(Continued)*

Base Shear & Overturning Moment (North/South Direction)							
Story	hx (ft)	Story Wt. (k)	$w_x h_x^k$	C_{vx}	Lateral Force (k)	Story Shear (k)	M_x (ft-k)
17	180	1610	8.4E+10	0.124	58.8	58.8	10282.17
16	170	1828	9.7E+10	0.142	67.6	126.3	11147.67
15	160	1828	8.6E+10	0.126	59.8	186.2	9276.28
14	150	1828	7.5E+10	0.111	52.6	238.8	7626.98
13	140	1828	6.5E+10	0.096	45.8	284.6	6185.75
12	130	1828	5.6E+10	0.083	39.5	324.1	4938.55
11	120	1828	4.8E+10	0.071	33.7	357.8	3871.35
10	110	1828	4.0E+10	0.060	28.3	386.0	2970.14
9	100	1828	3.3E+10	0.049	23.4	409.4	2220.88
8	90	1828	2.7E+10	0.040	18.9	428.4	1609.56
7	80	1828	2.1E+10	0.031	15.0	443.3	1122.13
6	70	1828	1.6E+10	0.024	11.5	454.8	744.58
5	60	1828	1.2E+10	0.018	8.4	463.2	462.88
4	50	1828	8.4E+09	0.012	5.8	469.0	263.00
3	40	1828	5.3E+09	0.008	3.7	472.8	130.92
2	30	1828	3.0E+09	0.004	2.1	474.9	52.60
1	20	658	1.7E+08	0.000	0.1	475.0	1.21
	Total	29688	6.8E+11			475	62907

SEISMIC LOAD *(Continued)*

Base Shear & Overturning Moment (East/West Direction)							
Story	hx (ft)	Story Wt. (k)	$w_x h_x^k$	C_{vx}	Lateral Force (k)	Story Shear (k)	M_x (ft-k)
17	180	1610	8.4E+10	0.124	51.46	51.46	9005.02
16	170	1828	9.7E+10	0.142	59.17	110.63	9763.01
15	160	1828	8.6E+10	0.126	52.41	163.04	8124.07
14	150	1828	7.5E+10	0.111	46.07	209.11	6679.63
13	140	1828	6.5E+10	0.096	40.13	249.24	5417.41
12	130	1828	5.6E+10	0.083	34.60	283.84	4325.13
11	120	1828	4.8E+10	0.071	29.48	313.32	3390.49
10	110	1828	4.0E+10	0.060	24.77	338.09	2601.22
9	100	1828	3.3E+10	0.049	20.47	358.57	1945.03
8	90	1828	2.7E+10	0.040	16.58	375.15	1409.63
7	80	1828	2.1E+10	0.031	13.10	388.25	982.75
6	70	1828	1.6E+10	0.024	10.03	398.29	652.10
5	60	1828	1.2E+10	0.018	7.37	405.66	405.38
4	50	1828	8.4E+09	0.012	5.12	410.78	230.33
3	40	1828	5.3E+09	0.008	3.28	414.05	114.65
2	30	1828	3.0E+09	0.004	1.84	415.89	46.07
1	20	658	1.7E+08	0.000	0.11	416.00	1.06
	Total	29688	6.8E+11			416	55093

From this analysis we can see the design seismic base shears for the North/South and East/West directions are 475 kips and 416 kips respectively. These values differ slightly from the given design base shears of 362 kips and 320 kips. This most likely comes from error in the preliminary approximations for the buildings self weight. Sample calculations for these values can be found in Appendix C.

