Biobehavioral Health Building

University Park, PA

Tech 3 Report: Lateral System Analysis and Confirmation Design

2012-2013 AE Senior Thesis

Rendering provided by BCJ

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Executive Summary

The purpose of Technical Report 3 is to do a thorough analysis of the BBH Building's lateral system to confirm that the design is suitable. With the assistance of the structural modeling program ETABS this purpose was achieved. ETABS was used to determine critical values such as deflections, story shears, story drifts, internal member forces, and support reactions based on lateral loads determined from ASCE7-05. Though not shown in this report, the ETABS model was able to show the movement of the building from the applied loads through animations.

Values such as the deflections were used to calculate the relative stiffness's of each frame. This aided in the understanding of how the lateral loads will be distributed when acting on the BBH Building. Because the BBH Building is fairly regular in its layout on each level, similar action will be observed on every level. Understanding this results in the luxury of only having to analyze one level to get a sense of how the whole lateral system will act in its entirety. . As expected the eccentric braced frame, which produced the larges stiffness, took most of the direct shear from the lateral loads applied in the Y direction.

Spot checks were done on members at the base of the braced frame to verify if they could resist the loads applied to that frame. That specific frame was chosen for spot checks because it was determined to be the more critical component of the lateral system for the BBH Building due to its significant influence on the position of the center of rigidity.

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.

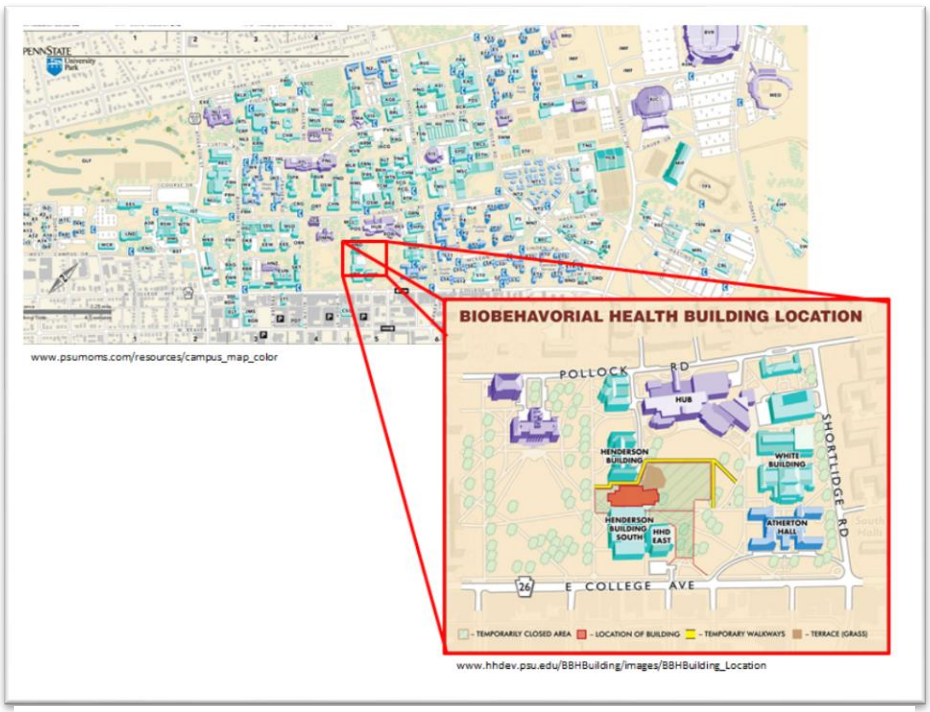


Figure 1: PSU Campus Map

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 northeast portion of the building the design is more modern to replicate HUB, which is a popular student hang out. 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. were 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. 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. Accordiong the the geotechnical engineer, “the bedrock must be free of clay seams or voids near the surface to provide a stable surface to place the foundations.” If bedrock is encountered before the required bearing elevations are met then over excavation is required and needed to be back filled with lean concrete. The bearing material must have a bearing capacity of 15 psf 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.

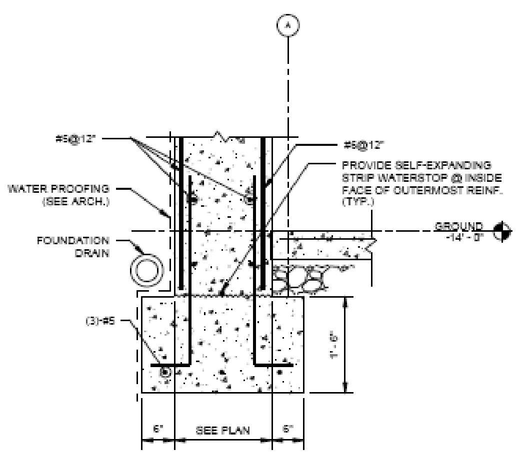


Figure 3: Typical Strip Footing

Floor/Framing System

The BBH Building floors are concrete slab on metal deck. The typical slab on deck consists 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 cuts 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.

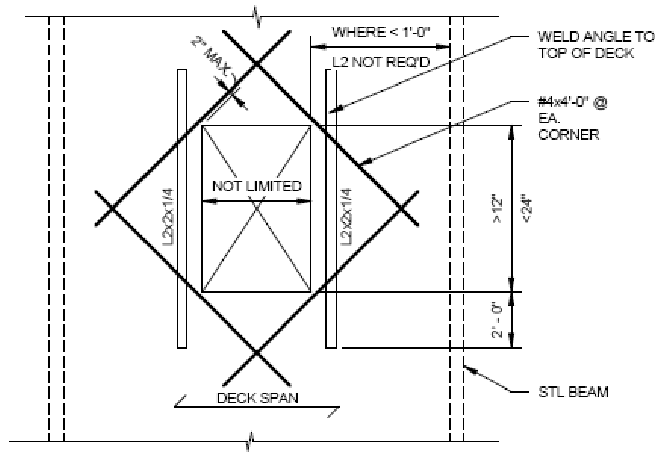


Figure 4: Openings in Slab on Steel Deck

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 Figure 6 for a typical floor plan.

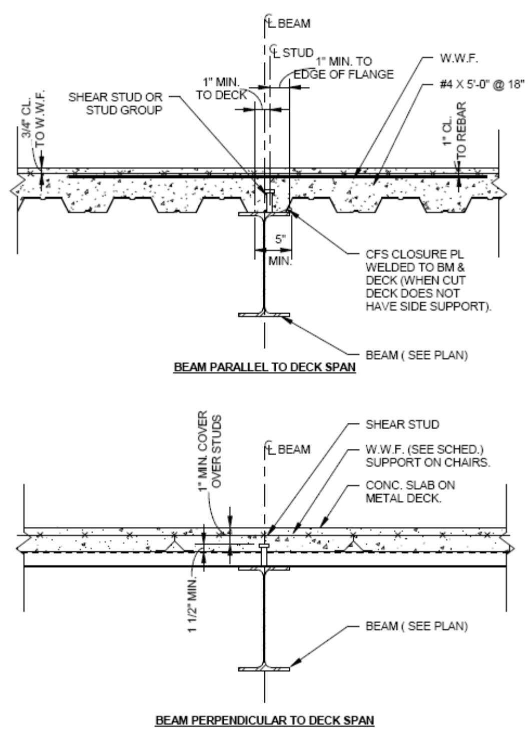


Figure 5: Typical Section Through Composite System

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.

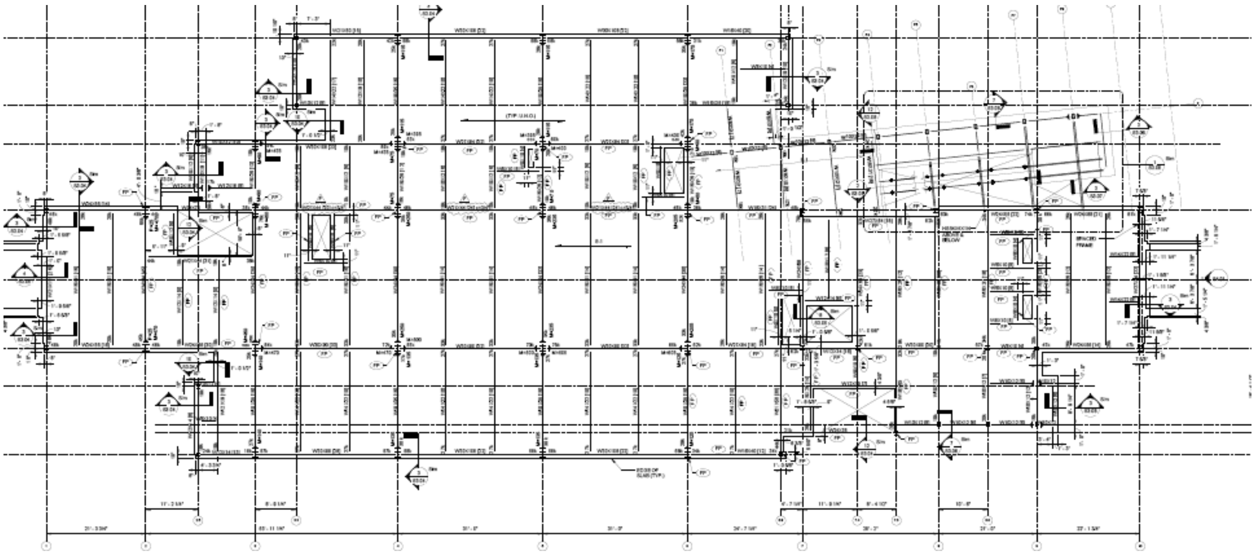


Figure 6: Typical Floor Framing Plan

Lateral System

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

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 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 shown below in Figure 8. Because the east wing of the BBH Building is exposed to the HUB lawn, it will experience higher wind loads. This could be the reason for using a duel lateral system consisting of both moment frames and eccentric braced frames (Figure 8).

