Temple University

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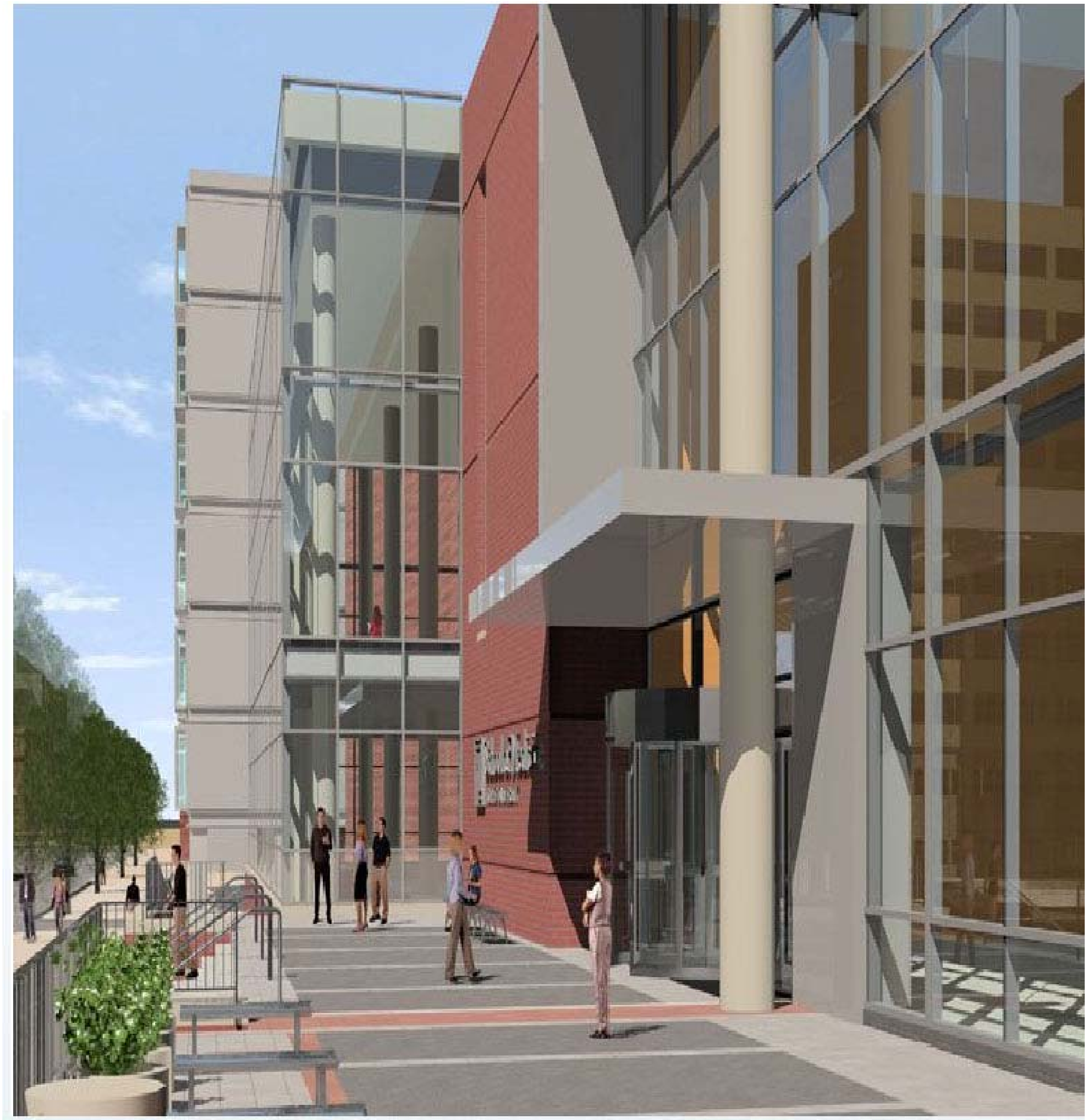

[TECHNICAL ASSIGNMENT 1]

3500 N. Broad St. Philadelphia, Pennsylvania 19140

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Executive Summary

The purpose of this report is to investigate the design of the Temple University Multipurpose Health Science Center's structural system. This summary is followed by a detailed structural description, wind and seismic lateral analysis, and spot checks for the braced frame, typical column, and typical beam. Design guides included the IBC2006, AISC Steel Construction Manual, ASCE7-05, and United Steel Deck design manual.

The Philadelphia site is located on a previous parking lot with intact bedrock located at about 50' boring depths. The building itself ties to preexisting campus buildings via a bridge and tunnel. The foundation consists of roughly 40% shallow foundations with 1'4" to 2'8" depths and 60% caissons with 15' to 35' depths. The superstructure is steel frame construction with W14 and W12 columns averaging about 100lbs and 30' in height. Beams and girders are typically W21 or W24 averaging about 50 to 100lbs. 2.5" slabs on 3" deep, 20 gage galvanized composite steel deck form the floor system. Braced frames providing lateral resistance are comprised of widely varying sized W-shapes and L-shapes.

The Equivalent Lateral Force Procedure was used for the seismic analysis with a resulting base shear equal to 1020k. The site classification used by the original designers was C; however, the USGS website's seismic program yielded a site classification B. I based my seismic calculations off of the site C classification.

The Method 2 – Analytical Procedure was used to determine the wind loads, with the sum equal to 2098k in the East-West direction. Since this number was so much higher than the seismic base shear, wind loading was combined with dead loads in a SAP 2000 model of the braced frame. The member axial forces obtained by the model were very close to those obtained from the original design. Variations were most likely due to how the frame was modeled. A secondary comparison was made by checking the first floor frame girder, which came out to be a conservative and efficient size.

Although the building shape varies considerably, a fairly typical bay was picked to perform the spot check of for a column and beam. The beam was analyzed for flexure and serviceability, with deflection controlling. The size I obtained then matched the size obtained by the original designers. The column size I obtained was significantly off compared to the original which was due to a lack of information concerning penthouse HVAC units and my simplification to not include wind loads.

Structural System - Foundation

General

The geotechnical survey justified a hybrid foundation system for the site. The upper layer of soil, between 19'to 35', consists of medium to very compact micaceous silty fines to coarse sands and varying gravel. Deeper soils, between 24' to 50', consist of more compact micaceous silty fines to coarse sands and gravel with borings terminating at intact mica bedrock. The building's excavation is between 78' to 83' with street level at approximately 100', placing the majority of the foundation between these two layers.

The expected column loadings are around 3,100 kips for the braced frame columns and about 1,000 kips for the majority of the columns. The higher bearing capacity of the lower layer of soil coupled with the required bearing of the capacity of the columns justified a hybrid system with braced frame columns resting on caissons.

Figure 1: View of structural systems

The concrete used is 28-day, normal weight concrete at f'c=4000 psi for most areas, with the primary exception being concrete exposed to weather-for example, the truck ramp- which should be air-entrained, normal weight at f'c=5000. Reinforcing is grade 60.

Slab

The typical basement slab consists 6" of concrete over a vapor barrier and 4" of crushed stone, with 6"x6" W4.0xW4.0 WWF. The primary areas where exceptions occur are underneath the library, mechanical and electrical equipment, the loading docks, and areas underneath the auditorium. Slab thicknesses in these areas are either 8" or 12".

Footings

The shallow foundation system consists of steel columns sitting on concrete piers and footings, which are connected by grade beams. Footing thickness ranges from 1'4" to 4'4", with most in the 1'10" to 2'4" range. Sizes generally range from 4'x4' to 9'x9'.

Caissons

The deep foundation system consists of steel columns sitting on concrete piers, caps and caissons. Sixty-six of the one-hundred thirteen basement columns rest on these caissons, which vary in diameter from 36" to 96". The top of the basement slab is at either 78' or 83' elevation, with caisson estimated bearing elevations ranging between 45' to 70', with the most around 60'.

Structural System – Columns

The framing system consists primarily of ASTM A992 Grade 50 rolled W-shapes with depths of 12" and 14". There are several 10" deep W-shapes in the basement through fourth floors and some HSS shapes in the auditorium. Sizes vary greatly with upper floor columns in the 100-120lb range, and lower floor columns in the 200lb range. The columns are spliced 4' above floor level and span two floors with lengths typically at 25' to 30'. A detailed floor by floor breakdown of typical column sizes can be found in the appendix.

Structural System – Floor System

Given the irregularities of the buildings shape, I decided to describe the framing system by dividing up the building into typical areas, which are schematically represented in figure 2 to the right. A simplified framing plan can be seen in figure 3 on the next page. Floor systems for the various areas are then described, with a detailed floor by floor breakdown of beam and girder sizes available in the appendix.

