TECHNICAL REPORT 1

S.T.E.P.S. Building

Lehigh University

Bethlehem, PA

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September 17th, 2012

Existing Conditions

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1. Executive Summary

This report aims to serve as a technical description and analysis of the structural system of the S.T.E.P.S. Building in Bethlehem, PA. The purpose of this report is to serve as a Capstone project for the Pennsylvania State University's Architectural Engineering (AE) program. It is known as Senior Thesis and is conducted by all 5th year students in the AE program.

First, the building will be introduced and described. This report will specifically describe the structural systems used in the building. It is constructed of a concrete slab with metal decking that transfers loads to wide-flange steel beams. These slabs are constructed compositely with the beams for added strength. The columns are also wide-flange sections which have concrete foundation piers. The piers have shallow footings that transfer loads into the ground.

The calculations made in this report examine the loadings that may have been used to design the building. In addition, there is a wind loading analysis and a seismic loading analysis. Lastly, some basic gravity spot checks were performed. In further reports, these calculations will be elaborated upon and used to test members for lateral loads and combined effects. Wind will in all likelihood control lateral loads used in design, because Bethlehem, PA is not in a seismic region. However, seismic response of this building must still be studied further.

In general, the loadings assumed in this report appear to make sense. Some communication must be made with the Structural Engineer of record to find out specifically what assumptions he or she made in design. This will also be explored in Tech 2 and Tech 3.

2. Building Introduction

Lehigh University envisioned the Science, Technology, Environment, Policy, and Society (S.T.E.P.S.) Building as a way to attract new students and retain existing students in the science and engineering fields. The university lacked a modern laboratory building with all the amenities that have come with increases in technology over the years. In an interesting and experimental fashion, the departments have been intermixed by Health, Education & Research Association, Inc. They believe it will lead to increased communication and collaboration among faculty and researchers of various disciplines.

The building is oriented on the corner of East Packer Ave. and Vine St. as shown in the photo below:

Figure 1:

Image Courtesy of Bing.com

Lehigh University slowly purchased the properties which were on the building site as they planned for a building to be put there. The building is also connected to an existing structure through the use of a raised pathway that is enclosed. The building is divided into three wings for the purpose of this analysis. These wings are diagramed in Figure 2 on the following page.

Figure 2:

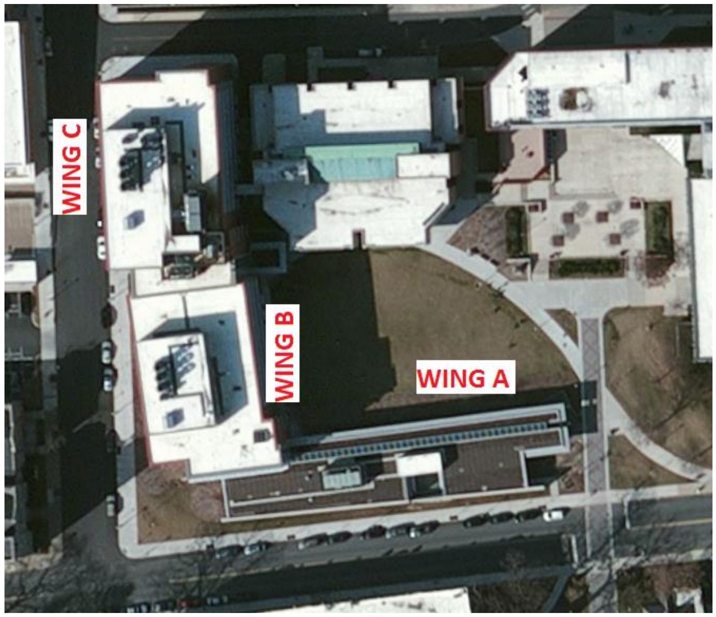


Image courtesy of Bing.com

Wing A is a one story structure with a lounge and entryway. It has raised clearstories to allow for natural daylight to illuminate the space. It also has a 12” deep green roof supported by structural wood which helped in earning LEED Certification. The building is LEED Gold certified by the United States Green Building Council (USGBC). Because of its limited building height, Wing A will not be analyzed in this report.

Wing B is a four story steel framed structure oriented along Packer Ave. Interestingly, Packer Ave. and Vine St. do not meet at a 90 degree angle. So, Wing B is aligned with Packer Ave., and Wing C is aligned with Vine St. There is a large atrium with lounge areas connecting the two structures on each floor.

Wing C is also steel framed and is 5 stories. The building’s lateral system consists of moment connections between columns and beams throughout the building. It should be noted that the load resisting elements are one structure as they continue uninterrupted through the atrium.

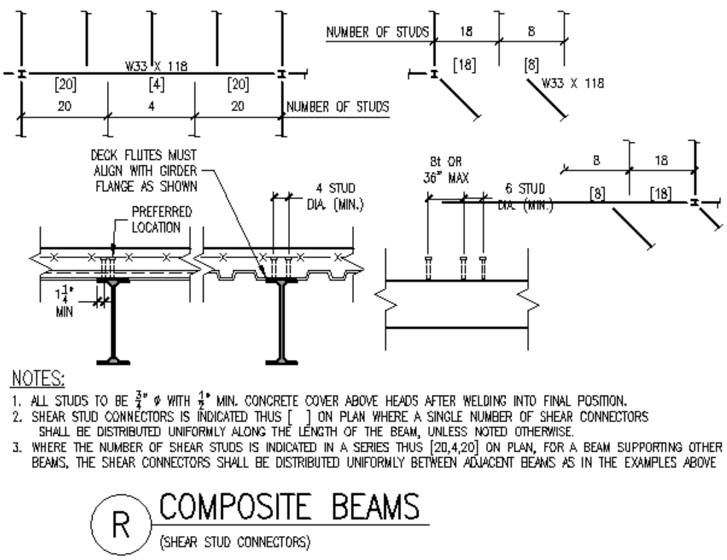
Sustainable features of the building include the green roof, high-efficiency glazing, sun shading, and custom mechanical systems.

