

[IPD/BIM Thesis]

Structural Option

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The Millennium Science Complex

University Park, Pennsylvania



[TECHNICAL REPORT 1]

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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: Aerial 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.

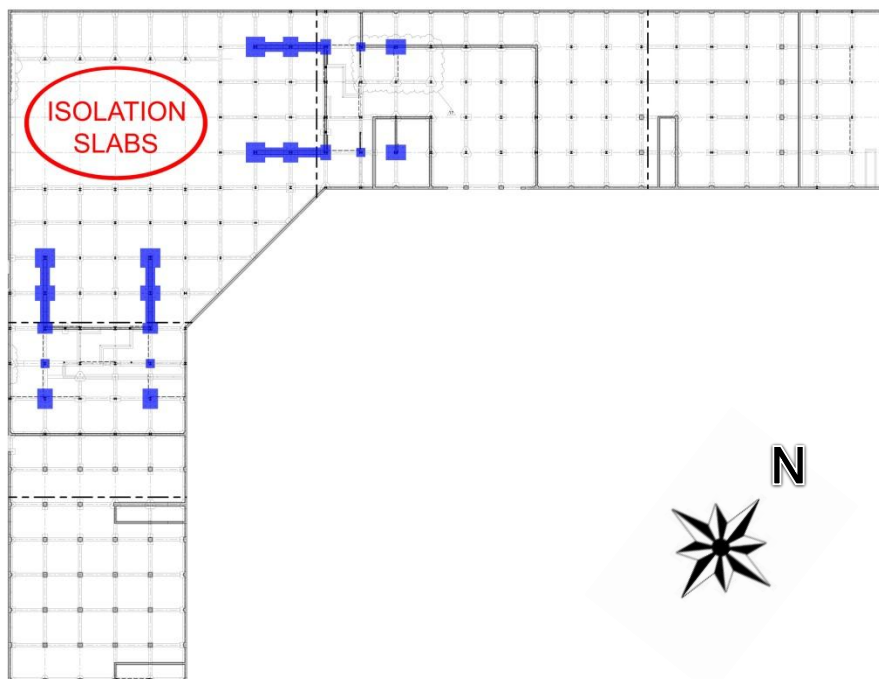


Figure 2 :

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.

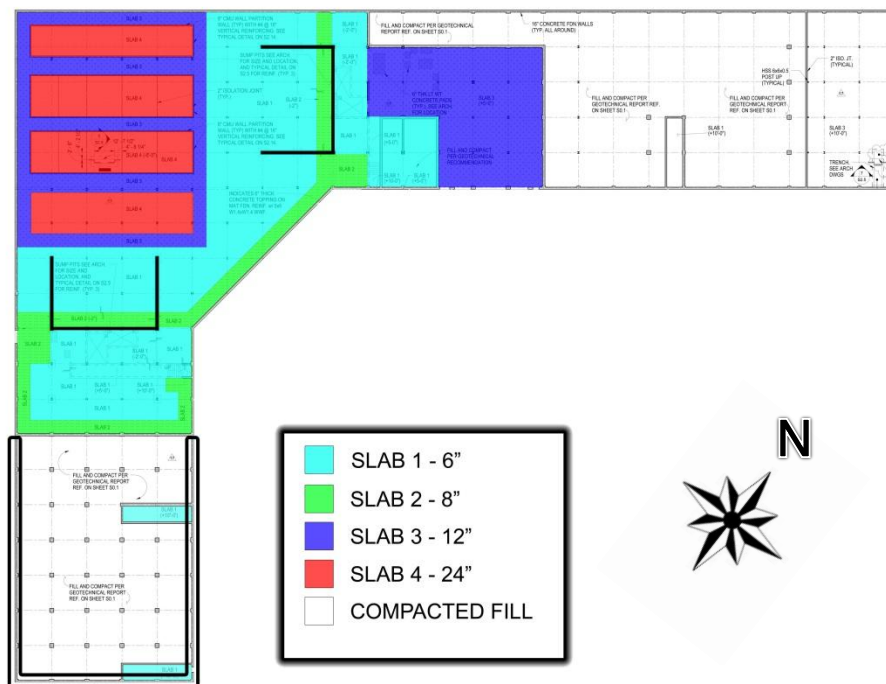


Figure 3:

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.

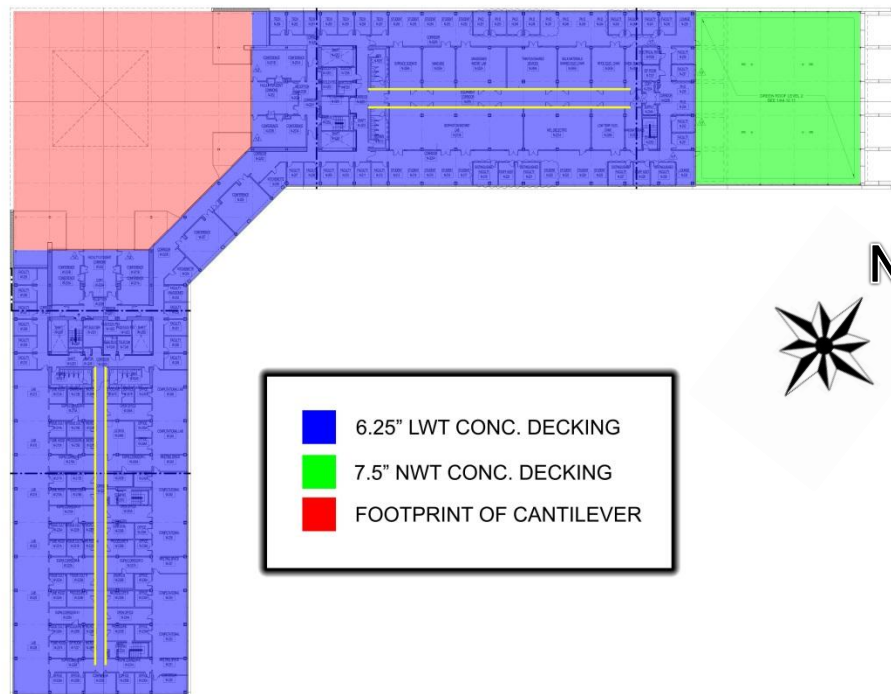


Figure 4:

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.

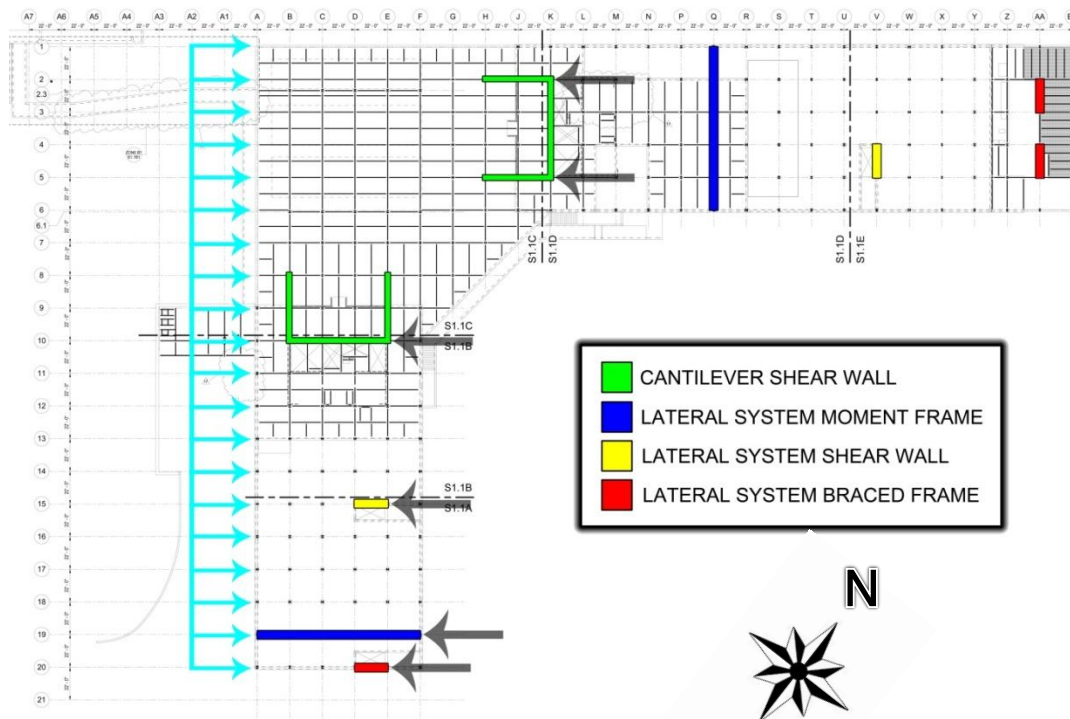


Figure 5:

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.

