# [IPD/BIM Thesis]

**Structural Option**

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# The Millennium Science Complex

## University Park, Pennsylvania

# [TECHNICAL REPORT 1]

[October 5th, 2010]

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## Executive Summary

The purpose of the Structural Concepts/Structural Existing Conditions Report is to describe the structural nature of the Millennium Science Complex, while also analyzing the functionality of the structural system to resist applicable loadings. This cutting edge research laboratory lends itself to faculties of both Life and Materials Sciences. Originally intended to be two separate buildings, further design by Rafael Viñoly Architects incorporated a large 150' cantilever merging the two buildings into one.

This research facility is located in University Park, Pennsylvania on The Pennsylvania State University campus. The building is composed of 4 stories with numerous cantilevered roofs that progressively step back. The smallest of levels is located over the cantilever and houses the mechanical equipment for the building's many laboratories. The primary structure of the building is a steel frame with 2 large steel-concrete composite truss systems, incorporating large C-shaped shear walls to resist the overturning moments created by the cantilever.

Gravity and Lateral load calculations were performed on the building. Spot checks of a typical frame were calculated to confirm the design of a typical composite deck, composite beam, composite girder, and column. The design of these typical gravity resisting members were confirmed and reported later in the spot check section of this report. Wind and seismic calculations were performed using ASCE7-05 and initial design terms were compared to those listed by the structural designer, Thornton Tomasetti.

# Introduction

The Millennium Science Complex is a brand new LEED certified science complex sanctioned by Penn State and OPP to house science laboratories and graduate student offices. It uses a massive cantilever as the main architectural and structural feature of the building reaching 150 feet over the main entrance. The Architect, Raphael Viñoly, aimed to design the building as if it were floating on air with cascading green roofs and trellises that stretch over the floors below them. It stands 4 floors high (approximately 75' measured from the ground) with two wings that join at the main entrance. Located at the corner of Pollock and Bigler road, the building will connect to the Life Sciences Building via an underground tunnel.

There are four occupiable floors, including the basement, with one level of mechanical on the fourth floor. The basement, directly accessed by the loading dock, contains three, fully isolated research labs. The first through third floors has a typical floor plan. Each wing has a central hallway surrounded by laboratories and student offices at the perimeter. Green roofs are located on the floors two, three, and four. Five of them in total are tiered on each wing integrating with the site landscape.

Taking advantage of the flexibility of steel design, the architect uses large, 20' floor to floor heights and a cantilever overhang that would far exceed the practical limits of concrete. With a building area of 275,000 square feet, the Millennium Science Complex combines modern building technologies and materials with the classic Penn State architectural style.

Figure 1: Arial View looking East towards the building from the roof of Pollock Residence Halls.

# Structural System Overview

## *Foundations*

The foundation of the Millennium Science Complex utilizes a system of pile caps, micropiles and grade beams. Each column ends at a pile cap on grid lines spaced twenty two feet apart in a square pattern, as seen in Figure 2 :. Groups of micropiles continue from the pile caps and make their descent through the soil allowing friction to carry the load of the building. Each of these pile caps are connected by grade beams which help to prevent differential settlement, a crucial design consideration for a laboratory building.

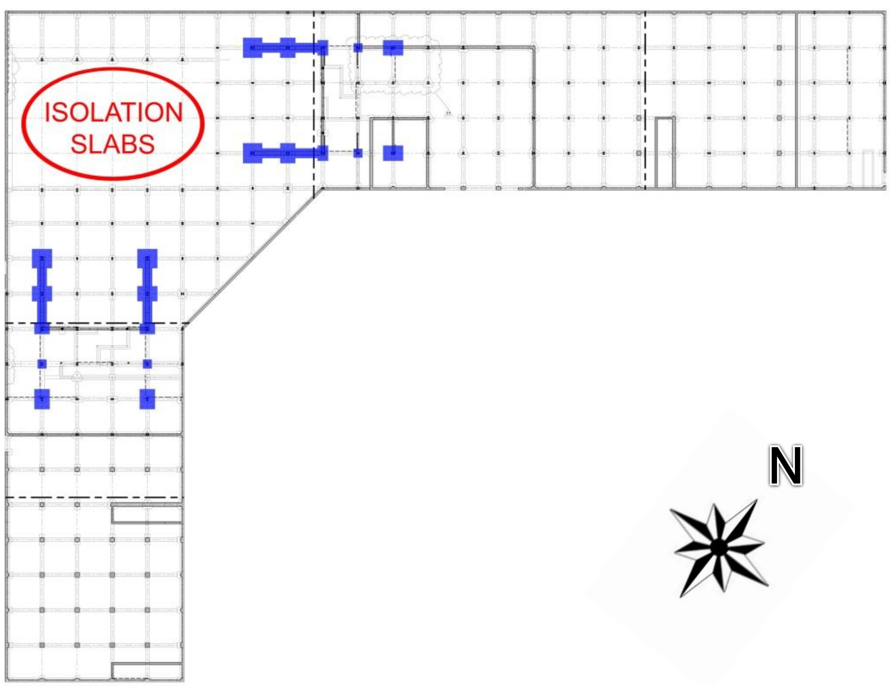



Figure 2 :

Seen here are pile caps positioned at every grid line corresponding to the location of the columns. Columns transfer their load into these pile caps and then into micropiles. Grade beams connect the pile caps in a grid pattern. Several of these pile caps are enlarged and highlighted in blue; they serve to distribute the load from the cantilever. Also seen here is a section circled in red which does not contain pile caps due to the presence of an isolation slab.

Forming the floor of the basement are four different slabs on grade in the occupiable area of the basement, shown on Figure 3:. The basement, extending 20 feet to the first floor of the building, covers only a portion of the entire footprint of the building. From approximately the halfway point of each wing (column lines R and 13) begins a compacted fill extending to the ends of each wing and to the first floor slab on grade. Columns and piers extend from the pile caps at the basement level up through the compacted fill, in this area of each wing, to the first floor. This was presumably designed in the event that the University would want to expand the basement level under each wing. Further evidence of this assumption can be found in the foundation walls, which enclose the compacted fill, and are in line with the exterior walls of the building. The accessible areas of the basement lie directly under the cantilever and extend to the edge of the compacted fill (column lines R and 13). Four isolation labs were placed at this level, designed to be completely disparate of the structural elements that make up the rest of the building. Slabs on grade, foundation walls, footings and piers use 4000 psi concrete; the pile caps are the only concrete items that use 6000 psi concrete. Reinforcement in the foundation and throughout the building is grade 60.

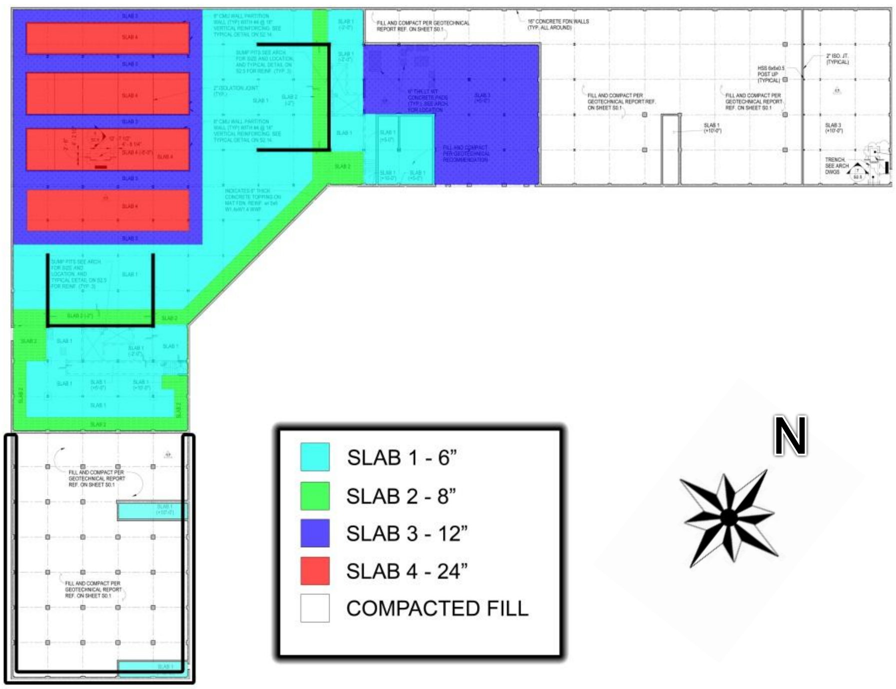



Figure 3:

This basement plan shows the occupiable areas in color, highlighting the four different slabs used in the basement level. This plan also shows areas where possible expansion could be made. The foundation walls encircled in black show the bounds of this possible expansion area.

## *Floor System*

A composite floor system with typical 22 foot square bays forms the floor system for the Millennium Science Building. A typical floor layout for the wings contains a centralized corridor surrounded by rooms on either side. Those perimeter spaces are generally divided into either laboratories or offices. The floor loads are handled by three types of composite decking used throughout the building, highlighted in Figure 4:, the most common of which is a 3 inch 18 gage deck with 3¼ inch light weight concrete topping. The concrete decking is supported by W21 beams and W24 girders which frame into W14 columns, at the intersection of each grid line. Beyond the typical dead and live loads, there are specialty loads from the green roof, mechanical equipment, and the pedestrian traffic at the entrance which call for increased slab strengths. A 3 inch metal deck is used with a 7 inch normal weight concrete topping immediately below the cantilever where pedestrian traffic is heaviest as people enter and exit the building, and a 4½ inch normal weight topping is used to support each green roof. These hallways call for a slightly higher ceiling so W18 beams are used in the center bay of each frame.

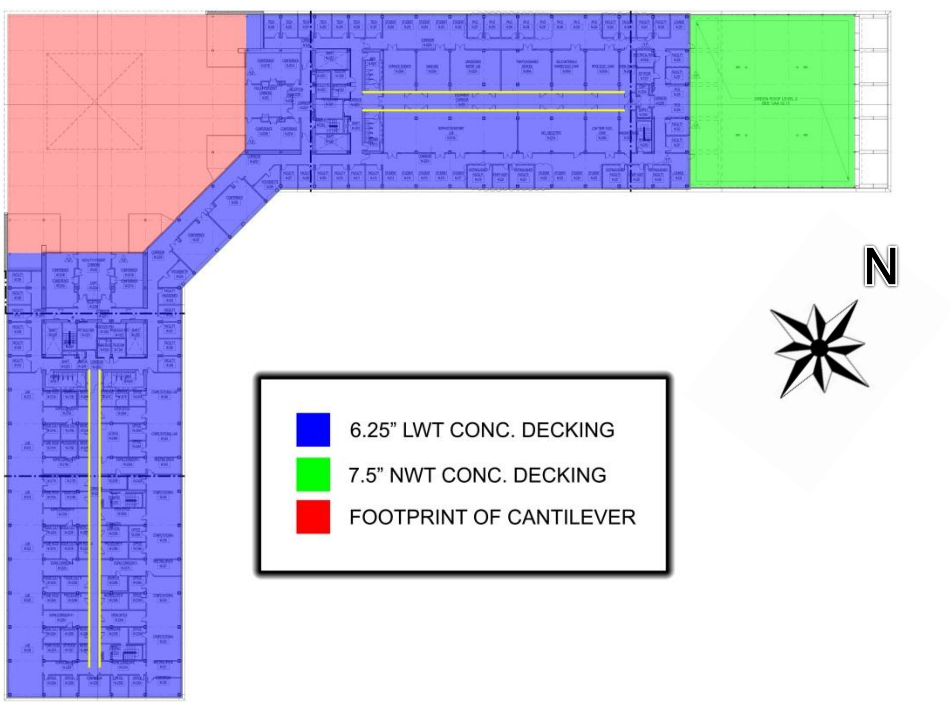



Figure 4:

Seen above is the second floor plan of the Millennium Science Building. Highlighted in green and blue are the different decks used on occupiable floors; they represent the green roof and interior floor, respectively, of the second floor. This plan is used as an example of a typical layout, being lightweight concrete used for the accessible spaces and normal weight concrete used for areas with specialty loads such as the green roof or mechanical penthouse. The area highlighted in red represents the plaza landscape under the cantilever. The yellow lines running through the center of each wing call out the central corridor.

## *Lateral System*

Two moment frames, several bays of braced frames, and two shear walls located at the stairwells make up the dedicated lateral system for the building. The moment frames are located at grid lines Q and 19, which are midway and at the end of their respective wings. The location of these moment frames correspond with shear walls placed in either wing several bays away, as shown in Figure 5:. The objective of these staggered frames and walls is to distribute the lateral forces over the entire floor, preventing excessive localized stresses in the diaphragm. State College itself does not suffer from large wind or seismic loads given building height restrictions and geographical location. Along with the large span trusses and C-shaped shear walls that support the cantilever, the dedicated lateral system more than suffices in resisting the maximum lateral loads State College has to offer.

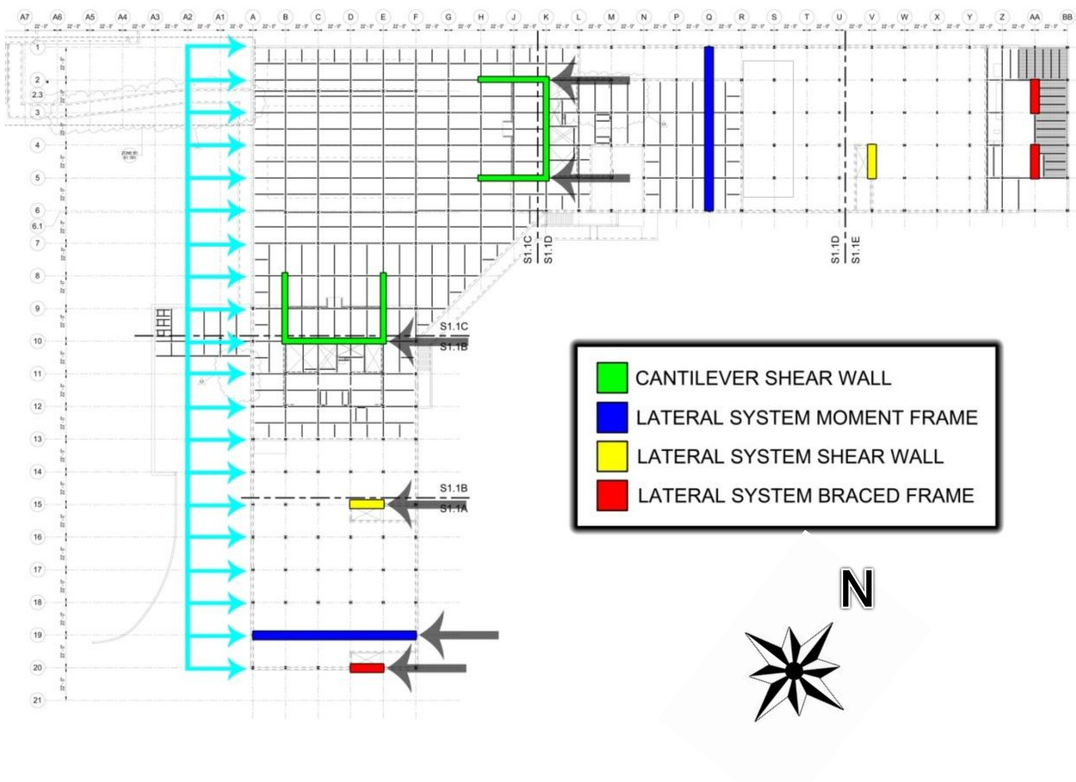



Figure 5:

As the wind hits the structure, loads are transferred from the exterior façade to the floors, acting as a diaphragm, which distribute the load to the columns.