3. Structural System

3.1 Floor System

There is a composite steel deck floor system in place for all floors in Wings B & C above grade. Basement floors are slab on grade. Below is a detail of a typical composite beam with shear studs indicated:

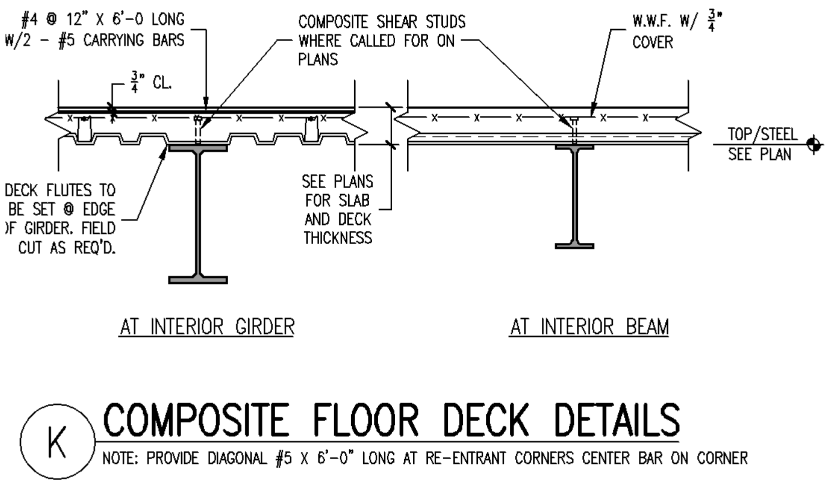
Figure 3:



Along Vine St., which will be considered the longitudinal direction of the building, typical girders have a span of 21’-4” with one intersecting beam at their midpoint. The transverse beams which run parallel to Packer Ave. have a span anywhere from 36’-11” to 42’8”.

The decking is a 3” 18 gauge steel deck with 4-1/2” concrete topping and welded wire fabric. The bulk of the decking is run longitudinally throughout Wings B & C and has a clear span of 10’8”. The exceptions to this are two bays to the very south of Wing B along Packer Ave. These bays are oriented transversely. The total thickness ends up being 7-1/2” with a 6x6” W2.9 x W2.9 welded wire fabric embedded ¾” from the top of the slab. On the following page is a typical detail of the composite floor slab:

Figure 4:



The floor system is supported by wide flange beams designed as simply supported. A combination of full moment connections, semi-rigid moment connections, and shear connections are used. Typical sizes for transverse beams are W24x55 and W24x76. The girders are W21x44. Most beams have between 28 and 36 studs to transfer shear. Figure 5 shows a typical Full Moment Connection with field welds noted. Figure 6 shows the entirety of the first floor system for Wing B. Figure 8 shows the entirety of the first floor system for Wing C.

Figure 5:

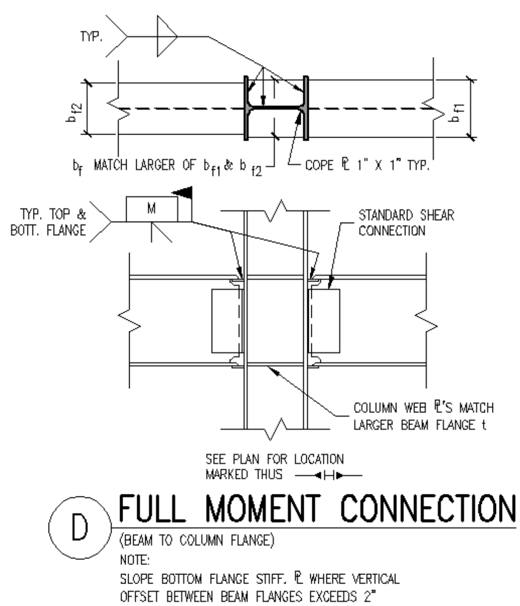


Figure 6:

