Biobehavioral Health Building

University Park, PA

Tech 1 Report: Structural Concepts and Existing Conditions Report

2012-2013 AE Senior Thesis

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Tech 1 Report

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Executive Summary

The following technical report was written to summarize the structural concepts and existing conditions of the Biobehavioral Health Building (BBH Building). In the report an overview of the different structural system will be given to better understand how certain loads are resisted. All of the construction documents were provided by Massaro CMS Services. All of the images (unless otherwise noted) in this report are the property of Bohlin Cywinski Jackson (Architect) and are being used for educational purposes.

Various loads such as wind, seismic, and gravity, were either estimated and or calculated using ASCE 7-05 or they were given on the first page of the structural drawings. In order to gain a better understanding, spot checks were made on a column, girder, beam, and deck with gravity loads only applied to them. It was then revealed that all the members passed with a very conservative design in some cases. This can be attributed to the fact the lateral loads were ignored in the analysis of these members and that the building shows redundancy in its design.

Through comparison of the base shears due to wind and seismic loads show that the wind load will control. This was expected and is common for structures located in this region. Due to geometry of the BBH Building it was found that wind loads in the N-S direction are much greater than that of the E-W direction.

Building Introduction

Located on the campus of the Pennsylvania State University in University Park, Pennsylvania is the Biobehavior Health Building(Figure 1). It is currently under construction and is scheduled to be finish in November 2012. When completed, it will house faculty and graduate students from the College of Health and Human Development. The overall project cost is approximately $40,000,000 and is being funded by the Pennsylvania Department of General Services. The BBH Building is comprised of 5 stories above grade (including a penthouse) and has a full basement 100% below grade.

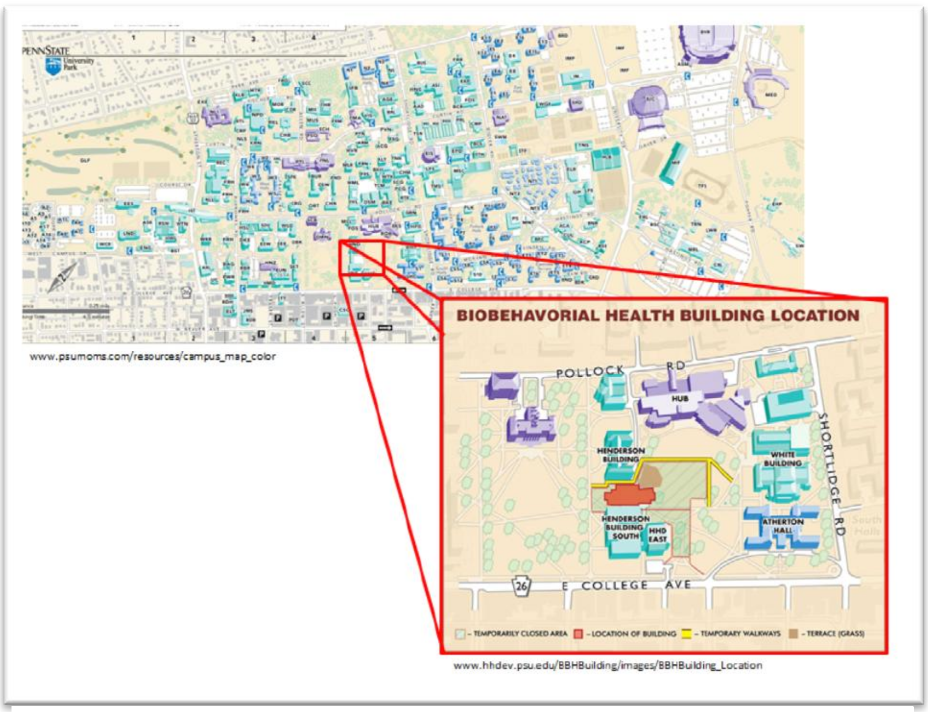


Figure 1: PSU Campus Map

The BBH Building was designed to blend with that existing architecture that surrounds it. The majority of the façade was designed to mimic Henderson North's Georgian style architecture with its large amount of hand placed brick and limestone. On the north east portion of the building the design is more modern to replicate that of the HUB. Since a portion of the BBH building protruded into the HUB Lawn, which is a popular student hangout, a terrace has been provided (Figure 2). Not only does this offer a relaxing place for students to lounge but it will also be used as a stage for future concerts. A majority of the interior space is made up of offices and conference rooms that will house faculty and graduate students from the college of health and human development. One of the key interior spaces is the lecture hall, which is located on the ground floor directly below the HUB lawn terrace. It is able to seat up to 200 people and has a ceiling designed to absorb any sounds or vibrations coming from the terrace above.

Figure 2: Rendered View from HUB Lawn

Structural Overview

Foundation

CMT Laboratories, Inc. was the geotechnical engineers hired to investigate the soil conditions on which the BBH building was to be placed. In order to better understand the soil located on the site CMT Laboratories took six test boring samples located around the site. With the information gathered from the test borings they were able develop recommendations for the structure below grade.

It was recommended that the foundations bear on sound dolomite bedrock. This bedrock must be free of clay seams or voids near the surface to provide a stable surface to place the foundations. If bedrock was run into before the required bearing elevations were met then over excavation was required and needed to be back fill with lean concrete. The bearing material must have a bearing capacity of 15 ksf minimum.

The BBH Building uses a shallow strip and spread footing foundation system. The strip footings are placed under the foundation walls around the perimeter of the building. These footings are at an elevation of -15’ and step down to -21’ around the lecture hall. A typical strip footing is 30” and 18” deep as shown in Figure 3. Normal weight concrete is used for all footings and must have minimum compressive 28 day strength of 4 ksi.

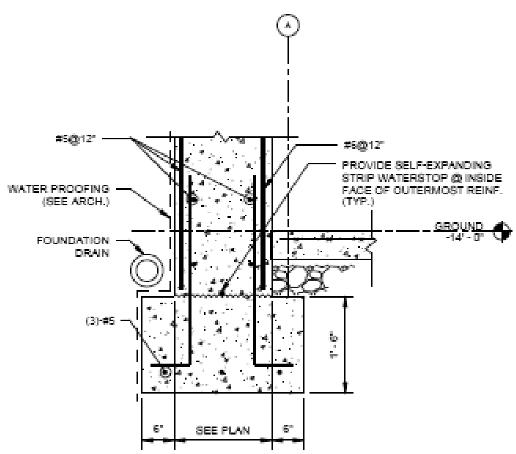


Figure 3: Typical Strip Footing

Floor/Framing System

The BBH Building floors are concrete slab on metal deck. The typical slab on deck is consist of 3 ¼" light weight concrete on 3" 18 gage galvanized composite steel deck that is reinforced with 6"x6" W2.0xW2.0 welded wire fabric. Any deck opening that cut through more than two deck webs needed to be reinforced. This was typically done with 4' long #4 rebar place at each corner as shown in Figure 4. This is typically done to keep the integrity of the slab and also prevents unwanted cracking in the concrete.

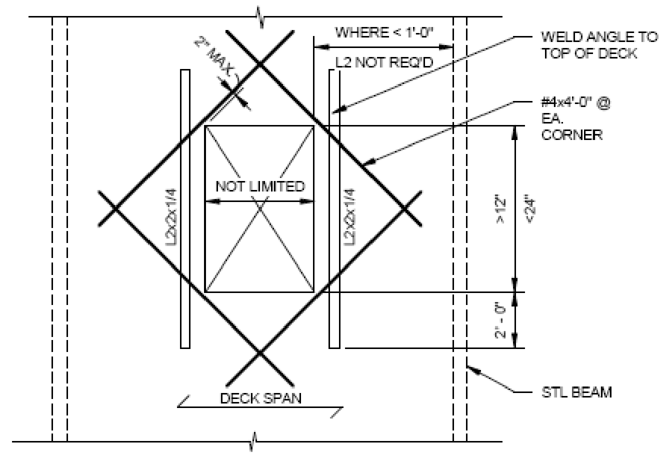


Figure 4: Openings in Slab on Steel Deck

In order to decrease beam depth the BBH building was designed as a composite steel system. Figure 5 shows a typical section through this composite system. ¾" diameter shear studs are welded to the top flange of the beam/girder. The number of shear studs varies per beam/girder. The typical floor plan has beams spanning N-S and girder spanning E-W. See fig x-x for a typical floor plan.