## *Specialty Systems*

To cope with the massive stresses induced by the 150 foot overhanging cantilever, a truss design was used to handle the gravity forces. Gravity loads start from the tip of the cantilever and are transferred into the diagonal compression members. Continuing on the load path, the truss feeds into a 30" shear wall integral with the truss frame. The loads from the diagonal compression members get carried into the shear wall and transfer into the foundation. The load is handled by 10 points in the foundation; one of the two identical frames is shown in Figure 6:. These enlarged pile caps and grade beams act in compression and tension on the soil, using the micropiles as an anchor.

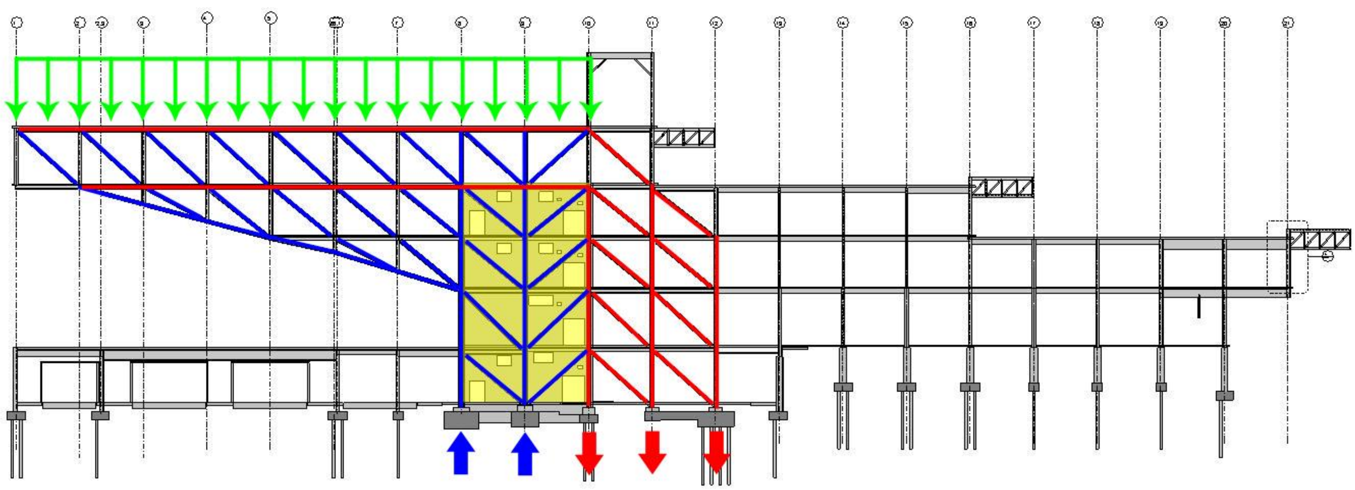


Figure 6:

Shown above is one of the four truss frames dedicated to supporting the cantilever. The members highlighted in blue are under compression; the red members are under tension. The shear wall is highlighted in yellow and provides added stiffness to the frame where foundational reactions change from positive to negative directions. The green distributed load represents gravity loads on the frame. This frame is located at grid line B.

## *Design Theory*

Due to the cantilever and possible cost constraints, steel framing over the entire building was chosen as the best structural solution by the structural designer. A concrete cantilever of the magnitude in the Millennium Science Building would not be feasible due to enormous member sizes and severe deflections. Concrete, however, does make sense as a way to increase stiffness and stability in anchoring the giant cantilever. A shear wall integral with the truss adds stiffness, weight (critical to dampening vibrations), and allows for smaller steel member sizes. Throughout the rest of the building, typical steel framing was used possibly to decrease sub-contractor costs and to avoid further complications in mixing two completely different types of materials and thus, framing types. Shear walls at the stairwells, moment frames and braced frames in the wings are fairly typical. In order to provide the required lateral stiffness, lateral systems were provided at the most convenient places. Overall, the building structure appears to be perfectly logical and few questions were raised on the relevance of a certain system or element. A redesign will prove to be fairly challenging based on limitations imposed by the cantilever and overall cost.

# Material Strengths

## Concrete

| | |
|---|---|
| Pile caps | f'c = 6000 psi |
| Foundation Walls | f'c = 4000 psi |
| Slab on Grade | f'c = 4000 psi |
| Footings and Piers | f'c = 4000 psi |
| Concrete on Metal Deck | f'c = 4000 psi (NWT)<br>f'c = 3000 psi (LWT) |
| Concrete Pads (LWT) | f'c = 3000 psi |
| Fill Slabs (LWT) | f'c = 3000 psi |

Table 1

## Metal Deck

| | |
|---|---|
| 3" 18 gage steel sheet | Fy = 33 ksi |

Table 2

## Structural Steel

| | |
|---|---|
| Rolled Shapes | ASTM A572 or A-992, Grade 50 |
| Channels | ASTM A572 or A-992, Grade 50 |
| Misc. Angles | ASTM A36, Grade 36 |
| Hollow Structural Steel | ASTM A500, Grade B |
| Pipes | Fy = 42 ksi |
| Tubes | Fy = 46 ksi |
| Bolts (3/4 ") | ASTM A325 or A490 |
| Base Plates | Vary, see plans |

Table 3

# Codes and References

## Design Codes:

### Model Codes

- Uniform Construction Code (UCC)
- International Building Code (IBC), 2006 Edition

### Structural Standards

- American Society of Civil Engineers (ASCE), ASCE7-05, Minimum Design Loads for Buildings and Other Structures

### Design Codes

- American Institute of Steel Construction (AISC), Steel Construction Manual, 13th edition, LRFD

## Thesis Codes:

### Model Codes

- Uniform Construction Code (UCC)
- International Building Code (IBC), 2006 Edition

### Structural Standards

- American Society of Civil Engineers (ASCE), ASCE7-05, Minimum Design Loads for Buildings and Other Structures

### Design Codes

- American Institute of Steel Construction (AISC), Steel Construction Manual, 13th edition, LRFD

# Gravity Loads

## *Dead and Live Loads*

Table 4 outlines the design dead and live loads used by Thornton Tomasetti, as well as a confirmation of these loads using ASCE7-05 and the United Steel Deck design guide*.

| Floor Loading Schedule | | | | | | | Confirmed Loads | |
|---|---|---|---|---|---|---|---|---|
| Floor | Elevation (ft-in.) | Occupancy | Slab Type | Loads | | | ASCE7-05 LL | USD* - DL |
| | | | | Slab (PSF) | SDL (PSF) | LL (PSF) | | |
| Level Roof | 1245'-6" | Roof | S1 | 50 | 25 | 30 | 30 | 48.8 |
| Mechanical Penthouse | 1226'-0" | Mechanical | S2 | 110 | 25 | 150 | -- | 106 |
| Third Floor | 1208'-0" | Green Roof | S3 | 76 | 120 | 30 | -- | 75.8 |
| | | Office | S1 | 50 | 30 | 50 | 50 | 48.8 |
| | | Material Science Labs | S1 | 50 | 30 | 150 | -- | 48.8 |
| | | Life Science Labs | S1 | 50 | 30 | 150 | -- | 48.8 |
| | | Corridors | S1 | 50 | 30 | Area Served | Area Served | 48.8 |
| | | Elevator Lobbies | S1 | 50 | 30 | 100 | 100 | 48.8 |
| Second Floor | 1190'-0" | Green Roof | S3 | 76 | 120 | 30 | -- | 75.8 |
| | | Office | S1 | 50 | 30 | 50 | 50 | 48.8 |
| | | Material Science Labs | S1 | 50 | 30 | 150 | -- | 48.8 |
| | | Life Science Labs | S1 | 50 | 30 | 100 | -- | 48.8 |
| | | Corridors | S1 | 50 | 30 | Area Served | Area Served | 48.8 |
| | | Elevator Lobbies | S1 | 50 | 30 | 100 | 100 | 48.8 |
| First Floor | 1170'-0" | Plaza Landscape | S2 | 110 | 300 | 100 | 100 | 106 |
| | | Office | S1 | 50 | 30 | 50 | 50 | 48.8 |
| | | Material Science Labs | S1 | 50 | 30 | 150 | -- | 48.8 |
| | | Life Science Labs | S1 | 50 | 30 | 100 | -- | 48.8 |
| | | Corridors | S1 | 50 | 30 | 100 | 100 | 48.8 |
| | | Mechanical Mezzanine | Grating | 10 | 10 | 150 | -- | -- |
| | | Elevator Lobbies | S1 | 50 | 30 | 100 | 100 | 48.8 |
| Basement Mezzanine | 1160'-0" | Retail | S1 | 50 | 30 | 50 | -- | 48.8 |

Table 4

## Snow Loads

Table 5 outlines the design snow loads used by Thornton Tomasetti, as well as a confirmation of these loads using ASCE7-05 and IBC 2006. It should be noted that the ground snow load found using ASCE7-05 is lower than that used by the structural designer. This is most likely due to a local standard referenced in the Uniform Construction Code which was not able to be obtained at the time this report was written, further confirmation will take place at a later date. It should also be noted that the Snow Importance Factor is pending confirmation because the information available to confirm the R between the Mechanical Penthouse and the heated floors below along with the $ft^2$/Btu, was not available at the time this report was written. Further confirmation from the mechanical option students participating in the IPD/BIM Thesis project will be necessary. For the spot check calculations listed later in the report, the snow load specified by the structural designer was used.

| Snow Loads | | | | Loading Check | | Applicable Standard | |
|---|---|---|---|---|---|---|---|
| Ground Snow Load | $p_g$ | 40 | psf | 30 | pending confirmation | ASCE7-05 | Fig 7-1 |
| Snow Exposure Factor | $C_e$ | 0.9 | | 0.9 | Exp. B | ASCE7-05 | 6.5.6.2 |
| Snow Importance Factor | I | 1.1 | | 1.1 | pending confirmation | ASCE7-05 | Table 7-4 |
| Thermal Factor | $C_t$ | 1 | | 1 | | ASCE7-05 | Table 7-3 |
| Flat Roof Snow Load | $p_f = 0.7C_eC_tIp_g$ | 28 | psf | 20.8 | | | |
| Snow Drifts | | Based on Sect. 1608.7 as applicable | | | | | |

Table 5

### Check min $p_f$:

$$p_g > 20\frac{lb}{ft^2} \therefore check\ p_f = 20I = 20 \times 1.1 = 22$$

$$p_f = 22 < 28 \therefore use\ p_f = 28\,psf$$

The column location spot checked below was assumed to be unaffected by the snow drift load and therefore drift calculations were not fully calculated. This assumption was justified based on the fact that the column spot checked was not adjacent to any boundary walls or roofs.

# Lateral Loads

## *Wind Loads*

Lateral loads were tabulated for wind loads using Method 2 per ASCE7-05 Chapter 6. An in depth calculation of all wind pressures and loads, including all relevant coefficients and equations, is reported in Appendix C: Wind Analysis. Worst case applied winds were determined to come from West and North directions. This is due to the larger windward pressures associated with the larger windward walls on these elevations as well as these directions being the typical windward sides on a daily basis. Windward and leeward pressures were calculated based on tributary heights of each floor. Based on windward and leeward pressures effective forces at each floor and a total base shear was determined for each direction of wind application.  Table 6 - Table 10 below report the calculated, windward, leeward, parapet, and roof wind pressures and floor forces due to these pressures.

| **Windward Walls** | | |
|---|---|---|
| | **P = qGCp + qi(Gcpi)** | **P = qGCp - qi(Gcpi)** |
| Height | Pressure 1 (psf) | Pressure 2 (psf) |
| z= 15ft | 11.39 | 4.46 |
| z= 20ft | 12.07 | 5.14 |
| z= 39ft | 13.87 | 6.94 |
| z= 57ft | 15.06 | 8.14 |
| z= 75.75ft | 16.05 | 9.12 |
| z= 87ft | 16.55 | 9.62 |

Table 6: Windward Wind Pressures

| **Leeward Walls** | | |
|---|---|---|
| | Pressure 1 (psf) | Pressure 2 (psf) |
| Wind- short side | 0.19 | -6.74 |
| Wind-Long Side | -4.72 | -11.65 |

Table 7: Leeward Wind Pressures

| **Parapets(Windward)** | | | |
|---|---|---|---|
| Heights | qp | GCpn | **Pp= qpGCpn (psf)** |
| z= 33ft | 13.87 | 1.5 | 20.81 |
| z= 51ft | 15.06 | 1.5 | 22.60 |
| z= 69ft | 16.05 | 1.5 | 24.07 |
| z= 87ft | 16.55 | 1.5 | 24.83 |

Table 8: Windward Parapet Wind Pressures

| Roof- First Value | | | |
|---|---|---|---|
| Roof Length | qp | | Pp= qpGCpn |
| 0-h | -11.26 | | -18.19 |
| h-2h | -4.72 | | -11.65 |
| >2h | -1.44 | | -8.37 |

Table 9: Roof Wind Pressures

| Final Story Forces | | | | | | |
|---|---|---|---|---|---|---|
| | Load | | Shear | | Moment | |
| Floor Level | E/W(K) | N/S(K) | E/W(K) | N/S(K) | E/W(K-ft) | N/S(K-ft) |
| First Floor | 140.24 | 185.22 | 763.68 | 874.70 | 1402.381 | 1852.21 |
| Second Floor | 204.54 | 254.70 | 623.44 | 689.47 | 6136.109 | 7640.975 |
| Third Floor | 194.15 | 199.12 | 418.90 | 434.77 | 9319.04 | 9557.669 |
| Mech. Pent. | 151.16 | 156.45 | 224.76 | 235.66 | 9976.463 | 10325.96 |
| Roof | 73.60 | 79.20 | 73.60 | 79.20 | 6292.719 | 6771.838 |
| | | Totals*(1.6) | 1221.887 | 1399.512 | 53002.74 | 57837.85 |

Table 10: Wind loads, shears, and moments at each floor level

Method 2 requires the calculation of two values for each pressure, as shown in the tables, with the addition of internal pressure. The larger of the two is chosen to then apply to the structure. Windward and leeward pressures, being calculated at explicit heights corresponding to the tributary width extents of each floor, can then be added together and multiplied by the effective area of an individual floor to achieve the story forces. These are combined to account for one design base shear and corresponding overturning moment, as shown in the tables above. As shown, the 1399.5 kip base shear of the North-South direction controls over the 1221.9 kip East-West base shear. This is due to the larger façade area in the North-South direction exposed to the wind pressures. No comparisons have been made to the wind analysis carried out by the structural engineer due to lack of contact and unavailable calculations. When contact is made and calculations are received this check can be made.