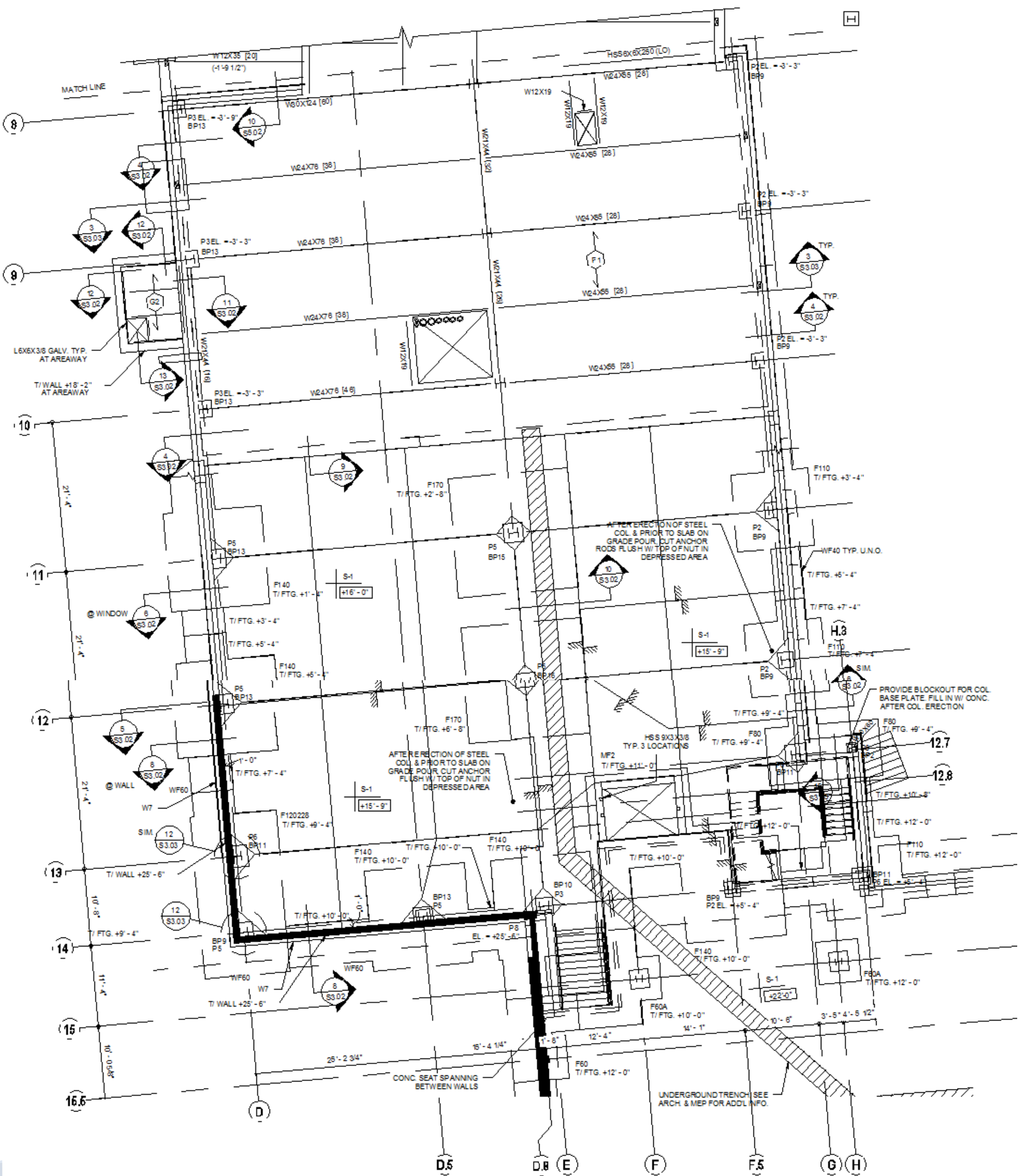
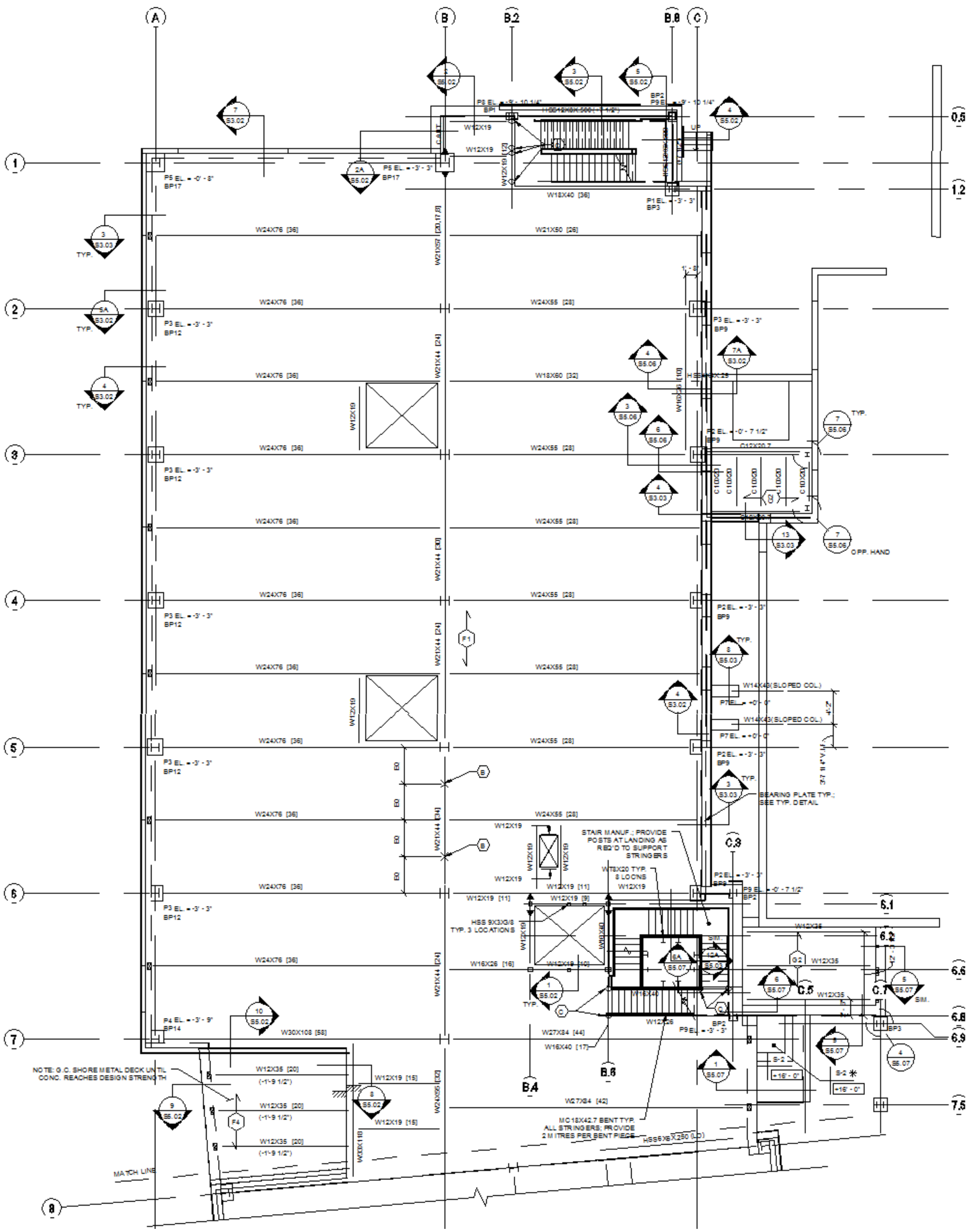


Figure 7:



3.2 Vertical Members

Wide flange columns are used throughout the building for gravity loads. They are arranged for strong axis bending in the transverse direction. Most spans have a column at either end with another at the midpoint.

W14 is the most common section size with weights varying from W14x90 all the way up to W14x192 on the lower floors.

3.3 Foundation

Schnabel Engineering performed a geotechnical analysis of the site in 2007. This concluded that the soil had sufficient bearing capacity to support the loads from the building.

Interior columns are supported by a mat foundation 18' wide and 3' deep. Exterior columns bear on square footings ranging from 11'x11' to 16'x16' with depths from 1'6" to 2'. These are tied into the foundation by base plates with concrete piers.