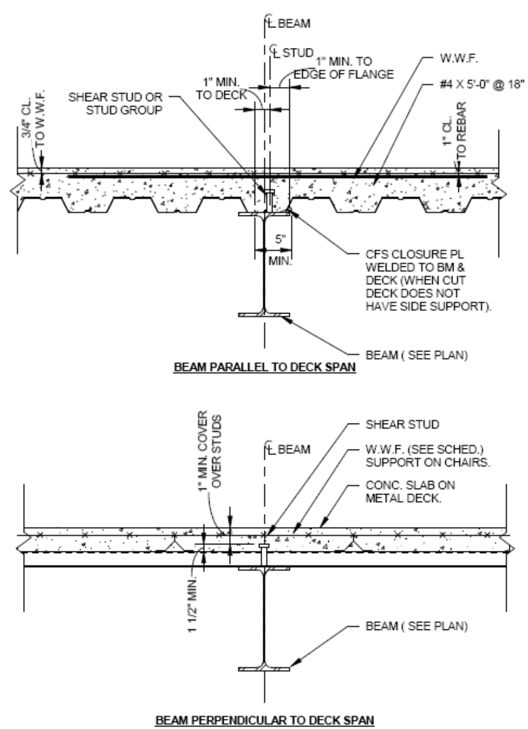


Figure 5: Typical Section Through Composite System

The composite slab supports gravity loads and transfers that load to the beams. The beams then transfer the load to the girders, which transfer the load to the columns. Finally the load is terminated at the foundations.

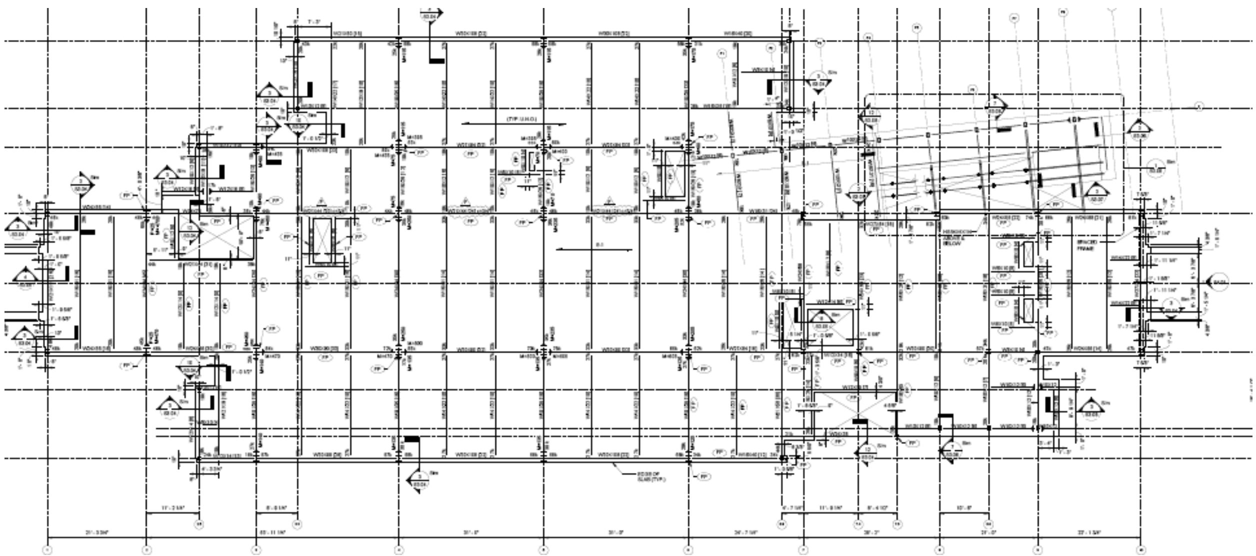


Figure 6: Typical Floor Framing Plan

Lateral System

The BBH Building uses two types of lateral force resisting systems, one being moment frames and the other an eccentric braced frame. These systems are used to resist lateral forces placed on the structure due to wind and seismic.

The moment frames are in both the N-S and E-W direction. Frames resisting N-S loads go from column line 2 to column line 6. Frames resisting E-W loads go are only located along column lines B and D. This type of system is use on every level above grade. These moment frames are accomplished by designing a rigid connection between the beams and columns. A rigid connection is created by welding the top and bottom flange of the beam to the column as shown in Figure 7. Location of the moment connections are located below in Figure 8. Because the east wing of the BBH Building is exposed to the HUB lawn, it will be exposed to higher wind loads. This could be the reason for why a duel lateral system was used and why it is configured as such (Figure 8).

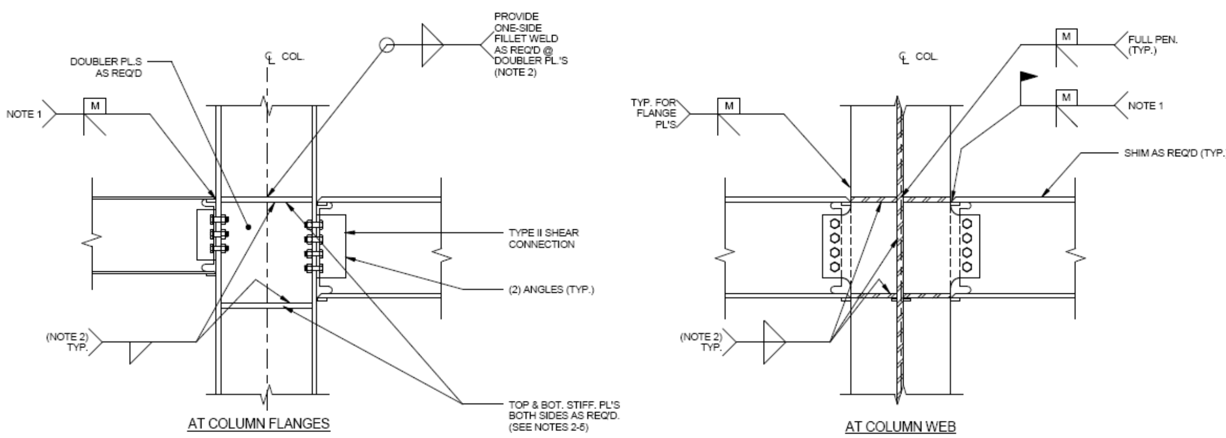


Figure 7: Typical Beam to Column Moment Connection

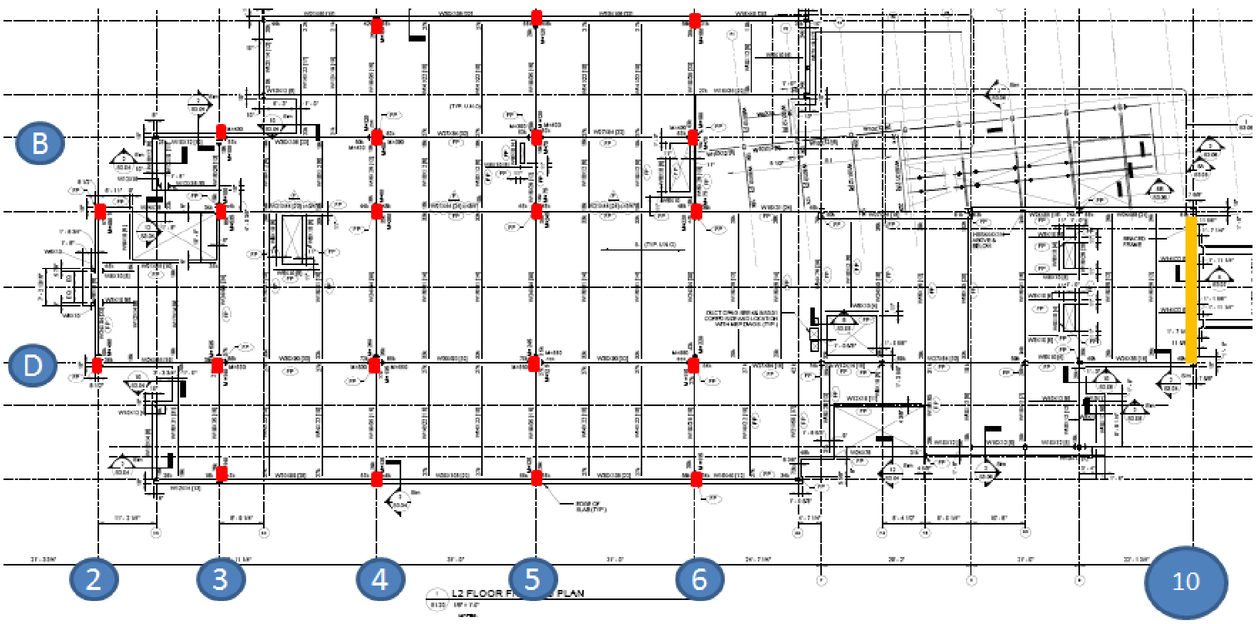


Figure 8: Location of Moment Connections (Red) and Braced Frame (Orange)