Diagrams showing the distribution of these wind pressures and forces are reported in the following Figure 7 - Figure 10. Due to the L-shape of the building a wind from either direction can either travel over the short side of the wing perpendicular to the wind force or over the full length of the wing parallel to the direction of the wind. Thus, yielding two different leeward pressures as reported in the tables above. Also the different amounts and heights of parapets of each roof on each wing causes slightly different parapet windward pressures.

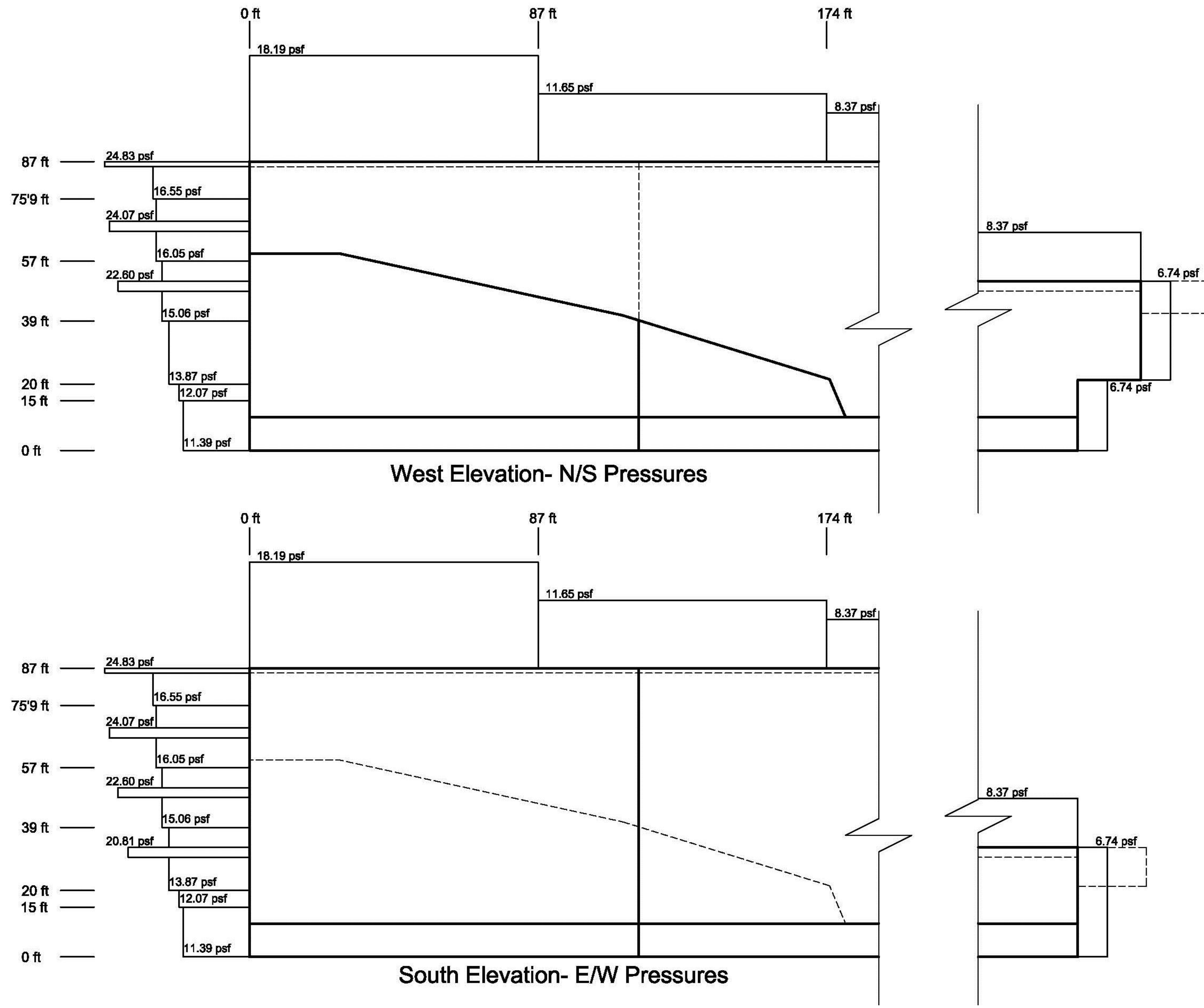
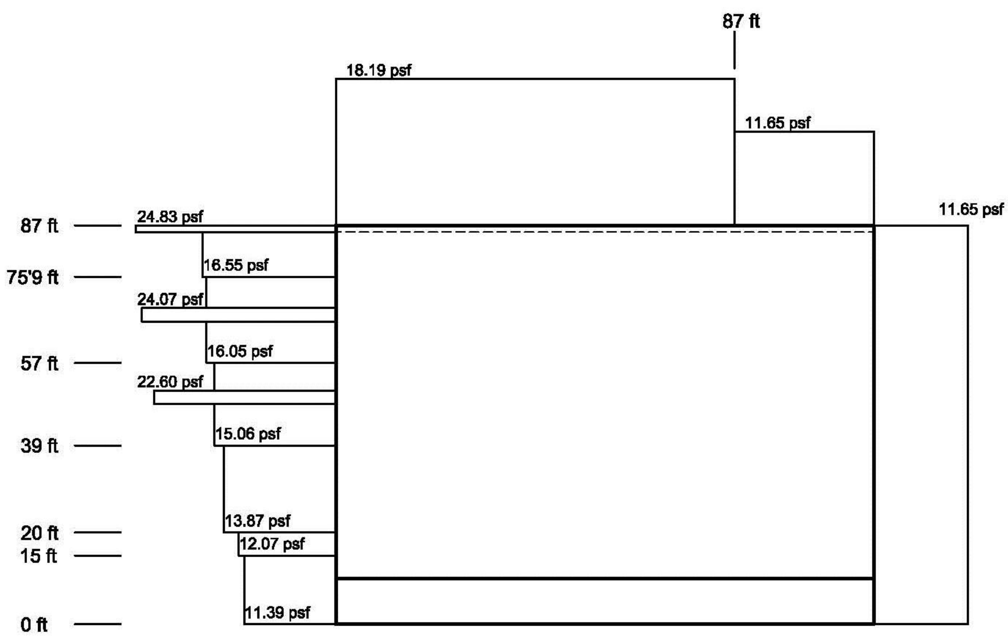
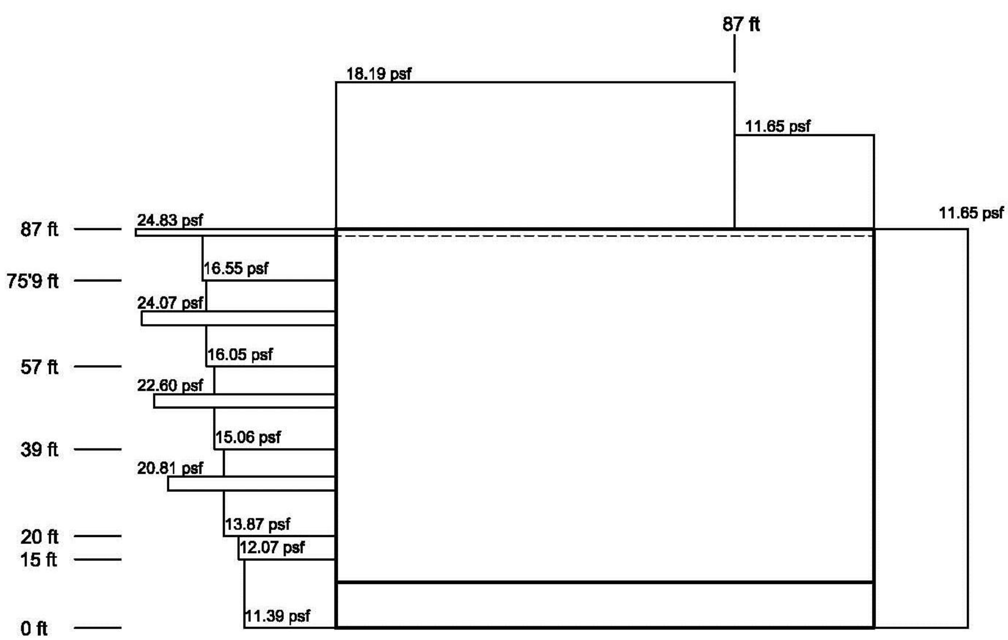

0 ft
87 ft
174 ft
18.19 psf
11.65 psf
8.37 psf
87 ft
24.83 psf
75'9 ft
16.55 psf
24.07 psf
57 ft
16.05 psf
22.60 psf
39 ft
15.06 psf
20 ft
15 ft
13.87 psf
12.07 psf
0 ft
11.39 psf
8.37 psf
6.74 psf
6.74 psf
West Elevation- N/S Pressures
0 ft
87 ft
174 ft
18.19 psf
11.65 psf
8.37 psf
87 ft
24.83 psf
75'9 ft
16.55 psf
24.07 psf
57 ft
16.05 psf
22.60 psf
39 ft
15.06 psf
20.81 psf
20 ft
15 ft
13.87 psf
12.07 psf
0 ft
11.39 psf
8.37 psf
6.74 psf
South Elevation- E/W Pressures