The reinforced foundation walls have strip footings ranging from 2' to 6' wide with depths between 1' and 2'. These are monolithically cast with the piers for the exterior columns.

3.4 Roof System

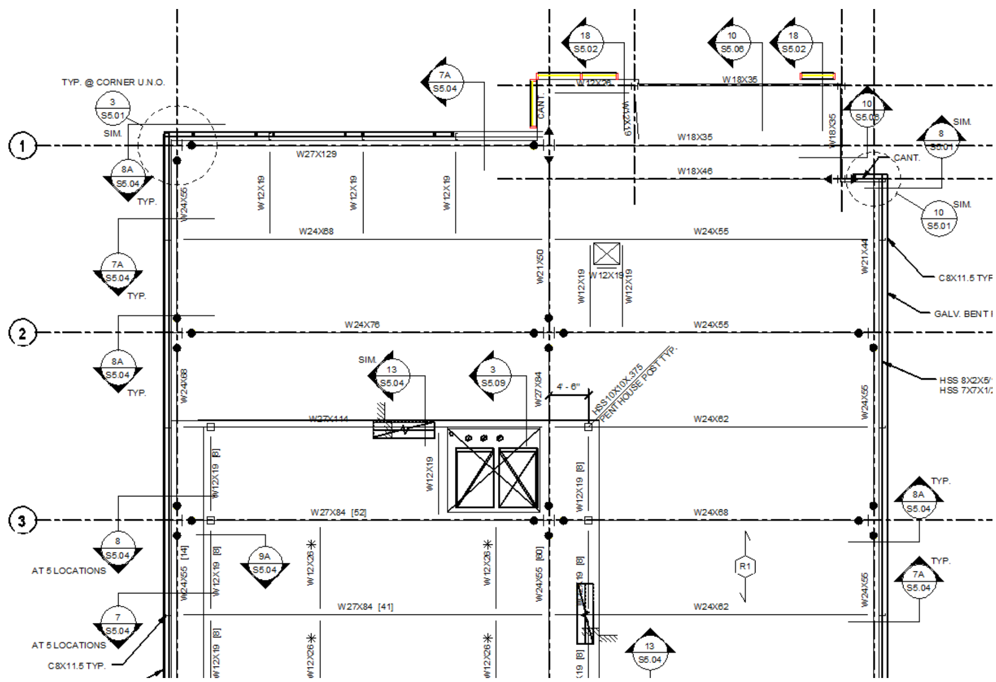
The roof decking consists of a 3" 16 gauge steel roof deck with a sloped roof for drainage. Topping ranges from ¼" to 4-1/2" to achieve a ¼":1' slope. Therefore, total thickness ranges from 3-1/4" to 7-1/2". Framing is similar to floor framing with wide flanges ranging from W24x55 to W24x68.

The floor system has increased loads where the mechanical penthouses are situated. The penthouse itself is framed with square HSS tubing. Heavier W27x84 wide flange beams support this area.

3.5 Lateral System

The building resists lateral loads by moment connections at the beam to column locations. They are continuous throughout the building and beams are designed as simply supported for gravity loads. The moment connections are designed only to take lateral loads. Many of these moment connections are semi-rigid connections to give the system more flexibility. An example of the two types of moment connections is shown below in a section of the roof plan for Wing C. The triangles are full moment connections and the dots are semi-rigid.

<u>Figure 7B:</u>



The lateral loads seen in the Penthouse are going to be the greatest based on height. At the highest Penthouse roof level, there are moment connections in the transverse direction and single angle braced frames in the longitudinal direction. The connections to the roof of the building are rigidly connected to the roof framing members. These members then transfer the load to flexible moment connections in the columns supporting the roof. These roof members are a larger W27x102 compared to adjacent members such as W24x68 or W27x84.

4. Design Codes

The Pennsylvania Uniform Construction Code (PUCC) is the code adopted by the city of Bethlehem, Pennsylvania. The PUCC is based on the International Code Council (ICC). When design was completed in 2008, the 2006 PUCC referenced the following codes:

2006 International Building Code

2006 International Electrical Code

2006 International Fire Code

2006 International Fuel Gas Code

2006 International Mechanical Code

ASCE 7-05, Minimum Design Loads for Buildings and Other Structures

AISC Steel Construction Manual, 13th Edition

ACI 318-05, Building Code Requirements for Structural Concrete

ACI 530-05, Building Code Requirements for Masonry Structures

The primary codes employed were the AISC Manual and ASCE 7-05

5. Design Loads

5.1 Live Loads

Table 1: Live Load Values

Occupancy	Design Load on Drawings	ASCE 7-05 Load (Tables 4-1, C4-1)
Office	50 PSF	50 PSF + 15 PSF (Partitions)
Classroom	40 PSF	40 PSF
Laboratory	100 PSF	100 PSF
Storage	125 PSF	125 PSF
Corridors/Lobbies @ Ground Level	100 PSF	100 PSF
Corridors Above Ground Level	80 PSF	80 PSF

5.2 Dead Loads

Table 2: Calculated Dead Load

	Dimension	Unit Weight	Load (PSF)
3" 18 Ga. Composite Deck			2.84
4-1/2" Topping	0.485 CF/SF	150 PCF	72.75
Self-Weight			5
MEP Allowance			10
Ceiling Allowance			5
TOTAL			**95.6 PSF**

5.3 Roof Live Load

Table 3: Roof Live Load

Occupancy	Design Load on Drawings	ASCE 7-05 Load (Tables 4-1, C4-1)	Design Load
Roof	N/A	20 PSF	**20 PSF**

5.4 Roof Dead Load

Table 4: Roof Dead Load

	Dimension	Unit Weight	Load (PSF)
3" 16 Ga. NS Roof Deck			2.46
3" Concrete Topping (Avg.)	0.290 CF/SF	150	43.5
Self-Weight			5
Roofing Allowance			10
TOTAL			**60.96 PSF**