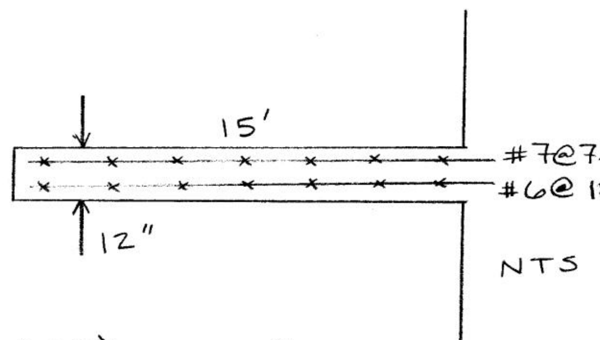
SPOT CHECK CALCULATIONS

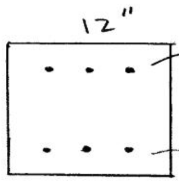
These simple calculations examine the strength for each of the members listed. Reinforcement sizes have been verified to provide the required strength.

TECH #1 | SPOT CHECK CALCS. 1/3

CANTILEVERED SLAB

I WILL ANALYZE THIS USING AN EQUIVALENT 12" WIDE DOUBLY REINF. BEAM FOR NEGATIVE BENDING MOMENT @ FACE OF SUPPORT





$A_s = (0.6)\left(\frac{12}{7.5}\right) = 0.96\ in^2$

$A_s' = (0.44)\left(\frac{12}{18}\right) = 0.293\ in^2$

$$0.85 f'_c b \beta_1 c + E_s \frac{0.003}{c}(c - d')A_s' = A_s F_y$$

$$(43.35)c + \frac{435}{c}(c - 1.5) = 57.6$$

$$(43.35)c^2 + (377.4)c - (652.5) = 0$$

$$c = 1.51''$$

$$\phi M_n = 0.9\left[0.85(5)(12)(0.85)(1.51)\left(10.5 - \frac{0.85(1.51)}{2}\right) + (0.293)(0.576)(10.5 - 1.5)\right]$$

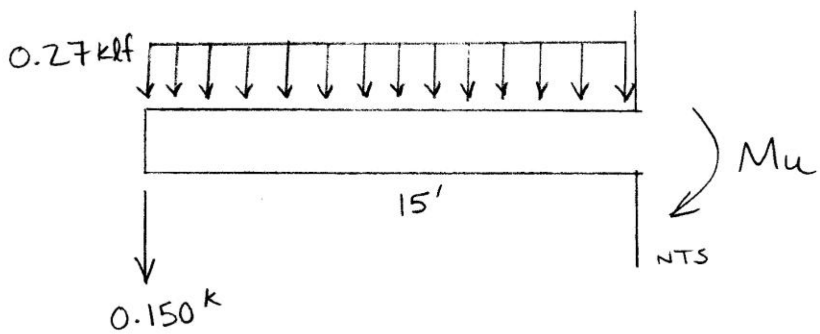
$$\phi M_n = 582\ ^{\prime k}/\text{FT WIDTH}$$

SPOT CHECK CALCULATIONS

TECH #1 SPOT CHECK CALCS. 2/3

		CANTILEVER	INTERIOR
SLAB	$(145)(1')(1') =$	145 plf	169 plf
L.L	$(40)(1') =$	40 plf	100 plf
PARTITIONS	$(20)(1') =$	20 plf	20 plf
MEP	$(15)(1') =$	15 plf	15 plf
TOPPING	$(50)(1') =$	50 plf	50 plf
		0.27 klf	0.354 klf

FACADE $(150\ plf)(1') = 0.150\ k$



$$M_u = \frac{w l^2}{2} + p l = \frac{0.27(15)^2}{2} + 0.15(15) = 32.6'^k$$

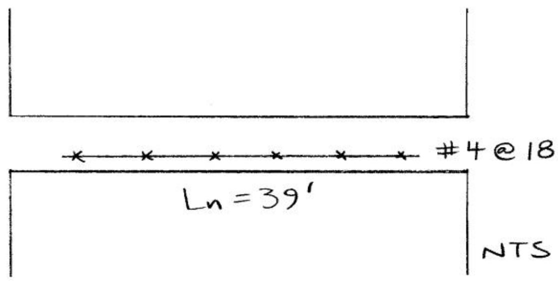
$$M_u = 32.6'^k < \phi M_n = 582'^k$$

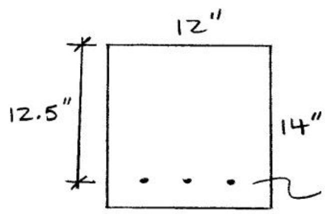
$\therefore$ OK

SPOT CHECK CALCULATIONS

INTERIOR SLAB

I WILL ANALYZE THIS SIMILAR TO BEFORE BUT WILL VERIFY BOTTOM STEEL RESISTING POSITIVE MOMENT





$$A_s = (0.2)\left(\frac{12}{18}\right) = 0.133\ in^2$$

$$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{(0.133)(60)}{0.85(5)(12)} = 0.156''$$