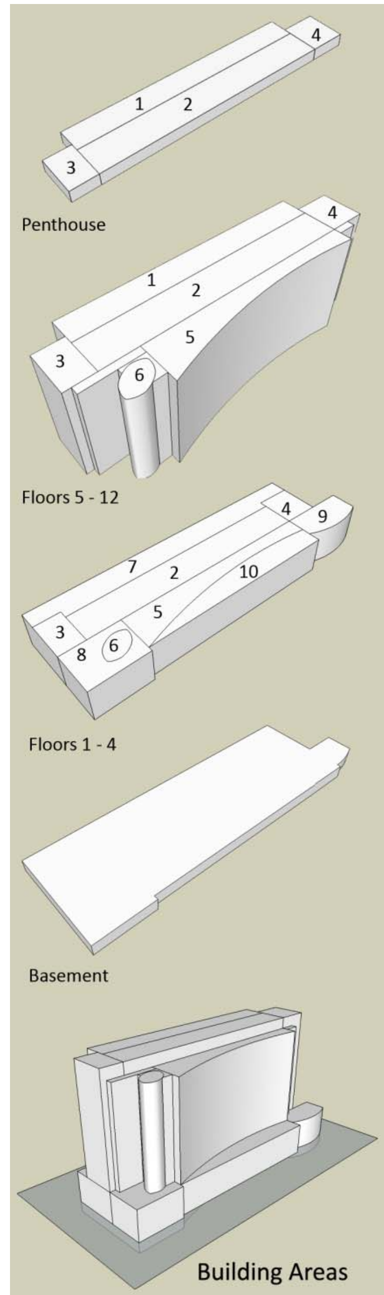


Figure 2: Building Areas

Slabs are typically 2.5" NWC on 3"deep, 20 gage, galvanized composite steel deck, with 6x6-W2.9xW2.9 WWF. The primary exception is penthouse mezzanine and roof level, where the slab is thinner.

This building also has three transfer trusses which take column point loads from above and redistribute them to offset columns at a lower level. Two of these trusses are located between the first and second floors, are 15'4" deep, and span 46.5' in order to clear space for the loading dock below. A third truss is located between the 5th and 6th floors, is 14'8" deep, and spans 62' in order to relocate columns for corridors on lower levels.

Area 1 typically has 25'x31' bays with beam sizes ranging from W12x14 to W21x14, with the most common size being the W21x14 and W18x40. The most common girder size is W24x68 and W21x50.

Area 2 contains an elevator core and riser openings. It typically has 38'x31'with beam sizes in the range of W24x44 to W24x94 spanning girders of a similar size.

Areas 3 and 4 contain greatly varying framing sizes due to openings. Area 3 contains openings for mechanical equipment and stairwells, while area 4 also contains an elevator core.

Area 5 contains the framing for the dramatic curved east façade. The curve itself is composed mostly of W21x44 or W24's members of various sizes with the curved bays typically spanned by W12x19's. Longer spans range from W14x22 to W24x84.

Area 6 is the oval tower, which is framed by a hexagon of W12 and 16 girders and beams. C shapes round out the shape of the oval. At the 4th floor and below, this area frames into area 8 which member sizes ranging from W14-W24.

At the 4th floor and below, area 1 becomes the larger area 7, with 25'x31' bays with W18x40 beams spanning W24x55 girders.

Area 9 is the auditorium with 44LH14 shapes spanning curved walls of W16,18 and 21 girders to form the roof deck. The floor is framed by sloped W30x90 beams for the seating area and W16 girders underneath the stage.

Area 10 is the atrium space with, which extends from the curved façade to form a straight edge facing the street. Beams varying from W16 to W24x68 span the curve girders to the straight W24x55 girders for the floor and roof.

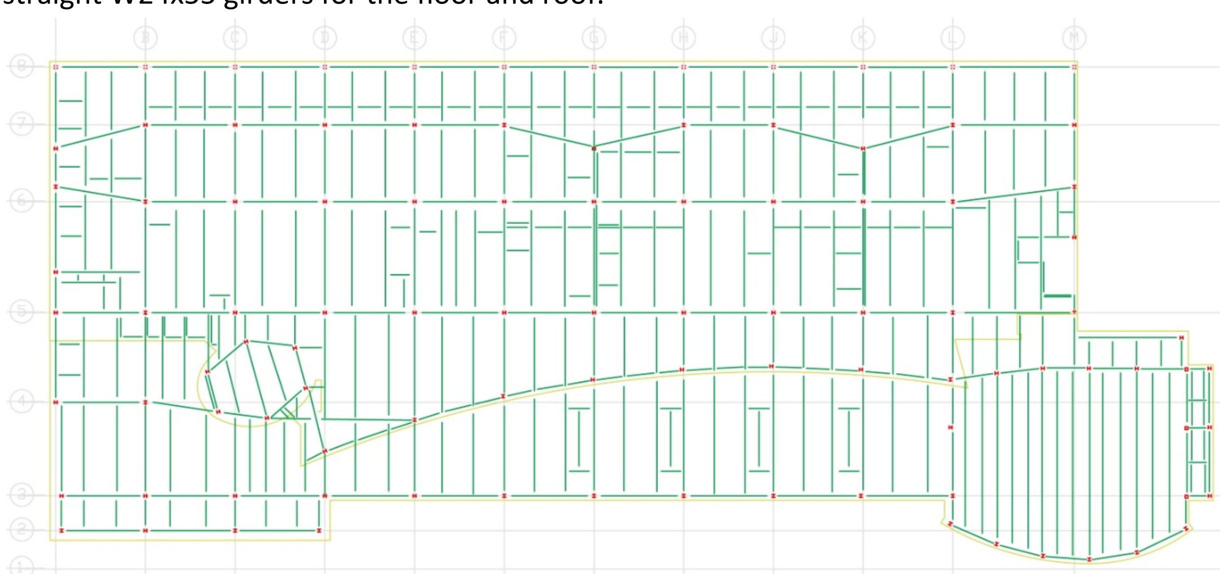


Figure 3: Simplified Framing Plan

Structural System – Lateral System

Due to the slender shape of the building lateral resistance is primarily needed in the East-West direction. This resistance is provided by four sets of braced frames which run the full height of the building. A review of detailed drawings of the connections did not indicate the use of moment connections. The vertical members range from W14x109 at the top to W14x550 at the bottom. Horizontal members are typically W24x55 but range from W21x44 to W27x161. Diagonal members range from W10x49 within the upper four floors to W12x190 at the bottom.

Three sets of North-South braced frames appear from the 12th, 13th mezzanine, and 13th penthouse levels in one line, with an additional set appearing in another line for only two levels. The member sizes are similar with the exception that diagonal members are comprised of 5x5L shapes.

Codes Applied

Below are listed the codes used by the original designers.

- IBC 2003 (Philadelphia building code)
- ASCE7-02
- Concrete:
  - ACI 318 “Building Code Requirements for Structural Concrete”
  - ACI 316 “Manual of Standard Practice for Detailing Concrete Structures”
  - ACI 301, 302, 304, 305, 306, 308, 311, 318, 347
- Steel:
  - AISC “Specifications for Design, Fabrication and Erection of Structural Steel for Buildings”
  - AISC “Code of Standard Practice for Steel Buildings and Bridges”
  - American Welding Society (AWS) D1.1 “Structural Welding Code – Steel.”
  - American Welding Society (AWS) D1.1 “Structural Welding Code – Steel.”
  - ASTM A6 “ General Requirements for Rolled Steel Plates, Shapes, Sheet Piling, and Bars for Structural Use.”
  - ASTM A325 “Specifications for Structural Joints”
  - Steel Deck Institute “Design Manual for Composite Decks, Form Decks, and Roof Decks”

For my design and analysis I used IBC 2006 and ASCE7-05

Loads: Live & Dead

The loads in tables 1 and 2 were determined by reviewing the building documents and noting the loads used by the original designers, who based their loading off of the IBC 2003, the adopted building code of Philadelphia, Pennsylvania.

Design dead loads, found in table 3 were not presented in the building documents, so material unit weights and ASCE 7-05 Minimum Design Dead Loads were used to make dead load assumptions. The calculations for these loads are available in the appendix.

Table 1: Live Loads

Area	Load (psf)
Slab on Grade	150
Truck Drive Aisle	300
High Density Storage Area	300
Elevated Frame Slabs	150
Office/corridor	100
Library	150
Roof	30
Penthouse	150

Table 2: Snow Loads

Flat-roof snow load	22 psf
Snow Exposure Factor	0.9
Snow Load Importance Facotr	1.1
Thermal Factor	1.0

Table 3: Dead Loads

	Load (psf)
Decking	50.1
Girders & Beams	7
Subtotal	60
Mech/Elec	20
Partitions	8
Ceiling	1
Floor	1
Total	85

Lateral Loads - Seismic

The ASCE7-05 code was used to investigate the seismic loads for the building which were expected to be relatively low, given the building's location in Philadelphia, Pennsylvania. The Equivalent lateral Force Procedure (12.8.2) was used to obtain a base sheer for the building. A direct comparison with the structural notes within the design documents was not possible since the base shear was not indicated in the notes or specifications; however, the notes did state that structural system was not specifically detailed for seismic loads. It also provided various seismic data including the site class, C, and the spectral response coefficients.

In my analysis, I initially used the USGS Earthquake Ground Motion Parameter Java Application (See notes in appendix for website, and figures 9 and 10 for the program results) to determine my site class and coefficients. After inputting latitude and longitude data, the program gave a site class B, which conflicted with the structural notes site class C classification. For my analysis I used the latter since it is more conservative and is provided by the actual geotechnical report. For the building weight I used my estimated dead weight of 85 psf.

I determined my lateral system to be an Ordinary Steel Concentrically Braced Frame, without moment connections. This was determined by reviewing the braced frame connection details, which had only sheer connections.

Although a direct comparison of the seismic base sheer was not possible, the value I obtained, V= 1020k is significantly lower than the base sheer value I obtained through my wind analysis, which was equal to 2098k in the East-West direction. In the North-South direction this value was roughly estimated to be 710k. This is somewhat low compared to the seismic value; however, three aspects make it still possible to assume that wind loading controls. The first is that the building has many bays and in the N-S direction which adds to its lateral resistance, secondly it is a relatively quiet seismic region, and lastly the site classification C was a conservative assumption. In sum, my results are in line with the original designers, and back up their decision to not design a lateral system based on seismic activity.