5.5 Roof Snow Load

5.5.1 Uniform Roof Snow Load

Table 5: Uniform Roof Snow Load

Design Factor	ASCE 7-05	Design Value
Snow Load (Pq)	Figure 7-1	30 PSF
Roof Exposure	Table 7-2	Fully Exposed
Exposure Type	Section 6.5.6.2	B
Exposure Factor (Ce)	Table 7-2	.9
Thermal Factor (Ct)	Table 7-3	1.0
Building Type	Table 1-1	III
Importance Factor (I)	Table 7-4	1.1
Flat Roof Snow Load (Pf)	Equation 7-1	20.8 PSF
Minimum Snow Load (Pf,min)	Section 7.2	22 PSF
Design Snow Load	Section 7.2	**22 PSF**

Pf = 0.7(Ce)(Ct)(I)(Pq)

Pf = 0.7(.9)(1.0)(1.1)(30) = 20.8 PSF

20.8 < Pf,min = 22 → Use 22 PSF as the Design Snow Load

5.5.2 Drift Snow Load

NOTE: For simplification of this analysis, snow drift was not considered. However, it will be necessary to consider snow drift later.

5.6 Penthouse Live Load

Table 6: Penthouse Live Load

Occupancy	Design Load on Drawings	ASCE 7-05 Load (Tables 4-1, C4-1)	Design Load
Mechanical Room	N/A	200 PSF	**200 PSF**

5.7 Penthouse Dead Load

Table 7: Penthouse Dead Load

	Dimension	Unit Weight	Design Load (PSF)
3” 18 Ga. Composite Deck			2.84
4-1/2” Concrete Topping	0.485 CF/SF	150 PCF	72.75
Self-weight			5
MEP Allowance			10
Ceiling Allowance			5
TOTAL			**95.6 PSF**

5.8 Brick Façade Load

Table 8: Brick Façade Load (Per Level)

	Height	Unit Weight (PSF)	Design Load (PLF)
Brick Veneer	10’-3”	35	357.8
2” Rigid Insulation	10’-3”	3	30.7
Steel Framing	10’-3”	6	61.3
Gypsum Wall Board	10’-3”	2	20.5
Window (Glass, Frame, Sash) (ASCE 7-05 Table C3-1)	5’-1”	8	40.8
TOTAL			**510.6 PLF**

5.9 Glass Curtain Wall Load

Table 9: Glass Curtain Wall Load (Per Level)

	Dimension	Unit Weight (PSF)	Design Load (PLF)
Window (Glass, Frame, Sash) (ASCE 7-05 Table C3-1)	15’-4”	8	**122.4 PLF**

5.10 Penthouse Wall Load

Table 10: Penthouse Wall Load

	Dimension	Unit Weight (PSF)	Load (PLF)
Metal Wall Panel	16'-4"	5	81.7
Steel Framing	16'-4"	7	114.3
Bracing Allowance	16'-4"	3	49
TOTAL			**246 PLF**

6. Wind Pressures

ASCE 7-05 was used for wind design. The Analytical Procedure in Chapter 6 is specifically what was instituted.

Table 11: Wind Design Factors:

Design Factor	ASCE 7-05	E/W Value	N/S Value
Design Wind Speed (V)	Figure 6-1C	90 mph	90 mph
Building Type	Table 1-1	III	III
Importance Factor (I)	Table 6-1	1.15	1.15
Exposure Type	6.5.6.2	Type B	Type B
Average Height (z)	6.5.8	84'	100'

Table 12: Design Wind Pressure by Level (Transverse Direction)

Level	Height	kz	qz	Pz (PSF) (Windward)	Ph (PSF) (Leeward)	Ptotal (PSF)
1	0'-0"	0.57	11.55	14.21	-11.26	25.47
2	15'-4"	0.58	11.76	14.46	-11.47	25.93
3	30'-8"	0.71	14.39	17.7	-14.03	31.73
4	46'-0"	0.79	16.01	19.69	-15.61	35.3
Roof/5th	60'-8"	0.85	17.22	21.18	-16.79	37.97
Roof/Penthouse	77'-0"	0.92	18.65	22.94	-18.18	41.12

NOTE: Assumed a partially enclosed building (qi=qz)

Figure 8: Elevation of Transverse Pressure Levels

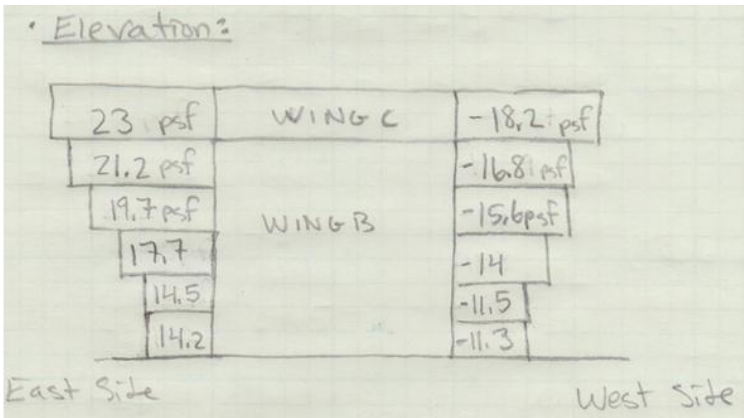
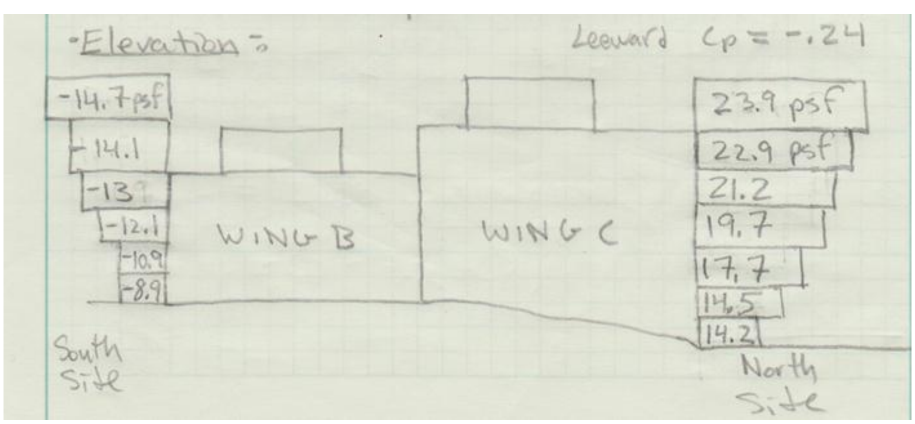