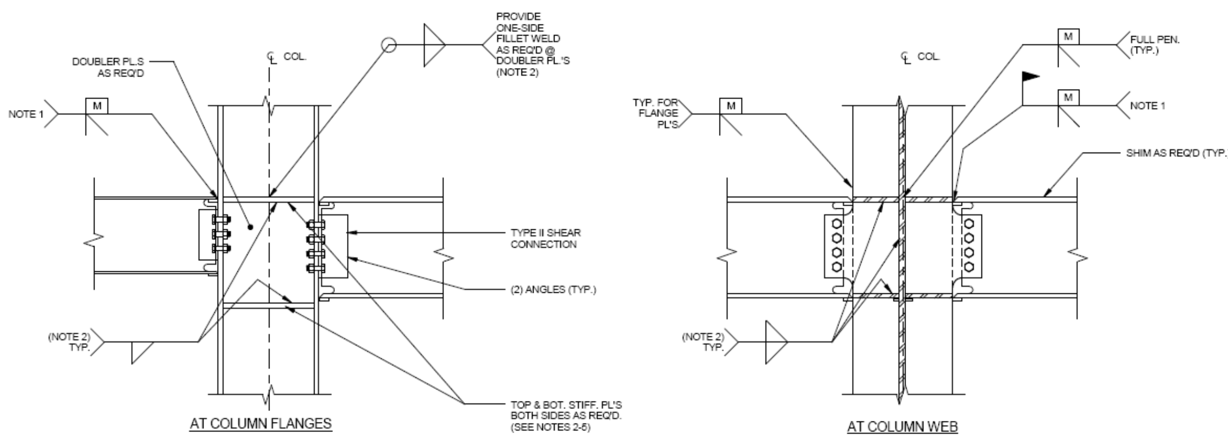


Figure 7: Typical Beam to Column Moment Connection

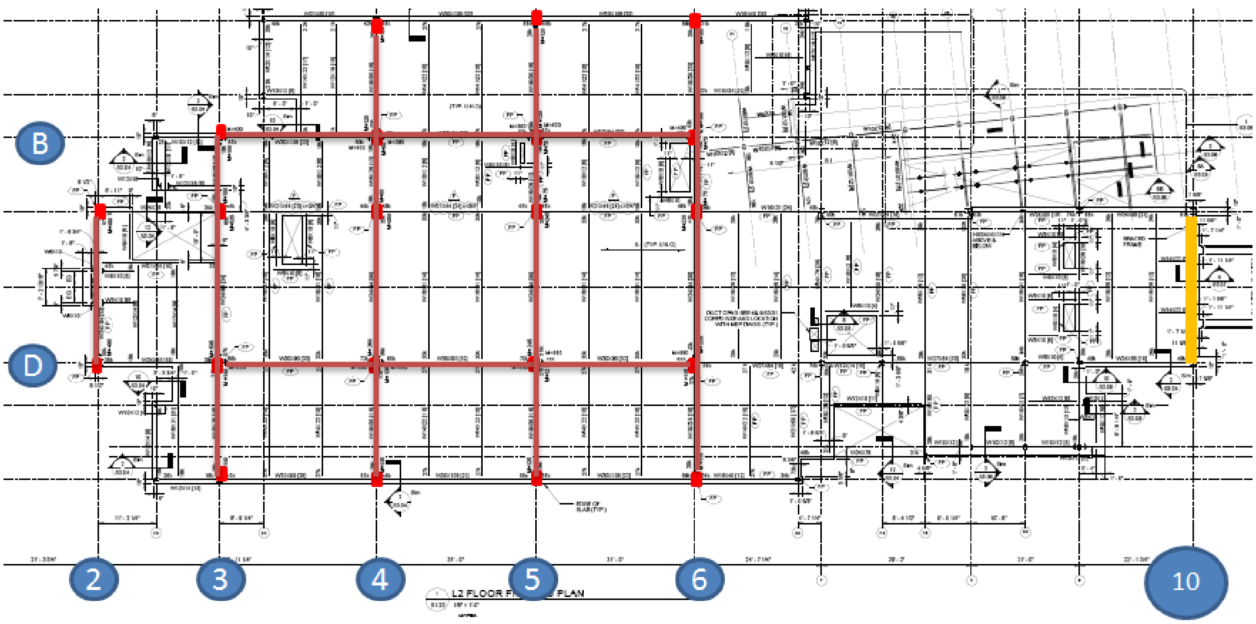


Figure 8: Location of Moment Connections (Red) and Braced Frame (Orange)

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.

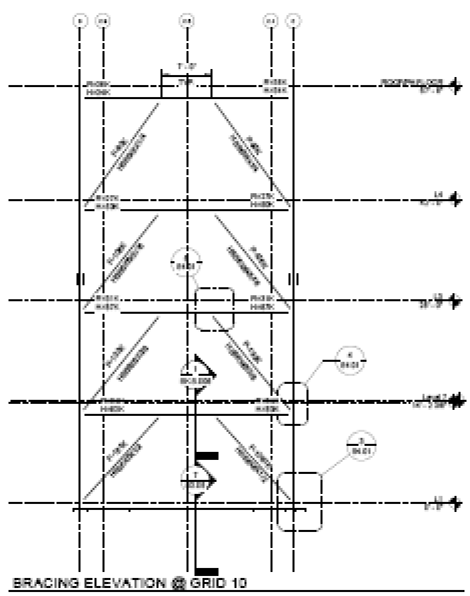


Figure 9: Eccentric Braced Frame

Design Codes

The BBH Building was designed using the following codes:

- IBC 2006 (as amended by Pennsylvania UCC administration)
- ASCE 7-05
- ACI 318-05
- 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 given by the designer.

Dead

Dead Loads * (psf)	
Slate roof assembly	32
Green roof assembly	60
Floor, typical	60
Floor, stone tile	85
Plaza (above auditorium)	212
* self-weight of steel framing members not included	

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 drift load was calculated 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 A). The gabled roof was neglected when calculating the wind load in the E-W direction due to the slenderness of it in that direction.

In summary, the base shear due to wind 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 A.