$$\phi M_n = A_s F_y (d - a/2) = (0.133)(60)\left(12.5 - \frac{0.156}{2}\right)(0.9)$$

$$\phi M_n = 89.2'^k$$

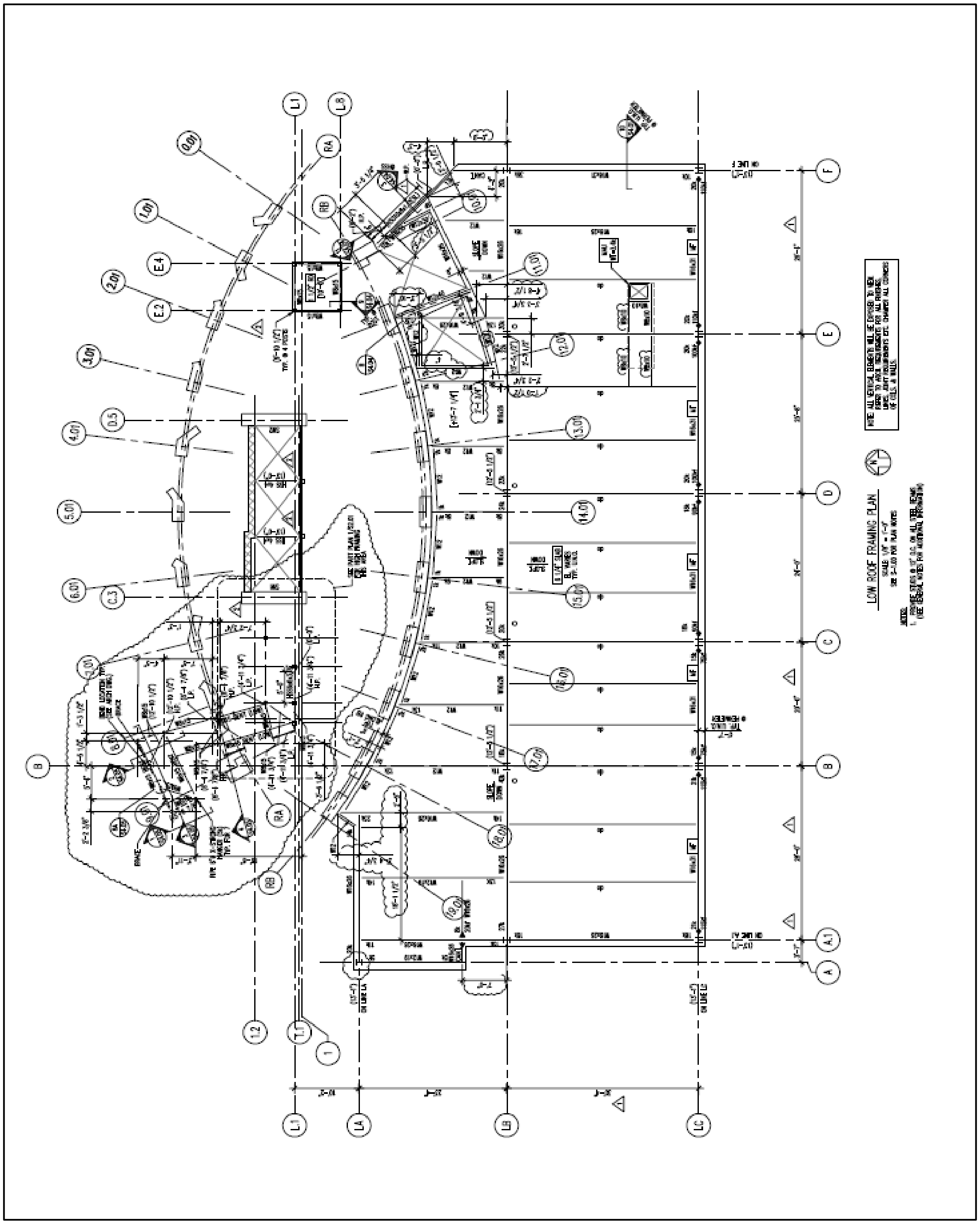