Table 13: Design Wind Pressure by Level (Longitudinal Direction)

Level	Height	kz	qz	Pz (PSF) (Windward)	Ph (PSF) (Leeward)	Ptotal (PSF)
G	0'-0"	0.57	11.55	14.21	N/A	14.21
1	15'-4"	0.58	11.76	14.46	-8.87	23.33
2	30'-0"	0.70	14.4	17.70	-10.85	28.55
3	45'-4"	0.79	16.01	19.69	-12.07	31.76
4	61'-0"	0.85	17.23	21.19	-12.99	34.18
Roof/5th	77'-4"	0.92	18.65	22.94	-14.06	37.00
Roof/Penthouse	92'-0"	0.96	19.46	23.94	-14.67	38.61

Figure 9: Elevation of Longitudinal Pressure Levels



7. Seismic Loads

Chapters 11 and 12 of ASCE 7-05 were used for seismic load design. The Equivalent Lateral Force procedure tests whether the building has the capability of handling a seismic event based on site and building properties.

Hand calculations can be found in Appendix A-2.

7.1 Seismic Design Factors

Design factors were the same for transverse and longitudinal directions since the building's lateral framing system consists of moment frames in both directions. Instead of determining the actual fundamental frequency through extensive calculation, the approximate fundamental period was determined using ASCE 7-05 Section 12.8.2.1.

Table 14: Seismic Load Design Factors

Design Factor	ASCE 7-05	Value
Short Period Spectral Response Acceleration (Ss)	USGS	0.291
One Second Spectral Response Acceleration (S1)	USGS	0.081
Site Class	Table 11.4-1	C
Short Period Site Coefficient (Fa)	Table 11.4-2	1.2
Long Period Site Coefficient (Fv)	Equation 11.4-1	1.7
Adjusted MCE Short Period Spectral Response Acceleration (Sms)	Equation 11.4-1	0.349
Adjusted MCE One Second Spectral Response Acceleration (SM1)	Equation 11.4-2	0.138
Design Short Period Spectral Response Acceleration (SMs)	Equation 11.4-3	0.233
Design One Second Spectral Response Acceleration (SM1)	Equation 11.4-4	0.0918
Maximum Height from Base (hn)	N/A	108.3'
Approximate Period Parameter (Ct)	Table 12.8-2	0.028

Approximate Period Parameter (x)	Table 12.8-2	0.8
Approximate Fundamental Period (Ta)	Equation 12.8-7	1.19 Hz
Building Type	Table 1-1	III
Importance Factor (I)	Table 11.5-1	1.25
Seismic Design Category	Table 6-2	B
Response Modification Coefficient (R)	Table 12.2-1	3.0
System Over-strength Factor (Omega)	Table 12.2-1	3.0
Deflection Amplification Factor (Cd)	Table 12.2-1	3.0
Flexible Diaphragm Condition	Section 12.3.1	Rigid
Long Period Translation Period (TL)	Figure 22-15	6
Seismic Response Coefficient (Cs)	Equation 12.8-3	0.0321

7.2 Effective Seismic Weight

Table 15: Effective Seismic Weight by Level

Level	Floor Area (SF) (96 PSF)	Roof Area (SF) (62.5 PSF)	Penthouse Floor Area (SF) (296 PSF)	Brick Façade (ft.) (510.6 PLF)	Glass Curtain Wall (ft.) (122.4 PLF)	Penthouse Wall (ft.) (246 PLF)	Effective Seismic Weight (k)
Penthouse		4497					281.06
Roof/Penthouse		7894	4497			288.7	1895.5
5	10832	9375	1557	421.3		161.3	2341.47
4	21814			589.7	89.5		2406.2
3	21814			589.7	89.5		2406.2
2	21814			589.7	89.5		2406.2
1	21814			589.7	89.5		2406.2
TOTAL	98088	21766	6054	2780.1	358	450	**14143**

7.3 Design Seismic Loads

Table 16: Seismic Design Loads by Level

Level	Effective Seismic Weight (wx)	Height from Base (hx)	$(wxhx)^k$	Vertical Distribution Factor (Cvx)	Lateral Seismic Force (Fx) (k)	Seismic Design Story Shear (Vx) (k)	Overturning Moment (k-ft.)
Penthouse	281.06 k	108.3'	3298348	0.0654	29.97	29.97	3217.11
Roof/Penthouse	1895.5 k	93'	16390547	0.3250	147.57	177.54	13724.53
5	2341.47 k	76.7'	13763837	0.2729	123.92	301.46	9501.36
4	2406.2 k	61.3'	9050606	0.1794	81.48	382.94	4997.72
3	2406.2 k	46'	5091519	0.1009	45.84	428.78	2108.76
2	2406.2 k	30.7'	2263389	0.0448	20.37	449.15	625.02
1	2406.2 k	15.3'	565478	0.0112	5.09	**454**	78.05
TOTAL	**14143 k**		50423724	1.0			**34252.54**

Seismic Base Shear = 454 k
Overturning Moment = 34252.5 k-ft.

Calculations for the earthquake analysis can be made available upon request.