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

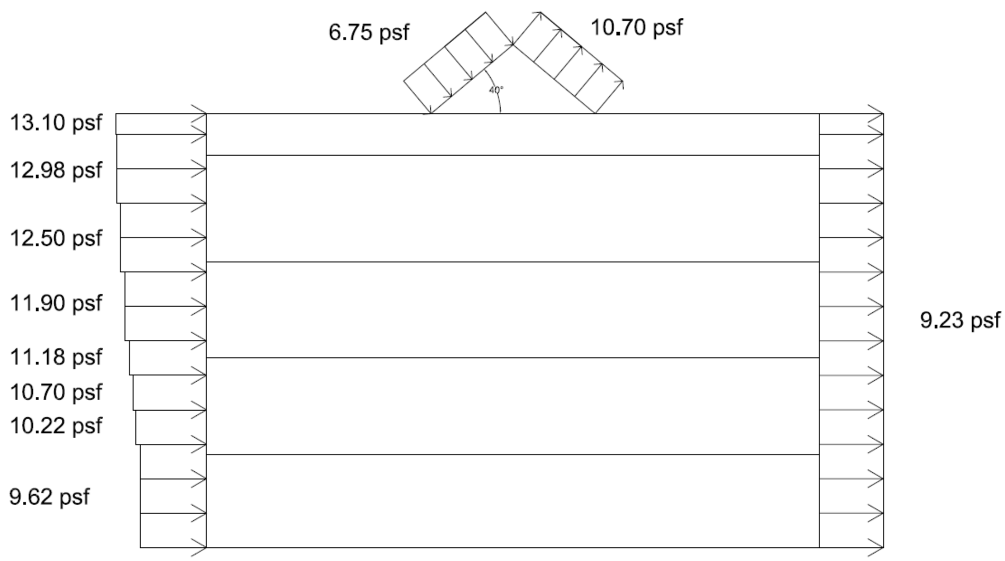


Figure 10: N-S Wind Pressure Diagram

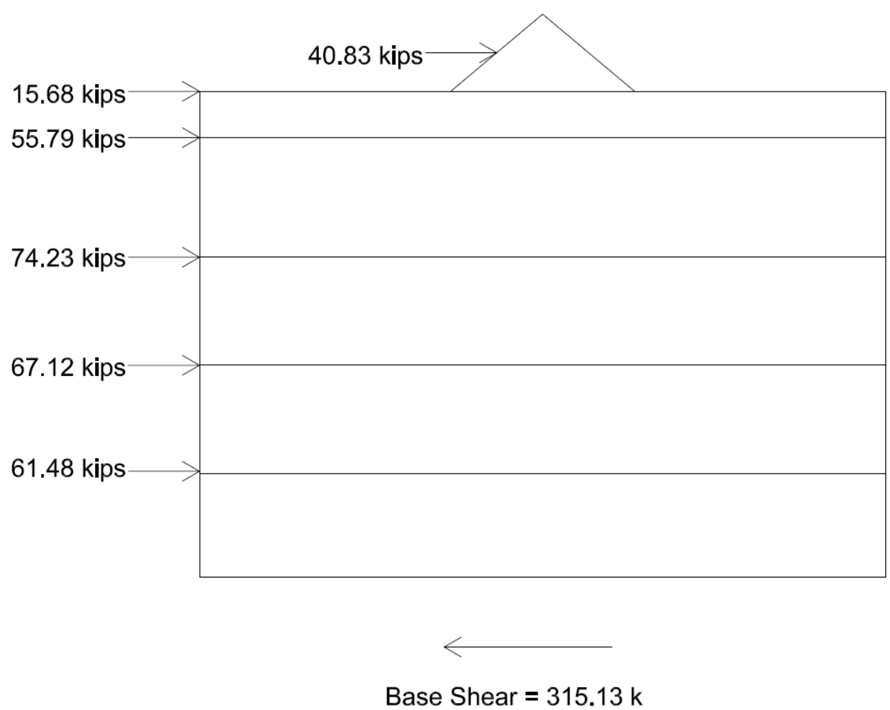


Figure 11: N-S Wind Story Force Diagram

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

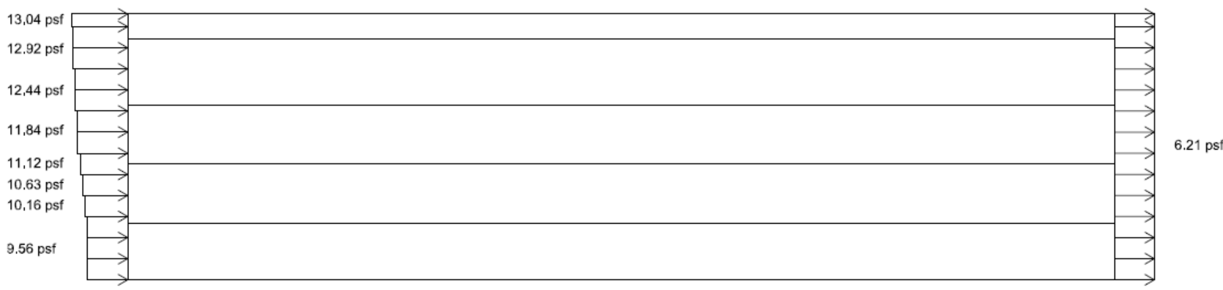


Figure 12: E-W Wind Pressure Diagram

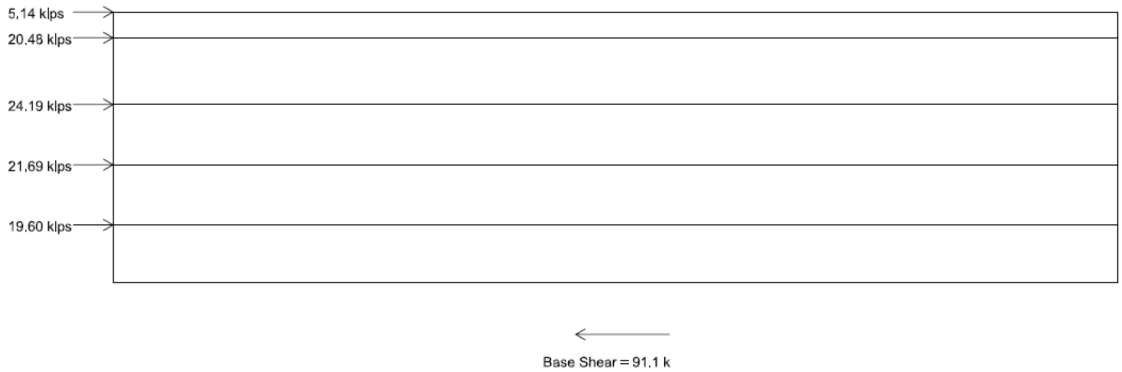


Figure 13: E-W Wind Story Force Diagram

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). Using the geotechnical testing reports it was determine by the geotechnical engineer that the soil would be classified as site class C – very dense soil and soft rock. According to the IBC a Cs value of .01 is allowed for buildings with a seismic design category A. See Appendix C for hand calculations. Vertical distribution of the seismic forces is shown below in Figure 14

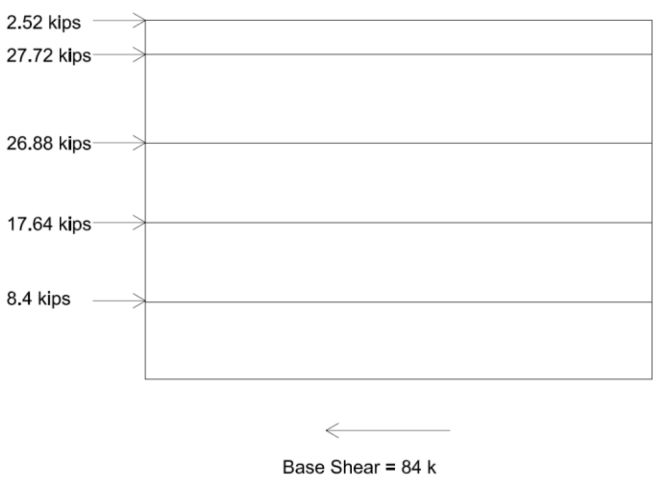


Figure 14: Vertical Distribution of seismic forces

Analysis

ETABS Model

The lateral system for the BBH Building was modeled using ETABS. The model was created to aid in the analysis of the lateral system. Since the lateral system for the BBH building is composed of steel moment and braced frames it was fairly simple to model. All of the frame members, such as columns, beams, and braces, were modeled using line elements. These line elements were each defined with the correct frame sections and material properties as specified in the drawings. It was assumed that all base connections would be fixed. When drawing in the moment frames, ETABS automatically assumes moment connections between members. Therefore no further steps needed to be taken in modeling the connections between the beams and columns in the moment frames. In order properly model the eccentric braced frame, located on the east side of the building, all the connections between the beams, columns, and braces needed to have their moments to be released. This would insure that only axial forces would act in the braces. Finally, rigid diaphragms were inserted at each floor and were given their respective weights. These diaphragms act as the concrete slab and provide a "link" between all the moment and braced frames at each level so they all deflect the same distance. It is important note that the penthouse walls and roof were not modeled because they were not part of the BBH's lateral system. Snap shots of the lateral system modeled in ETABS can be seen below.(Figures 15 & 16)

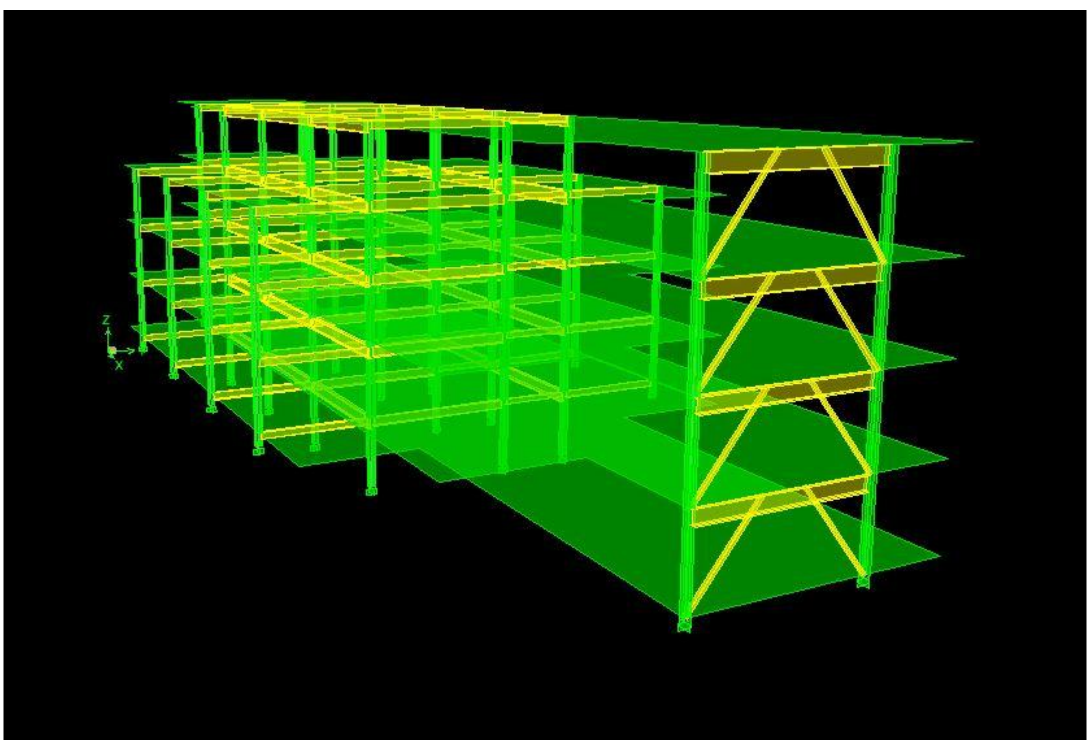


Figure 15: ETABS Model (view from east)