SEE PREVIOUS PAGE

$$M_u = \frac{w l^2}{8} = \frac{0.35(39)^2}{8} = 67.3'^k$$

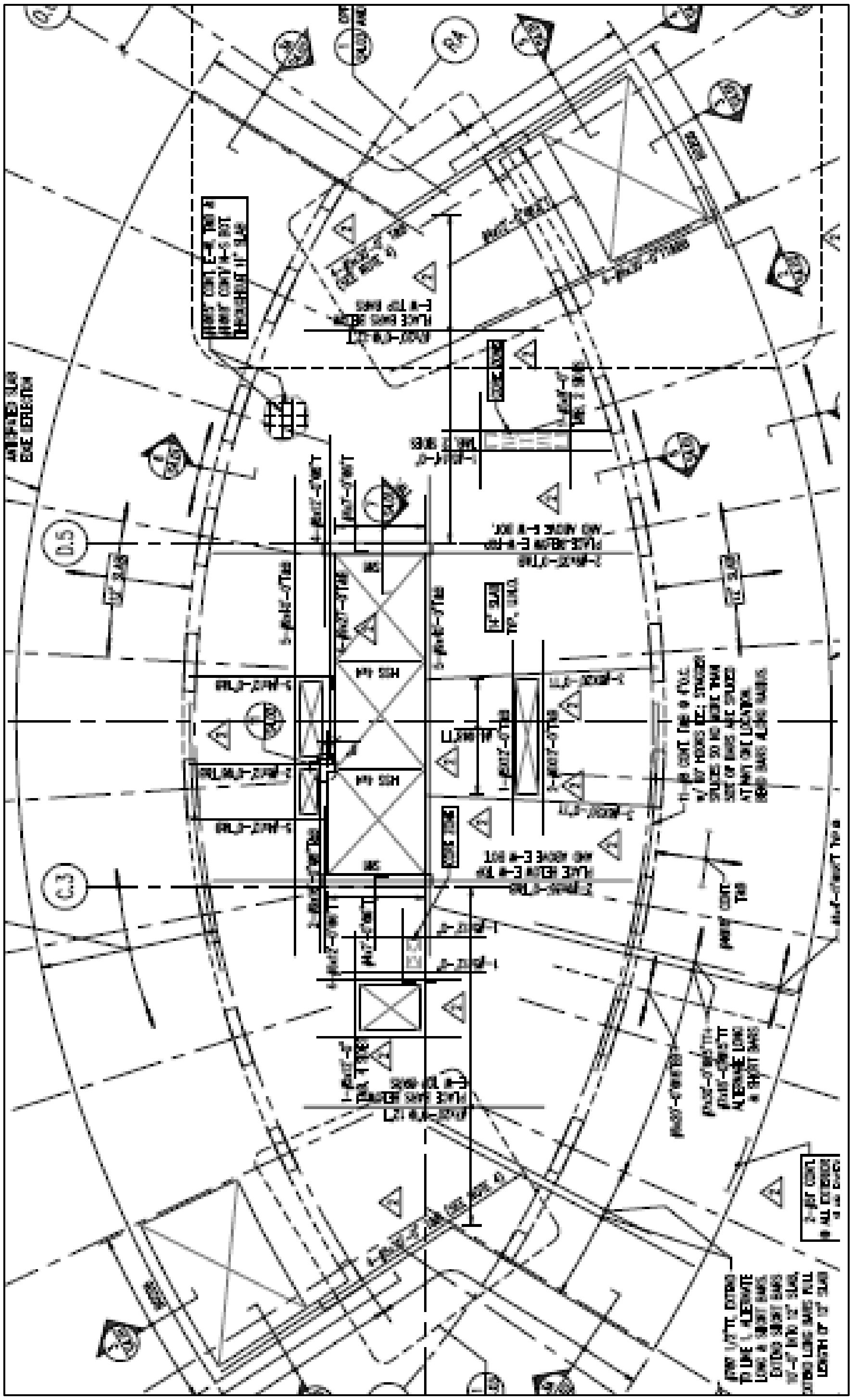