8. Gravity Member Spot Check

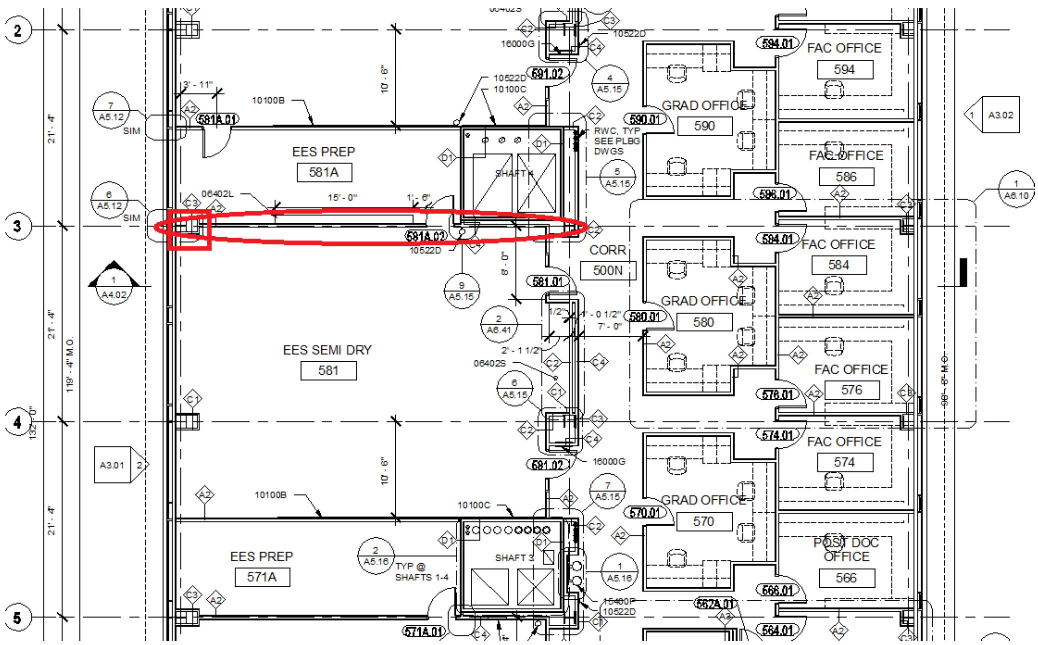
A floor slab, slab span, composite beam, and gravity column were inspected.

The floor slab seemed to be appropriately sized for both loading and deflection criteria. The span was also appropriate for unshored construction.

For the composite beam, the live load was not reduced as a conservative decision. However, after inspection, the loading imposed was greater than could be tolerated by the beam. This means that in all likelihood, live loads were reduced. The numbers are very close (1308>1300 k-ft.) and can be seen in Appendix A-2.

For the column, the column appeared to be extremely oversized even with an unreduced live load of 100 PSF. There must be a reason for this beyond pure axial strength. Perhaps this column was also designed for deflection requirements. Perhaps the building was designed for new equipment to be added at a later date or for expansion. Perhaps the column was designed to take some of the moment from the lateral system. These sort of combined effects will be examined in later reports.

Figure 10: Gravity Spot Check Elements



9. Conclusion

This process of analyzing the building has led me to believe that a much more in depth study must be undertaken for Tech 2. Combined effects must be studied and a much more intimate understanding of the loads used in design must be undertaken. It should also be considered that educational facilities frequently overdesign in preparation for expansion and growth. Also, dead or live loads may increase depending on specific equipment used in a laboratory setting.

A computer model and Excel sheets will be made for the upcoming assignment to limit hand calculations and better understand the building as a system, not just as individual parts. Some more complex and in depth calculations need to be undertaken as well, such as foundation analysis and a building enclosure analysis. Roof uplift must also be added to the wind analysis and snow drift must be considered.

Appendix A-1

ASCE 7-05: Section 6.5

V = 90 mph	(Figure 6-16)
Occupancy is 1490 > 500 for university, so TYPE III	(Table 1-1)
Importance TYPE III; V < 100 mph → therefore, I = 1.15	(Table 6-1)
Roughness Type B (Urban/Suburban)	(Section 6.5.6.2)

Figure A1: Plan View

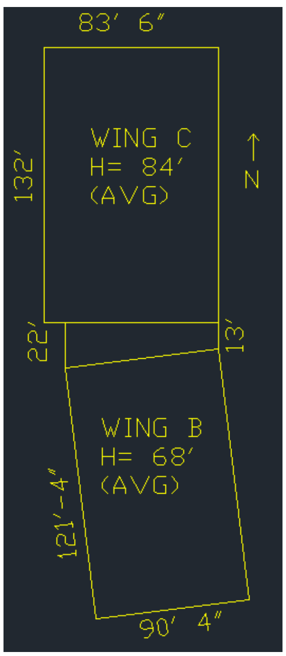
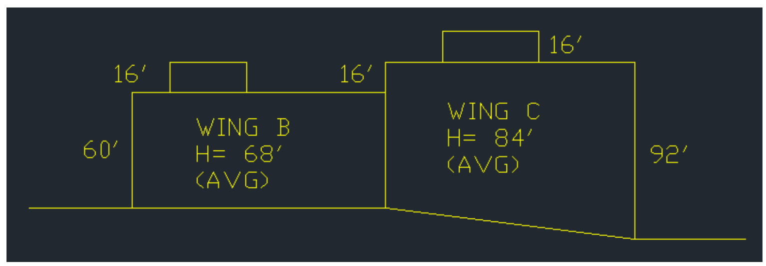


Figure A2: East Elevation



E/W: L = 86.9' B = 275.3'

$$H = \frac{(154)(84') + (121.3)(65')}{(154+121.3)} = 78'$$

N/S: L = 275.3' B = 86.9'

H = 100' to be conservative

- Building Category = III (Table 1-1)
- Exposure = B (Urban/suburban) (6.5.6.2)

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I \quad \text{(Eq. 6-15)}$$

- Determine z for top Level:

	E/W	N/S
$z =$	77'	108'-4"

- Determine K_z for Roof Level: (Table 6-3)

	E/W	N/S	
$K_z =$	.92	1.01	
$K_{zt} =$	1.0	1.0	(6.5.7)
$K_d =$	.85	.85	

$I = 1.15$ (Category III) (Table 6-1)

	E/W	N/S
$q_z =$	18.65 psf	20.47 psf

- Since $T_o < 1\text{ sec} \rightarrow G = .85$ (6.5.8.1)