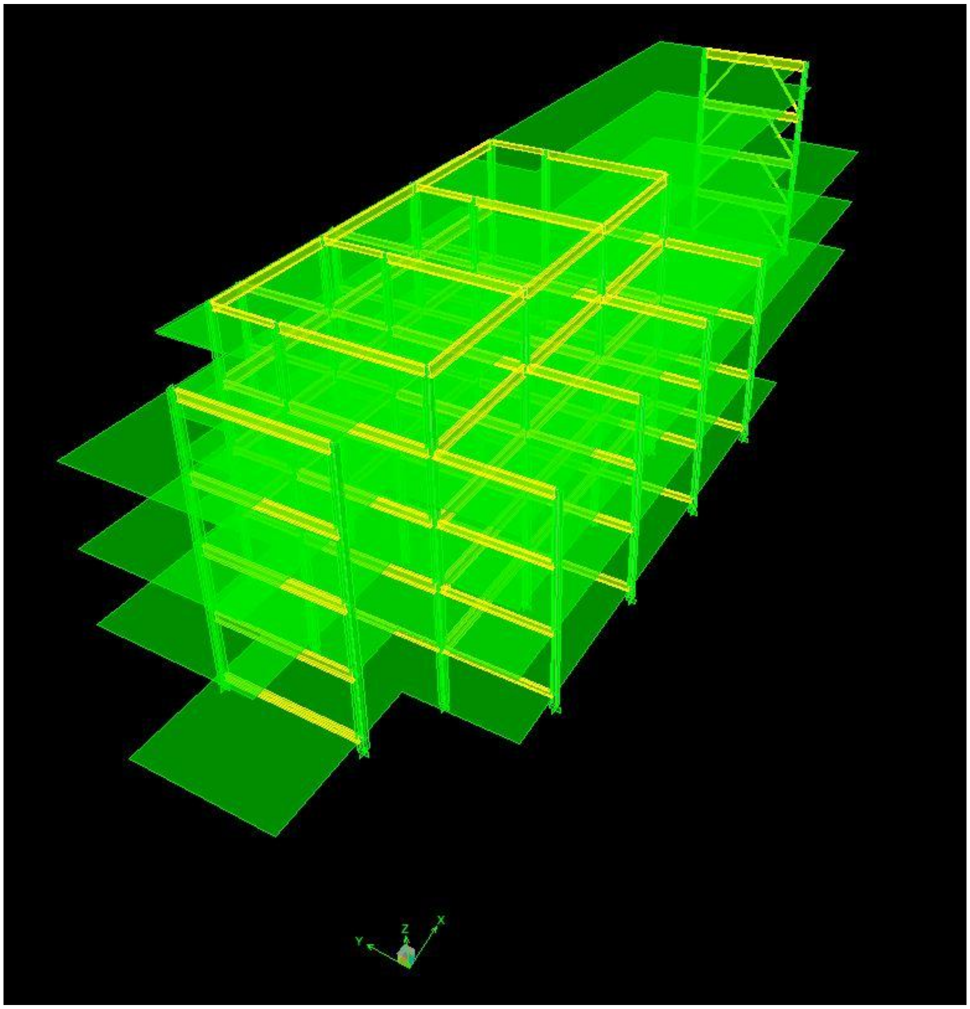


Figure 16: ETABS Model (view from west)

Relative Stiffness

In order to better understand how the lateral system of the BBH Building works, the relative stiffness of each frame needs to be determined. Stiffness is equal to a force (P) divided by the deflection (δ) caused by that force (K=P/δ). Using ETABS, a unit load of 1 kip was applied to the 5th story of the BBH Building. After running the analysis in ETABS, deflection measurements were taken at the 5th story of each frame. With the stiffness's calculated it was observed that the eccentric braced frame was the stiffest, which is to be expected. To find the relative stiffness's, each frames calculated stiffness was divided by the largest stiffness value, which in this case was the value calculated for the eccentric braced frame. These values give us a better sense of how the lateral forces get distributed to the frames at each level. See Figure 17 for the lateral frame layout and notation.

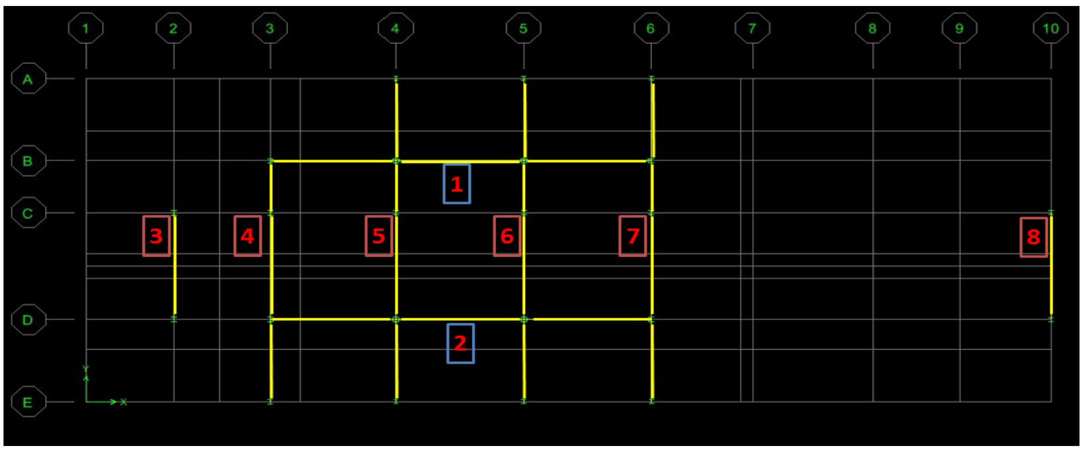


Figure 17: Lateral Frame Layout & Notation

Story 5						
Direction	Frame		P	Defl. (δ)	Frame Stiffness, K	Frame Relative Stiffness, K_{rel}
	#	Type	(kip)	(in)	(k/in)	(k/in)
X	1	Moment Frame	1	0.019464	51.38	0.1498
	2	Moment Frame	1	0.019523	51.22	0.1494
Y	3	Moment Frame	1	0.007022	142.41	0.4153
	4	Moment Frame	1	0.006570	152.21	0.4438
	5	Moment Frame	1	0.005981	167.20	0.4875
	6	Moment Frame	1	0.005383	185.77	0.5417
	7	Moment Frame	1	0.004786	208.94	0.6093
	8	Braced Frame	1	0.002916	342.94	1.0000

With each story of the BBH Building being fairly similar it is safe to assume that the behavior at the lower levels is consistent with the 5th story. This assumption was quickly checked and verified with the use of ETABS. Because the BBH exudes this behavior, technical report 3 will only analyze and study the forces at the 5th story.

Center of Rigidity and Mass

Using ETABS, the center of rigidity and center of mass were calculated in the model when the analysis was run. These values were then verified by hand using the relative stiffness and building weight(See appendix D). Below is an AutoCAD sketch showing the locations of the CR and CM for the 5th story (Figure 18).