Figure 7

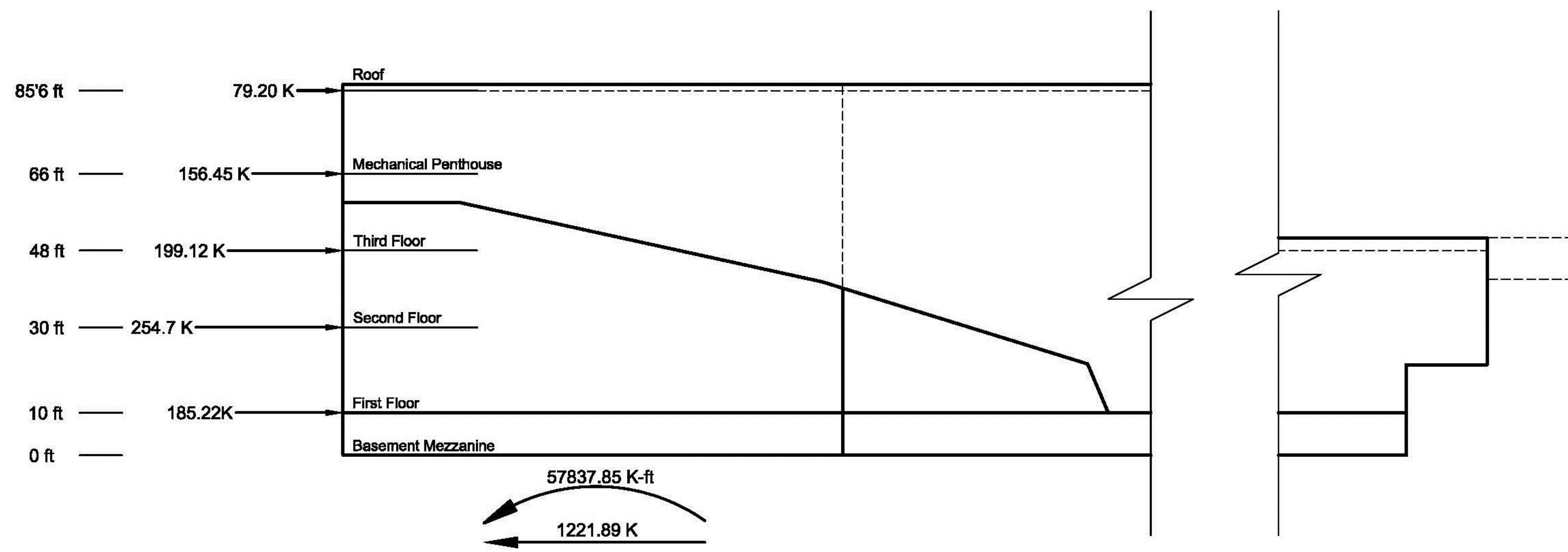



North Wing Section- N/S Pressures

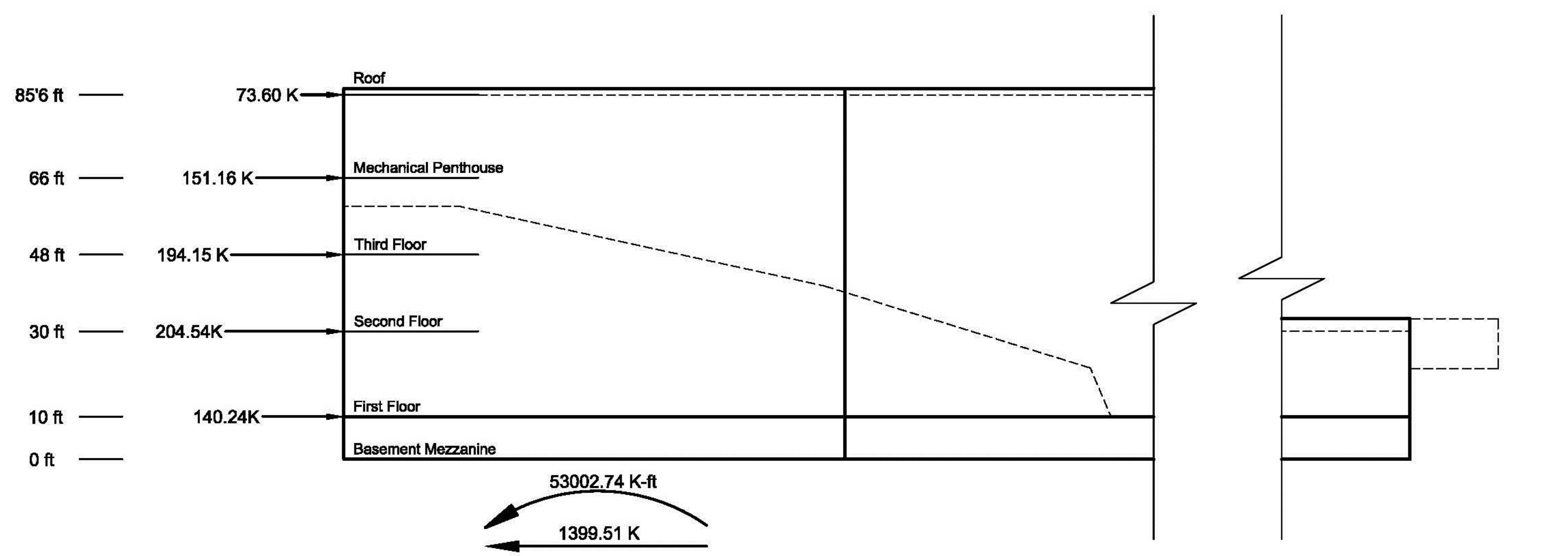



West Wing Section- E/W Pressures

Figure 8

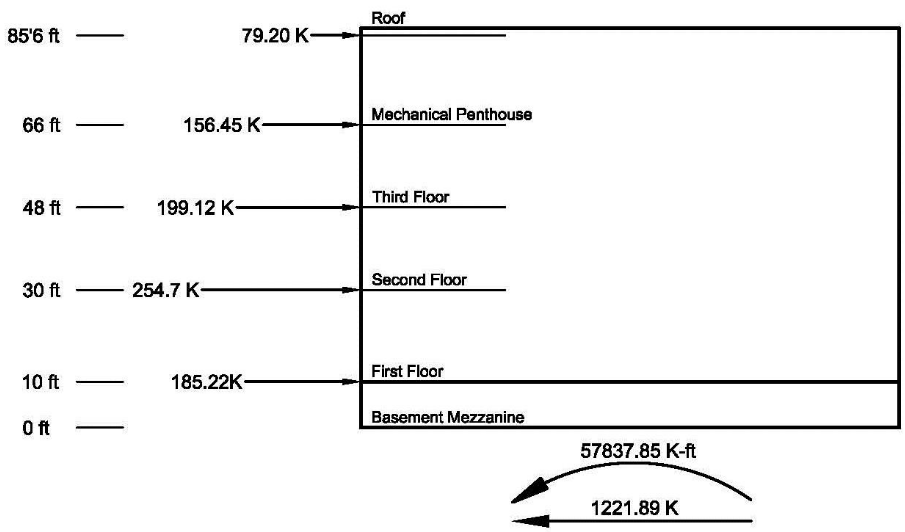



West Elevation- N/S Loads

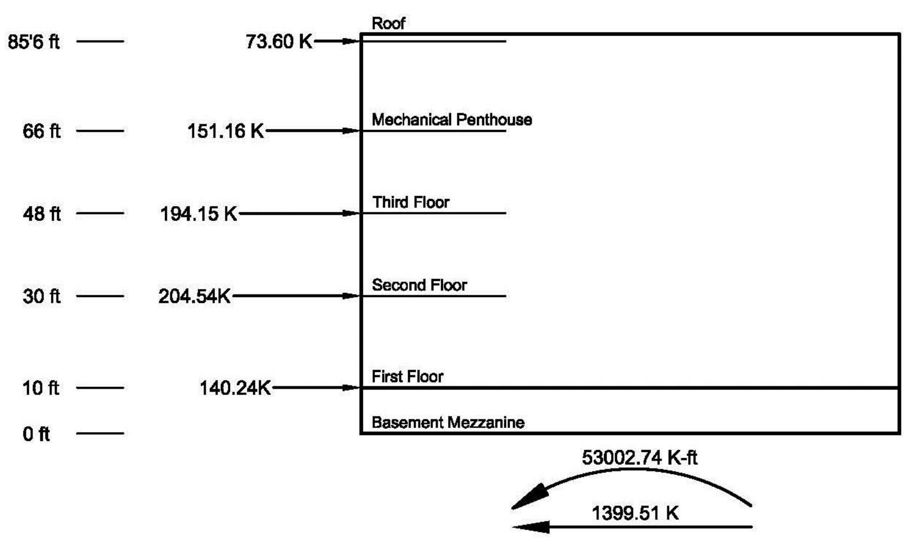



South Elevation- E/W Loads

Figure 9




North Wing Section- E/W Loads




West Wing Section- E/W Loads

Figure 10

## *Seismic Loads*

Lateral Loads due to seismic design requirements were calculated per ASCE7-05 chapters 11 and 12. Equivalent lateral force method was determined to be adequate for this analysis. Spectral design response coefficients for short and long periods were achieved using the USGS website and the specific site location in State College, PA. These coefficients are smaller than the ones used by the structural engineer potentially decreasing the design seismic base shear. The effective weight of the building was determined to be used in the calculation of seismic base shear including weight of columns, beams, diagonal bracing, slabs, precast façade panels, and all applicable superimposed dead loads including the green roof loads. The seismic design loads have been assumed equal in for both the North-South and East-West directions due to similar lateral systems in each wing, essentially symmetric about the cantilever system. An in depth seismic design load calculation including building weights and seismic coefficients is reported in Appendix D: Seismic Analysis.

Included below is a seismic load distribution of the design base shear to the floors above grade or without slab on grade to achieve effective shears at each floor. These lateral forces are assumed to be loaded at the center of mass of all floor systems, each of which is assumed to be a rigid diaphragm connecting all lateral resisting elements in the building.

**Seismic Load Distribution**

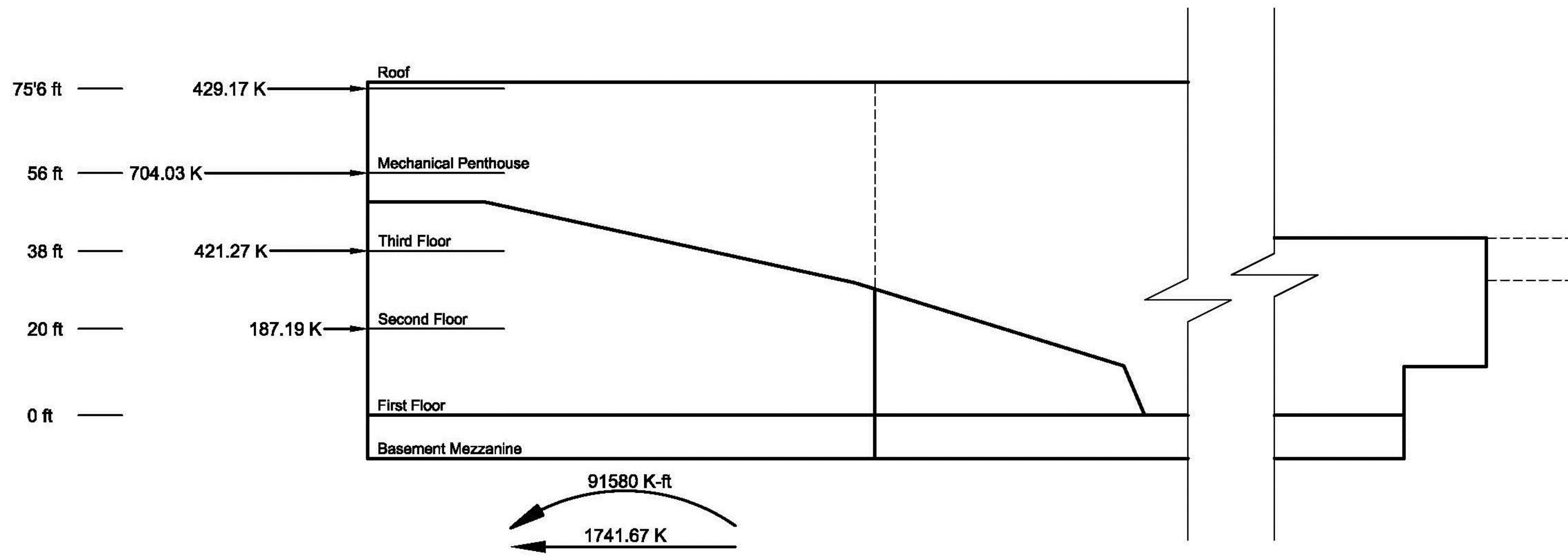
X/Y-Direction Loading

| | | |
|---|---|---|
| T= | 0.871 | s |
| k= | 1.185 | |
| $V_b$= | 1741.67 | kips |

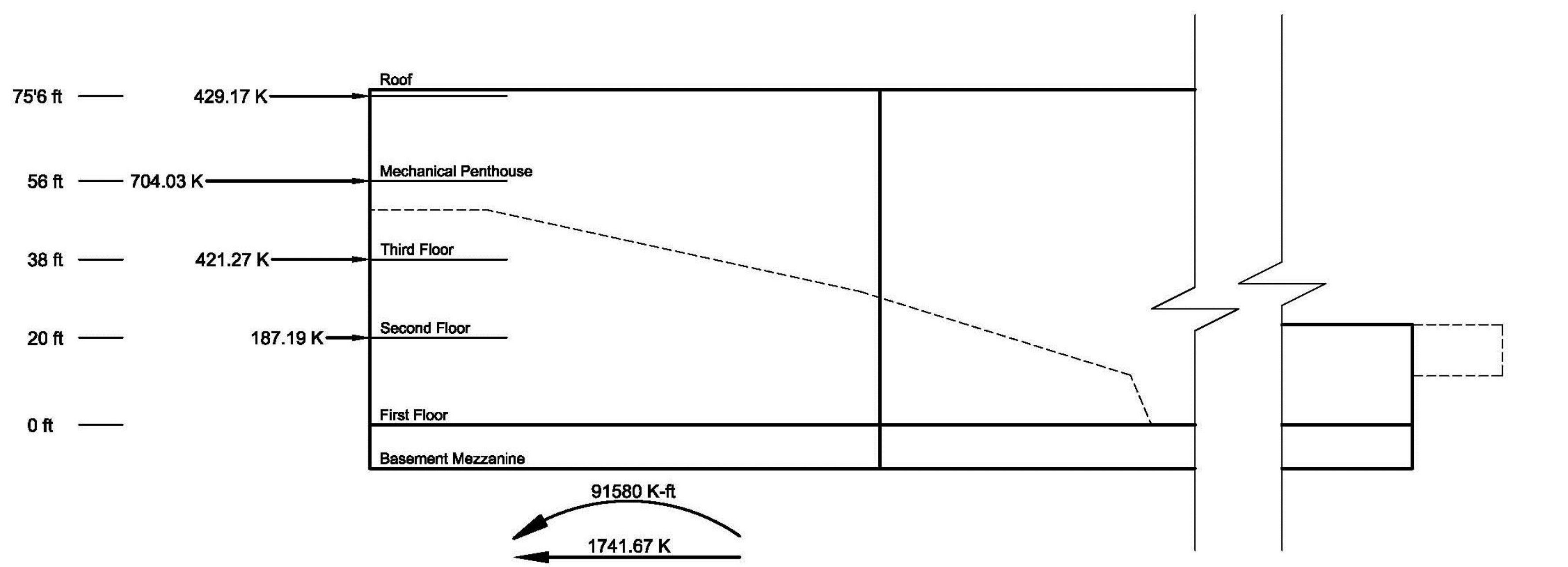
| Level i | Story Height $h_i$ (ft) | Effective Height h (ft) | Story Weight w (K) | $w*h^k$ | $C_{VX}$ | Lateral Force $f_i$ (K) | Story Shear $V_i$ (K) | Mi (K-ft) |
|---|---|---|---|---|---|---|---|---|
| Roof | 19.5 | 75.5 | 4165.67 | 701200 | 0.246 | 429.17 | 429 | 32402.14 |
| Mech. | 18.0 | 56.0 | 9738.01 | 1150294 | 0.404 | 704.03 | 1133 | 39425.91 |
| 3 | 18.0 | 38.0 | 9227.25 | 688304 | 0.242 | 421.27 | 1554 | 16008.44 |
| 2 | 20.0 | 20.0 | 8774.75 | 305846 | 0.107 | 187.19 | 1742 | 3743.844 |
| **Totals** | 75.5 | 75.5 | 31905.68 | 2845644 | 1.000 | 1741.67 | 1742 | 91580 |

From the seismic load distribution table above the total calculated design base shear due to seismic loading is 1742 kips. This is the North-South and East West direction seismic loading. Notice the seismic design base shear is larger than the controlling North-South design wind loading of 1399.5 kips. It is not typical that the base shear due to seismic design loads would control over wind in the State College area. It would be expected that wind loads should control the base shear. However, the calculated wind pressures are relatively low as the building is only 87 feet in height from Pollock Road. Also this is a laboratory building and as calculated it is a very massive building in all systems. The massiveness and small natural period of the building could have caused the seismic design loads to increase higher than expected, in this case higher than the total wind loading. No comparisons have been made to the seismic analysis carried out by the structural engineer due to lack of contact and unavailable calculations. When contact is made and calculations are received this check can be made.

Diagrams showing the distribution of these seismic forces are reported below in Figure 11 and Figure 12. No effective load was distributed to the first floor because the first floor is a slab on grade in both wings where no basement exists underneath. The remaining area of the first floor lies in the cantilever plaza which has no structure above it.