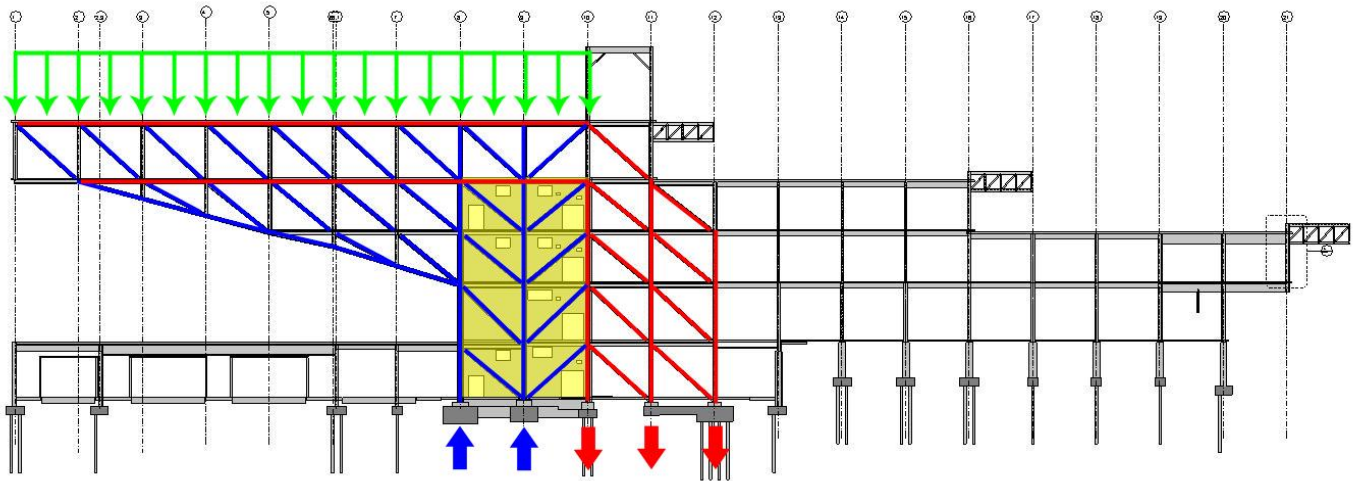


Figure 6:

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) 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
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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 ft²/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	$p_f = 0.7C_eC_tI p_g$	28	psf	20.8			
Snow Drifts		Based on Sect. 1608.7 as applicable					

Table 5

Check min p_f :

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

$$p_f = 22 < 28 \therefore \text{use } p_f = 28 \text{ psf}$$

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						
Floor Level	Load		Shear		Moment	
	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.

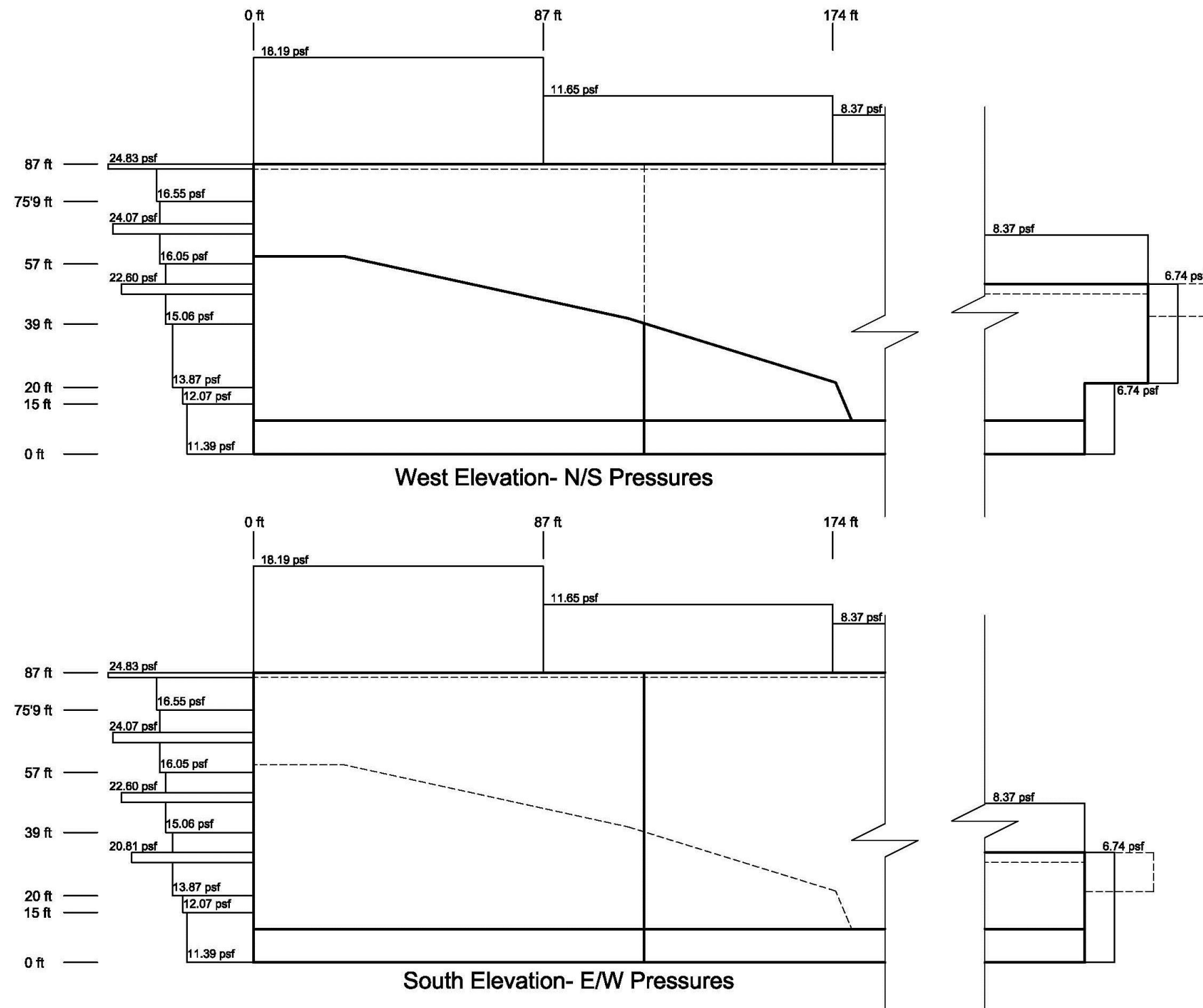
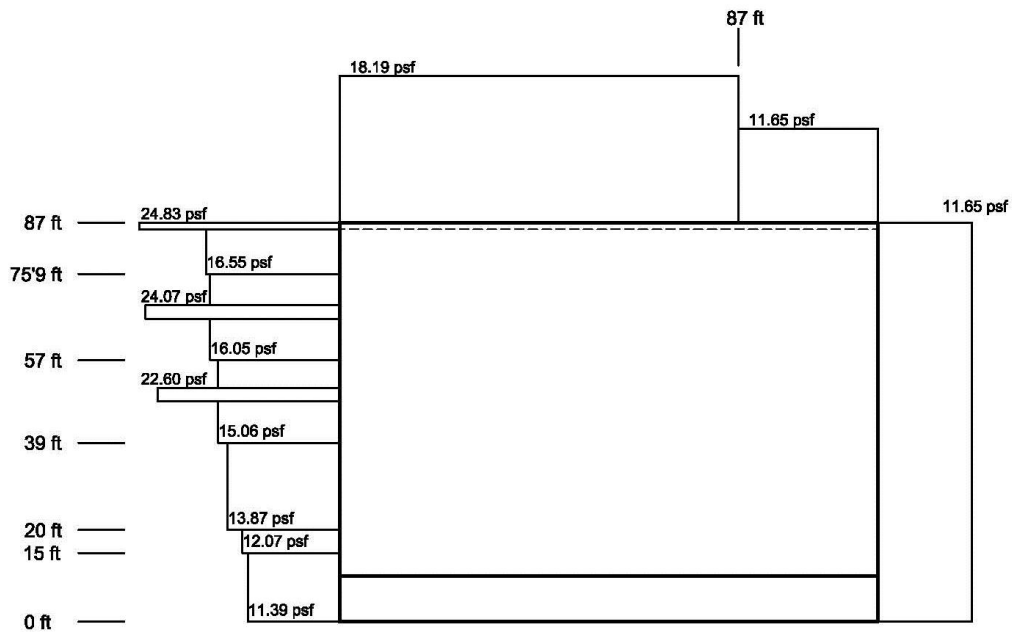
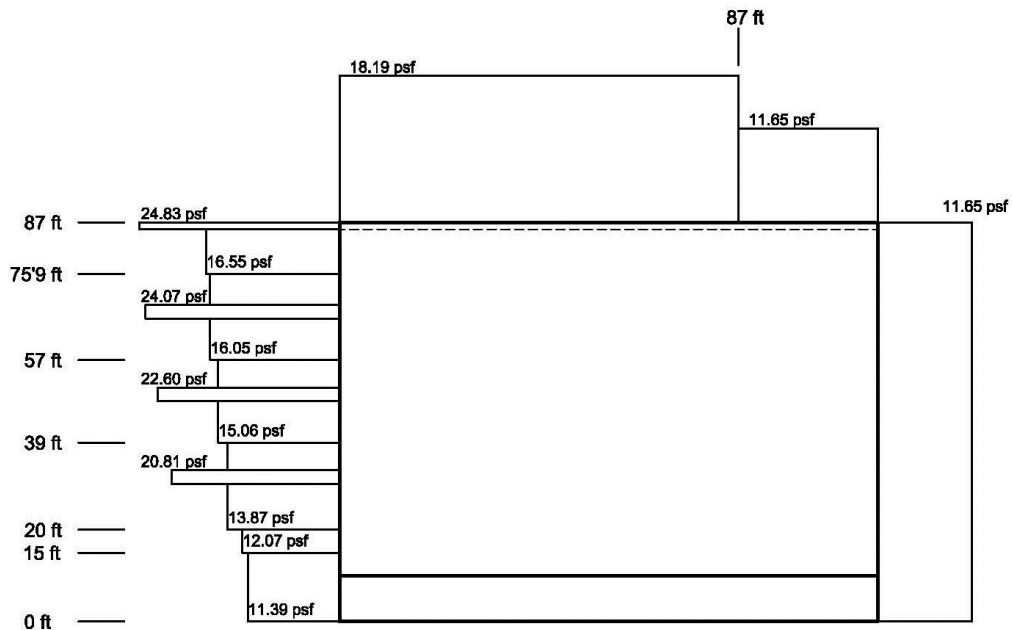