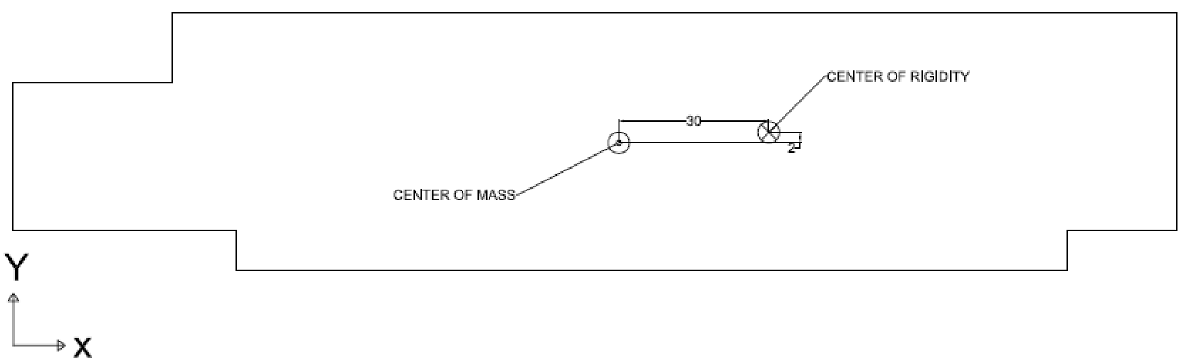


Figure 18: 5th Story CM & CR

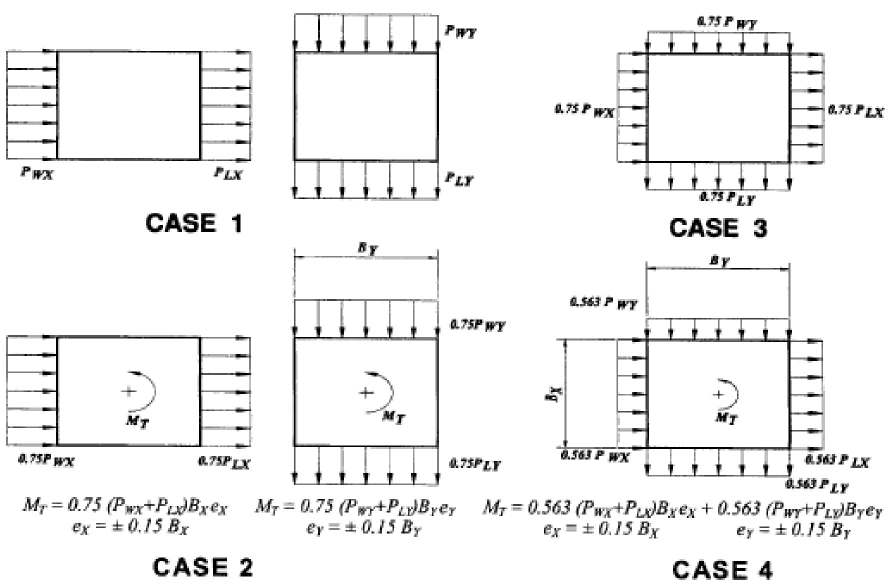
As you can see the CM and CR have an eccentricity in both the X and Y direction. Since forces act at the center of mass and rotation occurs about the center of rigidity, a torsional moment is now created from these eccentricities. These torsional moments will be calculated later in the report. See the table below for CM and CR values at each floor calculated by ETABS. (Note: Dimensions are in feet and are taken about the origin)

Story	XCM	YCM	XCR	YCR
STORY 5	121.866	40.774	152.043	42.892
STORY 4	118.548	40.857	149.644	43.078
STORY 3	118.52	40.855	147.683	42.47
STORY 2	122.692	40.986	136.431	41.499
STORY 1	113.61	43.118		

Wind Load Cases

Earlier in the report it was determined that wind forces would be the dominating force in both directions over the seismic forces. For this reason tech 3 will focus on wind forces only. In ASCE7-05 there are four wind load cases that need to be applied to the building in order to determine a worst case scenario for the design of the lateral system. Below are the four ASCE7-05 wind load cases.

Main Wind Force Resisting System–Method 2		All Heights
Figure 6-9	Design Wind Load Cases	



Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 6.5.12.2.1 and 6.5.12.2.3 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:

P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively.
P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.
$e\ (e_X, e_Y)$: Eccentricity for the x, y principal axis of the structure, respectively.
M_T: Torsional moment per unit height acting about a vertical axis of the building.

These load cases were applied to the 5th story of the lateral system of the BBH Building. Again, because every story is similar we can assume that the lower stories will behave in a similar fashion when their respective story load is applied to it. After completion of the calculations it was determined that Case 1 on average controls both in the N-S and E-W direction. Below are the results of each load case. These results show the distribution of the lateral load to each frame due to the direct shear and torsional shear. See appendix B for spreadsheet calculations performed to obtain these values. Be sure to notice that frame 8 takes the most direct shear force in every case. This confirms our observation that the eccentric braced frame (frame 8) is the stiffest component of the lateral system. It is only because of the torsional shear, in some cases, acting in the opposite direction that frame 8 does not always control when direct and torsional shear are summed together.

Case 1			
N-S Direction		E-W Direction	
Frame	Force (kips)	Frame	Force (kips)
1	0.52	1	12.81
2	-0.43	2	12.79
3	21.22	3	-0.12
4	21.13	4	-0.10
5	21.09	5	-0.08
6	21.00	6	-0.05
7	20.89	7	-0.02
8	20.23	8	0.18

Case 2 +e				**Case 2 -e**			
N-S Direction		E-W Direction		N-S Direction		E-W Direction	
Frame	Force (kips)	Frame	Force (kips)	Frame	Force (kips)	Frame	Force (kips)
1	-0.08	1	9.65	1	0.86	1	9.55
2	0.06	2	9.56	2	-0.71	2	9.58
3	8.82	3	0.49	3	23.01	3	-0.67
4	9.66	4	0.43	4	22.04	4	-0.59
5	10.92	5	0.34	5	20.72	5	-0.46
6	12.50	6	0.23	6	19.00	6	-0.31
7	14.47	7	0.08	7	16.86	7	-0.11
8	25.86	8	-0.74	8	4.48	8	1.01

Case 3	
Frame	Force (kips)
1	10.01
2	9.27
3	15.82
4	15.77
5	15.76
6	15.71
7	15.65
8	15.31

Case 4							
NS +e & EW +e		NS +e & EW -e		NS -e & EW -e		NS -e & EW +e	
Frame	Force (kips)	Frame	Force (kips)	Frame	Force (kips)	Frame	Force (kips)
1	7.19	1	7.13	1	7.83	1	7.89
2	7.23	2	7.28	2	6.69	2	6.64
3	6.99	3	6.11	3	16.76	3	17.64
4	7.57	4	6.81	4	16.10	4	16.87
5	8.46	5	7.85	5	15.20	5	15.81
6	9.56	6	9.15	6	14.03	6	14.43
7	10.93	7	10.78	7	12.57	7	12.72
8	18.85	8	20.18	8	4.12	8	2.80

Story Drift

In ASCE 7-05 the story drift limit was found to be H/400 where H is the story height. This was found in Chapter C, Appendix C. Keeping the story drift below H/400 is more for serviceability and will reduce any damage to the façade or nonstructural components. Unfactored loads were used in the analysis of the story drifts. The tables below show that each story has a story drift below the limit of H/400 in both the X and Y directions.

Story Drift - Y Direction					
Story	Story Height	Displacement	H/400	Story Drift	Pass?
	(ft)	(in)	(in)	(in)	Yes
PH	15	1.04	0.45	0.21	Yes
4	14	0.83	0.42	0.25	Yes
3	14	0.58	0.42	0.32	Yes
2	14	0.26	0.42	0.26	Yes

Story Drift - X Direction					
Story	Story Height	Displacement	H/400	Story Drift	Pass?
	(ft)	(in)	(in)	(in)	Yes
PH	15	1.07	0.45	0.18	Yes
4	14	0.89	0.42	0.26	Yes
3	14	0.63	0.42	0.35	Yes
2	14	0.28	0.42	0.28	Yes

Overturning Moment

The wind forces being applied to the BBH Building create a moment at the base of the structure making the building want to overturn hence the term "overturning moment." The controlling overturning moment in the BBH Building occurs about the plan East-West axis. This moment is calculated summing the product of the story shears with their corresponding moment arm. See the table below.

Overturning Moment			
Level	ht (ft)	Wind Force (K)	Moment (k-ft)
Roof	67	40.83	2736
Parapet	63	15.68	988
PH	57	55.79	3180
4	41.5	74.23	3081
3	27.5	67.12	1846
2	13.5	61.48	830
		Overturning Moment=	12,660 k-ft

This overturning moment is resisted by the buildings weight creating a moment in the opposite direction. This resisting moment can be estimated by taking 2/3 of the building weight times half of the buildings depth (dimension the moment is acting about). See the table below for resisting moment calculation. Fortunately in this case the resisting moment is enough to keep the building from overturning. If this were not the case then special consideration would need to be taken in the design of the foundation system (ex: increased reinforcement, wider spread footing base, increased anchor bolt strength, etc.).

Resisting Moment
M_{resist}=8,352k x 89'/2 x .67 = 249,015 k-ft

Spot Checks

Spot checks were done on the column and brace at the base of the eccentric braced frame. This frame is the more critical frame in the lateral system of the BBH Building. Because it is the only frame on the east portion of the building, a failure in this frame would cause the center of rigidly to drastically shift to the west and would cause the building to experience an amplified torsional force, which would most likely cause the significant damage to the BBH Building. See Appendix E for spot check calculations.