$$\phi M_n = 89.2'^k > M_u = 67.3'^k$$

$\therefore$ OK

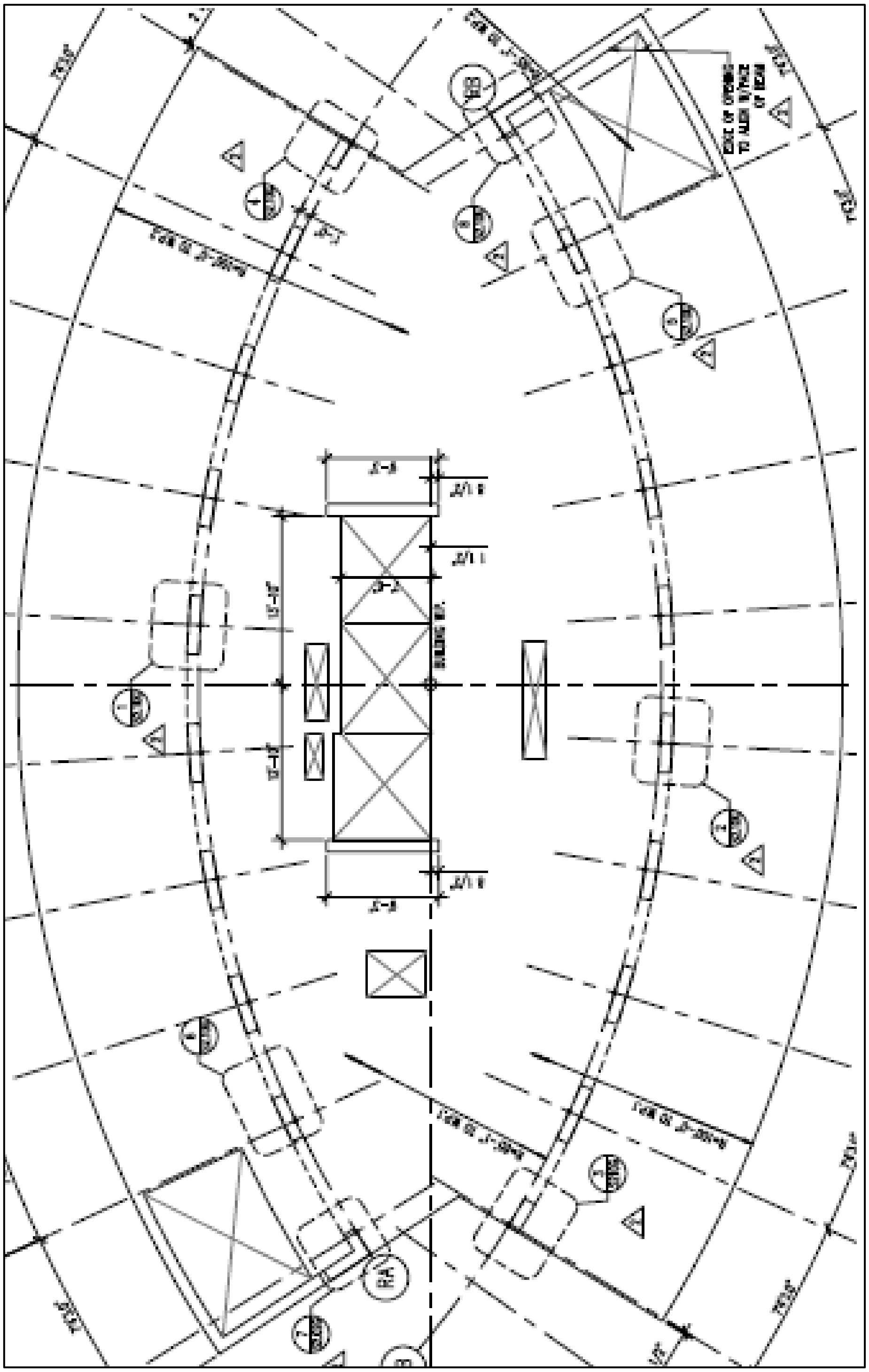
APPENDIX A: *Ground Floor Steel Framing Plan*