*Assuming # stories/10 = 6/10 = .6 = T_o

- $GC_{pi} = \pm .55$ partially enclosed buildings (Fig. 6-5)

$$p = qGC_p - q_i(GC_{pi})$$

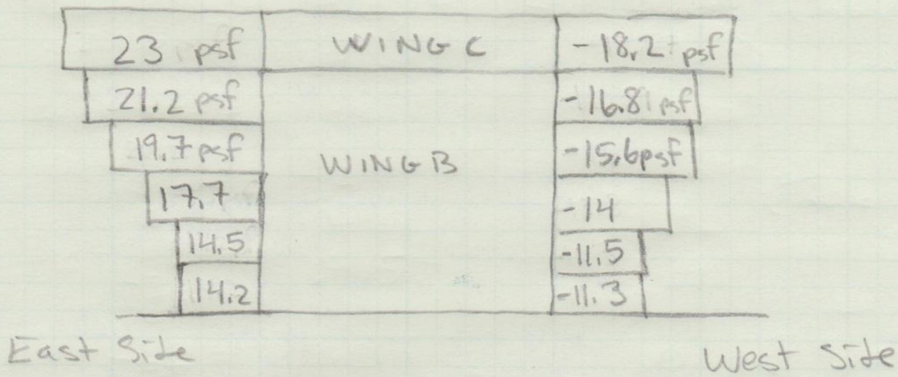
- East / West Pressures (Figure 6-6)

  Windward $Cp = .8$

  Leeward Cp is a function of L/B

  $$L/B = \frac{86.9}{275.3} = .316 \rightarrow Cp = -.5$$

- Elevation:

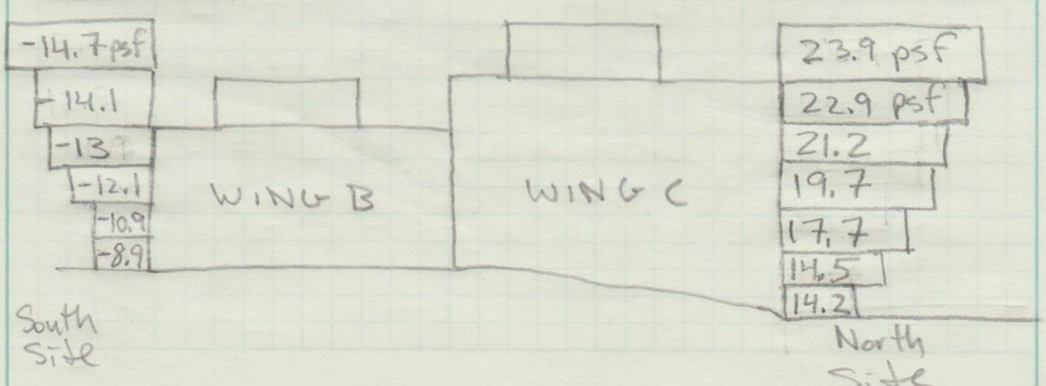


- Sample Calc (wind ward)

  $$p = 18.65\,(.85)(.8) - (18.65)(\pm .55)$$

  $$p = 22.94 \text{ psf}$$

- Elevation: Leeward $Cp = -.24$



Appendix A-2:

· Check Floor Slab

Worst Total Super. Load on Floor Slab is
125 psF Live + 20 PSF misc. Dead
= 145 PSF

For 3" 18 Ga., use Vulcraft 3VLI18

It can support 210 psf with 11' clear span
and the 4 1/2" topping as specified
210 psf > 145 psf $\boxed{\text{OK}}$

· Check Span:

Design Clear Span is 10'8"

3VLI18 can be unshored up
to 12'-0"
12' > 10'8" $\boxed{\text{OK}}$

The Deck and Slab are suitable.

· Check Composite Beam

Beam selected is between
A3 and B3 on 5th Floor
of Wing C.

Live Load for Lab = 100 psf
*Could be reduced

Dead Load = 95.6 PSF

$1.2(95.6) + 1.6(100) = 274.7$ PSF

Trib. width = 21'4"

$\frac{274.7}{1000} \times 21.33 = 5.86$ klf

• Simply Supported

$M_{max} = \frac{wl^2}{8} = \frac{5.86(42.25')^2}{8}$

$= 1307.6$ k-ft

• Existing Beam is 24x76 [50]

* Assuming 17.4 k is shear stud strength, $50(17.4) = \Sigma Q_n = 870$ k

$b_{eff} = \min \left| \begin{array}{l} \frac{42.25(12)}{8}(2) = 126.75" \\ \frac{21.33(12)}{2}(2) = 255.96" \end{array} \right.$

$b_{eff} = 126.75"$

$A_s = 22.4$ in^2

$A_s f_y = 22.4(50) = 1120$ k

$V'c = .85(126.75)(4000)(4.5)/1000$

$= 1939.3$ k

$\Sigma Q_n < V'c$
$< A_s f_y \Rightarrow Y_2 = 4.5 - \frac{a}{2}$

$Y_2 = 4.5 - \frac{870}{2(.85)(4)(126.75)} = 3.49 \approx 3.5"$

For W24x76 w/ $\Sigma Q_n = 870k$
and $Y_2 = 3.5''$,

$$\phi M_p = 1300 \text{ k-ft}$$

$$1300 \text{ k-ft} < 1308 \text{ k-ft}$$
design moment

* Must have
chosen a higher loading

* Could have reduced live loads

* Numbers are close.

- Check Column (Pure Axial)
Column A3 below Level 5

Brick Load = 510.6 PLF
Roof DL = 60.96 PSF
Snow = 22 PSF > Roof LL of 20 PSF
Laboratory LL = 100 PSF

$$1.2(60.96) + 1.2(510.6) + 1.6(100) + .5(22)$$

$$= 244.15 \text{ PSF} + 612.7 \text{ PLF}$$

$$612.7\,\frac{(21.33)}{1000} = 13.07 \text{ k from veneer}$$

$$\frac{244.15}{1000}\left(21.33 \times \frac{42.25}{2}\right) = 110 \text{ k}$$

$$P_u = 123.07 \text{ K}$$

Column is W 14x109 @ A4
between 4 and 5

$KL = 15'4''$

For $KL = 16'$ $\phi P_n = 1190K > 123.07k$

$\boxed{OK}$

Why is the column so oversized?

Could be for serviceability

Could be for future expansion