There is only a single eccentric braced frame in the BBH Building. It is located on the east side of the building along column line 10 (See Figure 8 above). Figure 9 shows the chevron bracing system used. Lateral movement in the frame is resisted through tension and compression in the HSS braces.

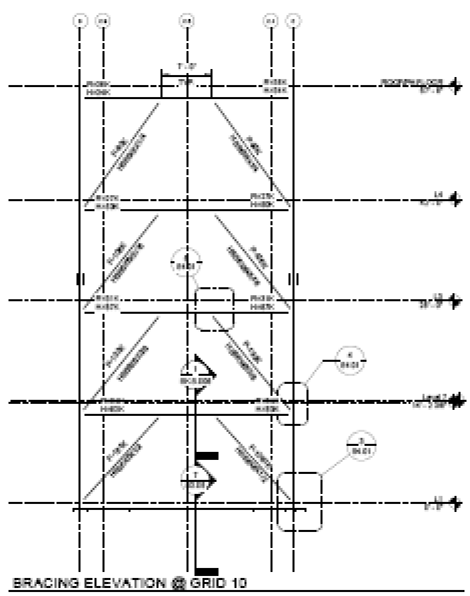


Figure 9: Eccentric Braced Frame

Design Codes

The BBH Building was designed using the following codes:

- IBC 2006 (as amended by Pennsylvania UCC administration)
- ASCE 7-05
- ACI 318
- ACI530/ASCE 5
- AISC, 13th Edition

For this thesis the following codes were used in the analysis for the BBH Building:

- AISC, 14th Edition
- ASCE 7-05

Material Properties

Steel	
Wide flange shapes	A992 or A572, fy=50ksi
Square and round steel tubing	ASTM A500, Grade B
Miscellaneous shapes, channels and angles	A36, or A572, fy=50ksi
Round pipes	A53, Grade B, fy=35ksi
Plates	A36, fy=36ksi
Anchor Rods	ASTM F1554, Grade 55
Bolted connections for beams and girders	A325 or F1852, 3/4" diameter
Welded headed shear studs	A108 3/4" diameter
Stainless steel hanger rods	ASTM A564 Type 17-PH fy=50ksi

Concrete	
Type	28 day compressive strength
Foundations	4000 psi
Slabs and beams	4000 psi

Reinforcement	
Deformed Bars	ASTM A615, Grade 60
Welded Reinforcing Steel	ASTMA706 Grade 60
Welded Wire Fabric	ASTM A185

Design Loads

The following design loads were either given by the designer on the general notes page or estimated using ASCE 7-05.

Dead

Dead Load	Uniform (psf)
Floor Slab on Deck	46
Roof Deck	3.3
Green Roof	25
Superimposed	5
Structural Steel	5
Façade	45
CMU (fully grouted	83
Interior brick walls	40
Interior stone floors	20
Slate Roof	10

Live

Live Load	Uniform (psf)	Concentrated (lbs)
Offices/Classrooms	80(1)	-
Lobbies/Assembly	100	2000(5)
Corridors, Stair	100	2000(5)
Mechanical Rooms	150(3)	-
Roof	30(2)	-
Plaza	125(4)	-
Assembly (fixed seats)	60	-
Heavy storage	250	2000(5)

1. Includes 20 psf partition load
2. Or Snow Load whichever is greater
3. Used in absence of actual weight of mechanical equipment
4. Used for roof over lecture Hall
5. Concentrated load shall be uniformly distributed over a 2.5 sq ft area and shall be located so as to produce maximum load effects in the structural members

Snow

The calculations for the design snow load can be found in Appendix A. The drift load was designed for the penthouse green roof as that is where the most drift would accumulate.

Snow Load Type	Uniform (psf)
Flat Roof Load	21
Sloped Roof Load	24
Drift Load	89.5

Wind

The wind design loads were found using the MWFRS Analytical Procedure found in ASCE 7-05. In order to do the analysis the building shaped was simplified to a rectangle (see Appendix). The gabled roof was ignored when calculating the wind load in the E-W direction due to the slenderness of it in that direction.

In summary, the base shear in the N-S direction (315 kips) controlled over the base shear in the E-W direction (91 kips). This outcome was expected due to the large surface area the wind encounters in the N-S direction as opposed to the E-W direction. Below are tables and diagrams summarizing the distribution of wind pressures and forces. Hand calculations done for this procedure can be found in Appendix B.

MWFRS Pressures (N-S)			
ht	qz (psf)	Windward Pressure (psf)	Leeward Pressure (psf)
0-15	10.04	9.62	-9.23
20	10.93	10.22	-9.23
25	11.63	10.7	-9.23
30	12.34	11.18	-9.23
40	13.4	11.9	-9.23
50	14.28	12.5	-9.23
60	14.98	12.98	-9.23
63	15.16	13.1	-9.23
67	15.51	6.75	-10.7

Forces on Building (N-S)	
floor	Force (k)
2	61.48
3	67.12
4	74.23
PH	55.79
Bottom of roof	15.68
gabled roof	40.83
Base Shear	315.13