Conclusion

The analysis done in this tech report has confirmed that the BBH's lateral system is adequate to resist the applied wind loads determined from ASCE7-05. Spot checks were done on members at the base of the braced frame to verify if they could resist the loads applied to that frame. That specific frame was chosen for spot checks because it was determined to be the more critical component of the lateral system of the BBH Building due to its significant influence on the position of the center of rigidity.

In this report the use of ETABS assisted in the determination of critical values used to calculate the relative stiffness of the individual lateral frames. Such values consisted of deflections, story shears, story drifts, internal member forces, and support reactions. Though not shown in this report, the ETABS model was able to show the movement of the building from the applied loads through animations.

The main objective of understanding the load distribution/path was achieved through this report. Through the use of the calculated relative stiffness's it became clear which frames were given the task of carrying the majority of the lateral load. As expected the eccentric braced frame, which produced the larges stiffness, took most of the direct shear from the lateral loads applied in the Y direction.

Appendix A: 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

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

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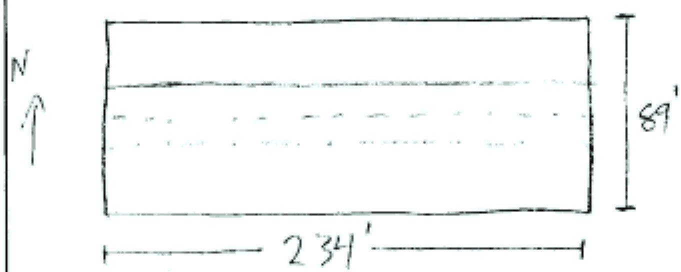
6.5.11.1 Internal Pressure Coefficient, GC_{pi}

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

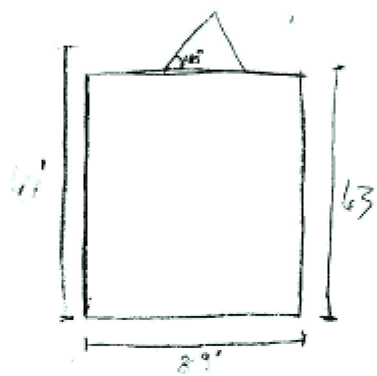
6.5.12.2.1 Design wind pressures for the MWFRS

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

2: 13.5'
3: 22.5'
4: 41.5'
PH: 57'
[illegible]: 63'
mech [illegible] ht: 67'



plan



elevation

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

$C_p = -0.27$

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Roof (N-S only)

$\theta = 40° \geq 10°$

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

Interpolate to find Cp

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

LW $C_p: -0.6$

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

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
65	15.16	13.10
roof 67	15.51	16.75

LW

$P_{wall} = -9.23$ 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	11.43	10.63
30	12.34	11.12
40	13.40	11.84
50	14.28	12.44
60	14.98	12.92
65	15.14	13.04

LW

$P = -6.21$ psf

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Wind Force at each story (N-S)

2nd

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

3rd

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

4th

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

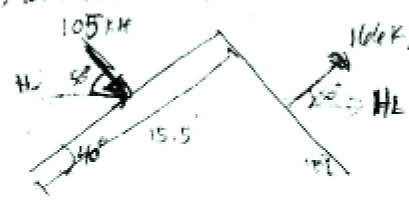
PH

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

Bottom of roof

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

Horiz Force On gabled roof



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

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

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

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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

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

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Overturning Moment

N-S

$M_{N-S} = (61.48K)(13.5') + (67.12K)(22.5) + (74.23)(41.5)$
$+ (55.79)(57') + (15.68)(63) + (40.83)(67)$
$= \boxed{12659.8 \text{ K-ft} \quad \text{Controls}}$

E-W

$M_{E-W} = (19.6)(13.5) + (21.69)(27.5) + (24.19)(41.5)$
$+ (20.48)(57) + (3.14)(63)$
$= 3356.1 \text{ K-ft}$

Appendix B: Wind Load Cases Excel Calculations

N-S	ft	E-W	ft			ft				
CR=	43	CR=	152		.15By=	36				CW(+)
CM=	41	CM=	122		.15Bx=	13				
CR-CP=	2.11	CR-CP=	30.29							

Case 1 NS									
F_{NS} (kip)	112.3			e_{NS} (ft)	-30				
F_{EW} (kip)	0			e_{EW} (ft)	0				
M_{NS} (k-ft)	-3369								
M_{EW} (k-ft)	0								
Frame	K (k/in)	$\sum K_{NS}$ (k/in)	$\sum K_{EW}$ (k/in)	Direct shear (kip)	d (ft)	Kd^2	∑Kd^2	Torsional Moment Shear (kip)	Total shear (kip)
1	0.1498	0.00	0.30	0.00	-24.00	86.29	23251.88	0.52	0.52
2	0.1494	0.00	0.30	0.00	20.00	59.74	23251.88	-0.43	-0.43
3	0.4153	3.50	0.00	13.33	-131.00	7126.39	23251.88	7.88	21.22
4	0.4438	3.50	0.00	14.25	-107.00	5081.47	23251.88	6.88	21.13
5	0.4875	3.50	0.00	15.65	-77.00	2890.65	23251.88	5.44	21.09
6	0.5417	3.50	0.00	17.39	-46.00	1146.25	23251.88	3.61	21.00
7	0.6093	3.50	0.00	19.56	-15.00	137.09	23251.88	1.32	20.89
8	1.0000	3.50	0.00	32.11	82.00	6724.00	23251.88	-11.88	20.23

Case 1 EW									
F_{NS} (kip)	0			e_{NS} (ft)	0				
F_{EW} (kip)	25.6			e_{EW} (ft)	2				
M_{NS} (k-ft)	0								
M_{EW} (k-ft)	51.2								
Frame	K (k/in)	$\sum K_{NS}$ (k/in)	$\sum K_{EW}$ (k/in)	Direct shear (kip)	d (ft)	Kd^2	∑Kd^2	Torsional Moment Shear (kip)	Total shear (kip)
1	0.1498	0.00	0.30	12.82	-24.00	86.29	23251.88	-0.01	12.81
2	0.1494	0.00	0.30	12.78	20.00	59.74	23251.88	0.01	12.79
3	0.4153	3.50	0.00	0.00	-131.00	7126.39	23251.88	-0.12	-0.12
4	0.4438	3.50	0.00	0.00	-107.00	5081.47	23251.88	-0.10	-0.10
5	0.4875	3.50	0.00	0.00	-77.00	2890.65	23251.88	-0.08	-0.08
6	0.5417	3.50	0.00	0.00	-46.00	1146.25	23251.88	-0.05	-0.05
7	0.6093	3.50	0.00	0.00	-15.00	137.09	23251.88	-0.02	-0.02
8	1.0000	3.50	0.00	0.00	82.00	6724.00	23251.88	0.18	0.18

Case 2 NS + .15By									
$.75F_{NS}$ (kip)	84.225			e_{NS} (ft)	6				
$.75F_{EW}$ (kip)	0			e_{EW} (ft)	0				
M_{NS} (k-ft)	505.35								
M_{EW} (k-ft)	0								
Frame	K (k/in)	$\sum K_{NS}$ (k/in)	$\sum K_{EW}$ (k/in)	Direct shear (kip)	d (ft)	Kd^2	∑Kd^2	Torsional Moment Shear (kip)	Total shear (kip)
1	0.1498	0.00	0.30	0.00	-24.00	86.29	23251.88	-0.08	-0.08
2	0.1494	0.00	0.30	0.00	20.00	59.74	23251.88	0.06	0.06
3	0.4153	3.50	0.00	10.00	-131.00	7126.39	23251.88	-1.18	8.82
4	0.4438	3.50	0.00	10.69	-107.00	5081.47	23251.88	-1.03	9.66
5	0.4875	3.50	0.00	11.74	-77.00	2890.65	23251.88	-0.82	10.92
6	0.5417	3.50	0.00	13.04	-46.00	1146.25	23251.88	-0.54	12.50
7	0.6093	3.50	0.00	14.67	-15.00	137.09	23251.88	-0.20	14.47
8	1.0000	3.50	0.00	24.08	82.00	6724.00	23251.88	1.78	25.86