Lateral Loads – Wind

Wind lateral loads were based off of the ASCE7-05-6.5 Analytical Procedure, since the building's height exceeded sixty feet. Simplified results are presented on the following page with a wind diagram (Figure 4), and Table 4, which shows varying height of floors, windward and leeward loads, and the total loads. Detailed calculations are presented in the appendix. An excel spread sheet follows and completes the hand calculations, providing the wind pressures for the various heights (Table 6). The diagram which follows the table was used to simplify tributary area calculations (Figure 11), which was used to obtain the final wind forces in Table 7 and Figure 4. The total combines the maximum of the windward pressure and the maximum possible leeward pressure, rather than using the base values. Table 5 provides the same data for the braced frame analyzed in the spot check. Figure 4 is the wind loading diagram.

The basic wind speed value, wind importance factor, exposure category, and internal pressure coefficients obtained by code analysis matched those of the original designers. The building is fully enclosed, with the calculated building frequency indicating a flexible structure.

Since the total shear value of wind was nearly double that of seismic, a wind analysis of the lateral system was performed which provided an opportunity to compare my wind analysis with the original designers. This is discussed in the next section.11

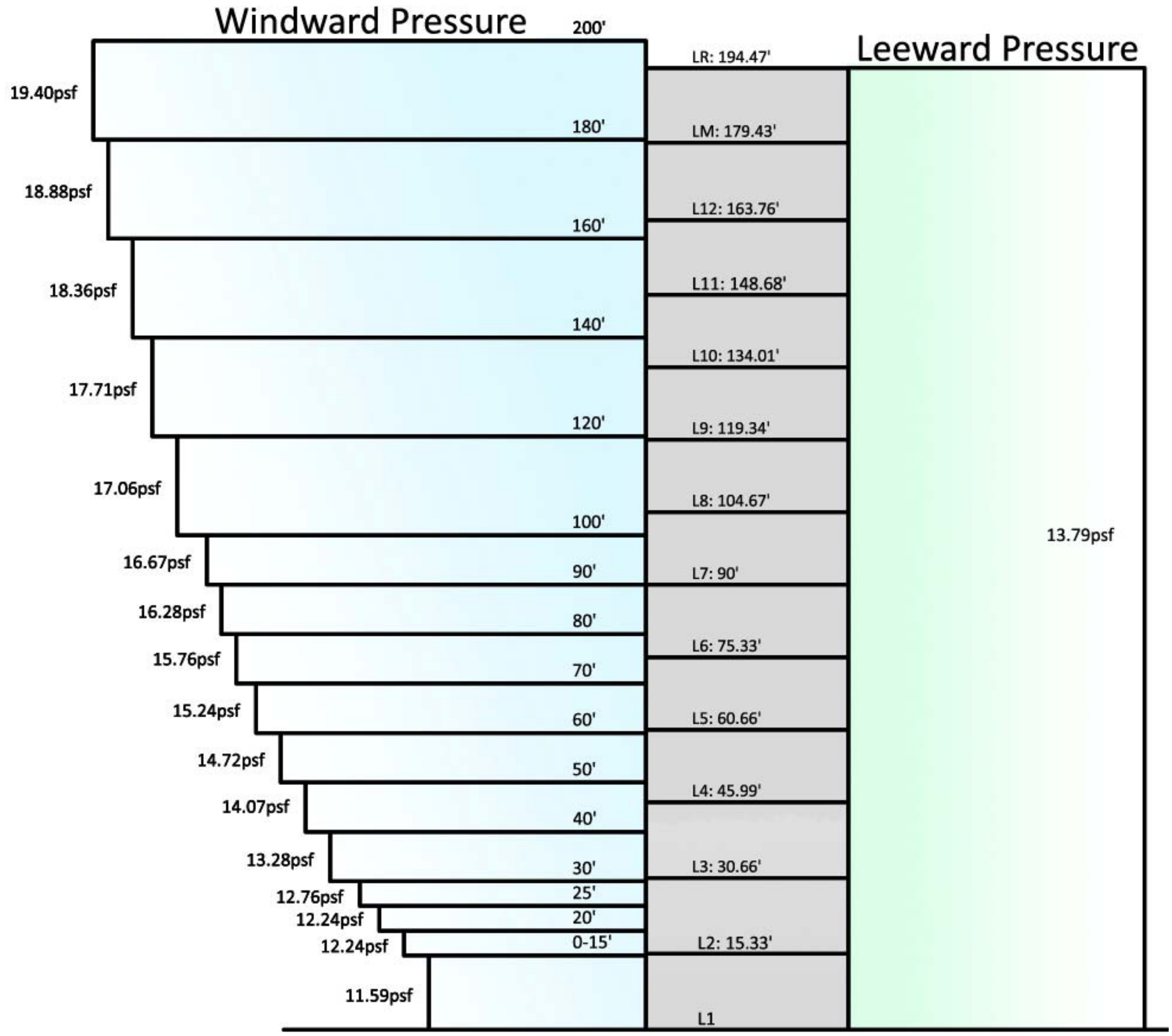


Figure 4: Wind Diagram

Table 4: Wind Loads (kips)- Building						
Floor	Height f	Elevation f	Width f	Windward	Leeward	Total
1	15'4"	0'	400'	0.00	0.00	0.00
2	15'4"	15'4"	400'	108.80	126.80	235.60
3	15'4"	30'8"	400'	79.60	84.40	164.00
4	14'8"	46'	352'	74.62	72.86	147.48
5	14'8"	60'8"	352'	77.44	71.10	148.54
6	14'8"	75'4"	352'	81.66	71.10	152.76
7	14'8"	90'	352'	85.18	71.10	156.28
8	14'8"	104'8"	352'	87.65	71.10	158.75
9	14'8"	119'4"	352'	89.76	71.10	160.86
10	14'8"	134'	352'	91.52	71.10	162.62
11	15'1"	148'8"	352'	96.10	72.16	168.26
Penthouse	15'8"	163'9"	352'	101.38	74.62	176.00
Mezzanine	15'0.5"	179'5"	352'	100.67	75.68	176.35
Roof	0	194'5.5"	352'	54.21	36.26	90.47
Total				1128.59	969.41	2098.00

Spot Check – Lateral System

After the wind loads were determine for each floor, the SAP 2000 modeling program was used to find the axial forces in the building members. This was then compared with the axial design values presented in the building documents. A second comparison with the original was made checking the one of the 1st floor girders of the braced frame.

The LRFD load combination of 0.9D + 1.6W was used to alter the original wind frame loads seen in Table 5, and the dead load of 85psf. For an initial trial, I had just used the unfactored wind loads without the dead loads, which resulted in axial values significantly less than those presented in the building documents; however, the factored load combination yielded accurate results at the base with increased variation at the top of the frame, which can be seen by comparing the axial loading in figures 5 thru 7. These discrepancies can be accounted for by differences in modeling. The most likely source is some of the column joints which had been modeled as pin connections but would have been more appropriately designated as fixed.

The second comparison was made by checking the 1st floor girder. I decided to check the girder as opposed to a column, due to the aforementioned discrepancy. It was appropriate to do so since it still underwent combined loading due to the significant axial and flexural loads. The AISC interaction equation H1-1a was used to check the girder. The value obtained from this equation was under but near the limiting value, indicating a conservative, yet efficient design.

Table 5: Wind Loads (kips) - Frame L						
Floor	Height	Elevation f	Trib. Width	Windward	Leeward	Total
1	15'4"	0'	73'	0.00	0.00	0.00
2	15'4"	15'4"	73'	19.86	23.14	43.00
3	15'4"	30'8"	73'	14.53	15.40	29.93
4	14'8"	46'	73'	15.48	15.11	30.59
5	14'8"	60'8"	73'	16.06	14.75	30.81
6	14'8"	75'4"	73'	16.94	14.75	31.69
7	14'8"	90'	73'	17.67	14.75	32.42
8	14'8"	104'8"	73'	18.18	14.75	32.93
9	14'8"	119'4"	73'	18.62	14.75	33.37
10	14'8"	134'	73'	18.98	14.75	33.73
11	15'1"	148'8"	73'	19.93	14.97	34.90
Penthouse	15'8"	163'9"	73'	21.02	15.48	36.50
Mezzanine	15'0.5"	179'5"	73'	20.88	15.70	36.58
Roof	0	194'5.5"	73'	11.24	7.52	18.76
Total				229.37	195.79	425.16