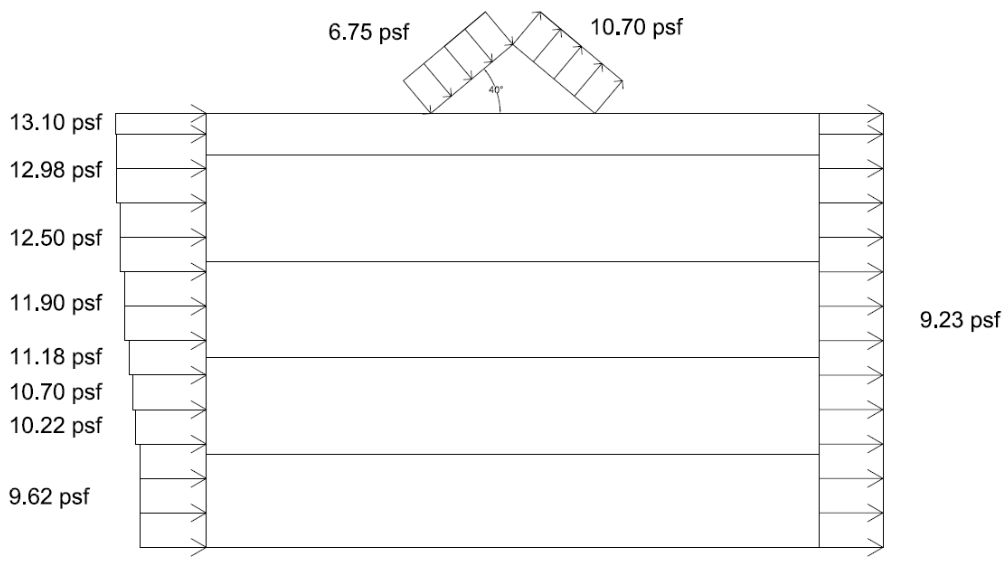


Figure 10: N-S Wind Pressure Diagram

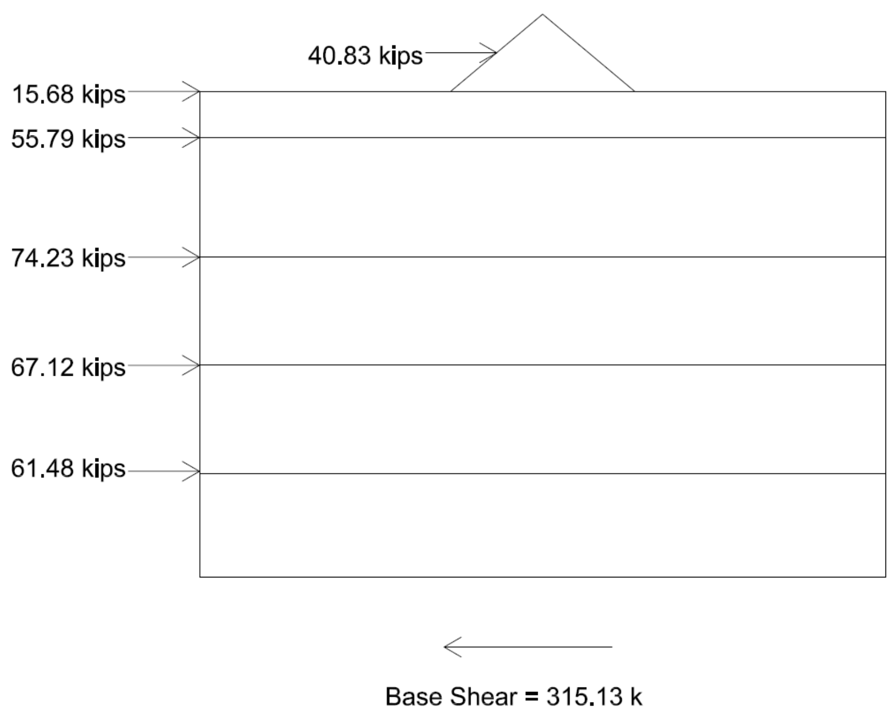


Figure 11: N-S Wind Story Force Diagram

MWFRS Pressures (E-W)			
ht	qz (psf)	Windward Pressure (psf)	Leeward Pressure (psf)
0-15	10.04	9.56	-6.21
20	10.93	10.16	-6.21
25	11.63	10.63	-6.21
30	12.34	11.12	-6.21
40	13.4	11.84	-6.21
50	14.28	12.44	-6.21
60	14.98	12.92	-6.21
63	15.16	13.04	-6.21

Forces on Building (E-W)	
floor	Force (k)
2	19.6
3	21.69
4	24.19
PH	20.48
Bottom of roof	5.14
Base Shear	91.1

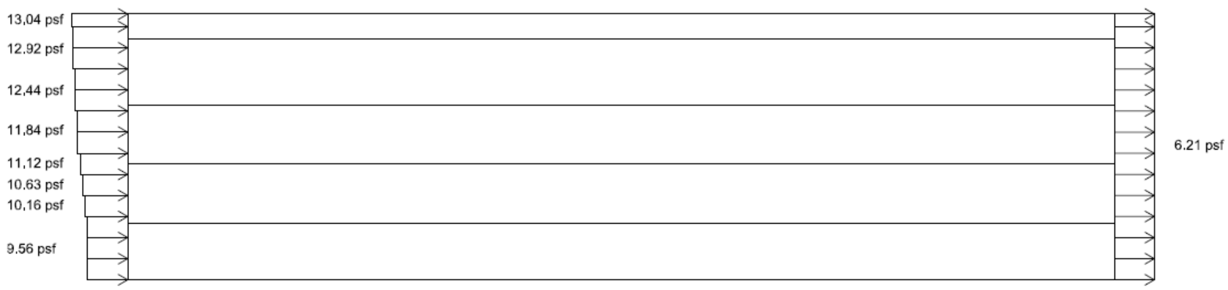


Figure 12: E-W Wind Pressure Diagram

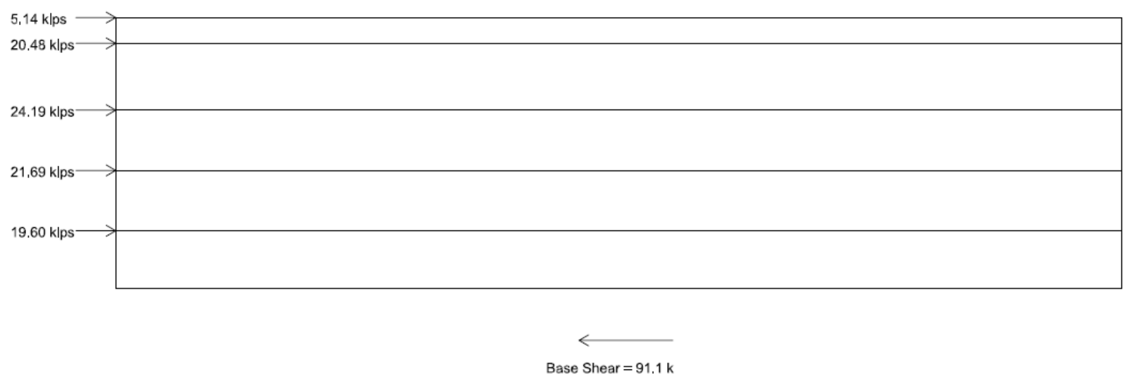


Figure 13: E-W Wind Story Force Diagram

Seismic

Chapters 11, 12, and 22 of ASCE 7-05 were used to find the seismic design load for the BBH Building. More specifically section 12.8 was used to calculate the base shear. In order to calculate the base shear the total building weight needed to be estimated. This was done using estimated square footages and the dead loads (Appendix C). Through the geotechnical testing it was determine by the geotechnical engineer that the soil would be classified as site class C – very dense soil and soft rock. Due to unknown errors in my assumptions/calculations my Cs value calculated was 5 times that of what the designer found (.01), which greatly increased the base shear. Further discussion with the design professional will be done to better understand how they came up with a Cs of .01. In order to move forward with the seismic load design the design professional's value of Cs was used to calculate the base shear. See Appendix C for hand calculations. Vertical distribution of the seismic forces is shown below in Figure 14.

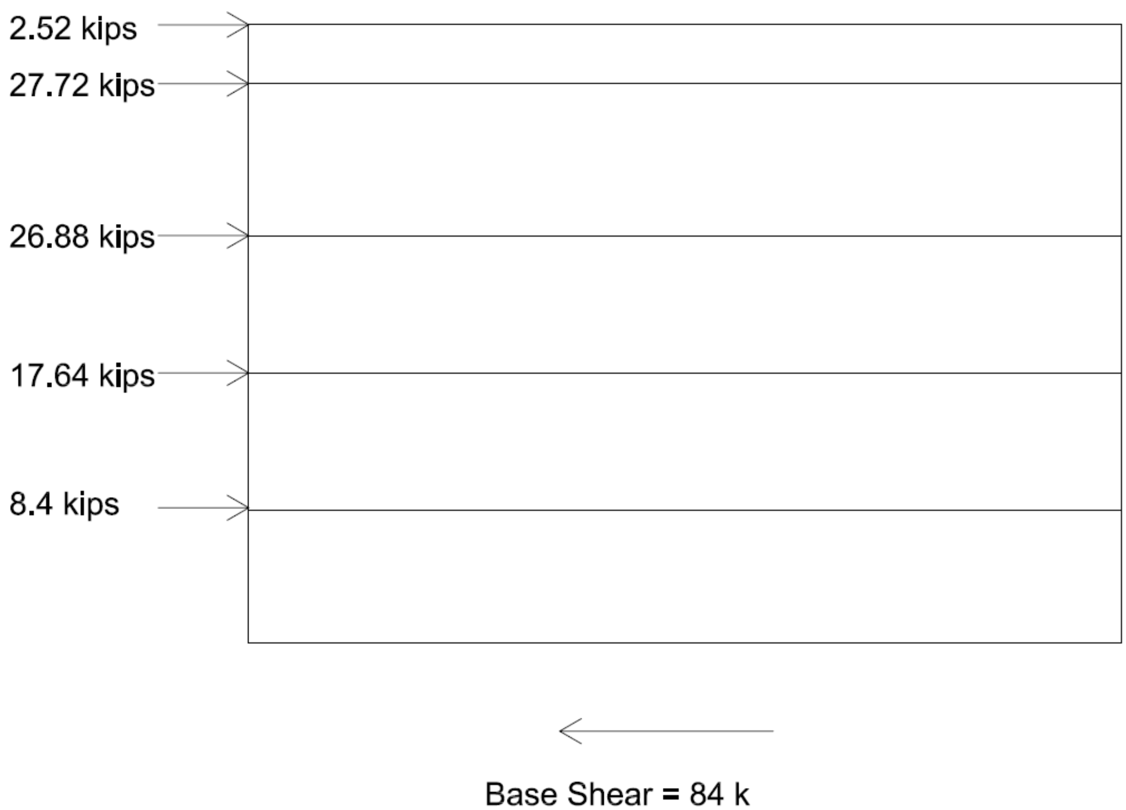


Figure 14: Vertical Distribution of seismic forces

Spot Checks

LRFD load combinations were used in the analysis of the following spot checks.