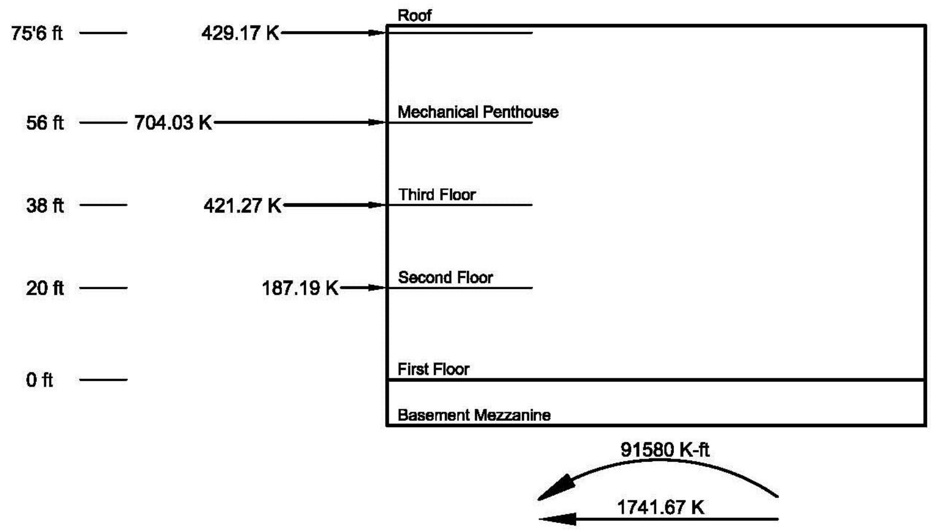



West Elevation- N/S Loads

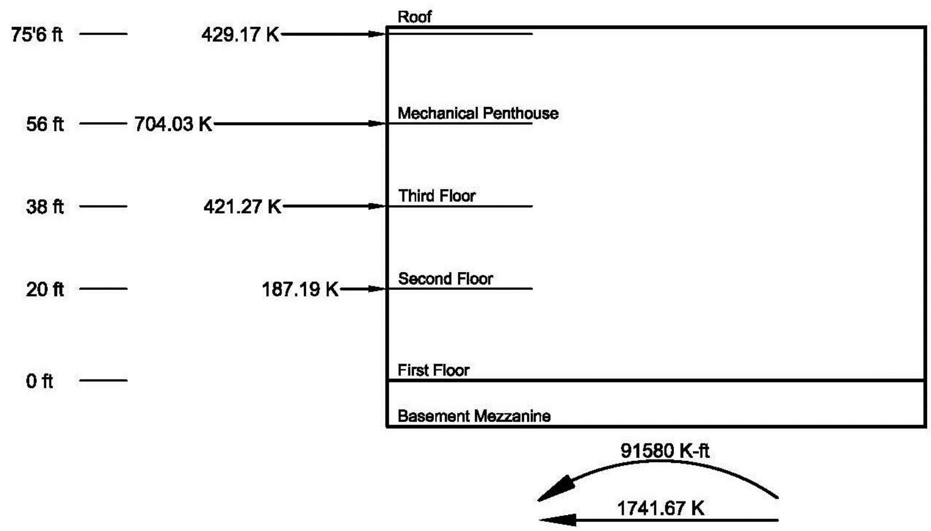



South Elevation- E/W Loads

Figure 11




North Wing Section- N/S Loads




West Wing Section- E/W Loads

Figure 12

# Typical Spot Checks

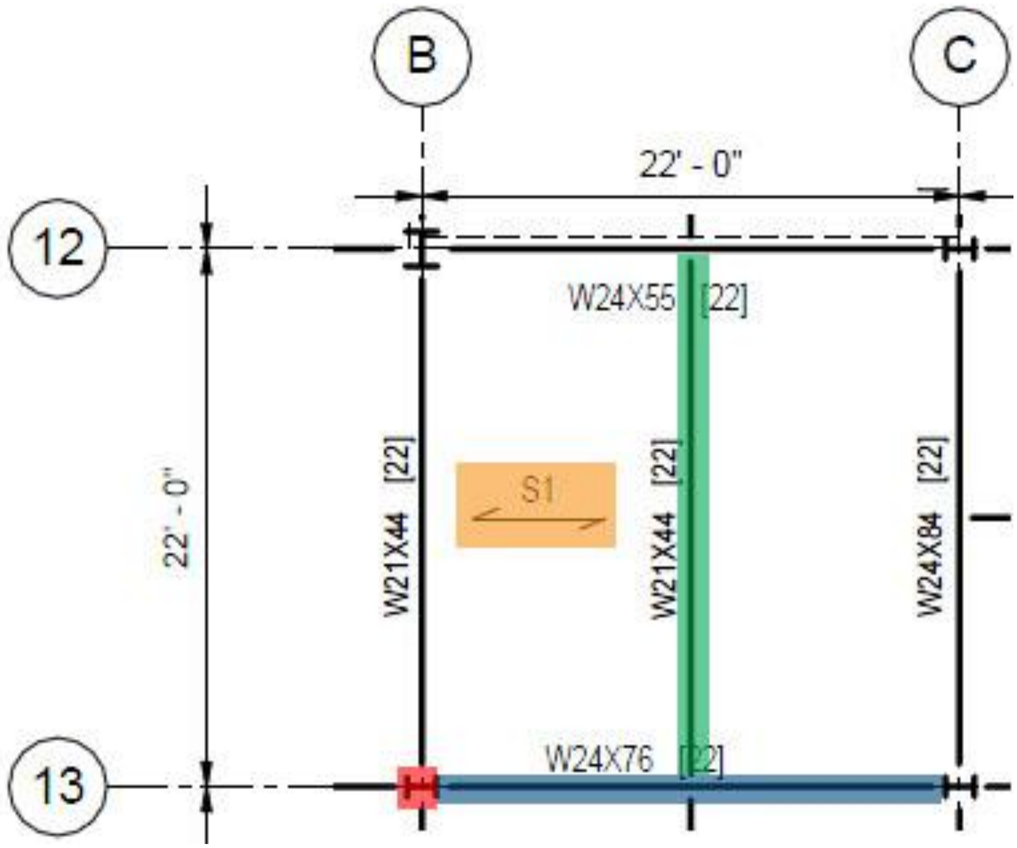



Figure 13: Figure of typical interior gravity frame used for spot checks. Orange: composite metal deck, green: composite beam, blue: composite girder, and red: column.

## Metal Decking

As indicated in the structural drawings, the slab for this typical bay is slab S1. This designates a total slab depth of 6 ¼" lightweight concrete on 3"-18 gage metal deck with WWF6x6 – W1.4xW1.4 and 4 ½" stud length. Section 05300 of the specification indicates that United Steel Deck, Inc. shall be used as the manufacturer for steel deck on this project unless otherwise specified. From the drawings we can see that the max span shall be 11' in a 3-span or more condition and the fire rating of this assembly must be 2 hours. From the tables provided in the United Steel Deck Design Manual and Catalog of Products: Steel Decks for Floors and Roofs (See Figure 14: USD deck chart: LWT concrete on 3" x 12" metal deck), it can be seen that the ideal gage and depth of decking for the given unshored 3-span condition of 11' is 18 gage 3" LOK – Floor with lightweight concrete. After finding the ideal deck gage and size we consult the provided U.L. Fire Ratings table for Composite Deck. Given no fireproofing on the deck, min 2-hr restrained assembly rating, and 3 LOK –Floor decking we can determine that the minimum concrete topping required is 3 1/4". Checking for adequate strength in the deck tables we can see that this deck assembly will support a 300 psf Uniform Live Service Load, when 1 stud/ft is used. Given the loads used by Thornton Tomasetti Engineers, Live Load of 100 and Superimposed Dead Load of 30, the deck strength of 300 psf is adequate to carry the design loads of 130 psf.

DECK PROPERTIES

| Gage | t | w | As | I | $S_p$ | $S_n$ | $R_b$ | $\phi V_n$ | studs |
|---|---|---|---|---|---|---|---|---|---|
| 22 | 0.0295 | 1.7 | 0.505 | 0.797 | 0.454 | 0.500 | 718 | 2190 | 0.49 |
| 20 | 0.0358 | 2.1 | 0.610 | 0.993 | 0.583 | 0.620 | 1020 | 3220 | 0.59 |
| 19 | 0.0418 | 2.4 | 0.710 | 1.158 | 0.708 | 0.726 | 1350 | 4310 | 0.69 |
| 18 | 0.0474 | 2.8 | 0.810 | 1.324 | 0.832 | 0.832 | 1720 | 4880 | 0.79 |
| 16 | 0.0598 | 3.5 | 1.020 | 1.666 | 1.045 | 1.045 | 2540 | 6130 | 0.99 |

COMPOSITE PROPERTIES (18 gage)

| Slab Depth | $\phi M_d$ in.k | $A_c$ $in^2$ | Vol. $ft^3/ft^2$ | W psf | $S_c$ $in^3$ | $I_{av}$ $in^4$ | $\phi M_{no}$ in.k | $\phi V_{nt}$ lbs. | Max. unshored spans, ft. 1span | 2span | 3span | $A_{wwf}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 5.50 | 80.96 | 37.6 | 0.333 | 38 | 1.94 | 9.1 | 54.28 | 5250 | 11.48 | 13.61 | 14.07 | 0.023 |
| 6.00 | 92.32 | 42.0 | 0.375 | 43 | 2.23 | 11.6 | 62.43 | 5870 | 10.94 | 13.07 | 13.51 | 0.027 |
| 6.25 | 98.00 | 44.3 | 0.396 | 46 | 2.38 | 13.0 | 66.67 | 6180 | 10.70 | 12.83 | 13.26 | 0.029 |
| 6.50 | 103.68 | 46.6 | 0.417 | 48 | 2.53 | 14.5 | 70.99 | 6510 | 10.48 | 12.59 | 13.01 | 0.032 |
| 7.00 | 115.04 | 51.3 | 0.458 | 53 | 2.85 | 17.9 | 79.88 | 7170 | 10.07 | 12.16 | 12.57 | 0.036 |
| 7.25 | 120.72 | 53.8 | 0.479 | 55 | 3.01 | 19.8 | 84.42 | 7510 | 9.88 | 11.96 | 12.36 | 0.038 |
| 7.50 | 126.40 | 56.3 | 0.500 | 58 | 3.17 | 21.8 | 89.03 | 7860 | 9.71 | 11.77 | 12.16 | 0.041 |
| 8.00 | 137.76 | 61.3 | 0.542 | 62 | 3.51 | 26.2 | 98.39 | 8570 | 9.43 | 11.42 | 11.80 | 0.045 |
| 8.25 | 143.44 | 63.9 | 0.563 | 65 | 3.68 | 28.6 | 103.15 | 8930 | 9.33 | 11.25 | 11.62 | 0.047 |
| 8.50 | 149.12 | 66.6 | 0.583 | 67 | 3.85 | 31.1 | 107.94 | 9300 | 9.23 | 11.09 | 11.46 | 0.050 |

L, Uniform Live Service Loads, psf * (18 gage)

| Slab Depth | $\phi Mn$ in.k | 9.00 | 9.50 | 10.00 | 10.50 | 11.00 | 11.50 | 12.00 | 12.50 | 13.00 | 13.50 | 14.00 | 14.50 | 15.00 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 5.50 | 80.96 | 385 | 345 | 305 | 275 | 250 | 225 | 205 | 185 | 170 | 155 | 140 | 130 | 120 |
| 6.00 | 92.32 | 400 | 390 | 350 | 315 | 285 | 255 | 235 | 210 | 195 | 175 | 160 | 150 | 135 |
| 6.25 | 98.00 | 400 | 400 | 370 | 335 | 300 | 275 | 245 | 225 | 205 | 190 | 170 | 160 | 145 |
| 6.50 | 103.68 | 400 | 400 | 395 | 355 | 320 | 290 | 260 | 240 | 220 | 200 | 180 | 165 | 155 |
| 7.00 | 115.04 | 400 | 400 | 400 | 395 | 355 | 320 | 290 | 265 | 240 | 220 | 205 | 185 | 170 |
| 7.25 | 120.72 | 400 | 400 | 400 | 400 | 370 | 335 | 305 | 280 | 255 | 235 | 215 | 195 | 180 |
| 7.50 | 126.40 | 400 | 400 | 400 | 400 | 390 | 355 | 320 | 290 | 265 | 245 | 225 | 205 | 190 |
| 8.00 | 137.76 | 400 | 400 | 400 | 400 | 400 | 385 | 350 | 320 | 290 | 265 | 245 | 225 | 205 |

Figure 14: USD deck chart: LWT concrete on 3" x 12" metal deck

## *Typical Composite Beam*

One typical beam size for the Millennium Science Complex is a W21x44 [22]. Figure 13 shows the location of this beam in the typical frame with a span of 22ft and spacing of 11ft. The beams were checked for shear strength, flexural strength, and deflection requirements. A thorough outline of these calculations can be found in Appendix B: Spot Checks.

From composite beam calculations it was determined that the member specified is more than adequate for shear and flexural strength, as well as, deflection requirements. The number of shear studs calculated was less than the number specified by the structural designer however it is assumed that 1 stud per foot was used for ease of constructability.

## *Typical Composite Girder*

One typical girder size for the Millennium Science Complex is a W24x76 [22]. Figure 13 shows the location of this girder in the typical frame with a span of 22ft and spacing of 22ft. Along with the beam, the girder was also checked for shear strength, flexural strength, and deflection requirements. Calculations pertaining to this composite girder can be found in Appendix B: Spot Checks.

From the typical composite girder calculations all member strengths and deflection requirements were determined to be adequate, with the exception of the number of shear studs specified. The flexural strength of the member is adequate and very efficient having only an additional available load capacity of 1% of the member strength. The controlling case for this member is assumed to be the member strength. This is based on minimal available load capacity noted in the previous statement. The discrepancy of the required shear studs being greater than the shear studs specified is speculated to be due to the fact that the calculations were performed neglecting the area of concrete below the top of steel deck. If the concrete below the top of steel deck was taken into consideration this could balance the shear stud strength thereby decreasing the required amount of shear studs.

## *Typical Column*

One typical column for the Millennium Science Complex is a W14x68. The specific column location analyzed for this spot check is located at column lines B and 13. This column extends from the basement level, where it is supported by a concrete pier, to the floor of the Mechanical Penthouse level which, because of the progressive floor setbacks, is also the base of a green roof above the 3rd floor. The column spot checked for this report is between the 2nd and 3rd floor, supporting both the 3rd floor and the exposed green roof. The most economical W14 section that could be used for this location was determined to be a W14x61 which is one size smaller than what was specified in the structural drawings. This discrepancy is most likely due to constructability purposes, because columns on lower floors would necessitate a larger section due to the increasing load. Thereby it is easiest just to use the same section throughout the height of the building at this column location. Another possible reason for this discrepancy could be due to the fact that the live load at the green roof level is greater than 100 psf making it unreducible. With this in mind the structural engineer could have chosen to neglect the effects of live load reduction throughout the height of the building. Column load takedowns and spot check calculations can be found in Appendix B: Spot Checks.

# IPD/BIM Thesis Existing Conditions and Modeling

IPD/BIM Thesis, exclusive to Penn State University's Architectural Engineering fifth year thesis, is an interdisciplinary program focused on the future of the building industry. Using integrated project delivery (IPD) as a delivery process and a main theme throughout the year long process, each group of four AE students, one from each of the discipline options, will analyze a building as the typical thesis project would, but the main focus instead of option specific will be centered around the coordination and optimization of all building systems as one collaborative effort. The use of building information modeling (BIM) through this process will help to facilitate the collaborative effort. Through the use and interoperability of multiple computer program platforms that contain inherent building component information, analysis, design, and 3-dimensional building coordination can become one process. Achieving building optimization through the use of BIM with IPD as the backbone is the essence of the IPD/BIM Thesis.