APPENDIX A: *Typical Floor Framing Plan*



APPENDIX A: *Typical Floor Column Layout*



APPENDIX B: *Wind Analysis*

TECH #1 WIND CALCULATIONS 1/2

$K_Z \Rightarrow$ FROM TABLE, SOME INTERPOLATED

EX. FLOOR 16 : $y_2 = \dfrac{(170-160)(1.17-1.13)}{(180-160)} + 1.13$

$y_2 = 1.15$

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$

EX. FLOOR 2: $0.00256(0.7)(1.0)(0.85)(90)^2(1.0) = 12.338$

$\bar{z} = 180'$

$q_h = 0.00256(1.17)(1.0)(0.85)(90)^2(1.0) = 20.622$

C_p WINDWARD $= 0.8$

C_p LEEWARD $= -0.5$ N/S

C_p LEWARD $= -0.372$ E/W

$GC_{pi} = \pm 0.18$

$P_z = q_z G C_p - q_h G C_{pi}$ WINDWARD

$P_h = q_h G C_p - q_h G C_{pi}$ LEEWARD

EX. FLOOR 5: $p_z = (14.982)(0.85)(0.8) - (20.622)(-0.18)$

$= 13.900$ psf

APPENDIX B: *Wind Analysis*

TECH #1 | WIND CALCULATIONS | 2/2

EX. FLOOR 5: $p_h = (20.622)(0.85)(-0.5) - (20.622)(0.18)$

$= -12.476\ psf\ (N/S)$

$p_h = (20.622)(0.85)(-0.372) - (20.622)(0.18)$

$= -10.233\ psf\ (E/W)$

TOTAL PRESSURE $= |p_z| + |p_h|$

EX. FLOOR 8: $|15.218| + |-12.476| = 27.694\ psf\ (N/S)$

TOTAL FORCE $= T_p \times$ AREA

EX. FLOOR 10: $28.353\,(10' \times 110')/1000 = 31.189\ K$

TOTAL SHEAR $= T_{F\ \text{OF LEVEL}} + T_{F\ \text{LEVELS ABOVE}}$

EX. FLOOR 16: $(32.969) + (33.232) = 66.201\ K$

TOTAL MOMENT FROM LEVEL $= T_F \times$ MEAN FLOOR HT.

EX. FLOOR 12: $(31.848) \times \left(\frac{130+120}{2}\right) = 3980.998\ ^{\prime K}$

APPENDIX C: Seismic Analysis

Building Self Weight

Ground Floor (Story 1)			
SLAB (SF)			
12" Thick		14" Thick	
	Do Not Include		Do Not Include
	Slab Self Wt.		Slab Self Wt.
0 k		0 k	
VERTICAL CONCRETE			
Column		Shear Wall	
9.17	Col. (Typ)	16.5	Shear Wall
x 20	Number of Col.	x 2	Number of Walls
x 145	Reinf. Concrete	x 8.83	Wall Height
x 8.83	Column Height		
235 k		291 k	
FAÇADE			
	Perimeter =	312	(ft)
		x 150	Façade
		47 k	
SUPERIMPOSED D.L.			
	Suspended MEP	15	
		x 5688	Gross Area
		85 k	
TOTAL STORY WEIGHT			
0 + 0 + 235 + 291 + 47 + 85 =		658 k	