Composite Deck

A quick spot check was done on the composite steel deck system used in the BBH Building. The check was done for the deck spanning inside column lines 5, 6, D, and E. The Vulcraft 2008 catalog was used to confirm that the 3” 18 GA composite deck with 3” LW concrete topping was adequate. It was determined that this design was adequate to support the required loads. Redundancy and fire rating could be factors causing the conservative design. See Appendix D for hand calculations.

Composite Beam & Girder

One of the interior composite beams used to support the deck was checked for acceptable unshored strength, wet concrete deflection, and live load deflection. It was found that a W 12x19 beam with 14 shear studs meets all of the above strength and deflection requirements. This is slightly conservative compared to the W14x22 [10] specified on the structural drawings. Being that a typical floor plan has some redundancy it is possible for overdesign in some members. Results were found to be similar for a typical exterior girder that supports the beams described above. See Appendix D for hand calculations

Column

Column A-5 is an exterior column that supports offices located on levels 2&3 and the green roof at level 4. Below are tables that were developed to determine the loads acting on the column due to only gravity. Live load reduction was taken advantage of in the determination of the loads.

floor	trib area (ft^2)	Façade Area ($ft^{2)}$	DL (psf)	Façade DL (psf)	LL (psf)	LL Reduced	Pu_{story} (k)	ΣPu (k)
2	703	434	51	128	80	33	114	252
3	703	434	51	128	80	33	114	138
4(roof)	703	0	28	128	30	16	24	24

The column specified to carry these loads was a W12x106. This column has an unbraced length of 14 feet and has a ϕP value well over the required to support the gravity load (see table below).

floor	Column	Unbraced Length (ft)	ϕPn	Adequate Strength?
2	W12x106	14	1130	Yes
3	W12x106	14	1130	Yes
4(roof)	W12x106	14	1130	Yes

Because this spot check only analyzed the column under gravity loads, it was expected that the analysis would show the column being extremely over designed. Further investigation, in Tech Report 3, due to lateral loads will show that the column used is of an economical design. See Appendix D for hand calculations.

Conclusion

Through this initial investigation of the existing structural system it was determined that the deck, beams, girders, and columns are adequately designed to carry the gravity loads applied to them. Analysis shows that the design is very conservative in some cases if these members were to only be subjected to gravity loads. Lateral loads will be considered in the analysis of these members in tech report 3.

Though lateral forces were not used to do spot checks, they were calculated. Both wind and seismic were determined using ASCE 7-10. Once completed it was revealed that lateral loads from wind would be the controlling factor in the design of the BBH Building. Discrepancies were found in the calculation of the seismic response coefficient. A follow up discussion with the design engineer will need to be done in order to determine what assumptions were made when using the equivalent lateral force procedure.

Upon completion of this report, a better understanding of the structural system for the BBH Building was acquired.

Appendix A: Snow Load & Drift Calculations

Daniel Bodde	Tech I	Snow load	1

Snow load

ASCE 7-05

7.3 flat roof

$p_f = 0.7\,C_e\,C_t\,I\,p_g$

$p_g = 30$ lb/ft² (Fig 7-1)

$C_e = 1.0$ (table 7-2)

$C_t = 1.0$ (table 7-3)

$I = 1.0$

$p_f = (0.7)(1.0)(1.0)(1.0)(30) = 21$ lb/ft²

7.4 Sloped roof

$p_s = C_s p_f = (0.8)(30) = 24$ lb/ft²

$p_f = 30$ lb/ft²

$C_s = 0.8$ (fig 7-2)

7.7 Drifts on lower roofs

$\gamma = 0.13 p_g + 14 = (0.13)(30) + 14 = 17.9\,pcf < 30\,pcf$ ✓

$h_b = \frac{p_s}{\gamma} = \frac{24}{17.9} = 1.34'$

$h_{PH\,roof} = 13'$

$h_c = 13' - 1.34' = 11.66'$

$\frac{h_c}{h_b} = \frac{11.66}{1.34} = 8.7 > .2$ Drift can't be ignored

LW Drift:

$l_u = 15'$

$p_g = 30$

From fig 7-9 $h_d = 1.5'$ ← controls

WW Drift

$l_u = 15'$

$p_g = 30$ lb/ft²

WW drift will not control

Daniel Bodde	Tech 1	Snow load	2

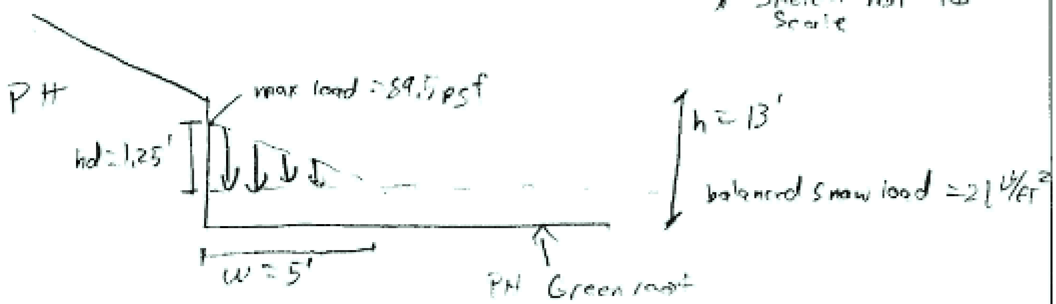
$hd = 1.25' < hc = 11.66'$

then $w = 4hd = (4)(1.23) = 5' < 8hc$ ✓

available width for drift $= 23'$

$Pd = (5')(17.9\ pcf) = 89.5\ psf$

* Sketch not to Scale



Appendix B: Wind Load Calculations

Daniel Bodde | Tech 1 | Wind Calc

ASCE 7-05

Building: Biobehavioral Health Building.

Location: University Park

6.5.4 Basic Wind Speed, V

For University Park $V = 90$ mph (See fig 6-1)

6.5.4.4 Wind Directionality Factor, Kd

$K_d = 0.85$

6.5.5 Importance Factor, I

Building category – II ⇒ $I = 1.00$ (See table 6-1)

6.5.6 Exposure: B

6.5.7 Topographic Effects

$K_{zt} = 1.0$ for homogeneous topography

6.5.8.1 Gust Effect Factor – Rigid Structures, G

use $G = 0.85$

6.5.10 Velocity Pressure, q_z

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

height (ft)	K_z (B, Case 2)	q_z (psf)
0–15	0.57	10.04
20	0.62	10.93
25	0.66	11.63
30	0.70	12.34
40	0.76	13.40
50	0.81	14.28
60	0.85	14.98
63	0.86	15.16
mean roof ht → 67	0.88	15.51

$$\frac{.89 - .85}{70 - 60} = \frac{K_{z67} - .85}{67 - 60} \Rightarrow K_{z67} = 0.88$$