The first step in this process is to report on the existing conditions of the building. In general this consists of Technical Report 1, which for the most part is discipline specific. Even though it is a collaborative thesis the existing conditions from each system are important before moving on to attempt the redesign of any part of such systems. Therefore the authors of this technical report have compiled all of the available information and design criteria for the existing structural system. However, the IPD process need not be discarded. It is important to think ahead and prepare for the collaborative effort.

To ensure that the IPD process has roots in this project from the beginning, it benefits all three teams involved to set initial goals focused on collaboration to carry through the entire process. The availability and inherent requirement to attempt to use BIM where applicable is the best tool at hand to set up a foundation for future collaboration. Therefore as one large group, all three teams combined, a decision has been made about how each team will execute 3-d collaboration throughout the process. Autodesk Revit 2011 is to be this foundation. Disciplinary models have been received from the Whiting Turner, the construction management contract, which have been used for 3-d collaboration through the design and construction process on the actual project. Each team will keep and update a central Revit Architecture Model. Site, structural, and MEP models will be linked in to this model as overlays. Each of the disciplinary models will also simultaneously link to the models of other disciplines. These simultaneous links will ensure 3-d confirmation of any changes immediately and provide instant feedback to the individual and the team. Each of the disciplinary models will be organized themselves with internal worksets to organize individual components into categorical groups that can be edited individually allowing two separate individuals in two separate locations on the same network to work on the same model simultaneously if need be. This framework has been set up and is active. At this point each discipline has embarked on an existing conditions modeling journey necessary before moving forward with the models at hand.

3-d collaboration and coordination is the most important use of our modeling. The Millennium Science Complex will take all the coordination possible. A laboratory building with complex systems is complicated enough just to decide what to model. Therefore the students of IPD/BIM Thesis as a whole have selected the third floor as the sole location to study the coordination of all existing systems and to come back to when coordinating new systems or changes to the existing systems. Although this does not consider the coordination issue in the rest of the building, simplifying to only the third floor will emphasize the importance of 3d coordination while allowing enough time to focus on optimizing and redesigning the building systems. The coordination of the entire building would be an entire project on its own and would need full time attention. The reason behind the choice of the third floor is simple; there is some of everything the building has to offer. It resembles the typical floor of Millennium Science Complex. The occupancy of the third floor consists of laboratories and offices in both wings connected with a café at the intersection of the wings. Structurally it includes part of the typical gravity frames, the cantilever truss and shear walls and has a typical floor assembly.

Each discipline and or group will decide what is necessary to model in order to remain in line with individual and group BIM goals. Also in the same respect other platforms may be used to interface with the database of the Revit models. This will most likely vary by discipline by at large will assist individuals in creating an information link between analysis and conceptual ideas to the actual model used for 3-d coordination. In the specific case of structure it will be necessary to create an analytical model of the building structure in a structural analysis program. This will be used to apply forces to a modeled structure and obtain feedback about the behavior of the structure from member forces, shears, and moments, as well as dynamic characteristics. This can be used in redesign of the structure and then interfaced with Revit Structure 2011 to update the collaborative BIM model. The structural students have selected SAP 2000 as the structural modeling program of choice. The next step will be to model the existing structure in SAP.

# Conclusion

After performing an in depth load analysis, gravity member checks, and existing conditions study, the authors have obtained a thorough grasp of the structural system of the Millennium Science Complex. This report has shown that the structural designers have developed an adequate design that elegantly supports the ideas of the architect.

Using the knowledge gained from Technical Report 1: Existing Conditions the authors are prepared to move forward onto Technical Report 2. The focus for Tech. 2 will be the design of 3 alternative floor systems as plausible attempts to optimize the structural characteristics of the building while monitoring the overall building performance and interconnectivity of other building systems. This research facility will pose significant challenges in redesign due to high tolerances necessary for this building type. Collaboration among options will be the key to success in completing Tech. Report 2 and the rest of the work throughout the IPD/BIM Thesis Project.

# Appendix A: Lateral System Plans and Elevations

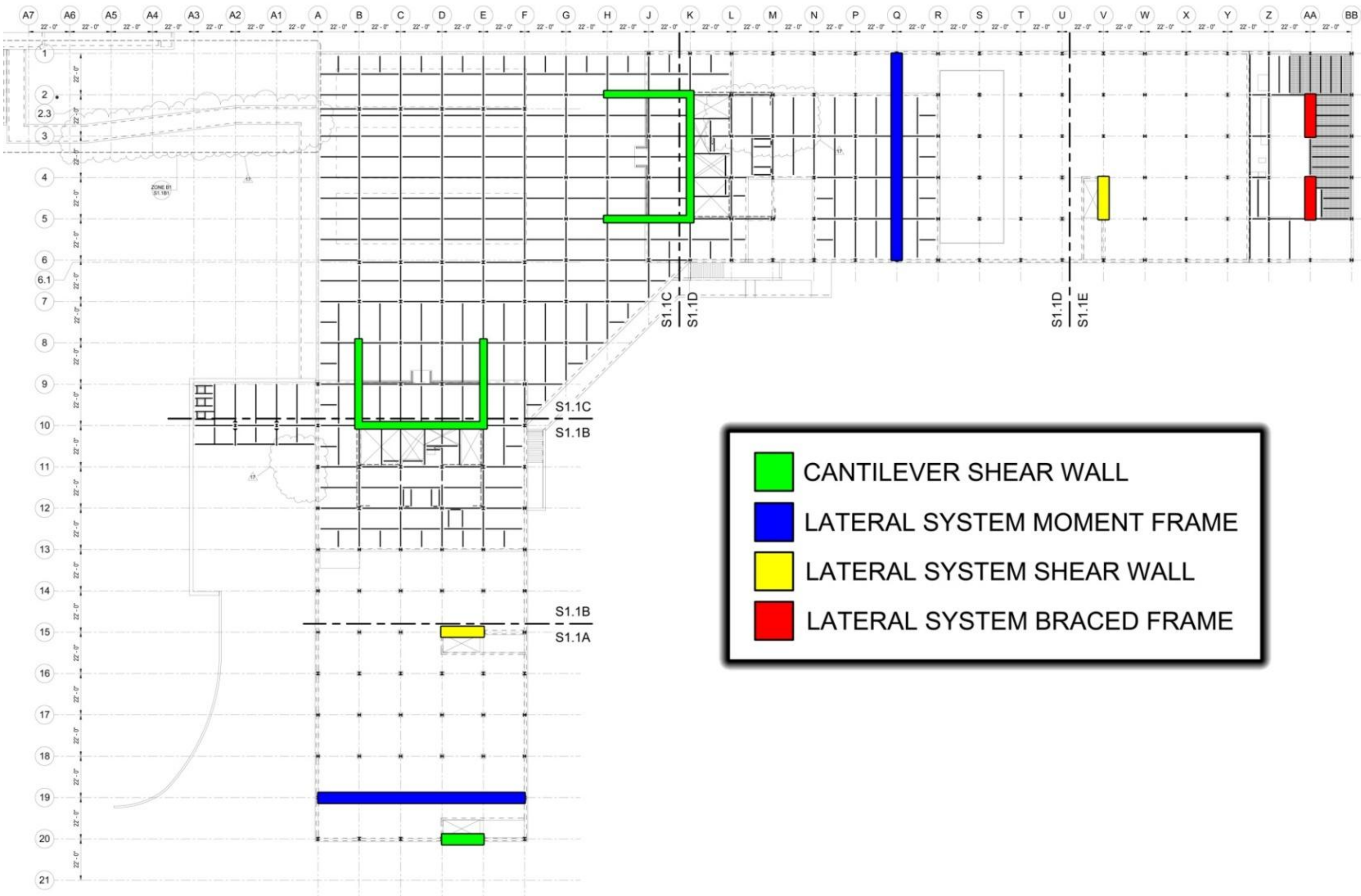



Figure 15: First floor lateral system elements

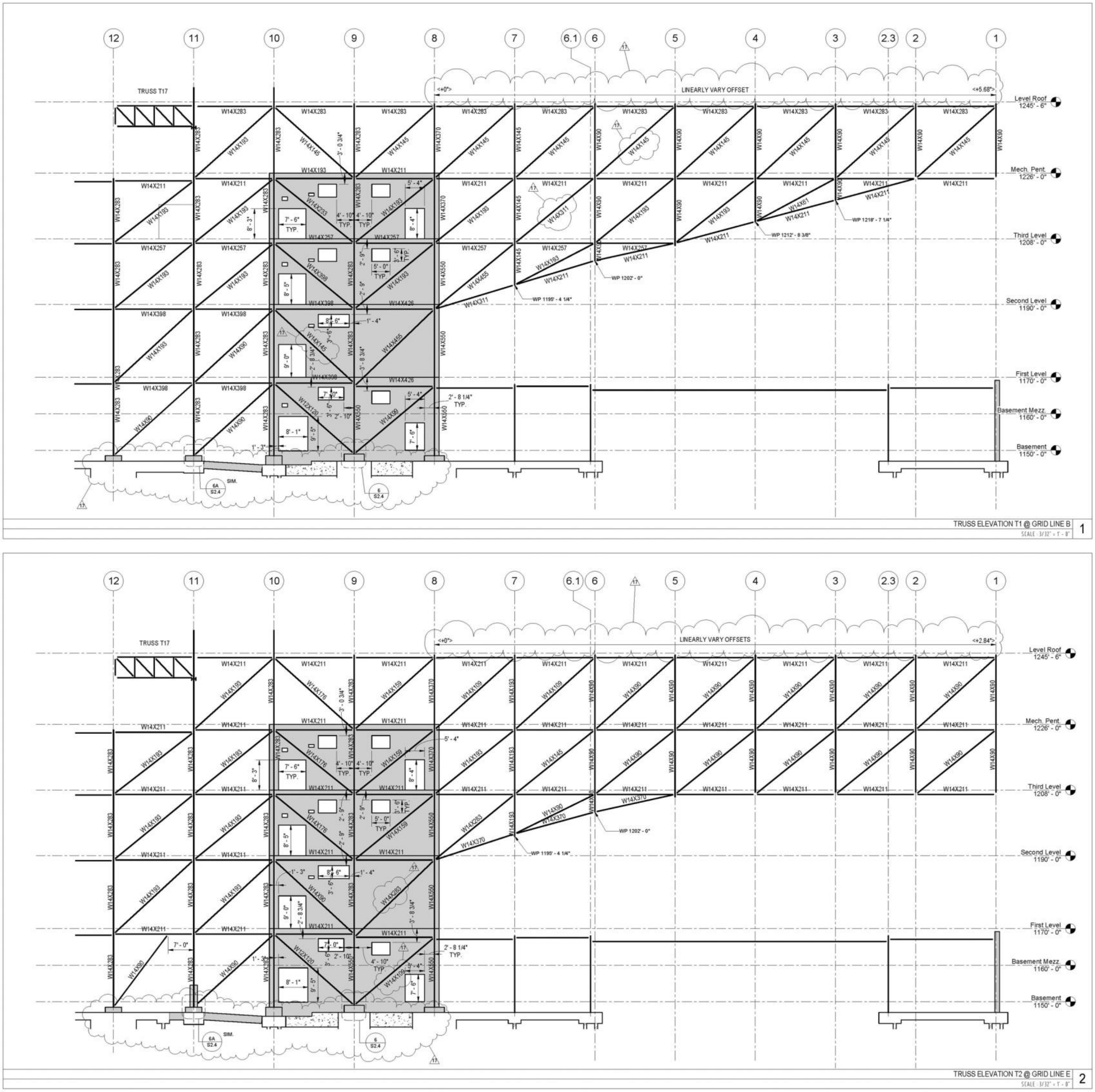



Figure 16: Truss frames at column lines 2 and 5; also replicated at column lines B and E.