Figure 7



North Wing Section- N/S Pressures



West Wing Section- E/W Pressures

Figure 8

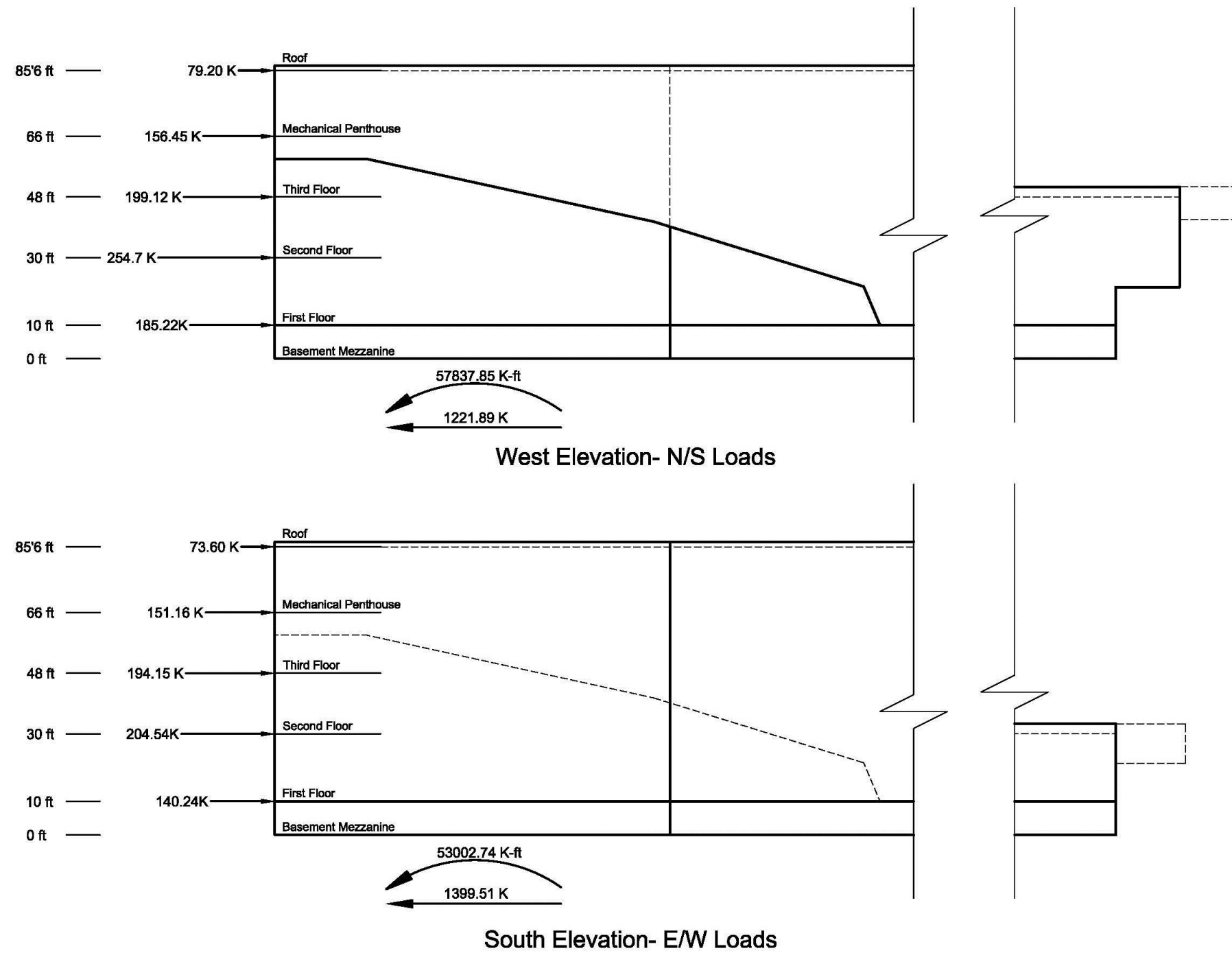
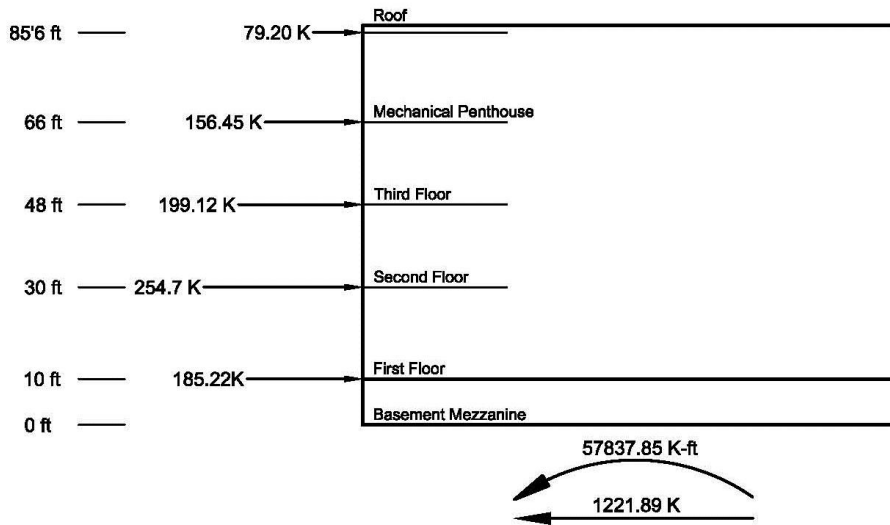
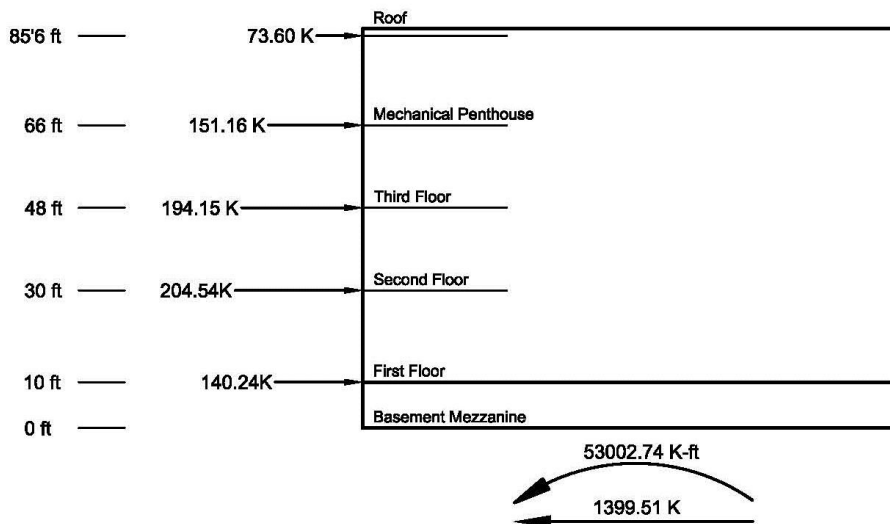