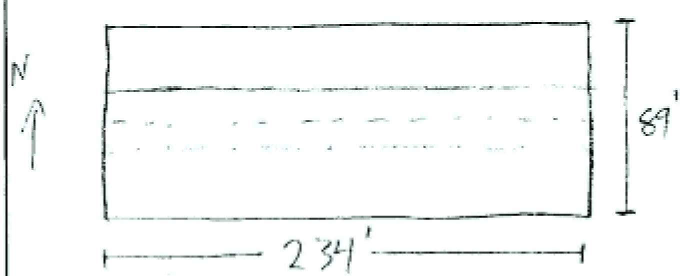
Daniel Bodde	Tech 1	Wind Calc	2

6.5.11.1 Internal Pressure Coefficient, GC_{pi}

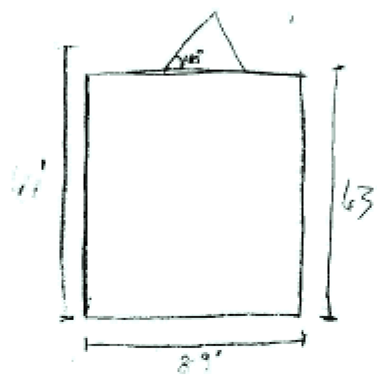
$GC_{pi} = \pm 0.18$ for enclosed buildings (Fig 6-5)

6.5.12.2.1 Design wind pressures for the MWFRS

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



plan



elevation

2: 13.5'
3: 27.5'
4: 41.5'
PH: 57'
63'
mean roof ht: 67'

Find External Pressure Coeff., C_p (Fig 6-6)

N-S Wall

Windward Wall $C_p = 0.8$

Leeward Wall:

$L/B = 89/234 = .38$

$C_p = -0.5$

E-W Wall

WW Wall $C_p = 0.8$

LW Wall:

$L/B = \frac{234}{89} = 2.63$

Interpolate to find C_p

$C_p = -0.27$

Daniel Bodde	Tech 1	Wind Calc	3

<u>Roof</u> (N-S only)

* Since the gabled roof spans E-W direction it will only be resisting wind forces in the N-S direction

$\theta = 40° \geq 10°$

$h/L = \frac{67}{89} = .75$

Interpolate to find Cp

WW $C_p = -.1$ & $.3$

LW $C_p: -0.6$

MWFRS Pressures (N-S)

ht	qz (psf)	WW p (psf)
0-15	10.04	9.62
20	10.93	10.22
25	11.63	10.70
30	12.34	11.18
40	13.40	11.90
50	14.28	12.50
60	14.98	12.98
63	15.14	13.10
roof 67	15.51	6.75

<u>LW</u>

$P_{wall} = -9.23$ psf

$P_{roof} = -10.7$ psf

MWFRS Pressures (E-W)

ht	qz (psf)	WW p (psf)
0-15	10.04	9.54
20	10.93	10.14
25	11.63	10.63
30	12.34	11.12
40	13.40	11.84
50	14.28	12.44
60	14.98	12.92
63	15.14	13.04

<u>LW</u>

$P = -6.21$ psf

Daniel Bodde	Tech 1	Wind Calc	4

Wind Force at each story (N-S)

2nd

$$\left[\left[(9.62\text{ psf})(6.75') + (9.62\text{ psf})(1.5') + (10.22\text{ psf})(5') + (10.7\text{ psf})(.5')\right](234') + (9.23\text{ psf})(13.75')(234')\right] / 1000 = \boxed{61.48\text{ K}}$$

3rd

$$\left[(10.7)(4.5') + (11.18)(3') + (11.9)(4.5') + (9.23)(14')\right]\frac{234'}{1000} = \boxed{67.12\text{ K}}$$

4th

$$\left[(11.9)(5.5) + (12.5)(9.25) + (9.23)(14.75)\right]\frac{234'}{1000} = \boxed{74.23\text{ K}}$$

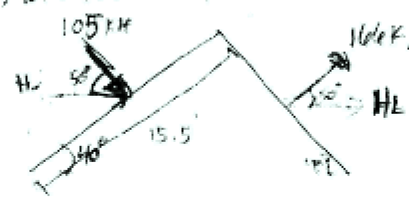
PH

$$\left[(12.5)(.75') + (12.98)(10') + (9.23)(10.75')\right]\frac{234'}{1000} = \boxed{55.79\text{ K}}$$

Bottom of roof

$$\left[(13.1)(3) + (9.23)(3)\right]\frac{234}{1000} = \boxed{15.68\text{ K}}$$

Horiz Force on gabled roof



$H_w = 105\cos(50) = 67.5$ klf

$H_L = 166\cos(50) = 107$ klf

$$(67.5 + 107)\frac{234}{1000} = \boxed{40.83\text{ K}}$$

Daniel Bodde | Tech 1 | Wind calc | 5

Wind Force at each Story (E-W)

2nd

$$\left[(9.56)(8.25') + (10.16)(5') + (10.63)(.5') + (6.21)(13.75')\right]\frac{89}{1000}$$

$$= \boxed{19.60K}$$

3rd

$$\left[(10.63)(4.5') + (11.12)(5') + (11.84)(4.5') + (6.21)(14')\right]\frac{89}{1000}$$

$$= \boxed{21.69K}$$

4th

$$\left[(11.84)(5.5') + (12.44)(9.25') + (6.21)([illegible])\right]\frac{89}{1000}$$

$$= \boxed{24.19K}$$

PH

$$\left[(12.44)(.75') + (12.92)(10') + (6.21)(14.75')\right]\frac{89}{1000}$$

$$= \boxed{20.42K}$$

Bot of roof

$$\left[(13.09)(3') + (6.21)(3')\right]\cdot\frac{89}{1000} = \boxed{5.14K}$$

Base Shear

N-S

$$V_{N\text{-}S} = 61.48 + 67.12 + 74.23 + 55.79 + 15.68 + 40.83$$

$$= \boxed{315.13K \quad \text{Controls}}$$

E-W

$$V_{E\text{-}W} = 19.60 + 21.69 + 24.19 + 20.48 + 5.14 = 91.1K$$

Daniel Bodde	Tech 1	Wind Calc	6

Overturning Moment

N-S

$$M_{N-S} = (61.48K)(13.5') + (67.12K)(22.5) + (74.23)(41.5) + (55.79)(57') + (15.48)(63) + (40.83)(67)$$
$$= \boxed{12659.8 \text{ K-ft} \quad \text{Controls}}$$

E-W

$$M_{E-W} = (19.6)(13.5) + (21.69)(22.5) + (24.19)(41.5) + (20.48)(57) + (3.14)(63)$$
$$= 3356.1 \text{ K-ft}$$

Appendix C: Seismic Calculations

Daniel Bodde | Tech I | Seismic Load | 1

Location: University Park, PA

Site Soil Classification:

Site Class C - Very Dense Soil & Soft Rock

Occupancy Category: III

$S_S = 0.147$ Fig. 22-1

$S_1 = 0.049$ Fig. 22-2

table 11.4-1

$S_{DS} = \frac{2}{3} F_a S_S = \frac{2}{3}(1.2)(0.147) = \boxed{0.1176}$

table 11.4-2

$S_{D1} = \frac{2}{3} F_v S_1 = \frac{2}{3}(1.7)(0.049) = \boxed{0.056}$

Seismic Design Category: A

(According to tables 11.6-1 & 11.6-2)

* see excel spread sheet for total weight

$V = C_s W$

$T = C_t h_n^x = (0.02)(67)^{0.75} = 0.47$ sec

$h_n = 73'$
$x = 0.75$
$C_t = 0.02$
} table 12.8-2

$T_L = 6$ sec $\qquad T < T_L$

$C_s = \frac{S_{D1}}{(R/I)} = \frac{(0.056)}{(3/1.25)} = .049$

does not match designer's C_s value of 0.01. Can't find mistake ∴ proceeding with $C_s = 0.01$ per designer

$V = (0.01)(8,351,893 \text{ lb}) = 83.5 \text{ K} \approx 84 \text{ K}$

Daniel Bodde	Tech 1	Seismic load	2

Vertical Distribution of Seismic Forces

$F_x = C_{vx} V$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad \text{where } k = 1$$