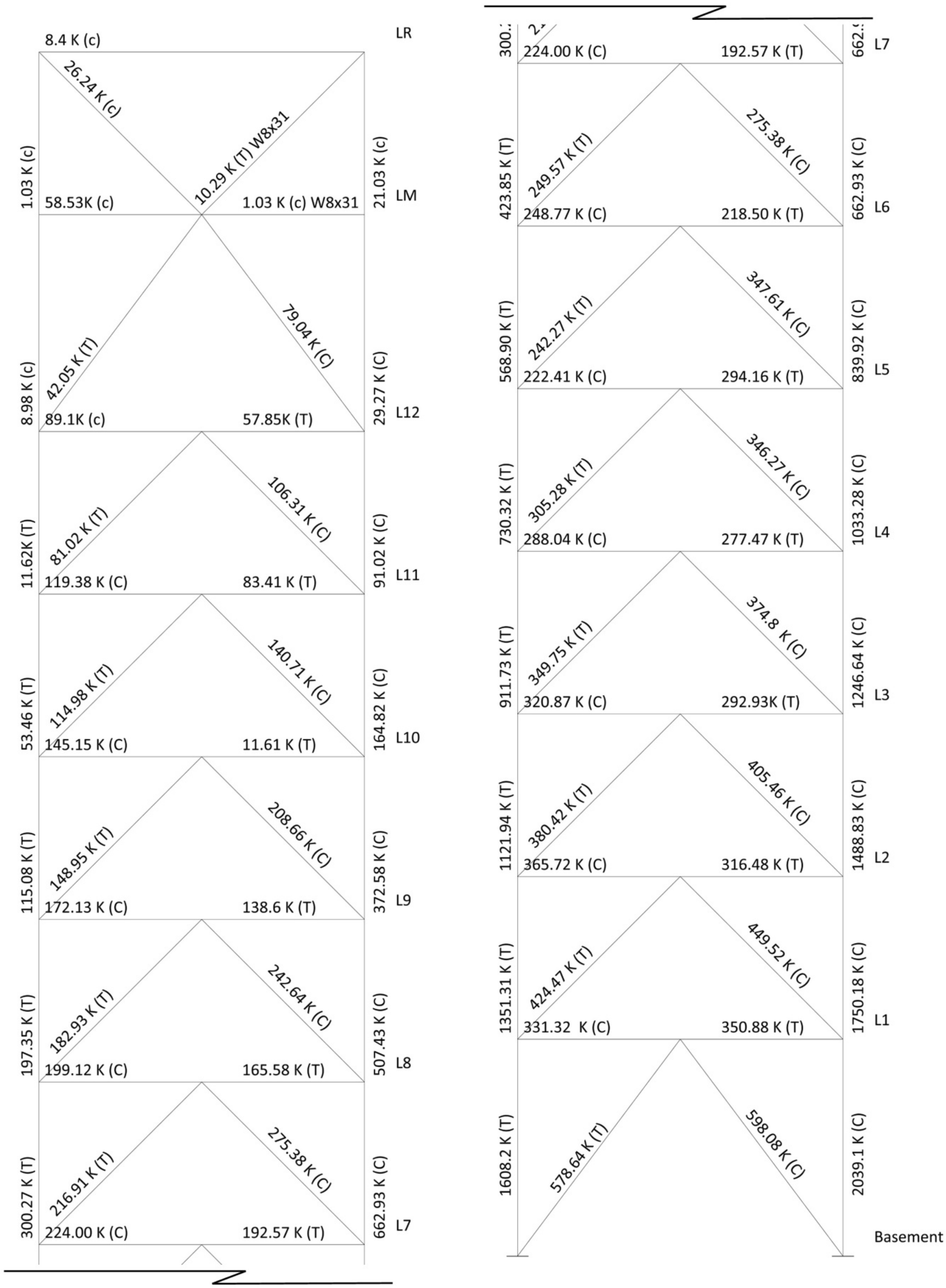
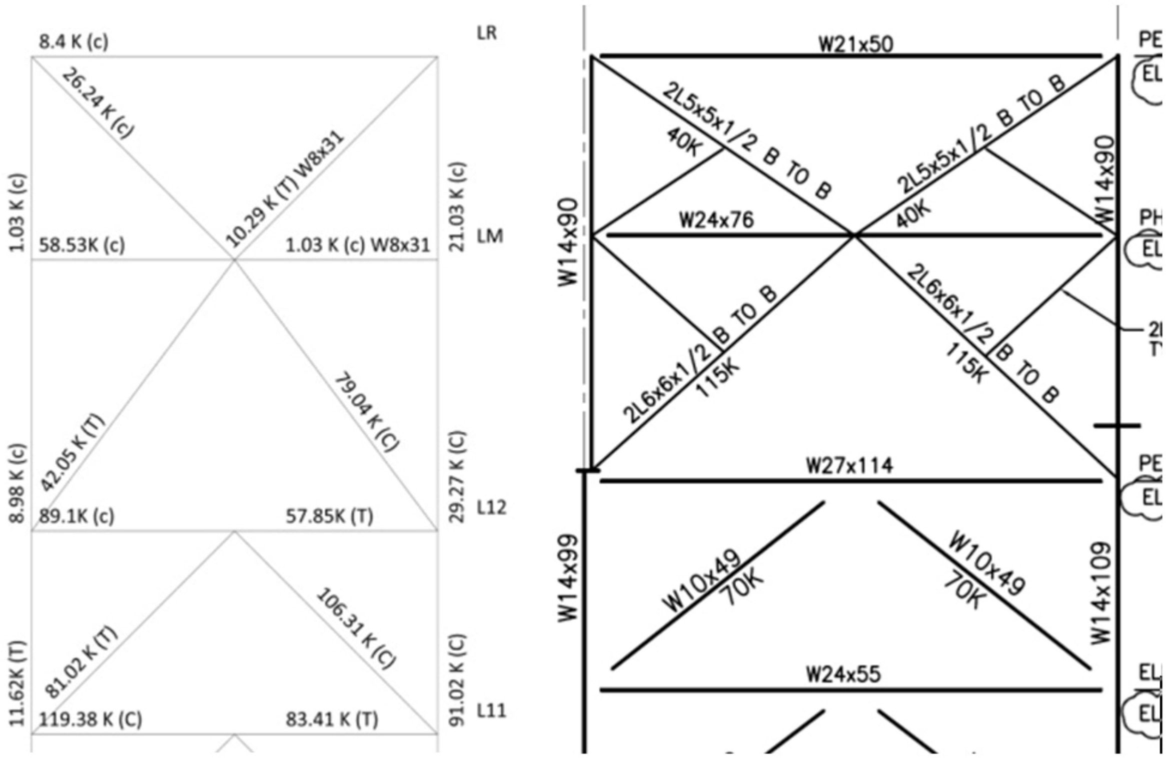


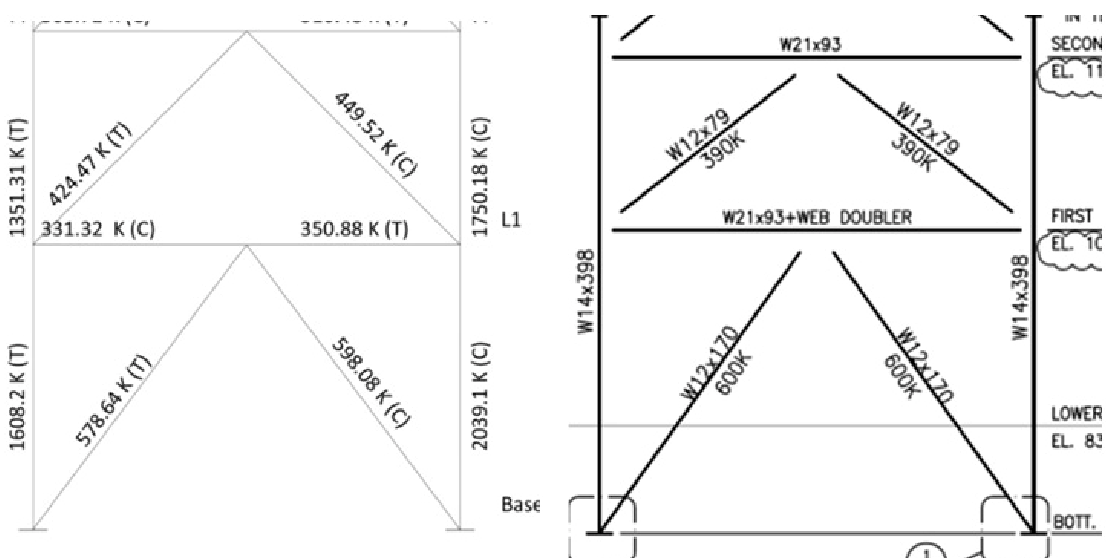
Figure 5: Results of Wind Modeling



Lateral Check

Original System

Figure 6: Comparison of Top of Braced Frame



Lateral Check

Original System

Figure 7: Comparison of Bottom of Braced Frame

Spot Check – Typical Beam

Although there are quite a few irregularities in this building, such as the curved façade and oval tower, the area I analyzed has a fairly common bay size. I decided to analyze one of the beams within this bay since it would be a heavily repeated member size. The assumed dead load of 85 psf and live corridor/office load of 100psf were used in the LRFD 1.2D + 1.6L load combination to design the composite beam for flexure. This yielded a beam significantly smaller than the original; however, I performed an analysis for deflections which yielded the same beam size as the original (see appendix).

Spot Check – Typical Column

The column that I checked was selected for the same reasons as the beam. I used the same loading and load combination to find the axial load. Table 4-1 (Available Strength in Axial Compression) of the AISC design manual was then used to choose a column size (see appendix). The column I found was significantly smaller than the column originally used. There are two factors contributing to this. The first is that the weights of the Penthouse HVAC equipment were not available in the drawings and specifications, which would add a significant dead load at the rooftop level. Secondly, wind loads were not taken into account. Another analysis should be performed using a different load combination including wind. Modeling the frame in question and applying the wind loads with a combination of either live or dead load will add end moments to the column. This combined axial and flexural loading would be more accurate in sizing the column.