Figure 9



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

Table 11

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.

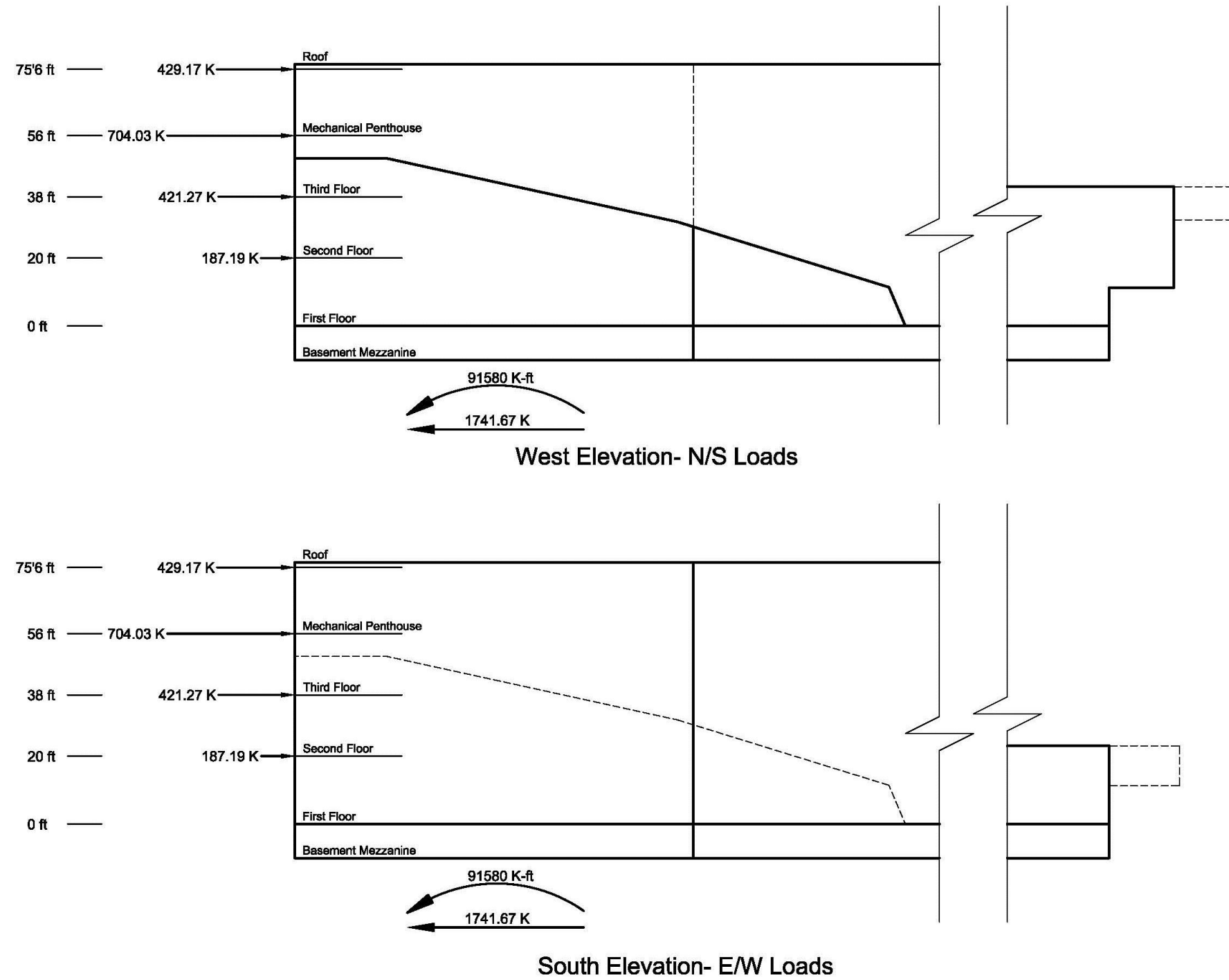


Figure 11

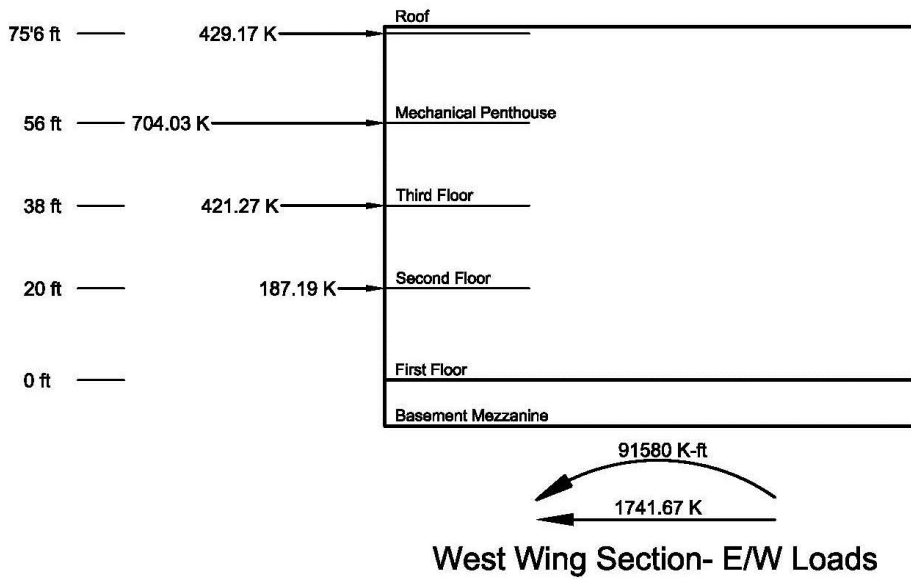
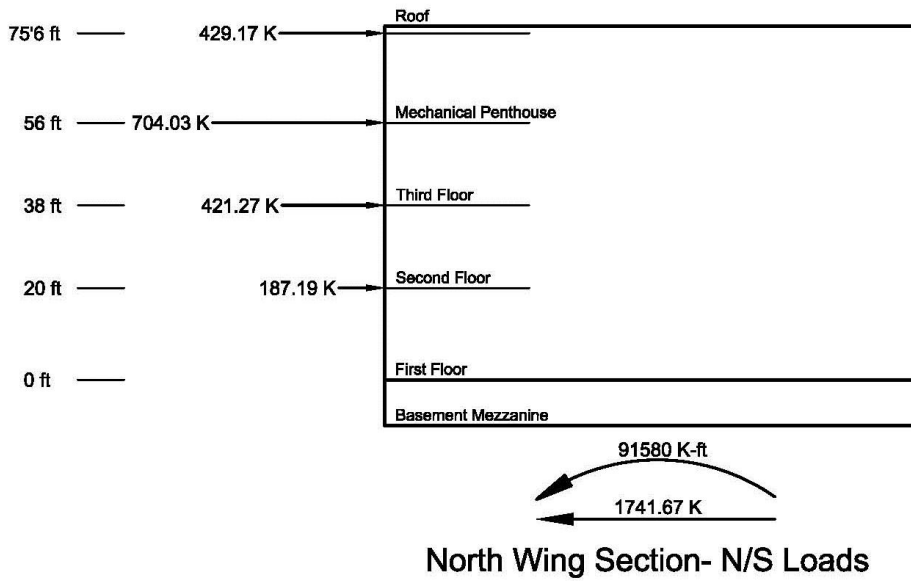


Figure 12

Typical Spot Checks

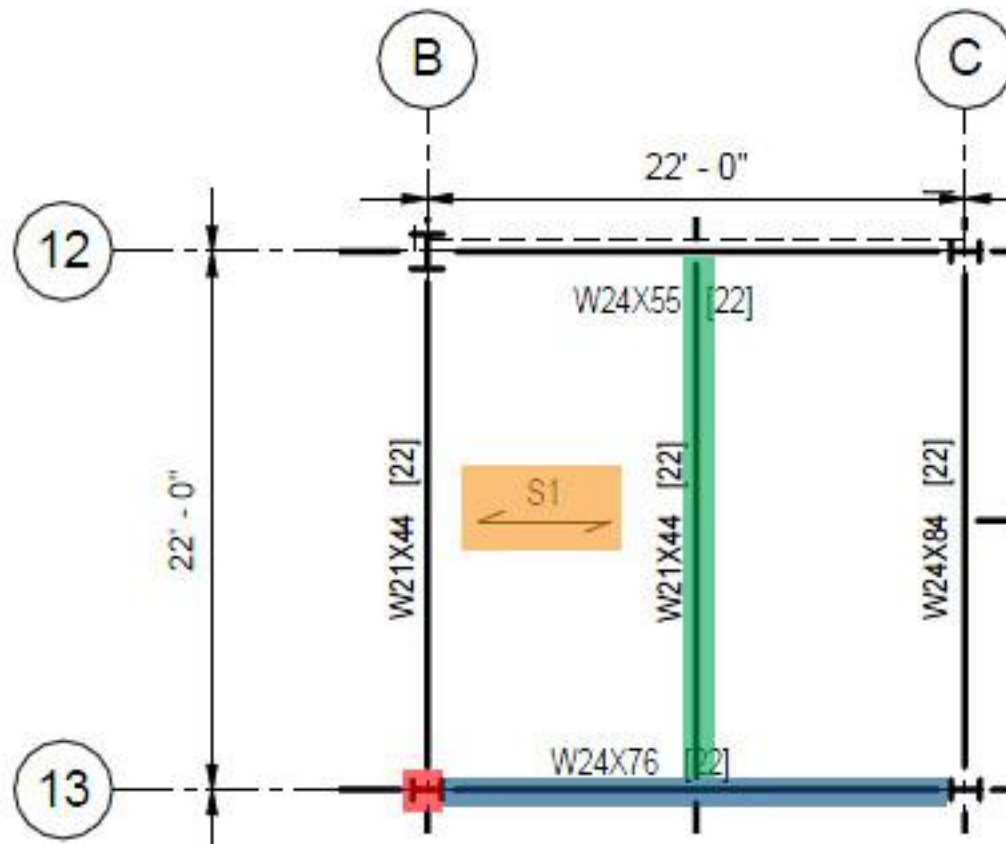


Figure 13: Figure of typical interior gravity frame used for spot checks. Orange: composite metal deck, green: composite beam, blue: composite girder, and red: column.

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 ¼". 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 _y	S _x	R _s	φV _s	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	4890	0.79	
16	0.0598	3.5	1.020	1.666	1.045	1.045	2540	6130	0.99	

COMPOSITE PROPERTIES												
Slab Depth	φM _{cr} in.k	A _c in ²	Vol. ft ³ /ft ²	W psf	S _c in ³	I _{cr} in ⁴	φM _{no} in.k	φV _{re} lbs.	Max. unshored spans, ft.			A _{req}
									1span	2span	3span	
5.50	80.96	37.6	0.333	38	1.94	9.1	54.28	8250	11.48	13.61	14.07	0.023
6.00	92.32	42.0	0.375	43	2.23	11.6	62.43	5670	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	13.28	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 *														
Slab Depth	φM _n 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	295	255	235	210	195	175	160	150	135
6.25	98.00	400	400	370	335	310	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	375	320	290	270	240	220	205	185	170
7.25	120.72	400	400	400	400	370	335	305	278	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	355	320	290	260	245	225	205	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