See spread sheet for C_{vx} values

$F_{re} = (0.03)(84) = 2.52$ K

$F_{PH} = (0.33)(84) = 27.72$ K

$F_{lvl4} = (0.32)(84) = 26.88$ K

$F_{lvl3} = (0.21)(84) = 17.64$ K

$F_{lvl2} = (0.10)(84) = 8.4$ K

Lvl 2	Area	DL	Weight
Slab	16600	46	763600
superimposed	16600	5	83000
Steel	16600	5	83000
Façade	8663	45	389812.5
CMU	8663	83	719029
Int Brick	2590	40	103600
Stone Floor	1700	20	34000
Total			2,176,042

Lvl 3	Area	DL	Weight
Slab	16600	46	763600
superimposed	16600	5	83000
Steel	16600	5	83000
Façade	8820	45	396900
CMU	8820	83	732060
Int Brick	1400	40	56000
Stone Floor	1700	20	34000
			2,148,560

Lvl 4	Area	DL	Weight
Slab	16600	46	763600
superimposed	16600	5	83000
Steel	16600	5	83000
Façade	9293	45	418162.5
CMU	9293	83	771319
Int Brick	1500	40	60000
Stone Floor	1700	20	34000
			2,213,082

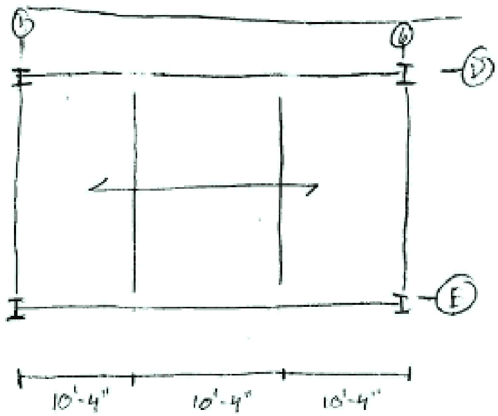
PH	Area	DL	Weight
Slab	6000	46	276000
Roof Deck	4700	3.3	15510
superimposed	10700	5	53500
Steel	10700	5	53500
Façade	9000	45	405000
CMU	9000	83	747000
Green Roof	4700	25	117500
			1,668,010
Roof	Area	DL	Weight
Slate	7310	10	73100
steel	7310	5	36550
superimposed	7310	5	36550
			146,200
Bld weight (lbs)			8,351,893

Level	weight, w	height, h	k	$w_i h_i^k$	C_{vx}
Lvl 2	2,176,042	13.5	1.0	29,376,560	0.10
Lvl 3	2,148,560	27.5	1.0	59,085,400	0.21
Lvl 4	2,213,082	41.5	1.0	91,842,882	0.32
PH	1,668,010	57	1.0	95,076,570	0.33
Roof	146,200	67	1.0	9,795,400	0.03
			$\Sigma w_i h_i^k$	285,176,813	

Appendix D: Gravity Load Spot Checks

Daniel Bodde | Tech 1 | Spot check

Steel Deck Spot check



LW Concrete

Slab: 3 1/4" topping

3 Spans (10'-4" span)

Unshored

3" 18 GA Composite deck

Loads:

LL = 100 psf

SDL = 5 psf

105 psf

2008 Vulcraft

3VLI18

SDI Max Unshored Clr Span

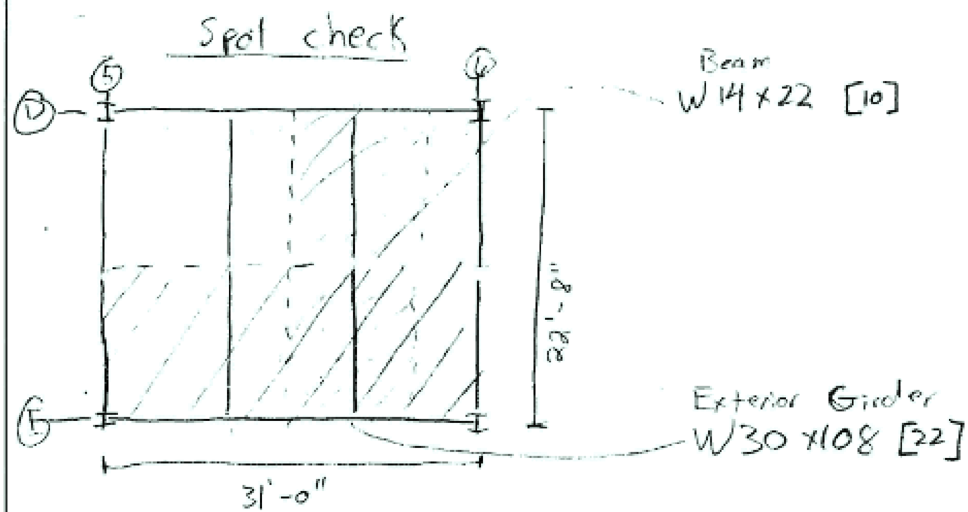
3 span = 15' > 10'-4" OK

Superimposed LL at 10'-6" < 10'-4" clr span

= 218 psf > 105 psf OK

Daniel Bodde | Tech 1 | Spot check

Spot check



Load Combonations: 1.2D + 1.6L

Dead loads

Slab on Deck = 46 psf

SDL = 5 psf

Brick & CMU = 45 + 83 = 128 psf

Live loads

Corridor : 100 psf & 2.0K concentrated load

Beam check

$A_T = 10.33' \times 22.67' = 234\ ft^2$

$K_{LL} = 2$

$K_{LL}A_T = 2(234) = 468\ ft^2 > 400\ ft^2$ ✓

$LL = L_o\left[0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right] = 100\left[0.25 + \frac{15}{\sqrt{468}}\right]$

$= 94\ psf$

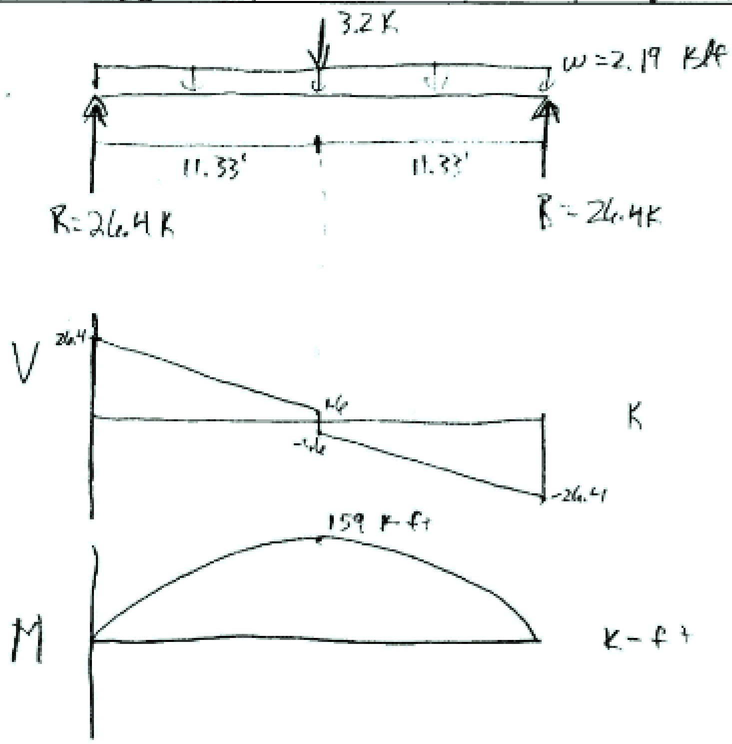
$w_u = 1.2D + 1.6L$

$= 1.2\left[(46+5)(10.33')\right] + 1.6\left[(94)(10.33')\right]$

$= 2.19\ Klf$

$P_u = 1.6 \times 2 = 3.2\ K$

Daniel Bodde | Tech 1 | Spot check



$M_u = 159$ K-ft

Daniel Bodde	Tech 1	Spot check

$$b_{eff} = \min \begin{cases} \frac{(22.67)(12)}{8} = 34'' \times 2 = 68'' \\ \left(\frac{1}{2}\right)(10.33)(12) = 62 \end{cases}$$

Q_n (From Table 3-21)

- Deck is ⊥ to bm
- assume studs in weak position
- one stud per rib
- 3/4" dia studs
- LW conc w/ $f'c = 4$ ksi