Case 2 NS - .15By									
$.75F_{NS}$ (kip)	84.225			e_{NS} (ft)	-66				
$.75F_{EW}$ (kip)	0			e_{EW} (ft)	0				
M_{NS} (k-ft)	-5558.85								
M_{EW} (k-ft)	0								
Frame	K (k/in)	$\sum K_{NS}$ (k/in)	$\sum K_{EW}$ (k/in)	Direct shear (kip)	d (ft)	Kd^2	∑Kd^2	Torsional Moment Shear (kip)	Total shear (kip)
1	0.1498	0.00	0.30	0.00	-24.00	86.29	23251.88	0.86	0.86
2	0.1494	0.00	0.30	0.00	20.00	59.74	23251.88	-0.71	-0.71
3	0.4153	3.50	0.00	10.00	-131.00	7126.39	23251.88	13.01	23.01
4	0.4438	3.50	0.00	10.69	-107.00	5081.47	23251.88	11.35	22.04
5	0.4875	3.50	0.00	11.74	-77.00	2890.65	23251.88	8.97	20.72
6	0.5417	3.50	0.00	13.04	-46.00	1146.25	23251.88	5.96	19.00
7	0.6093	3.50	0.00	14.67	-15.00	137.09	23251.88	2.18	16.86
8	1.0000	3.50	0.00	24.08	82.00	6724.00	23251.88	-19.60	4.48

Case 2 EW-.15Bx									
$.75F_{NS}$ (kip)	0			e_{NS} (ft)	0				
$.75F_{EW}$ (kip)	19.2			e_{EW} (ft)	15				
M_{NS} (k-ft)	0								
M_{EW} (k-ft)	288								
Frame	K (k/in)	$\sum K_{NS}$ (k/in)	$\sum K_{EW}$ (k/in)	Direct shear (kip)	d (ft)	Kd^2	∑Kd^2	Torsional Moment Shear (kip)	Total shear (kip)
1	0.1498	0.00	0.30	9.61	-35.50	188.80	23299.17	-0.07	9.55
2	0.1494	0.00	0.30	9.59	-5.50	4.52	23299.17	-0.01	9.58
3	0.4153	3.50	0.00	0.00	-131.00	7126.39	23299.17	-0.67	-0.67
4	0.4438	3.50	0.00	0.00	-107.00	5081.47	23299.17	-0.59	-0.59
5	0.4875	3.50	0.00	0.00	-77.00	2890.65	23299.17	-0.46	-0.46
6	0.5417	3.50	0.00	0.00	-46.00	1146.25	23299.17	-0.31	-0.31
7	0.6093	3.50	0.00	0.00	-15.00	137.09	23299.17	-0.11	-0.11
8	1.0000	3.50	0.00	0.00	82.00	6724.00	23299.17	1.01	1.01

Case 2 EW+.15Bx									
$.75F_{NS}$ (kip)	0			e_{NS} (ft)	0				
$.75F_{EW}$ (kip)	19.2			e_{EW} (ft)	-11				
M_{NS} (k-ft)	0								
M_{EW} (k-ft)	-211.2								
Frame	K (k/in)	$\sum K_{NS}$ (k/in)	$\sum K_{EW}$ (k/in)	Direct shear (kip)	d (ft)	Kd^2	∑Kd^2	Torsional Moment Shear (kip)	Total shear (kip)
1	0.1498	0.00	0.30	9.61	-24.00	86.29	23251.88	0.03	9.65
2	0.1494	0.00	0.30	9.59	20.00	59.74	23251.88	-0.03	9.56
3	0.4153	3.50	0.00	0.00	-131.00	7126.39	23251.88	0.49	0.49
4	0.4438	3.50	0.00	0.00	-107.00	5081.47	23251.88	0.43	0.43
5	0.4875	3.50	0.00	0.00	-77.00	2890.65	23251.88	0.34	0.34
6	0.5417	3.50	0.00	0.00	-46.00	1146.25	23251.88	0.23	0.23
7	0.6093	3.50	0.00	0.00	-15.00	137.09	23251.88	0.08	0.08
8	1.0000	3.50	0.00	0.00	82.00	6724.00	23251.88	-0.74	-0.74

Case 3 NS & EW									
$.75F_{NS}$ (kip)	84.225			e_{NS} (ft)	-30				
$.75F_{EW}$ (kip)	19.215			e_{EW} (ft)	2				
M_{NS} (k-ft)	-2526.75								
M_{EW} (k-ft)	38.43								
Frame	K (k/in)	ΣK_{NS} (k/in)	ΣK_{EW} (k/in)	Direct shear (kip)	d (ft)	Kd^2	∑Kd^2	Torsional Moment Shear (kip)	Total shear (kip)
1	0.1498	0.00	0.30	9.62	-24.00	86.29	23251.88	0.38	10.01
2	0.1494	0.00	0.30	9.59	20.00	59.74	23251.88	-0.32	9.27
3	0.4153	3.50	0.00	10.00	-131.00	7126.39	23251.88	5.82	15.82
4	0.4438	3.50	0.00	10.69	-107.00	5081.47	23251.88	5.08	15.77
5	0.4875	3.50	0.00	11.74	-77.00	2890.65	23251.88	4.02	15.76
6	0.5417	3.50	0.00	13.04	-46.00	1146.25	23251.88	2.67	15.71
7	0.6093	3.50	0.00	14.67	-15.00	137.09	23251.88	0.98	15.65
8	1.0000	3.50	0.00	24.08	82.00	6724.00	23251.88	-8.78	15.31

Case 4 NS+.15By & EW+.15									
$.563F_{NS}$ (kip)	63.2249			e_{NS} (ft)	6				
$.563F_{EW}$ (kip)	14.42406			e_{EW} (ft)	-11				
M_{NS} (k-ft)	379.3494								
M_{EW} (k-ft)	-158.66466								
Frame	K (k/in)	ΣK_{NS} (k/in)	ΣK_{EW} (k/in)	Direct shear (kip)	d (ft)	Kd^2	∑Kd^2	Torsional Moment Shear (kip)	Total shear (kip)
1	0.1498	0.00	0.30	7.22	-24.00	86.29	23251.88	-0.03	7.19
2	0.1494	0.00	0.30	7.20	20.00	59.74	23251.88	0.03	7.23
3	0.4153	3.50	0.00	7.51	-131.00	7126.39	23251.88	-0.52	6.99
4	0.4438	3.50	0.00	8.02	-107.00	5081.47	23251.88	-0.45	7.57
5	0.4875	3.50	0.00	8.81	-77.00	2890.65	23251.88	-0.36	8.46
6	0.5417	3.50	0.00	9.79	-46.00	1146.25	23251.88	-0.24	9.56
7	0.6093	3.50	0.00	11.01	-15.00	137.09	23251.88	-0.09	10.93
8	1.0000	3.50	0.00	18.08	82.00	6724.00	23251.88	0.78	18.85

Case 4 NS-.15By & EW-.15E									
$.563F_{NS}$ (kip)	63.2249			e_{NS} (ft)	-66				
$.563F_{EW}$ (kip)	14.42406			e_{EW} (ft)	15				
M_{NS} (k-ft)	-4172.8434								
M_{EW} (k-ft)	216.3609								
Frame	K (k/in)	ΣK_{NS} (k/in)	ΣK_{EW} (k/in)	Direct shear (kip)	d (ft)	Kd^2	ΣKd^2	Torsional Moment Shear (kip)	Total shear (kip)
1	0.1498	0.00	0.30	7.22	-24.00	86.29	23251.88	0.61	7.83
2	0.1494	0.00	0.30	7.20	20.00	59.74	23251.88	-0.51	6.69
3	0.4153	3.50	0.00	7.51	-131.00	7126.39	23251.88	9.26	16.76
4	0.4438	3.50	0.00	8.02	-107.00	5081.47	23251.88	8.08	16.10
5	0.4875	3.50	0.00	8.81	-77.00	2890.65	23251.88	6.39	15.20
6	0.5417	3.50	0.00	9.79	-46.00	1146.25	23251.88	4.24	14.03
7	0.6093	3.50	0.00	11.01	-15.00	137.09	23251.88	1.56	12.57
8	1.0000	3.50	0.00	18.08	82.00	6724.00	23251.88	-13.95	4.12

Case 4 NS+.15By & EW-.15E									
$.563F_{NS}$ (kip)	63.2249			e_{NS} (ft)	6				
$.563F_{EW}$ (kip)	14.42406			e_{EW} (ft)	15				
M_{NS} (k-ft)	379.3494								
M_{EW} (k-ft)	216.3609								
Frame	K (k/in)	ΣK_{NS} (k/in)	ΣK_{EW} (k/in)	Direct shear (kip)	d (ft)	Kd^2	ΣKd^2	Torsional Moment Shear (kip)	Total shear (kip)
1	0.1498	0.00	0.30	7.22	-24.00	86.29	23251.88	-0.09	7.13
2	0.1494	0.00	0.30	7.20	20.00	59.74	23251.88	0.08	7.28
3	0.4153	3.50	0.00	7.51	-131.00	7126.39	23251.88	-1.39	6.11
4	0.4438	3.50	0.00	8.02	-107.00	5081.47	23251.88	-1.22	6.81
5	0.4875	3.50	0.00	8.81	-77.00	2890.65	23251.88	-0.96	7.85
6	0.5417	3.50	0.00	9.79	-46.00	1146.25	23251.88	-0.64	9.15
7	0.6093	3.50	0.00	11.01	-15.00	137.09	23251.88	-0.23	10.78
8	1.0000	3.50	0.00	18.08	82.00	6724.00	23251.88	2.10	20.18