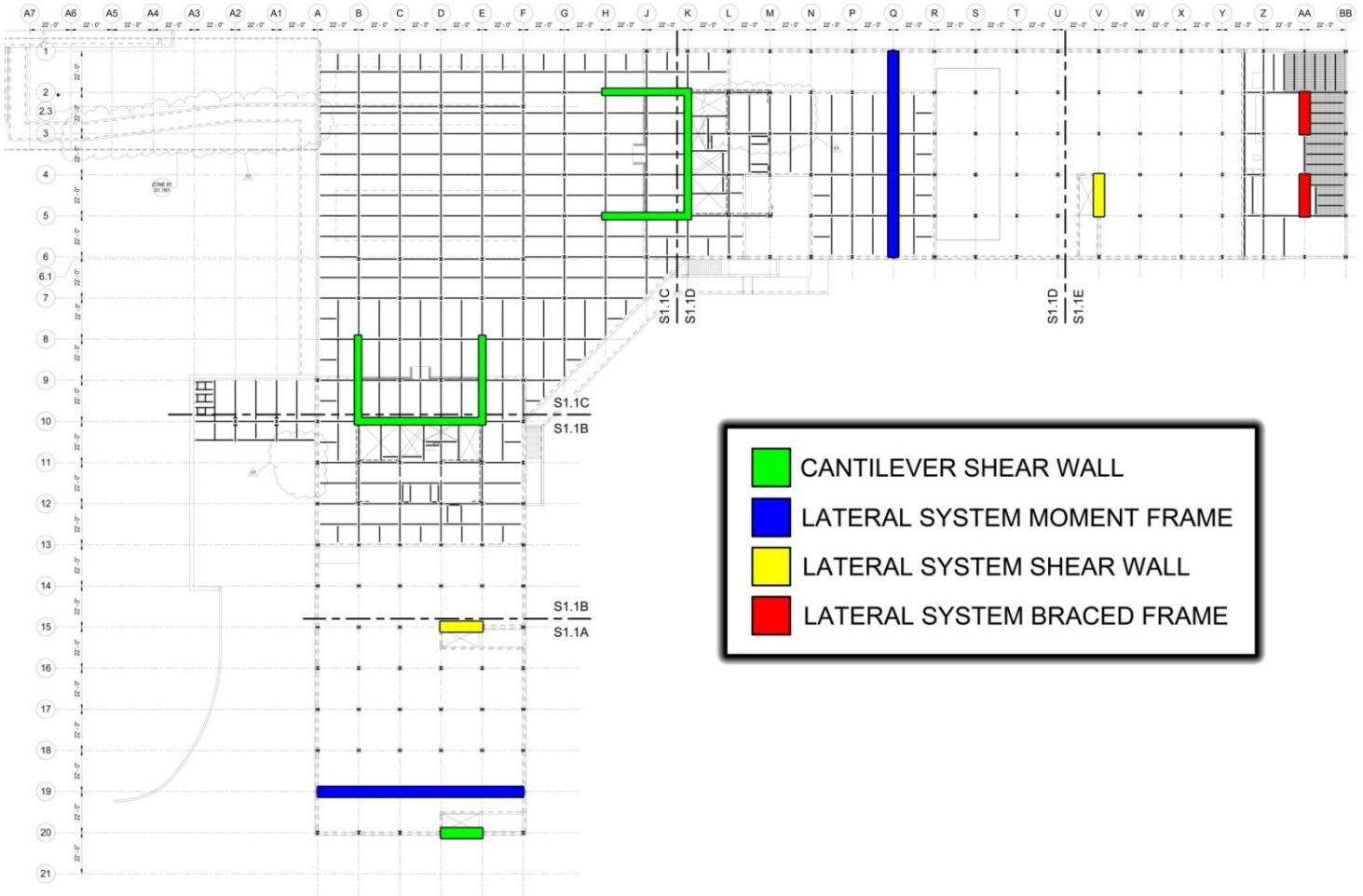
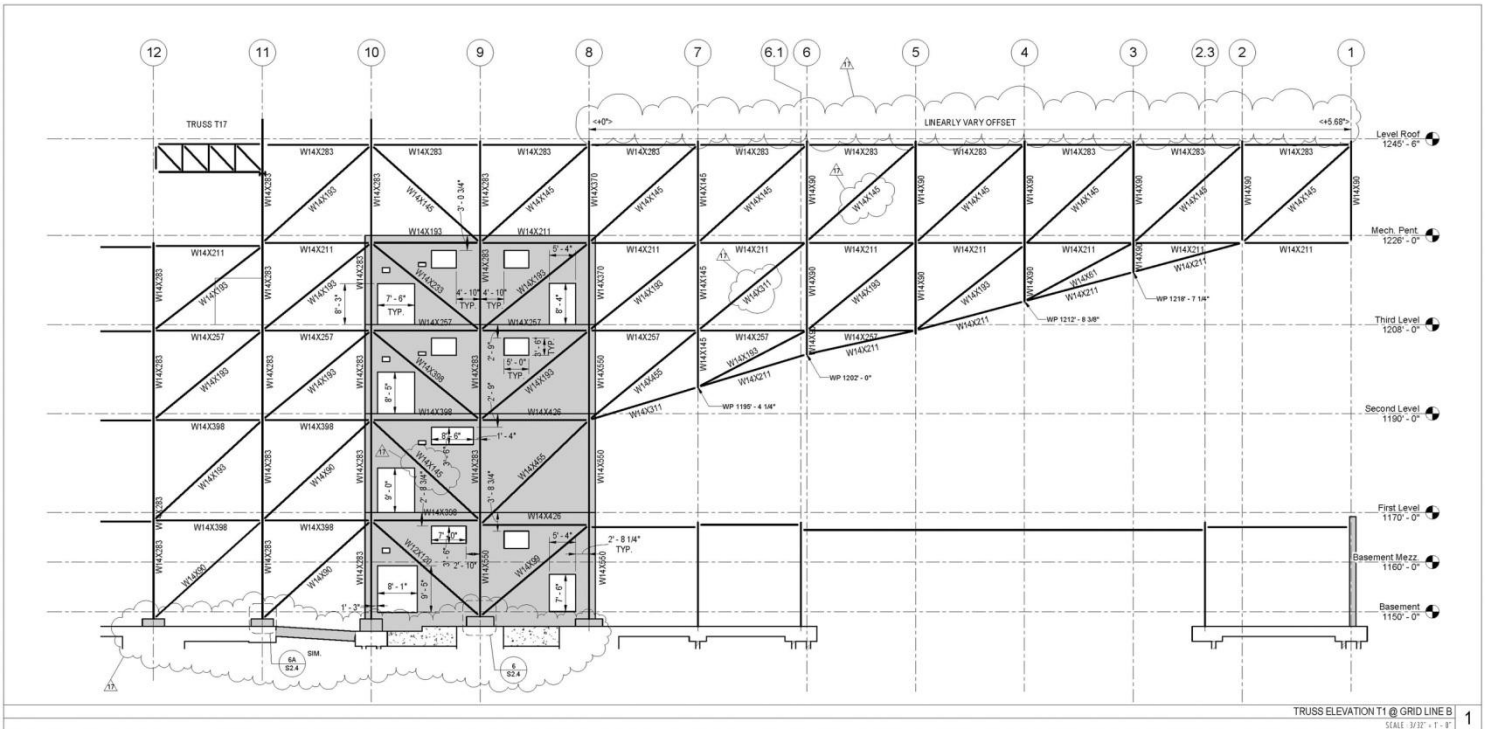
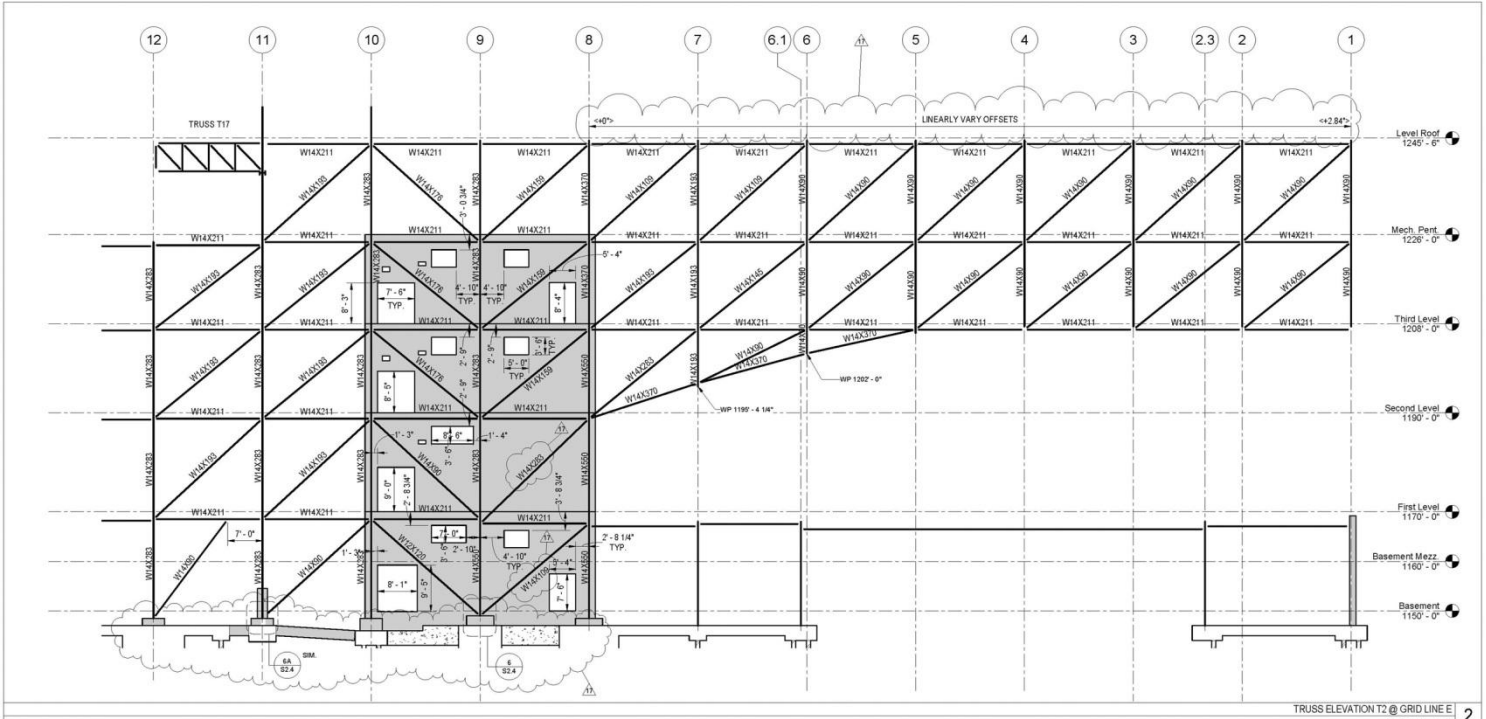