$Q_n = 17.2$ K

$$Q_n = \min \begin{cases} 17.2\text{ K} \\ R_g R_p (28.7) = (1.0)(0.6)(28.7) = 17.2\text{ K} \end{cases} \quad \therefore \underline{Q_n = 17.2\text{ K}}$$

Say $a = 1.5$ then $Y2 = 6.25'' - \frac{1.5}{2} = 5.5''$

Try W12x19 $\phi M_n = 164$ k-ft
$\Sigma Q_n = 104$ K ← most economical

$\frac{104}{17.2} = 6.05$ round to $7 \times 2 = 14$ studs

$19\#/\text{ft} \times 22.67' + 10\#/\text{stud} \times 14 = \underline{571 \text{ lb}}$

Try W12x22 $\phi M_n = 171$ k-ft
$\Sigma Q_n = 81$ K

$\frac{81}{17.2} = 4.7$ round to $5 \times 2 = 10$ studs

$22 \times 22.67 + 10 \times 10 = 598$ lb

Try W14x22 $\phi M_n = 188$ k-ft
$\Sigma Q_n = 81$ K

$\frac{81}{17.2} = 4.7$ round to $5 \times 2 = 10$ studs

$22 \times 22.67 + 10 \times 10 = 598$ lb

Daniel Bodde	Tech 1	Spot check

Check a:

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{104K}{(0.85)(4ksi)(68'')} = .45'' < 1.5'' \text{ OK}$$

Check unshored strength

W 12x19 $\quad \phi M_p = 92.6$ K-ft

$w_u = 1.4(46)(10.33) + 1.4(19) = .692$ Klf

$w_u = 1.2[(46)(10.33) + 19] + 1.6(20)(10.33) = \underline{.924 \text{ Klf}}$

$$M_u = \frac{(.924 \text{ Klf})(22.67)^2}{8} = 59.4 \text{ K-ft} < 92.6 \checkmark$$

Check wet conc deflection

$W_{wc} = (46)(10.33) + 19 = .494$ K/f $\qquad I_x = 130 in^4$

$$\Delta_{wc} = \frac{(5)(.494 \text{ Klf})(22.67)^4(1728)}{384(29000)(130)} = .78''$$

$$\max \Delta_{wc} = \frac{l}{240} = \frac{(22.67)(12)}{240} = 1.13'' > .78'' \checkmark$$

Check LL deflection

$W_{LL} = (94 psf)(10.33') = .971$ K/f

$I_{LB} = 334 in^4$ @ $Y2 = 5.5''$ & $Y1 = 1.68''$

$$\Delta_{LL} = \frac{(5)(.971)(22.67)^4(1728)}{384(334)(29000)} = .60''$$

$$\max \Delta_{LL} = \frac{L}{360} = \frac{22.67 \times 12}{360} = .76'' > .6'' \checkmark$$

W 12x19 [14] works

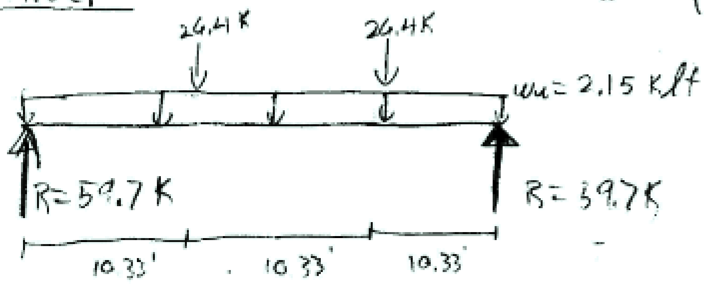
Designer was more conservative with the

W 14x22 [10]

Daniel Bodde | Tech 3 | Spot check

Girder

$w_u = 1.2(128\text{ psf} \times 14') = 2.15\text{ k/lf}$



W 30 x 108 [22]

$$M_u = \frac{wl^2}{8} + Px = \frac{(2.15\text{ k/f})(31')^2}{8} + (24.4\text{k})(10.33')$$

$= 531$ k-ft

$$b_{eff} = \min\begin{cases} \frac{31' \times 12}{8} = 46.5 \\ 10'' \end{cases} + \min\begin{cases} \frac{31' \times 12}{8} = 46.5 \\ \frac{22.67 \times 12}{2} = 136 \end{cases}$$

$= 10 + 46.5 = 56.5''$

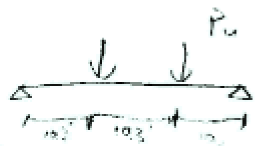
Same stud size & location as beam

Say $a = 2.5$ then $Y2 = 6.25 - \frac{2.5}{2} = 5''$

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{394}{(.85)(4)(56.5)} = 2.06'' < 2.5'' \quad \text{OK}$$

$\phi M_n = 1780$ k-ft > 531 k-ft OK

Check unshored strength



$P_u = [1.2(527 + 108) + 1.6(200)](11.33')$

$= 12.24$ K

$M_u = 12.26 \times 10 = 122$ k-ft $< \phi M_p = 1300$ k-ft ✓

Daniel Bodde	Tech 1	Spot check

Check LL defl

$I_{LB} = 6980 \, in^4$

$$\Delta_{LL} = \frac{11.7(31)^3(1728)}{28(29000)(6980)} = .11''$$

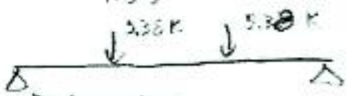
$$\max \Delta_{LL} = \frac{31 \times 12}{340} = 1.03'' > .11'' \quad OK$$

Check wet conc

$$\delta_{max} = \frac{l}{240} = \frac{31 \times 12}{240} = 1.55''$$

$I_x = 4470 \, in^4$

$P = 5.38\,K$

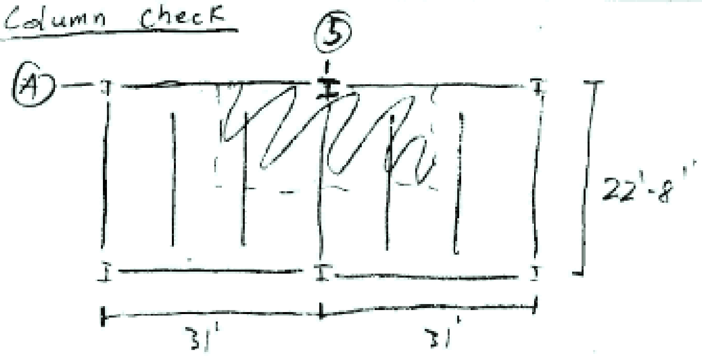


$$\Delta_{wc} = \frac{Pl^3}{28EI} = \frac{(5.38)(31)^3(1728)}{(28)(29000)(4470)} = .07'' < 1.55'' \quad \underline{OK}$$

W30 x108 works

Daniel Bodde	Tech 1	Spot check	10

Column check



DL
slab/deck = 46 psf
roof deck = 3.3 psf
Green roof = 25 psf
SDL = 5 psf
Facade & CMU = 128 psf

LL
office = 80 psf
roof = 30 psf > Snow load

$A_T = 31' \times 22.67' = 703 \text{ ft}^2 > 400 \text{ ft}^2$ LL Reduction allowed

$K = 4$

LL reduction (roof)

$$LL_{roof} = 30 \left| \begin{matrix} .5 \\ 0.25 + \frac{15}{\sqrt{(703)(4)}} \end{matrix} \right._{max} = .53$$

$LL_{roof} = 16$ psf

LL reduction (floors)

$$LL = 80 \left| \begin{matrix} .4 \\ .25 + \frac{15}{\sqrt{(703)(4)(3)}} \end{matrix} \right._{max} = .41$$

$LL = 33$ psf

See exel spreadsheet for final calcs

floor	trib area (ft^2)	Façade Area (ft^2)	DL (psf)	Façade DL (psf)	LL (psf)	LL Reduced	Pu_{story} (k)	ΣPu (k)
2	703	434	51	128	80	33	114	252
3	703	434	51	128	80	33	114	138
4(roof)	703	0	28	128	30	16	24	24

floor	Column	Unbraced Length (ft)	ϕPn	Adequate Strength?
2	W12x106	14	1130	Yes
3	W12x106	14	1130	Yes
4(roof)	W12x106	14	1130	Yes