Appendix

Columns: Detailed Description

Penthouse Flr.	-	Mostly W14x90, some W12x65
10th-Penthouse.	-	Mostly W14x90 and W12x65, a few W12x86 and W14 in the 100-120 lb range
8th-10th	-	Similar, with more W14 in the 100-120 lb range, and some W12 in the 70-120 lb range
6th-8th	-	Mostly W14 in the 100-230 lb range and W12x65, with some W12 in the 70-150 lb range
4th-6th	-	Mostly W14 in the 130-280 lb range and W12 in the 60-100 lb range, with some W12 in the 100-200 lb range
2nd-4th	-	Several W14 in the 160-34 lb range, mostly W12 in the 60-100 lb range, some W12 in the 100-200 lb range, and a few W10x49
Basement-2nd	-	Mostly W14 in the 100-550 lb range and W12 in the 60-120 lb range, some W12 in the 200-250 lb range, a few W10x49/79, and a few 8x8x12 HSS shapes

Beams & Girders: Detailed Description

General Superstructure (2-13):

- Areas 3 and 4 contain greatly varying framing sizes due to openings. Area 3 contains openings for mechanical equipment and stairwells, while area 4 also contains an elevator core.
- Area 2 contains another elevator core and additional riser openings. At the lower levels this includes an atrium and stairwell. Area 5 contains the framing for the dramatic curved east façade. The curve itself is composed mostly of W21x44 or W24's members of various sizes with the curved bays typically spanned by W12x19's. Longer spans range from W14x22 to W24x84.

13th Floor (Roof):

- Area 1 has several small 8'x31' regular bays with W12x14 beams spanning girders of various dimensions. Another regular bay of 25'x31' contains mostly W14x22 beams.
- Area 2 consists mostly of 31'x38' regular bays with W21x44 beams spanning girders ranging from W21x44 to 62.

Mezzanine Level:

- Area 1 contains small cantilevered 5’x31’ bays with W21x14 beams spanning W18x40 and W16x26 girders. Another typical bay is 31’x17’ with W21x19 beams spanning W18x(55,50) girders. Several of these bays contain openings for which W8’s, 10’s and 12’s are utilized.
- Area 2 is open to below

12th Floor:

- Area 1 contains mostly 31’x 25’ bays with W16x(26,31) beams spanning W 27x94 and W24x55 girders. The bays size remains the same until the 4th floor.
- Area 2 38’x31’ bays use W24x(55,62) beams to span W27x(84,94) girders. There are now several openings in the floor as well. Otherwise the bay sizes remain the same for the entire structure.

6th-11th Floors:

- Area 1’s cantilevered 5’x31’ bays with W21x14 beams spanning W21x(44,50) girders continues until the 4th floor. The other bay size also remains the same with W18x(40,71) beams spanning W24x68 and W21x50 girders.
- Area 2’s W24x76 beams span W24x68 girders.
- Area 6 is the oval tower which is framed by a hexagon of W12 and 16 girders and beams. C shapes round out the shape of the oval.

5th Floor:

- Area 1 W18x35 beams frame into W24x94 and W21x50 girders.
- Area 2 uses W24x94 beams and girders with L shaped beams creating a diagonal pattern in three of the bays.
- Area 6 is open to the floor below on this floor

4th Floor:

- Area 1 is replaced by area 7 and continues with the

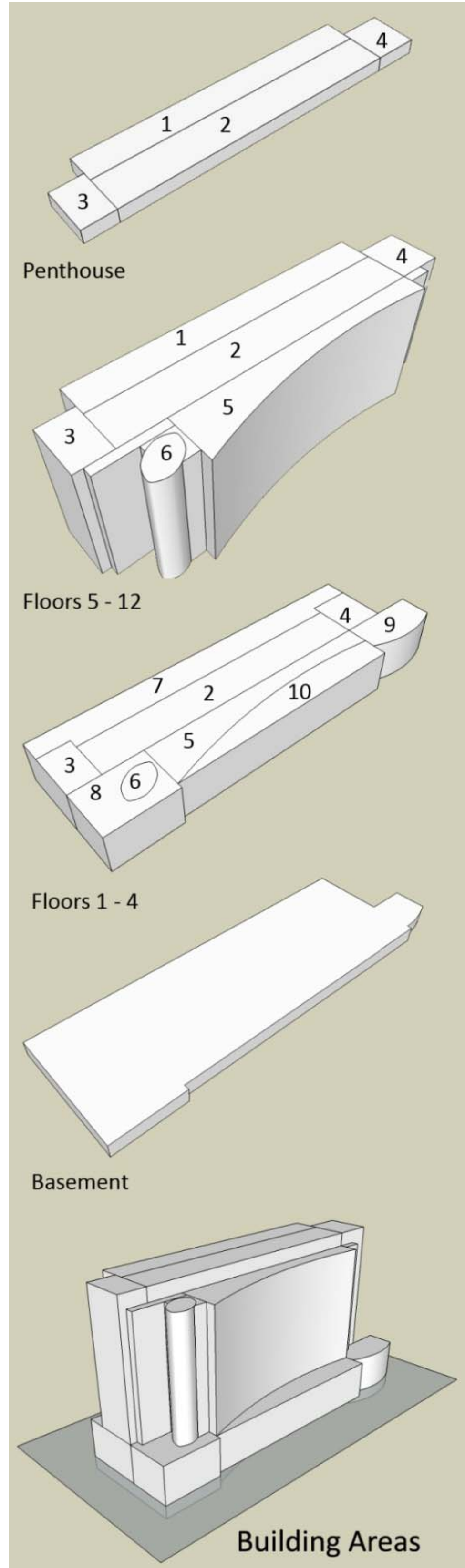


Figure 8: Building Areas

25'x31' bays with W18x40 beams spanning W24x55 W24x76 girders. The Western 5'x31' bay is extended to 20'x31' and consists of W12x19 beams spanning between W21x44 and W24x55 girders. W8x10's span in between the beams. Some of these bays are trapezoidal in shape in order to connect previously unconnected columns with girders.

- Area 2 uses W24x76 beams to span W24x76 and W27x84 girders
- Area 6 framing is integrated with area 8 so that its member sizes range from W14-W24.
- Area 8 consists of a W16 beams framing into W24 girders, and W12 beams framing into W12 and W21 girders.
- Area 9, the auditorium roof, consists of 44LH14 shapes spanning the curved exterior and interior walls framed by W16,18 and 21 girders.
- Area 10 is the atrium roof, which extends from the curved façade to form a straight edge facing the street. Beams varying from W16 to W24x68 span the curve girders to the straight W24x55 girders.

3rd Floor:

- Area 7 and 2 bay sizes continue the same as the upper level, while using slightly larger beam and girder sizes.
- Area 6 becomes independent again, using the same framing as on the 6th floor.
- Areas 8,9 and 10 are open to below.

2nd Floor:

- Little variation from floor 3 or 4, except that area 8 is framed again.

1st Floor:

- Area 1 decreases to one 25'x31' bay again with W16 beams spanning W24 girders.
- Area 2 is similar to floor 4.
- Area 8 is similar as floor 2.
- Area 9 is framed by W30x90 beams sloped for the seating area, with W12 beams framing into W16 girders for the stage.
- Area 10 is similar to floor 4.

Basement:

- See foundation description.

Dead Load Calculations

Dead Loads

Selfweight

Decking (USD Design Manual)

2.5" nwc, 3" deep 20 gage galvanized composite (3"LOK floor)

$w_c = 48$ psf

$w_s = 2.1$ psf

$w_t = 50.1$ psf

Girders + Beams

Average unit weights of typical bay beams + girders

Avg = 4×40 plf + 2×71 plf + 35 plf + 4×94 plf = 67 plf

67/10 = 7 psf

Sum = 60 psf

Mechanical / Electrical

Typical lab area has 8 lines running through (dwg P107-2)

Assume water line (waste + gas + lab lighter per specs) (Spec 15062)
1.5"-2" copper pipe

Copper 150 lb/ft3 $[\pi(\frac{2.125}{4})^2 - \pi(\frac{1.985}{4})^2]$ = 68 plf

8×68 plf / 30' span = 20 psf

Partitions (ASCE 7-05)

8 psf steel frame + gwb

Ceiling (ASCE 7-05)

1 psf Acoustic Fiber Board

Floor

1 psf Vinyl composition tile

TOTAL

85 psf

Seismic Load Calculations

Seismic 1

The USGS Earthquake Ground Motion Parameter Java Application at http://earthquake.usgs.gov/research/hazmaps/design was used to determine SDS and SD1.