Figure 15: First floor lateral system elements



TRUSS ELEVATION T1 @ GRID LINE B
SCALE: 1/8" = 1'-0"



TRUSS ELEVATION T2 @ GRID LINE E
SCALE: 1/8" = 1'-0"

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

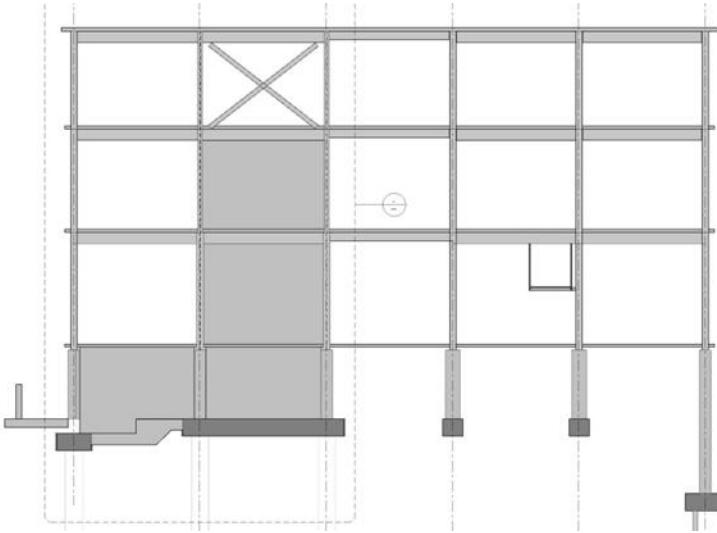


Figure 19: Shear wall at column line 15

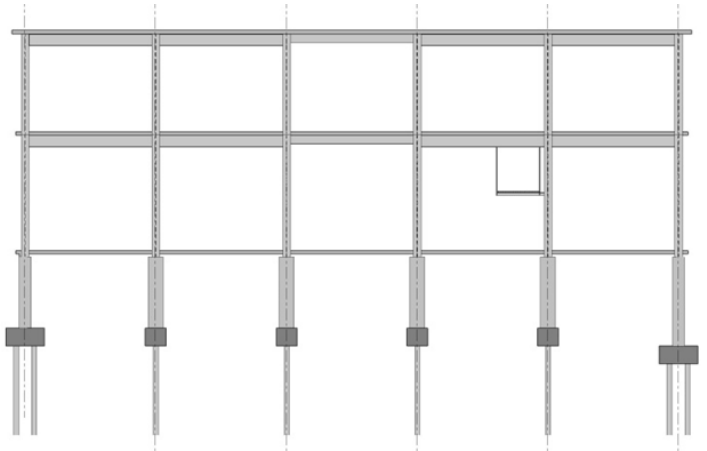


Figure 18: Moment frame at column line 19

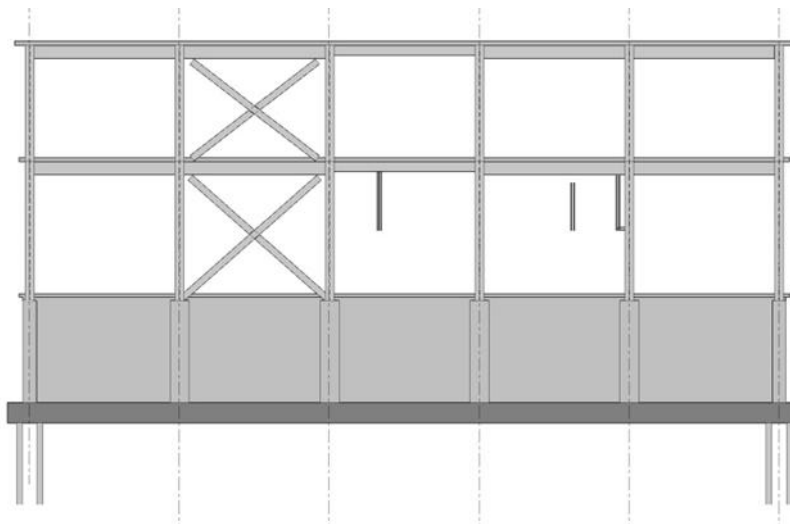


Figure 17: Braced frame at column line 20

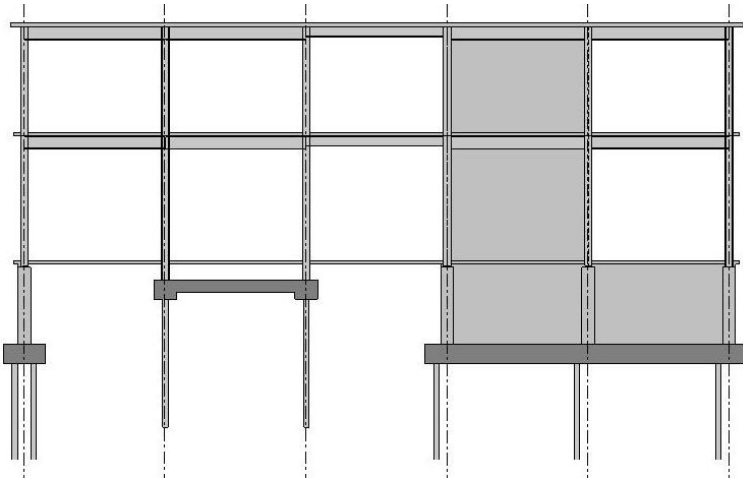


Figure 20: Shear wall at column line V

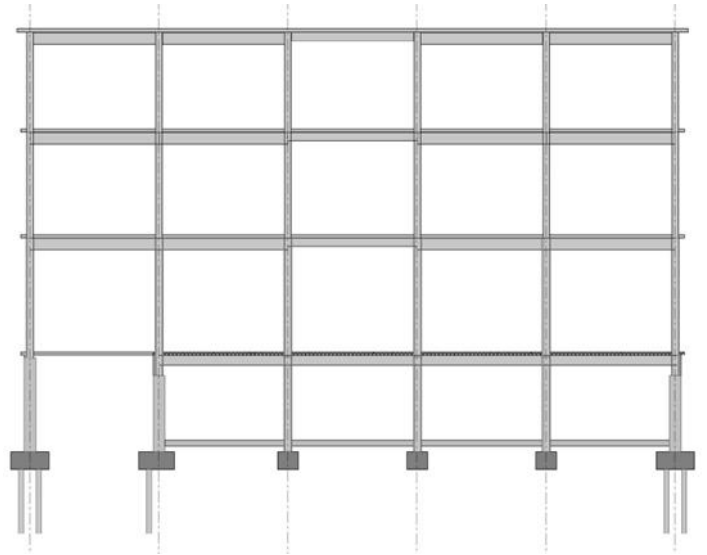


Figure 21: Moment frame at column line Q

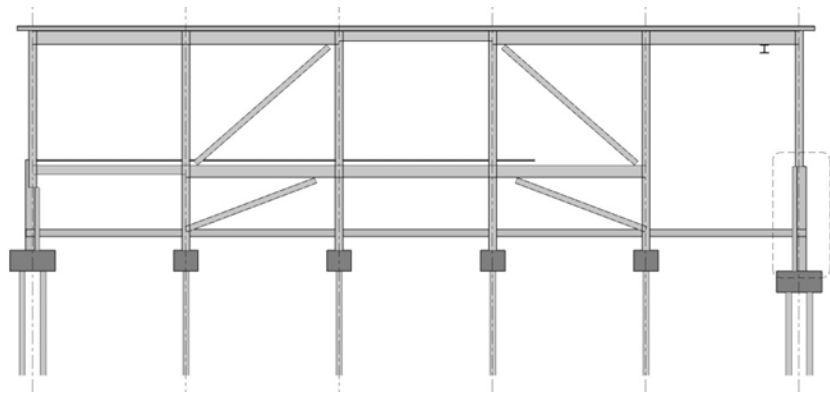


Figure 22: Braced frame at column line Z

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	

TECHNICAL REPORT 1

October 5, 2010

Check Member Strength

$w_u=1.2D+1.6L$	4.2768	klf					
$V_u=w_uL/2$	47.0		$\leq \phi V_n$	217	k	OK	Table 3-2
$M_u=w_uL^2/8$	258.7		$\leq \phi M_n$	522	ft-k	OK	Table 3-19

Check Shear Studs

ΣQ_n	162				k		Table 3-19
Q_n	17.2				k		Table 3-21
# of Studs = $(\Sigma Q_n/Q_n)*2$	20		\leq	# Studs Specified	22	OK	

Check a

a	0.721925134		\leq	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		$\leq \phi_b M_p$	358	ft-k	OK	Table 3-19

Deflection Check

$\Delta_{LL}=5w_{LL}L^4/384EI_{LB}$	0.201		\leq	$\Delta_{LL}=L/360$	0.73	in	OK
$\Delta_{CDL}=5w_{CDL}L^4/384EI$	0.223		\leq	$\Delta_{LL}=L/240$	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

TECHNICAL REPORT 1

October 5, 2010

Check Member Strength

$P_u = w_{u\text{Beam}} L_{\text{Beam}}$	94.6	k					
$V_u = P_u / 2$	47.3	\leq	ϕV_n	316	k	OK	Table 3-2
$M_u = P_u L / 4$	1040.6	\leq	ϕM_n	1050	ft-k	OK	Table 3-19

Check Shear Studs

ΣQ_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	\leq	assumed a	1.25	in	OK	
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Check Unshored Strength

$w_u = 1.4D$	0.143	klf					
$P_u =$	23.4	k					
$M_u = (w_u L^2 / 8) + (P_u L / 4)$	137.5	\leq	$\phi_b M_p$	750	ft-k	OK	Table 3-19

Deflection Check

$\Delta_{LL} = PL^3 / 48EI_{LB}$	0.360	\leq	$\Delta_{LL} = L / 360$	0.73	in	OK	
$\Delta_{CDL} = PL^3 / 48EI$	0.083	\leq	$\Delta_{CDL} = L / 240$	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
			ft	A _T ft ²	K _{LL} A _T ft ²		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		
							k/ft	k	ksf	ksf	ksf	ksf	ksf	kips	kips	kips	kips	
4 Mech.	1	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
φP _n	512 @ KL=8	k
P _u = 301.08 ≤ φP _n = 512	OK	
Min W14	W14x61	
φP _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