APPENDIX C: Seismic Analysis

Typical Floor (Stories 2 - 16)			
SLAB (SF)			
12" Thick		14" Thick	
3076	Gross Area	2612	Gross Area
-117	Openings	-247	Openings
2959 x 145	Reinf. Concrete	2365 x 145	Reinf. Concrete
429 k		343 k	
VERTICAL CONCRETE			
Column		Shear Wall	
9.17	Col. (Typ)	16.5	Shear Wall
x 20	Number of Col.	x 2	Number of Walls
x 145	Reinf. Concrete	x 8.83	Wall Height
x 8.83	Column Height		
235 k		291 k	
FAÇADE			
	Perimeter =	312	(ft)
		x 150	Façade
		47 k	
SUPERIMPOSED D.L.			
	Partitions	20	
	Suspended MEP	15	
	Slab Topping	50	
		85 x 5688	Gross Area
		483 k	
TOTAL STORY WEIGHT			
	429 + 343 + 235 + 291 + 47 + 483 =	1828 k	

APPENDIX C: Seismic Analysis

Top Floor/Roof (Story 17)			
SLAB (SF)			
12" Thick		14" Thick	
3076	Gross Area	2612	Gross Area
-117	Openings	-247	Openings
2959 x 145	Reinf. Concrete	2365 x 145	Reinf. Concrete
429 k		343 k	
VERTICAL CONCRETE			
Column		Shear Wall	
9.17	Col. (Typ)	16.5	Shear Wall
x 20	Number of Col.	x 2	Number of Walls
x 145	Reinf. Concrete	x 8.83	Wall Height
x 8.83	Column Height		
235 k		291 k	
FAÇADE			
	Perimeter =	220	(ft)
		x 150	Façade
		33 k	
SUPERIMPOSED D.L.			
	Partitions	20	
	Suspended MEP	15	
	Slab Topping	50	
		85 x 2612	Gross Area
	Roofing	10 x 5688	
		279 k	
TOTAL STORY WEIGHT			
429 + 343 + 235 + 291 + 33 + 279 =		1610 k	

APPENDIX C: Seismic Analysis

TECH #1 SEISMIC CALCS. 1/2

EX. N/S

$S_{MS} = F_a S_s = 1.2(0.28) = 0.336$

$S_{M1} = F_v S_1 = 1.7(0.06) = 0.102$

$S_{DS} = 2/3\, S_{MS} = 2/3\,(0.336) = 0.224$

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

$T_a = C_x h_n^{x} = 0.02(180)^{0.75} = 0.95s$

$T = T_a C_u = (0.95)(1.7) = 4.00$

TOTAL BLDG WT. $= 658 + 15(1828) + 1610 = 29{,}688$ K

BASE SHEAR $(V) = C_s W_T$

$V = 0.016(29{,}688) = 475$ K

$W_x h_x^{k}$

EX. LEVEL 2: $[(30)(1828)]^2 = 3.0 \times 10^9$

$C_{vx} = \dfrac{W_x h_x^{k}}{\Sigma W_x h_x^{k}}$ $\Sigma W_x h_x^{k} = 6.8 \times 10^{11}$

EX. LEVEL 2: $\dfrac{3.0 \times 10^9}{6.8 \times 10^{11}} = 0.004$

APPENDIX C: Seismic Analysis

TECH #1 SEISMIC CALCS. 2/2

$F_x = C_{vx} V$

EX. LEVEL 9: $(0.049)(475) = 23.4 \text{ k}$

SHEAR STORY = $F_{x \text{ OF LEVEL}} + \sum F_{x \text{ LEVELS ABOVE}}$

EX. LEVEL 16: $(67.6) + (58.8) = 126.3 \text{ k}$

$M_x = F_x \cdot$ MEAN FLOOR HT

EX. LEVEL 3: $(3.7)\left(\frac{40+30}{2}\right) = 130.92 \text{ 'k}$