A Site Class B was obtained by inputting these coordinates
Lat: 40.006164
Long: -75.151794

However, the geotechnical report classified the site as C so this was used for my analysis since the source is more accurate and the calculations are more conservative

The ASCE 7-05.12.8 Equivalent Lateral Force Procedure was used

$S_{DS} = 0.214$
$S_{D1} = 0.068$

12.8-2 Seismic Response Coeff.

$$C_s = \frac{S_{DS}}{(R/I)}$$

$R = 3.25$ (Table 12.2-1): Drawing S520 states that structural was not specifically detailed for seismic. Also, a review of braced frame connections did not indicate moment connections. Values were picked for B4: Ordinary Steel Concentrically braced frames

$I = 1.25$ for Occupancy III using Tables 1-1, 11.5.1

$$C_s = \frac{0.214}{(3.25/1.25)} = 0.084$$

Limits

$$T = C_t h_n^x = 0.02(195ft)^{0.75} = 1.044s \text{ (Table 12.8-2)}$$

$$T_L = 6s \text{ (Figure 22.15)}$$

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} \text{ for } T \leq T_L$$

$$= \frac{0.068}{1.044\left(\frac{3.25}{1.25}\right)} = 0.025$$

Seismic 2

Seismic Weight

Dead Load (85psf)(480,000sf) = 40,800 k

$V = C_s W = (0.025)(40,800k) = 1020\ k$

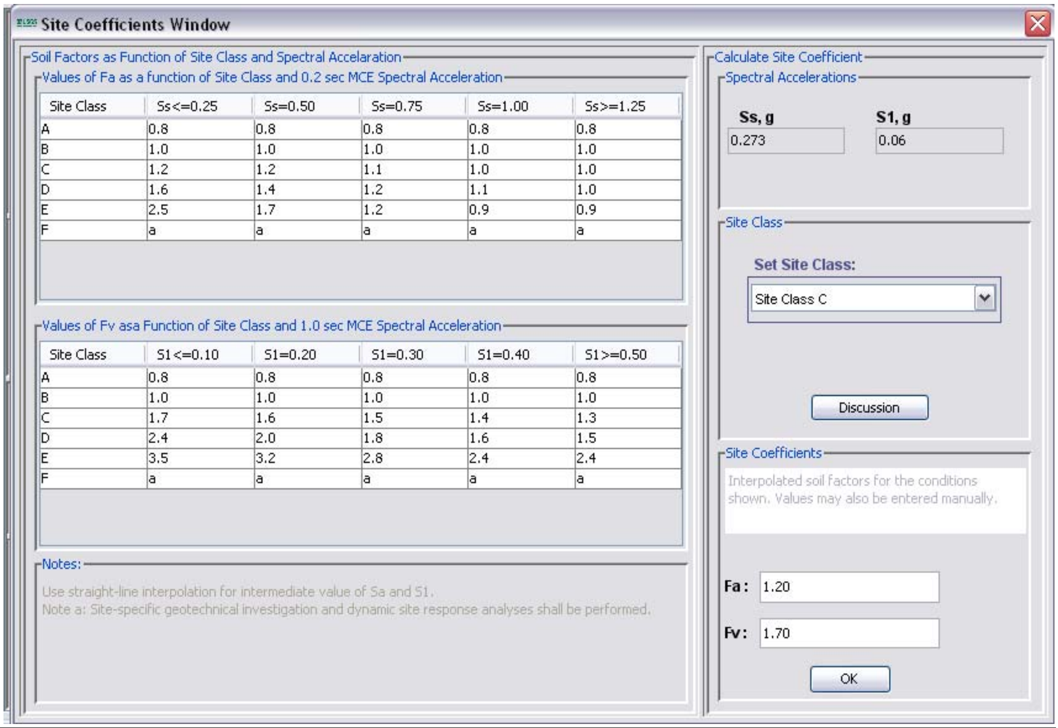


Figure 9: USGS results

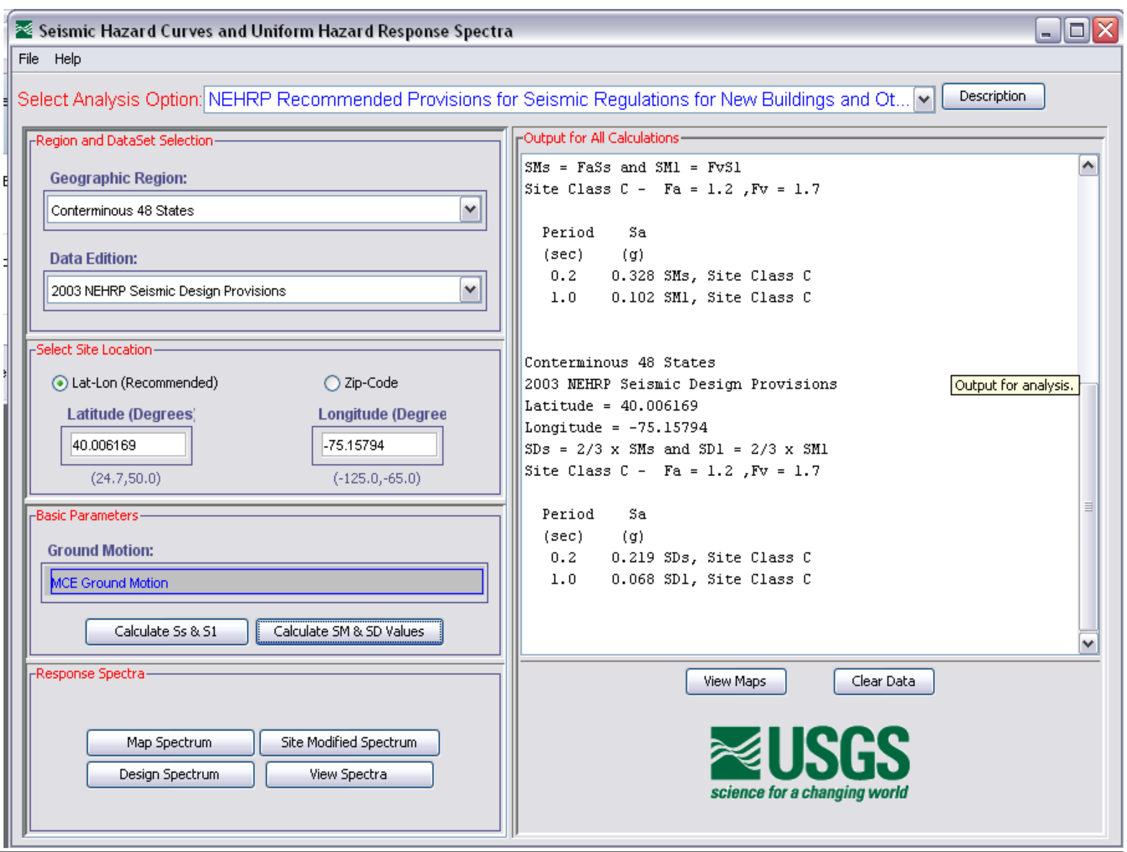


Figure 10: USGS Results

Wind Load Calculations

Wind : ASCE 7-05 | Method 2 : Analytical Procedure | 1

This method was used since height > 60'

① Basic windspeed (Fig 6-1)
$V = 90$ mph

Wind Directionality Factor (Table 6-4)
$K_d = 0.85$

② Importance Factor
Occupancy III Table 1-1

$I = 1.15$

③ Exposure Category
Urban : B

④ Topographic Factor
$K_{zt} = 1$

⑤ Gust Effect Factor
$T = 1.044s$ (from Table 12.8-2)
$n_1 = \frac{1}{1.044} = 0.96 < 1 \longrightarrow$ Flexible Structure

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

$g_Q = g_v = 3.4$

$$g_R = \sqrt{2 \ln(3600 n_1)} + \frac{0.577}{\sqrt{2 \ln(3600 n_1)}} = 4.180$$

Resonant Response Factor

$$L_{\bar{z}} = \ell \left(\frac{\bar{z}}{33}\right)^{\bar{\varepsilon}} = 487.948$$

$\bar{z} = 0.6h = 0.6(195') = 117'$
$\ell = 320$ ft (Table 6-2)
$\bar{\varepsilon} = 1/3.0$
$\bar{b} = 0.45$
$\bar{\alpha} = 1/4.0$

$$\bar{V}_{\bar{z}} = \bar{b} \left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V \left(\frac{88}{60}\right) = 0.45 \left(\frac{117'}{33}\right)^{1/4.0} (90)\left(\frac{88}{60}\right) = 81.509$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{0.96(487.948)}{81.509} = 5.747$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47(5.747)}{(1 + 10.3(5.747))^{5/3}} = 0.046$$

2

R_h : $\eta = 4.6 n_1 h / \bar{V}_{\bar{z}} = 4.6(0.96)(195')/81.509 = 10.565$

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

$= \frac{1}{10.565} - \frac{1}{2(10.565)^2}(1-e^{-2(10.565)}) = 0.090$

R_B : $\eta = 4.6 n_1 B / \bar{V}_{\bar{z}} = 4.6(0.96)(352')/81.509 = 19.071$

* $B \approx 352'$ for most of building

$R_B = \frac{1}{19.071} - \frac{1}{(2)(19.071)^2}(1-e^{-(2)(19.071)})$

$= 0.051$

R_L : $\eta = 15.4 n_1 L / \bar{V}_{\bar{z}} = 15.4(0.96)(85)/81.509 = 15.417$

* $L \approx 85'$ for most of building

$R_L = \frac{1}{15.417} - \frac{1}{2(15.417)^2}(1-e^{-2(15.417)}) = 0.063$

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

* Assume $\beta = 0.05$

$= \sqrt{\frac{1}{0.05}(0.046)(0.090)(0.051)(0.53+0.47(0.063))}$

$= 0.049$

Background Response

$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{352+195}{487.948}\right)^{0.63}}}$

$= 0.772$

Intensity of Turbulence

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.30\left(\frac{33}{117}\right)^{1/6} = 0.243$

$c = 0.30$ Table 6-2

$G_f = 0.925\left(\frac{1 + 1.7(0.243)\sqrt{(3.4)^2(0.772)^2 + (4.180)^2(0.049)^2}}{1+1.7(3.4)(0.243)}\right) = 0.803$

(6) Enclosure Factor : Enclosed

(7) Internal Pressure Coefficient Figure 6-5

$GC_{pi} = +0.18$ (Pressure acting toward internal surface)
-0.18 (away)

3

(8) External Pressure Coefficient Fig 6-6

Windward wall $C_p = 0.8$ use q_z
Leeward wall $C_p = -0.5$ use q_h $(L/B = 0.241)$