Building 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 Flooring Membrane tapered. The lower roofs are all green roofs. All roofs flat.

a) Basic Wind Speed (Fig. 6-1): $V = 90\text{mph}$

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: $qz = 0.000256kzktkdV^2I$

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: $GC_{pi} = +/- 0.18$ (assume Enclosed Building)

g) Design Wind Pressures: $P = qG_{Cp} - q_i(G_{Cpi})$
 $q = qz$ (windward, depends on height)
 $q = qh$ (leeward, taken at height-h)
 $G = 0.85$
 $q_i = qh$ (windward, leeward, and roofs for enclosed buildings)
 $GC_{pi} = +/- 0.18$

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)	\hat{a}	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
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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
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Wall Areas

Height	Windward		Leeward-Long Side		Leeward-Short Side
	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

Height	Windward	
	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

Floor Level	Windward		Leeward-Long Side		Leeward-Short Side
	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

Floor Level	Load		Shear		Moment	
	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)

$V = C_s W$ 12.8-1
 W: Effective Weight- 12.7.2
 C_s : Seismic Coeff.- 12.8.1.1

$$C_s = \min \left\{ \begin{array}{l} S_{ds}/(R/I) \\ S_{D1}/(T^*R/I) \\ S_{D1} * T / (T^2 * R/I) \end{array} \right\} > 0.01$$

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

F_a, F_v - Table 11.4-1, 11.4-2
 S_s, S_1 - 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

$$T = \min \left\{ \begin{array}{l} C_u * T_a \\ T_b \end{array} \right\} \quad \begin{array}{l} S_{ds} = 2/3(S_{MS}) \\ S_{D1} = 2/3(S_{M1}) \end{array} \quad \begin{array}{l} S_{MS} = F_a * S_s \\ S_{M1} = F_v * S_1 \end{array}$$

C_u - Table 12.8-1
 T_a - 12.8.2.1
 T_b - 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

$S_s =$ 0.147 g (Site Class B)
 $S_1 =$ 0.049 g (Site Class B)

$F_a =$ 1.6
 $F_v =$ 2.4

$S_{MS} =$ 0.2352 g $S_{D5} =$ 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

$$\begin{array}{l}
 T_a = 0.512 \text{ s} \\
 C_u = 1.7 \\
 C_u * T_a = 0.871 \text{ s}
 \end{array}
 \quad
 C_s = \text{Min} \left\{ \begin{array}{l} 0.0603 \\ 0.0346 \\ 0.2386 \end{array} \right\} = 0.0346$$

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

$$\begin{array}{l}
 V = C_s * W \\
 C_s = 0.0346 \\
 W = 50120.12 \text{ K} \\
 V = 1735.46 \text{ K}
 \end{array}$$