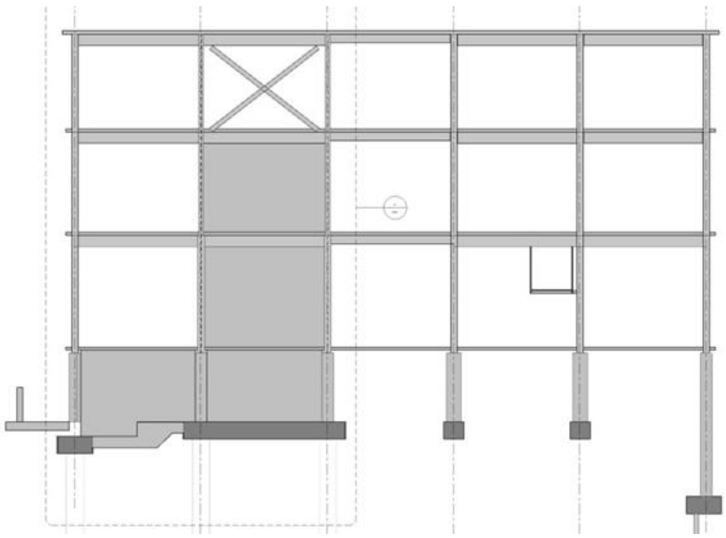


Figure 17: Shear wall at column line 15

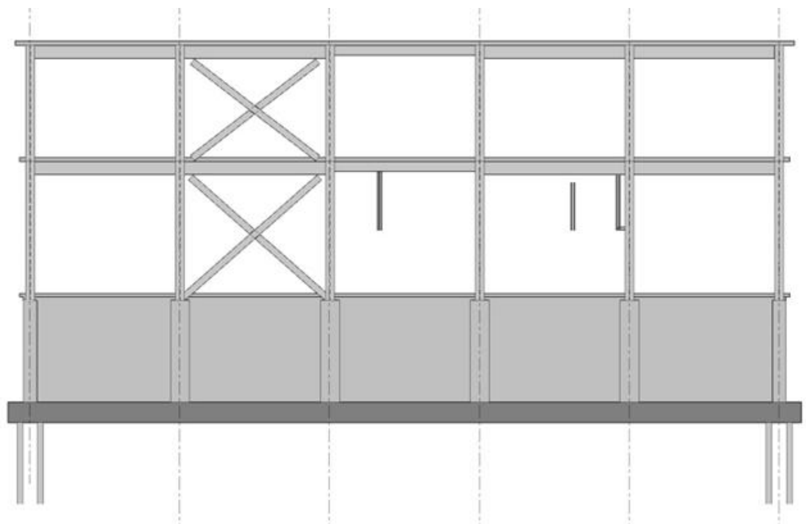


Figure 19: Braced frame at column line 20

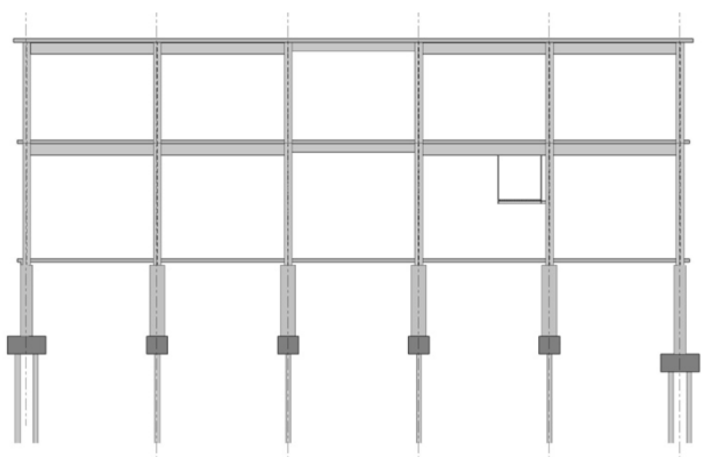


Figure 18: Moment frame at column line 19

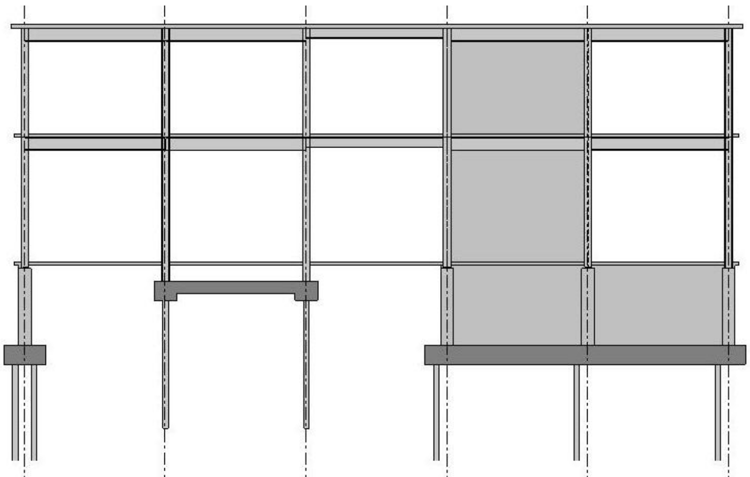


**Figure 20:** Shear wall at column line V

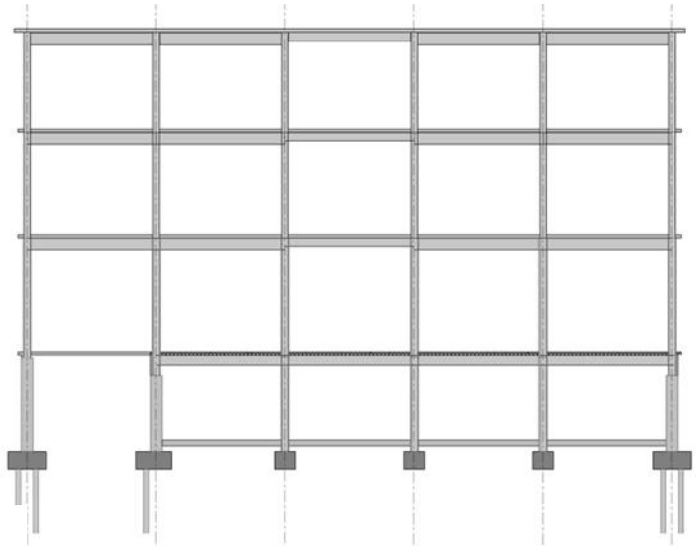


**Figure 21:** Moment frame at column line Q

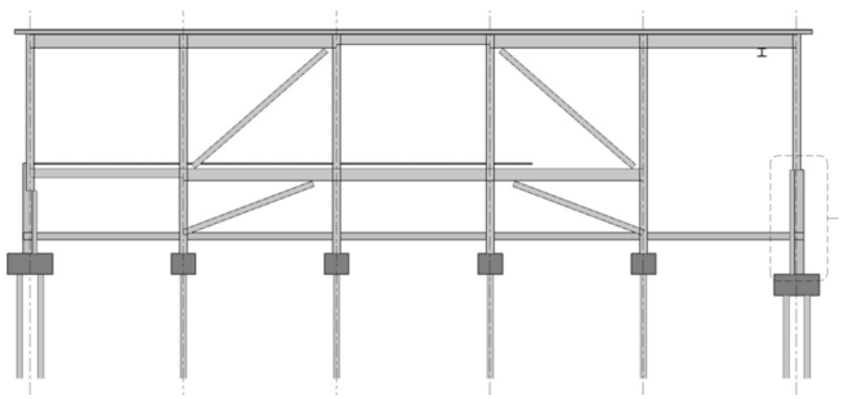


**Figure 22:** Braced frame at column line Z

# Appendix B: Spot Checks

## *Typical Composite Beam Calculations*

| | | | | |
|---|---|---|---|---|
| Bay 12-13,B-C 3rd Floor | | | | |
| Member ID | | | W21x44 | |
| | Shear Studs | | 22 | |
| Concrete | $f'_c$ | | 4 | ksi |
| | Density | | 115 | pcf |
| Beam | $F_y$ | | 50 | ksi |
| | $F_u$ | | 65 | ksi |
| | E | | 29000 | ksi |
| | I | | 843 | $in^4$ |
| | $I_{LB}$ | | 1490 | $in^4$ |
| | spacing | | 11 | ft |
| | span | | 22 | ft |
| | $b_{eff}$ | | 66 | in |
| | t | | 6.25 | in |
| | Assumed a | | 1 | in |
| | $Y_2$=t-a/2 | | 5.5 | |
| Loads | Dead Load | | | |
| | | Slab SW | 0.0500 | ksf |
| | | Beam SW | 0.0440 | klf |
| | | SDL | 0.0300 | ksf |
| | | Total DL | 0.1240 | |
| | Live Load | | | |
| | | floors supporting | 1 | |
| | | $L_o$ | 0.150 | ksf |
| | | $K_{LL}$ | 2 | |
| | | $A_T$ | 242 | sf |
| | | $K_{LL}A_T$ | 484 | |
| | | Reduction Coeff. | Unreducible | |
| | | L | 0.150 | ksf |
| | | Construction LL | 0.02 | |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| **Check Member Strength** | | | | | | | | |
| $w_u$=1.2D+1.6L | 4.2768 | klf | | | | | | |
| $V_u=w_uL/2$ | 47.0 | ≤ | $\phi V_n$ | 217 | k | OK | Table 3-2 |
| $M_u=w_uL^2/8$ | 258.7 | ≤ | $\phi M_n$ | 522 | ft-k | OK | Table 3-19 |
| **Check Shear Studs** | | | | | | | |
| $\Sigma Q_n$ | 162 | | | | k | | Table 3-19 |
| $Q_n$ | 17.2 | | | | k | | Table 3-21 |
| # of Studs = $(\Sigma Q_n/Q_n)$*2 | 20 | ≤ | # Studs Specified | 22 | | OK | |
| **Check a** | | | | | | | |
| a | 0.721925134 | ≤ | assumed a | 1 | in | OK | |
| **Check Unshored Strength** | | | | | | | |
| $w_u$=1.4D | 0.832 | klf | | | | | |
| $w_u$=1.2D+1.6L | 1.065 | klf | | | | | |
| $M_u=w_uL^2/8$ | 64.4 | ≤ | $\phi_b M_p$ | 358 | ft-k | OK | Table 3-19 |
| **Deflection Check** | | | | | | | |
| $\Delta_{LL}=5w_{LL}L^4/384EI_{LB}$ | 0.201 | ≤ | $\Delta_{LL}$=L/360 | 0.73 | in | OK | |
| $\Delta_{CDL}=5w_{CDL}L^4/384EI$ | 0.223 | ≤ | $\Delta_{LL}$=L/240 | 1.10 | in | OK | |

## *Typical Composite Girder Calculations*

| | | | | |
|---|---|---|---|---|
| Bay 12-13,B-C 3rd Floor | | | | |
| Member ID | | | W24x76 | |
| | Shear Studs | | 22 | |
| Concrete | $f'_c$ | | 4 | ksi |
| | Density | | 115 | pcf |
| Steel | $F_y$ | | 50 | ksi |
| | $F_u$ | | 65 | ksi |
| | E | | 29000 | ksi |
| Girder | I | | 2100 | $in^4$ |
| | $I_{LB}$ | | 3470 | $in^4$ |
| | $b_{eff}$ | | 66 | in |
| | t | | 6.25 | in |
| | span | | 22 | ft |
| | Assumed a | | 1.25 | in |
| | $Y_2$=t-a/2 | | 5.5 | |
| | | | | |
| Beam | spacing | | 11 | ft |
| | span | | 22 | ft |
| | | | | |
| Loads | Dead Load | | | |
| | | Slab SW | 0.0500 | ksf |
| | | Girder SW | 0.0760 | klf |
| | | SDL | 0.0300 | ksf |
| | | Total DL | 0.1560 | |
| | Live Load | | | |
| | | floors supporting | 1 | |
| | | $L_o$ | 0.150 | ksf |
| | | $K_{LL}$ | 2 | |
| | | $A_T$ | 484 | sf |
| | | $K_{LL}A_T$ | 968 | |
| | | Reduction Coeff. | Unreducible | |
| | | L | 0.150 | ksf |
| | | | | |
| | | Construction LL | 0.02 | ksf |
| | | Construction DL (on beam) | 0.594 | klf |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Check Member Strength** | | | | | | | |
| $P_u=w_{uBeam}L_{Beam}$ | 94.6 | k | | | | | |
| $V_u=P_u/2$ | 47.3 | ≤ | $\phi V_n$ | 316 | k | OK | Table 3-2 |
| $M_u=P_uL/4$ | 1040.6 | ≤ | $\phi M_n$ | 1050 | ft-k | OK | Table 3-19 |
| **Check Shear Studs** | | | | | | | |
| $\Sigma Q_n$ | 280 | | | | k | | Table 3-19 |
| $Q_n$ | 21.2 | | | | k | | Table 3-21 |
| # of Studs = $(\Sigma Q_n/Q_n)*2$ | 28 | > | # Studs Specified | 22 | | No Good | |
| **Check a** | | | | | | | |
| a | 1.248 | ≤ | assumed a | 1.25 | in | OK | |
| **Check Unshored Strength** | | | | | | | |
| $w_u=1.4D$ | 0.143 | klf | | | | | |
| $P_u=$ | 23.4 | k | | | | | |
| $M_u=(w_uL^2/8)+(Pl/4)$ | 137.5 | ≤ | $\phi_b M_p$ | 750 | ft-k | OK | Table 3-19 |
| **Deflection Check** | | | | | | | |
| $\Delta_{LL}=PL^3/48EI_{LB}$ | 0.360 | ≤ | $\Delta_{LL}=L/360$ | 0.73 | in | OK | |
| $\Delta_{CDL}=PL^3/48EI$ | 0.083 | ≤ | $\Delta_{CDL}=L/240$ | 1.10 | in | OK | |

## *Typical Column Calculations*

| Col B-13 | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Story Level | Columns Below Level | Floor to Floor Height | Tributary Area | Live Load Influence Area | Live Load Reduction Factor | Dead Load | | | Roof Live Load | | Snow Load | Floor Live Load | | Column Load | Column Load | Column Load | Column Design Load |
| | | | $A_T$ | $K_{LL}A_T$ | | | Col SW | Total | Unreduced | Reduced | | Unreduced | Reduced | 1.4D | 1.2D + 1.6L + 0.5(Lr or S) | 1.2D + 1.6(Lr or S) + 0.5L | |
| | | ft | $ft^2$ | $ft^2$ | | ksf | k/ft | k | ksf | | ksf | ksf | | kips | kips | kips | kips |
| 4 Mech. | 1 | 18 | 484 | 1936 | 0.59 | 0.2 | 0.068 | 96.088 | 0.03 | 0.018 | 0.028 | 0.000 | 0.000 | 134.52 | 122.08 | 136.99 | 136.99 |
| 3 | 2 | 18 | 484 | 1936 | Unreducible | 0.08 | 0.068 | 39.944 | | 0 | | 0.150 | 0.150 | 55.92 | 164.09 | 84.23 | 301.08 |
| 2 | 3 | 20 | 484 | 1936 | 0.59 | 0.08 | 0.068 | 40.08 | | 0 | | 0.100 | 0.059 | 56.11 | 93.86 | 62.40 | 394.94 |
| 1 | 4 | 20 | 242 | 968 | 0.73 | 0.08 | 0.099 | 21.34 | | 0 | | 0.100 | 0.073 | 29.88 | 53.96 | 34.47 | 448.89 |
| Basement | 5 | | 0 | Unreducible | Unreducible | | | | | 0 | | | 0.000 | 0.00 | 0.00 | 0.00 | 448.89 |

| B-13 Supporting FL3 | W14x68 | |
|---|---|---|
| $F_y$ | 50 | ksi |
| L | 18 | ft |
| $KL_x$ | 18.0 | ft |
| $KL_y$ | 18.0 | ft |
| $P_u$ | 298.1 | k |
| $\phi P_n$ | 512 @ KL=8 | k |
| $P_u = 301.08 \leq \phi P_n = 512$ | OK | |
| Min W14 | W14x61 | |
| $\phi P_n$ | 457 | k |

# Appendix C: Wind Analysis

**MWFRS Wind Analysis (ASCE7-05)- MSC Complex**

**Location:** University Park, PA

**Topography:** Campus Setting. Buildings to North, North West, West, and South. Mostly open terrain to East with small obstructions

**Bulding Dimensions:** L-Shaped. North Wing outside dimension = 550 ft, West Wing outside dimension = 440 ft.
Building Heights (From Pollock Road): 85'-6" ft to Roof level, 66 ft to Mechanical Penthouse, 48 ft to Third Floor, 30 ft to second floor, 10 ft to first floor.
Roof Step Backs: Roof steps to: North Wing- Steps Down to Mech. Penthouse level at 220 ft, Third Floor at 330 ft, and Second Floor at 440 ft.
- Same on West Wing except the last step down does not exist

**Framing:** Primarily Steel Framing- W-Flange columns, beams, and cross-bracing. The floor system is a composite beam and concrete slab on metal deck.