Sidewall $C_p = -0.7$ use q_h

Roof $C_p = -0.18$ $(h/L = 2.294)$

(9) Velocity Pressure q_z or q_h

$$q_z = 0.00256\, k_z\, k_{zt}\, k_d\, V^2 I$$
$$= 0.00256\, k_z\, (1)(0.85)(90)^2(1.15)$$
$$= 20.269\, k_z$$

Table 6-3 Exp. B, Case 2

(See Spreadsheet Table 4)

(10) MWFRS for Flexible Buildings

Windward

$$p = q\, G_f\, C_p \pm q_i (G C_{pi})$$
$$p = q(0.803)(0.8) \pm 23.71(0.18)$$
$$= 0.64q \pm 4.27$$

$q_i = q_h = 20.269(1.17) = 23.71$

Leeward

$$p = q_h\, G_f\, C_p \pm q_i (G C_{pi})$$
$$q_h = 20.269(1.17) = 23.715$$
$$= 23.715(0.803)(0.5) \pm 4.27$$
$$= 9.52 \pm 4.27$$

Max = 13.79
Min = 5.25

(11) Approximate Face Area E-W

$(352' \times 198' + 50' \times 45') = 70890$ sf

4

Floor	Above	+ Below	= Windward plf (vertical)	Leeward plf (vertical)
LR		153.38	= 154	103
LM	126.91 + 11.32	147.89	= 286	215
L12	147.89	70.80 + 69.62	= 288	212
L11	138.47	134.64	= 273	205
L10	24.42 + 106.26	129.81	= 260	202
L9	118.13 + 11.43	125.05	= 255	202
L8	125.05	79.67 + 44.51	= 249	202
L7	122.19	119.33	= 242	202
L6	43.47 + 73.60	84.00 + 30.48	= 232	202
L5	111.71	10.21 + 98.18	= 226	202
L4	49.02 + 56.25	84.42 + 22.18	= 212	207
L3	101.86	8.55 + 63.80 + 24.48	= 199	211
L2	93.88	4.04 + 173.85	= 272	317

I used the dimensions + loads from the loading diagram to calculate these pressures. (see Appendix)

Total windward base sheer = 1129 (see table 5)

Total Leeward base sheer = 969

2098 > seismic base shear so wind controls

E-W base sheer Rough Estimate

$$2098\left(\frac{4}{13}\text{ floors}\right)\frac{160'}{350'} + 2098\left(\frac{7}{13}\text{ floors}\right)\frac{130'}{350'} = 715\text{ k}$$

Table 6: Windward Pressure (psf)						
Height	kz	qz=20.269*kz	$p=q*G_f*C_p$	± qi*(GCpi)	Max	Min
0-15	0.57	11.55	7.42	± 4.17	11.59	3.25
20	0.62	12.57	8.07	± 4.17	12.24	3.90
25	0.66	13.38	8.59	± 4.17	12.76	4.42
30	0.7	14.19	9.11	± 4.17	13.28	4.94
40	0.76	15.40	9.90	± 4.17	14.07	5.73
50	0.81	16.42	10.55	± 4.17	14.72	6.38
60	0.85	17.23	11.07	± 4.17	15.24	6.90
70	0.89	18.04	11.59	± 4.17	15.76	7.42
80	0.93	18.85	12.11	± 4.17	16.28	7.94
90	0.96	19.46	12.50	± 4.17	16.67	8.33
100	0.99	20.07	12.89	± 4.17	17.06	8.72
120	1.04	21.08	13.54	± 4.17	17.71	9.37
140	1.09	22.09	14.19	± 4.17	18.36	10.02
160	1.13	22.90	14.71	± 4.17	18.88	10.54
180	1.17	23.71	15.23	± 4.17	19.40	11.06
200	1.2	24.32	15.62	± 4.17	19.79	11.45

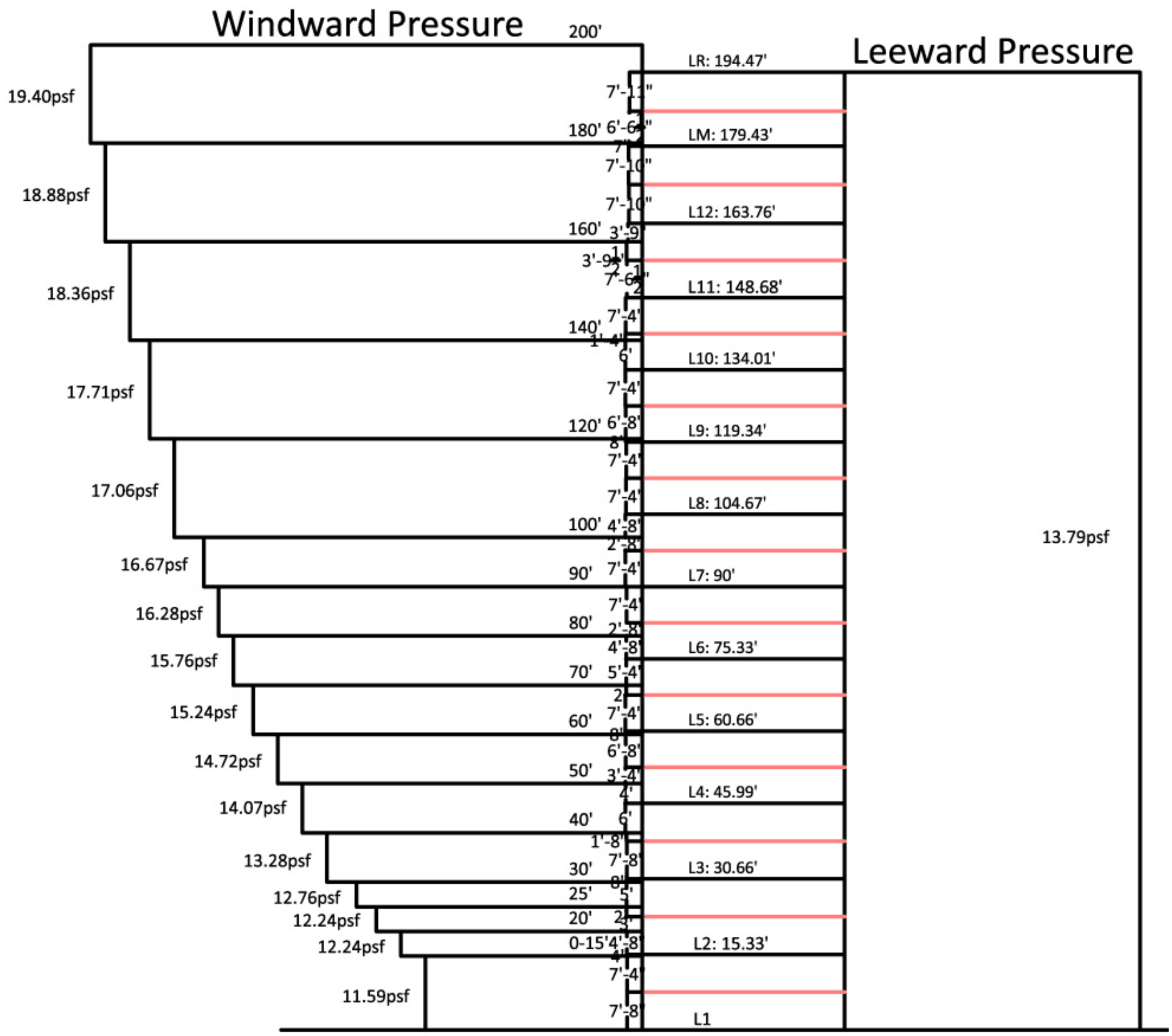


Figure 11: Wind Diagram for Tributary Area

Table 7: Detailed Calculations												
							Windward	Leeward	Windward kips		Leeward kips	
Floor	Height	Elevation	Bldtg Width	Trib. Width	Area Bldg	Area Fr	PLF vert W	PLF vert L	Bldg Load	Frame Load	Bldg Load pl	Frame Load
1	15.33	0	400	73	6132.00	1119.09	0	0	0.00	0.00	0.00	0.00
2	15.33	15.33	400	73	6132.00	1119.09	272	317	108.80	19.86	126.80	23.14
3	15.33	30.66	400	73	6132.00	1119.09	199	211	79.60	14.53	84.40	15.40
4	14.67	45.99	352	73	5163.84	1070.91	212	207	74.62	15.48	72.86	15.11
5	14.67	60.66	352	73	5163.84	1070.91	220	202	77.44	16.06	71.10	14.75
6	14.67	75.33	352	73	5163.84	1070.91	232	202	81.66	16.94	71.10	14.75
7	14.67	90	352	73	5163.84	1070.91	242	202	85.18	17.67	71.10	14.75
8	14.67	104.67	352	73	5163.84	1070.91	249	202	87.65	18.18	71.10	14.75
9	14.67	119.34	352	73	5163.84	1070.91	255	202	89.76	18.62	71.10	14.75
10	14.67	134.01	352	73	5163.84	1070.91	260	202	91.52	18.98	71.10	14.75
11	15.08	148.68	352	73	5308.16	1100.84	273	205	96.10	19.93	72.16	14.97
Penthouse	15.67	163.76	352	73	5515.84	1143.91	288	212	101.38	21.02	74.62	15.48
Mezzanine	15.04	179.43	352	73	5294.08	1097.92	286	215	100.67	20.88	75.68	15.70
Roof	0	194.47	352	73	5294.08	1097.92	154	103	54.21	11.24	36.26	7.52
Total									1128.59	229.37	969.41	195.79