Case 4 NS-.15By & EW+.15Bx									
$.563F_{NS}$ (kip)	63.2249			e_{NS} (ft)	-66				
$.563F_{EW}$ (kip)	14.42406			e_{EW} (ft)	-11				
M_{NS} (k-ft)	-4172.8434								
M_{EW} (k-ft)	-158.66466								
Frame	K (k/in)	ΣK_{NS} (k/in)	ΣK_{EW} (k/in)	Direct shear (kip)	d (ft)	Kd^2	ΣKd^2	Torsional Moment Shear (kip)	Total shear (kip)
1	0.1498	0.00	0.30	7.22	-24.00	86.29	23251.88	0.67	7.89
2	0.1494	0.00	0.30	7.20	20.00	59.74	23251.88	-0.56	6.64
3	0.4153	3.50	0.00	7.51	-131.00	7126.39	23251.88	10.13	17.64
4	0.4438	3.50	0.00	8.02	-107.00	5081.47	23251.88	8.85	16.87
5	0.4875	3.50	0.00	8.81	-77.00	2890.65	23251.88	6.99	15.81
6	0.5417	3.50	0.00	9.79	-46.00	1146.25	23251.88	4.64	14.43
7	0.6093	3.50	0.00	11.01	-15.00	137.09	23251.88	1.70	12.72
8	1.0000	3.50	0.00	18.08	82.00	6724.00	23251.88	-15.28	2.80

Appendix C: Seismic Load 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 $\quad T < T_L$

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

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

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

Structural Engineer responded saying that the IBC allows a Cs value of .01 for buildings with SDC: A

See flow chart in section 6.5 of IBC

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

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

$F_{rf} = (0.33)(84) = 27.72 \text{ K}$

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

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

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

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

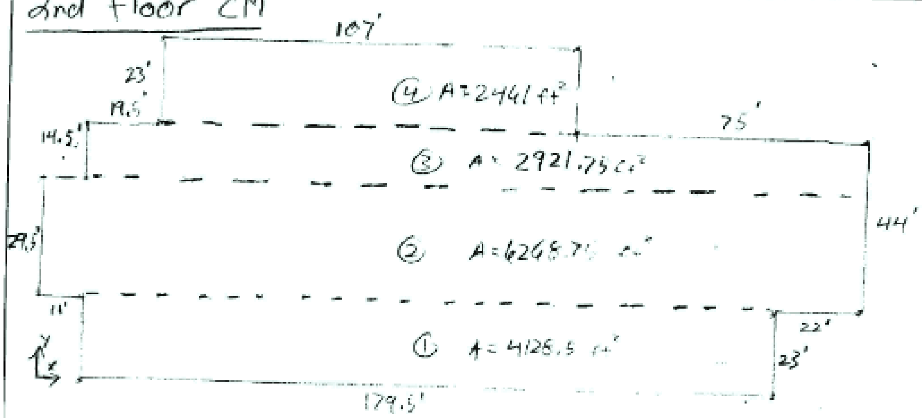
Lvl 4	Area	DL	Weight
Slab	16600	46	763600
superimposed	16600	5	83000
Steel	16600	5	83000
Façade	9293	45	418162.5
CMU	9293	83	771319
Int Brick	1500	40	60000
Stone Floor	1700	20	34000
			2,213,082

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

Appendix D: Center of Rigidity and Center of Mass Spot Checks

Daniel Bodde | Tech 3 | CM & CR Spot Checks

2nd floor CM



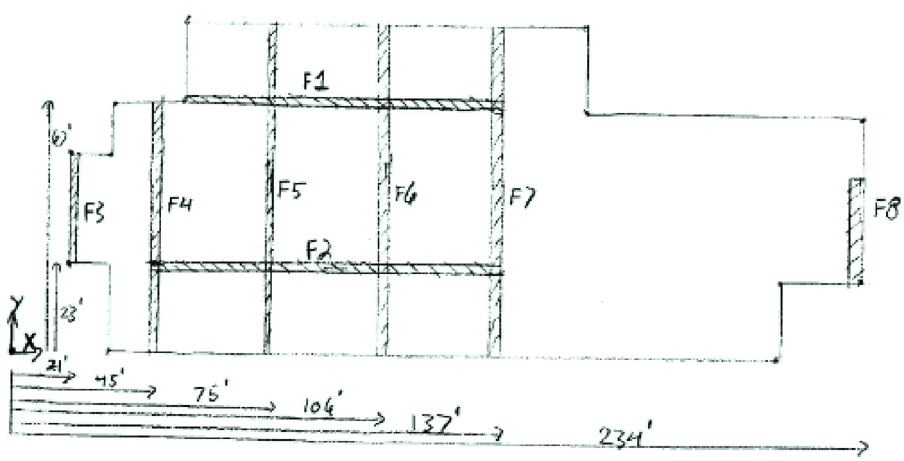
$$\bar{y}_{cm} = \frac{\Sigma \bar{y} A}{\Sigma A} = \frac{(11.5)(4126.5) + (37.75)(6268.75) + (59.75)(2921.75) + (78.5)(2441)}{15780}$$

$$= 41'$$

$$\bar{x}_{cm} = \frac{\Sigma \bar{x} A}{\Sigma A} = \frac{(122)(4128.5) + (127.5)(6268.75) + (133)(2921.75) + (105.25)(2441)}{15780}$$

$$= 124'$$

2nd floor CR



Daniel Bodde | Tech 3 | CM & CR Spot Checks

Relative Stiffnesses

$K_1 = 0.2117$ $K_5 = 0.6965$

$K_2 = 0.2116$ $K_6 = 0.7404$

$K_3 = 0.6309$ $K_7 = 0.7902$

$K_4 = 0.6376$ $K_8 = 1.0$

$$\bar{x} = \frac{\Sigma k_{iy} x_i}{\Sigma k_{iy}} = \frac{(0.6309)(21) + (0.6576)(45) + (0.6965)(75) + (0.7404)(106) + (0.7902)(137) + (1.0)(234)}{0.6309 + 0.6576 + 0.6965 + 0.7404 + 0.7902 + 1.0}$$

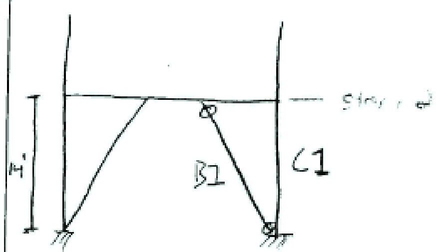
$$= 114'$$

$$\bar{y} = \frac{\Sigma k_{ix} y_i}{\Sigma k_{ix}} = \frac{(0.2117)(23) + (0.2116)(67)}{0.2117 + 0.2116} = 45'$$

Values differ slightly from ETABS model. This can be attributed to the assumption that frames have zero stiffness about the weak axis in the hand calculations.

Appendix E: Member Strength Spot Checks

Daniel Bodde | Tech 3 | Spot Checks



B1: HSS 6 x 6 x 1/2

C1: W 12 x 106

Column Check (C1)

Loads

$P_u = 310^K$

$M_{rx} = 3645$ K-ft

$M_{ry} = 0.81$ K-ft

Values from ETABS

Axial only:

W 12 x 106

KL = .7 x 14' = 9.8' ≈ 10'

$\phi P_u = 1240 K >> 150 K$ ✓

Combined loading

From table 6-1

$p = .794 \times 10^{-3}$

$b_x = 1.45 \times 10^{-3}$

$b_y = 3.16 \times 10^{-3}$

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

$(0.794 \times 10^{-3})(310) + (1.45 \times 10^{-3})(3645) + (3.16 \times 10^{-3})(0.81)$

$= 0.30 \leq 1.0$ OK

Daniel Bodde	Tech 3	Spot Checks

Brace check (B1)

KL = 18'

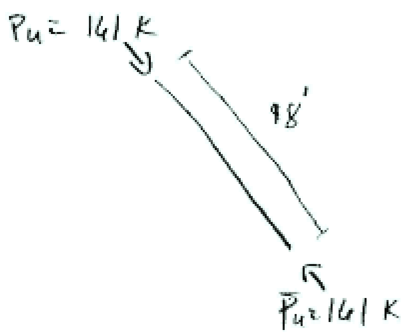


Table 4-4 for compression

$\phi P_n = 165\ K > 161$, Kips

OK

Check tension if load is reversed

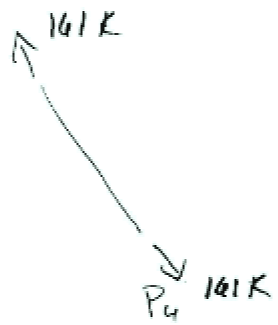


Table 5-5 for tension

$\phi P_n = 403\ K >>> 161$ Kips

OK