**Cladding:** Alternate horizontal strips of precast concrete panels and exterior glazing for each floor of elevation. Assume no debris resistant glazing.

**Roof Top:** Primary Roof consists of EPDM Walkway Pads and EODM Fooring Membrane tapered. The lower roofs are all green roofs. All roofs flat.

**a) Basic Wind Speed** (Fig. 6-1): V = 90mph

**b) Exposure:** (6.5.2.3) Exposure B: Urban/Suburban, wooded, numerous closely spaced obstructions- single family dwellings and larger.

**c)Building Classification:** Construction Type IIIB, Occ. Cat: B with special Occ. areas of H-5

**d) Velocity Pressure:** qz = 0.000256kzkztkdV2I

| kz (Table 6-3) = | 15 | 20 | 39 | 57 | 75.75 | 87 | (ft) |
|---|---|---|---|---|---|---|---|
| | 0.575 | 0.624 | 0.755 | 0.842 | 0.913 | 0.950 | |

| | | |
|---|---|---|
| kzt (Fig. 6-4) = | 1 | , assume homo-topo |
| kd (Table 6-4) = | 0.85 | , buildings |
| $V^2 = 90^2$ = | 8100 | |
| I (Table 6-1) = | 1.15 | |
| **qz =** | **20.27** | *kz **psf** *depends on height |

**e) Gust Effect Factor:** G= 0.85 (Rigid Structure T< 1.0s, refer to Seismic Analysis)

**f) Internal Pressure Coefficient:** GCpi = +/- 0.18 (assume Enclosed Building)

**g) Design Wind Pressures:** P = qGCp - qi(Gcpi)

q = qz (windward, depends on height)
q = qh (leeward, taken at height-h)
G = 0.85
qi = qh (windward, leeward, and roofs for enclosed buildings)
GCpi = +/- 0.18

**Cp values determined:**

**h) Wall Cp:** (Fig. 6-6 cont'd)

| | | |
|---|---|---|
| Cpw = | 0.8 | (windward, with qz) |
| Cpsw = | -0.07 | (side walls, with qh) |
| Cpl= | -0.5 | L/B = 0-1 (Leeward, with qh) |
| | -0.3 | L/B = 2 |
| | -0.2 | L/B = > 4 |

**i) Roof Cp:** (Fig. 6-6 cont'd)

angle < 10deg., h/L < 0.5

| Hor. Dist. From Wind. Edge | Cp (1st) | Cp (2nd) |
|---|---|---|
| 0-h/2, h/2-h | -0.9 | -0.18 |
| h-2h | -0.5 | -0.18 |
| > 2h | -0.3 | -0.18 |

**j) MWFRS Pressures** P = qGCp - qi(Gcpi)

**Terrain Exposure Constants**

| Exposure | α | Zg(ft) | â | b^ | α– | b- | c |
|---|---|---|---|---|---|---|---|
| B | 7.00 | 1200.00 | 0.14 | 0.84 | 0.25 | 0.45 | 0.30 |

| l (ft) | ε | Zmin (ft) |
|---|---|---|
| 320.00 | 0.33 | 30.00 |

**Windward Walls**

| Height | qz | G | Cp | **qzGCp** | qi=qh | (+GCpi) |
|---|---|---|---|---|---|---|
| z= 15ft | 11.65 | 0.85 | 0.8 | 7.92 | 19.25 | 0.18 |
| z= 20ft | 12.65 | 0.85 | 0.8 | 8.60 | 19.25 | 0.18 |
| z= 39ft | 15.31 | 0.85 | 0.8 | 10.41 | 19.25 | 0.18 |
| z= 57ft | 17.06 | 0.85 | 0.8 | 11.60 | 19.25 | 0.18 |
| z= 75.75ft | 18.50 | 0.85 | 0.8 | 12.58 | 19.25 | 0.18 |
| z= 87ft | 19.25 | 0.85 | 0.8 | 13.09 | 19.25 | 0.18 |

**Leeward Walls**

| | qh | G | Cp | **qhGCp** | qi=qh | (+GCpi) |
|---|---|---|---|---|---|---|
| Wind- short side | 19.25 | 0.85 | -0.2 | -3.27 | 19.25 | 0.18 |
| Wind-Long Side | 19.25 | 0.85 | -0.5 | -8.18 | 19.25 | 0.18 |
| **Side Walls** | 19.25 | 0.85 | -0.7 | -11.45 | 19.25 | 0.18 |

**Roof- First Value**

| Length | qh | G | Cp | **qzGCp** | qi=qh | (+GCpi) |
|---|---|---|---|---|---|---|
| 0-h | 19.25 | 0.85 | -0.9 | -14.73 | 19.25 | 0.18 |
| h-2h | 19.25 | 0.85 | -0.5 | -8.18 | 19.25 | 0.18 |
| >2h | 19.25 | 0.85 | -0.3 | -4.91 | 19.25 | 0.18 |

**Roof- Second Value**

| | qh | G | Cp | **qzGCp** | qi=qh | (+GCpi) |
|---|---|---|---|---|---|---|
| all lengths | 19.25 | 0.85 | -0.18 | -2.95 | 19.25 | 0.18 |

**Final Pressures**

**Windward Walls**

| | P = qGCp + qi(Gcpi) | P = qGCp - qi(Gcpi) |
|---|---|---|
| Height | Pressure 1 (psf) | Pressure 2 (psf) |
| z= 15ft | 11.39 | 4.46 |
| z= 20ft | 12.07 | 5.14 |
| z= 39ft | 13.87 | 6.94 |
| z= 57ft | 15.06 | 8.14 |
| z= 75.75ft | 16.05 | 9.12 |
| z= 87ft | 16.55 | 9.62 |

**Leeward Walls**

| | Pressure 1 (psf) | Pressure 2 (psf) |
|---|---|---|
| Wind- short side | 0.19 | -6.74 |
| Wind-Long Side | -4.72 | -11.65 |

**Parapets(Windward)**

| Heights | qp | GCpn | Pp= qpGCpn |
|---|---|---|---|
| z= 33ft | 13.87 | 1.5 | 20.81 |
| z= 51ft | 15.06 | 1.5 | 22.60 |
| z= 69ft | 16.05 | 1.5 | 24.07 |
| z= 87ft | 16.55 | 1.5 | 24.83 |

**Side Walls**

| | | | |
|---|---|---|---|
| All Heights | -7.99 | | -14.92 |

**Roof- First Value**

| Roof Length | qp | | Pp= qpGCpn |
|---|---|---|---|
| 0-h | -11.26 | | -18.19 |
| h-2h | -4.72 | | -11.65 |
| >2h | -1.44 | | -8.37 |

**Roof- Second Value**

| | | | |
|---|---|---|---|
| all lengths | 0.52 | | -6.41 |

**Wall Areas**

| | Windward | | | Leeward-Long Side | | Leeward-Short Side |
|---|---|---|---|---|---|---|
| Height | E/W(SF) | N/S(SF) | | E/W(SF) | N/S(SF) | E/W=N/S(SF) |
| z= 0-15ft | 6397.78 | 8335.00 | | 4636.78 | 6575.00 | 1760.00 |
| z= 15-20ft | 2135.55 | 2778.33 | | 1549.89 | 2191.67 | 586.67 |
| z= 20-39ft | 8443.92 | 10140.50 | | 6214.58 | 7911.17 | 2229.33 |
| z= 39-57ft | 7233.50 | 7563.50 | | 5451.50 | 5451.50 | 2112.00 |
| z= 57-75.75ft | 5422.75 | 5752.75 | | 3552.75 | 3552.75 | 2200.00 |
| z= 75.75-87ft | 2401.75 | 2740.25 | | 1420.25 | 1420.25 | 1320.00 |

**Parapet Areas**

| | Windward | |
|---|---|---|
| Height | West(SF) | North(SF) |
| z= 33ft | 0 | 330 |
| z= 51ft | 330 | 330 |
| z= 69ft | 330 | 330 |
| z= 87ft | 338.5 | 338.5 |

*Leeward- included in wall areas

**Floor Loads by Area**

| | Windward | | Leeward-Long Side | | Leeward-Short Side |
|---|---|---|---|---|---|
| Floor Level | E/W(K) | N/S(K) | E/W(K) | N/S(K) | E/W=N/S(SK) |
| First Floor | 74.33 | 96.79 | 54.04978 | 76.57222 | 11.85771 |
| Second Floor | 117.14 | 147.55 | 72.37482 | 92.13317 | 15.01977 |
| Third Floor | 116.43 | 121.40 | 63.48798 | 63.48798 | 14.22925 |
| Mech. Pent. | 94.96 | 100.26 | 41.3752 | 41.3752 | 14.82214 |
| Roof | 48.17 | 53.77 | 16.54018 | 16.54018 | 8.893282 |

**Final Story Forces**

| | Load | | Shear | | Moment | |
|---|---|---|---|---|---|---|
| Floor Level | E/W(K) | N/S(K) | E/W(K) | N/S(K) | E/W(K-ft) | N/S(K-ft) |
| First Floor | 140.24 | 185.22 | 763.68 | 874.70 | 1402.381 | 1852.21 |
| Second Floor | 204.54 | 254.70 | 623.44 | 689.47 | 6136.109 | 7640.975 |
| Third Floor | 194.15 | 199.12 | 418.90 | 434.77 | 9319.04 | 9557.669 |
| Mech. Pent. | 151.16 | 156.45 | 224.76 | 235.66 | 9976.463 | 10325.96 |
| Roof | 73.60 | 79.20 | 73.60 | 79.20 | 6292.719 | 6771.838 |
| | | **Totals*(1.6)** | **1221.887** | **1399.512** | **53002.74** | **57837.85** |

# Appendix D: Seismic Analysis

**Design Seismic Base Shear (ASCE7-05)**

V= CsW 12.8-1
W: Effective Weight- 12.7.2
Cs: Seismic Coeff.- 12.8.1.1

$$Cs = \text{Min}\left|\begin{array}{l} S_{ds}/(R/I) \\ S_{D1}/(T*R/I) \\ S_{D1}*Tl/(T^2*R/I) \end{array}\right| > 0.01$$

In addition, where S1 > 0.6
$Cs > 0.5S_1/(R/I)$ Eq. 12.8-6

Fa,Fv- Table 11.4-1, 11.4-2
Ss, S1- USGS website, using long./lat. of site location

R: Response Mod. Coeff.- Table 12.2-1
I: Importance Factor- 11.5
Occ. Cat.- Table 1-1

$$T = \text{Min}\left|\begin{array}{l} Cu*Ta \\ Tb \end{array}\right| \qquad S_{ds} = 2/3(S_{MS}) \qquad S_{MS} = Fa*Ss$$
$$S_{D1} = 2/3(S_{M1}) \qquad S_{M1} = Fv*S1$$

Cu- Table 12.8-1
Ta- 12.8.2.1
Tb- Fundamental Mode of Vibration from Modal Analysis w/Mass input

**Design Seismic Base Shear- MSC Complex, University Park, PA**

| | | |
|---|---|---|
| Latitude: | 40.802 | Site Class: D |
| Longitude: | -77.86 | Occ. Cat: III |

| | | |
|---|---|---|
| Ss = | 0.147 | g (Site Class B) |
| S1= | 0.049 | g (Site Class B) |
| Fa= | 1.6 | |
| Fv= | 2.4 | |

| | | | | | |
|---|---|---|---|---|---|
| $S_{MS}$= | 0.2352 | g | $S_{DS}$= | 0.1568 | g |
| $S_{M1}$= | 0.1176 | g | $S_{D1}$= | 0.0784 | g |

**Lateral Force Resisting System: Ordinary Steel Concentrically Braced Frames**

| | | |
|---|---|---|
| R= | 3.25 | |
| I= | 1.25 | |
| SDC: | B | (No Limitations) |
| Tl= | 6 | s (Fig. 22-15) |

**Design Base Shear**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Ta= | 0.512 | s | Cs= | Min | 0.0603 | = | 0.0346 |
| Cu= | 1.7 | | | | 0.0346 | | |
| Cu*Ta= | 0.871 | s | | | 0.2386 | | |

**Building Weight Calculation (above ground)**

**Frame Weights**

| Columns | | Beams | Braces | |
|---|---|---|---|---|
| | Weight(K) | W (K) | # braces | W (K) |
| 1st Floor | 503.31 | 770.26 | 32 | 358.35 |
| 2nd Floor | 440.87 | 889.46 | 72 | 806.28 |
| 3rd Floor | 325.36 | 1011.99 | 95 | 1063.84 |
| Mech. | 234.69 | 762.95 | 88 | 985.45 |
| Roof | 59.93 | 481.01 | 24 | 268.76 |
| | | Total | 311 | 3482.68 |

**Floor Weights (slabs, beams, columns, façade)**

| | slabs(K) | columns(K) | beams(K) | braces(K) | façade(K) | W(K) |
|---|---|---|---|---|---|---|
| Level-2 | 5030.64 | 472.09 | 889.46 | 582.3126 | 1800.24 | 8774.75 |
| Level-3 | 5536.31 | 383.12 | 1011.99 | 935.0597 | 1360.77 | 9227.25 |
| Mech. | 6436.63 | 280.02 | 762.95 | 1024.646 | 1233.77 | 9738.01 |
| Roof | 1977.54 | 147.31 | 481.01 | 761.4857 | 798.32 | 4165.67 |
| | | | | | Total | 31905.68 |

**Superimposed Dead Loads**

| | psf | Area (SF) | W (K) |
|---|---|---|---|
| Roof | 25 | 33202.82 | 830.07 |
| Pent | 25 | 40174.27 | 1004.36 |
| Third | 30 | 53742.49 | 1612.27 |
| Second | 30 | 64818.42 | 1944.55 |
| Green Roofs | 120 | 63584.00 | 7630.08 |
| | Total | 255522.00 | 13021.33 |

**Precast Panels- Unit Weight**

| | t (in) | width (ft) | length(ft) | pcf | W(K) |
|---|---|---|---|---|---|
| Legs(x2) | 8 | 4.00 | unit/ft | 145 | unit/ft |
| Level-2 | 7 | 11.72 | 1811.33 | 145 | 1800.24 |
| Level-3 | 7 | 9.72 | 1650.00 | 145 | 1360.77 |
| Mech. | 7 | 9.72 | 1496.00 | 145 | 1233.77 |
| Roof | 7 | 9.72 | 968.00 | 145 | 798.32 |
| | | | | Total= | 5193.10 |

**V= CS*W**

**Cs=** 0.0346

**W=** 50120.12 K

**V=** **1735.46** K