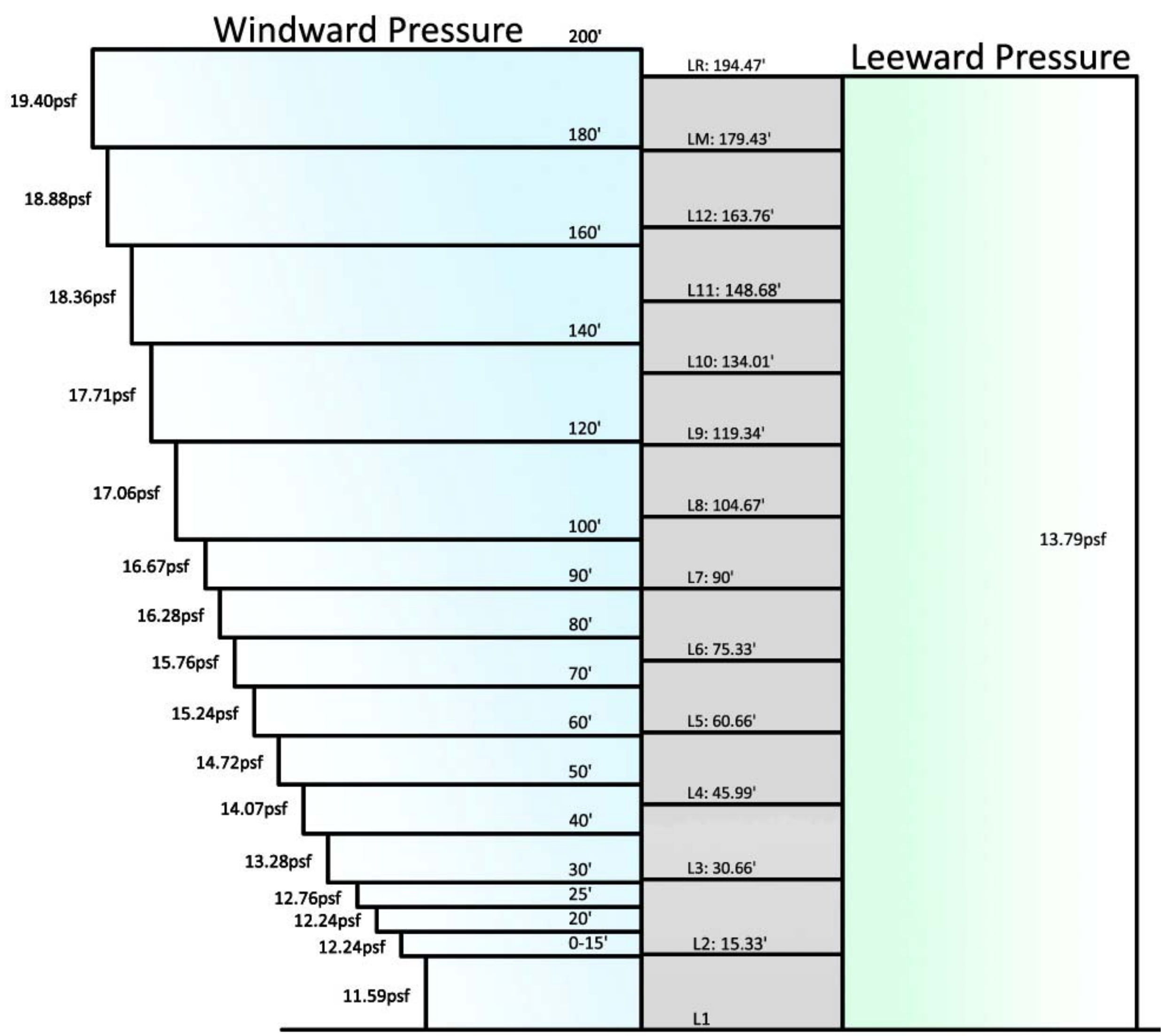


Figure 12: Wind Diagram

Frame Spot Check

Braced Frame - Spot Check 1

Frame L, 1^{st} floor beam
Note although in the model the beam was split into two sections, using the following combined loads for the whole beam should be conservative.

$P_r = 331.32$ k (c)
$M_r = 433.94$ k·in

Results from SAP model using LAFD combination $0.9D + 1.6W$ with $D = 55$ psf and tributary area for D ≈ 10'

Existing Beam W21×93 (non compact)
$r_x = 8.70$ $A_g = 27.3$ in²

(1) Find P_c (Section E of AISC)
① $k = 1$ (pin)
$L = 19'2" = 19.167$ (unbraced length)

$kL/r_x = 19.167(12)/8.70 = 26.44$

② $26.44 < 4.71\sqrt{\frac{E = 29{,}000}{F_y = 50 ksi}} = 113$

Use $F_{cr} = [0.658^{F_y/F_e}]F_y$

$$F_e = \frac{\pi^2 E}{\left(\frac{kL}{r}\right)^2} = \frac{\pi^2(29{,}000)}{(26.44)^2} = 409.43 \text{ ksi}$$

$$F_{cr} = [0.658^{(50 ksi/409.43 ksi)}]\, 50 ksi = 47.51 \text{ ksi}$$

③ $P_c = F_{cr} A_g$
$= 47.51$ ksi$(27.3 in^2)$
$= 1297$ k

(2) Combined Equation
① $P_r/P_c = 331.32/1297 = 0.26 > 0.2$

② Use H1-1a

$pP_r + b_x M_{rx} + b_y M_{ry} \leq 1.0$

Table 6-1: x-axis, unbraced length = 20' for W21×93
$p = 2.75\times10^{-3}$, $b_x = 1.67\times10^{-3}$

$2.75\times10^{-3}(331.32k) + 1.67\times10^{-3}(433.94 \text{ kin} \div 1/12") + 0 = 0.97 < 1.0$

The W21×93 is an efficient size

Beam Spot Check

Typical Beam - Spot Check 1

Loading: Beam Typical Col line F-H / 6-7 Floors 6-11 & Other Areas

$1.2D + 1.6L = 1.2(85psf) + 1.6(100psf) = 0.262$ ksf

Distributed $(0.262)(10'4"$ trib. width$) = 2.707$ klf

$Mu = \phi Mn = w\ell^2/8 = 2.707$ klf $(31)^2/8 = 325$ kft

Design: Flexure

Composite system 2.5" nwc 3/4" ϕ studs, 3" deep, 20 gage galvanized deck

Table 3-21
Deck Parallel, $\frac{wr}{hr} = \frac{12''}{3''} = 4'' > 1.5''$, 3/4" studs, nwc, $f'_c = 4$ ksi

$Qn = 18.3$

$\Sigma Qn = 18.3$ (25 studs) $= 457.5$ k

$a = \frac{\Sigma Qn}{0.85 f'_c b} = \frac{457.5}{0.85(4\text{ksi})(3.31' \times 12\text{in/ft})} = 3.39$

$b \le \begin{cases} 1/8\,(26.5') = 3.31' \\ 1/2\,(10.33') = 5.165' \\ \text{distance to edge of slab} \end{cases}$

$Y_2 = Y_{conc} - a/2 = 5.5'' - 3.39''/2 = 3.805''$

Table 3-19
$Y_2 = 3.8''$, $\phi Mn = 325$ kft

W16x26 with $\phi Mn = 327$ @TFL (Adding More SW would make no size difference)

Design: Deflections

$\Delta = \frac{\ell}{360} = \frac{21'4''}{360} = \frac{256}{360} = 0.71$ in

$\Delta = \frac{5w\ell^4}{384EI}$

$0.71 = \frac{5(2.707\text{klf} \times \frac{1}{12})(256'')^4}{384(29{,}000\text{ ksi})I}$

$I = 612.7\ in^4$

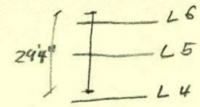
Table 3-3 W18x40 $I = 612 \sim 612.7$ (composite so I is actually greater)
same as original

Table 3-19
$Y_2 = 3.8''$, TFL $\phi M_n = 549$ kft > 325 ok

Column Spot Check

Typical Column - Spot Check 1

Examine W14x257 col. @ G6, level 6



Load

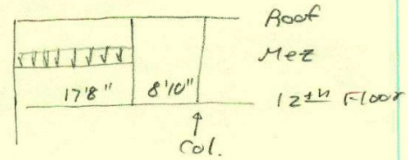
$Area_{5-12} = 31' \times (26.5'/2 + 24.333'/2) = 787.927\ ft^2$

$\times$ 8 floors = 6306.416 ft^2

$A_{mez} = 31' \times (17'8''/2) = 273.833\ sf$

Contribution to col.

$(273.833\ sf)\left(\frac{8'10''}{26'4''}\right) = 91.734\ sf$



$A_{roof} = 31' \times \left(\frac{38'4''}{2} + \frac{26.5'}{2}\right) = 1004.912\ sf$

$A_{total} = 7400.062\ sf$

$Load = 0.262\ ksf\ (7400.062) = 1938.82\ k$

Size Column

1938.82 k — Bracing in strong + weak axis

4' — kL: $0.5(4) = 2'$

14'8" — $0.5(14.667) = 7.334'$

10'8" — $0.5(10.667) = 5.334'$

Table 4-1 $kL = 7.334$ in y-axis, $\phi P_n = 1938.82$

W14x159 $\phi P_n = 2020$ $kL = 8$

Much smaller than original:

W14x257 $\phi P_n = 3270$ $Lk = 8$

See tech report.