University Health Building

Located in the Mid-Atlantic Region

Final Report

Evan Landis || Structural Option

Advisor - Heather Sustersic

UNIVERSITY HEALTH BUILDING

LOCATED IN THE MID-ATLANTIC REGION

PROJECT TEAM

- Structural: Tadjer Cohen Edelson Associates
- Architect: Payette Associates
- MEP/FP: Affiliated Engineers
- Construction: Whiting Turner
- Lighting: Atelier Ten

PROJECT INFO

Delivery Method: Design-Bid-Build
Cost: $ 56 Million
Size: 161,000 SF
Floors: 7

EVAN LANDIS || STRUCTURAL OPTION

ARCHITECTURE

- Saw tooth reveals on the northeastern façade give the building some bite
- Each floor is a mesh between offices and classrooms. The architect hopes that this mesh will help to bridge the gap between faculty and students.
- Centralized sky lit atrium that serves as the building's main staircase

STRUCTURE

- Predominantly constructed of 2-way post tensioned plate slabs and continuous drop panel systems, depending where you are located in the building
- Lateral system: concrete moment frames
- Foundation: a network of spread footings connected with grade beams

MECHANICAL

- Primary cooling units are two 350 ton chillers located in the basement run by VFD's
- Primary heating provided by four natural gas boilers located in the penthouse
- The design also utilizes four energy recovery wheels for energy savings

ELECTRICAL

- Lighting is a mix of fluorescents and LED's
- Panels distribute 3-phase 480Y/277 and 208Y/120 volt power
- 750kW generator for emergency power

Photos courtesy of Payette Associates

Table of Contents

Acknowledgments………………………………………………………………………….. 5

Executive Summary……………………………………………………………..………… 6

Building Introduction…………………………………………………………………….. 7

Structural Overview……………………………………………………………………… 8

Foundation……………………………………………………………………….. 8

Floor Slabs..…………………………………………………………………….... 8

Lateral System………………………………………………………………….... 9

Roof System…………………………………………………………………….. 10

Codes & References..………………………………………………………………… 11

Material Strengths………………………………………………………………….… 12

Proposed Structural Depth.………………………………………………………… 13

Structural Redesign…………………………………………………………………... 14

ETABS Model…………………………………………………………………….. 14

Shear Wall Design……………………………………………………………… 16

Shear Wall Placement.……………………………………………………….… 19

Story Drift………………………………………………………………………… 23

Foundation……………………………………………………………………… 25

Breadth I: Cost/Foundation Schedule Analysis…....………………………………………… 27

Proposed Breadth……………………………………………………………….. 27

Caisson Cost vs. Original Spread Footing Cost………………………………... 27

Shear Wall Cost…..…………………………………………………………….... 27

Caisson Schedule vs. Original Spread Footing Schedule………………………….. 29

Breadth II: Condensation Analysis of Building Envelope..………………………………….. 30

Proposed Breadth...…………………………………………………………….. 30

Condensation Point……………………………………………………………… 30

R-Value………………………………………………………………………… 31

MAE Requirements……………………………………………………………………. 31

Conclusion……………………………………………………………………..….. 32

Table of Contents

Appendix A: Wind Force Calculations………….…………………………………………….. 33

Appendix B: Sample Calculations for Shear Wall Flexure………………………………….. 34

Appendix C: Sample Calculations for Shear Wall Shear..………………………………….. 36

Appendix D: Typical Shear Wall Detail…………………………………………………….. 39

Appendix E: Tabulated Calculations for Shear Wall Flexure and Shear...……………….. 40

Appendix F: Spread Footing Calculation for Column……………………………………….. 50

Appendix G: Spread Footing Calculation for Shear Wall…….…………………………….. 52

Appendix H: Caisson/Caisson Cap Calculations for Shear Wall.…………….………….. 54

Appendix I: Caisson/Caisson Cap Calculations for Column..…………………………….. 57

Appendix J: CRSI Design Tables………………………………………….………………. 58

Appendix K: Shear Wall Cost Analysis…………………………………………………….. 61

Appendix L: Caisson/Caisson Cap Cost Analysis………………………………………….. 62

Appendix M: Existing Spread Footing Cost…….……………………………………………. 64

Appendix N: RSMeans Building Construction Cost Data 2013…..……………………….. 65

Appendix O: Condensation/R-Value Analysis……………………………………………….. 69

Acknowledgments

I would like to thank Whiting-Turner, especially Brian Acri, Randy Riesner, and Sobe Ahmed, for helping me to the obtain the drawings and reports necessary for completion of my senior thesis as well as answering my questions.

Additionally, I would like to thank Payette Architects for allowing my to use their renderings of the building throughout my report.

Lastly, I would like to thank the following Pennsylvania State University Architectural Engineering faculty for their invaluable advice and guidance.

Professor Heather Sustersic

Dr. Moses Ling

Professor Kevin Parfitt

Professor Robert Holland

The entire AE faculty and staff

Executive Summary

This report determined if the University Health Building could be relocated to Orlando, Florida. The determination of the cost, changes to the lateral system and foundation, and analysis of the building's envelope were carried out to answer the above statement.

The relocation of the UHB is feasible based on a scope of elements that were analyzed in this report. Other items such as availability of materials needed, and the cost associated with these changes would also need to be analyzed.

It was determined that the relocation of the building would require the addition of 7 new shear walls and modifications to the existing shear wall. The addition of the shear walls resulted in an increased cost to the building of $118,694. The foundation of the building was also analyzed and determined to be inadequate. The foundation system would not only have to be changed in size but also in type. Caissons were used to carry the new load. It was determined that each of the shear walls would need two caissons, and the columns would each need one, resulting in a total of 63 new caissons. The cost of the foundations change from spread footing to caissons including caisson caps was determined to be $570,954. This resulted in a total increase to the building's cost of $689,648.

Lastly, a typical section of the building's envelope was checked to determine whether or not it would be able to perform in Orlando, Florida. After analyzing both the walls condensation point and R-value, it was determined that they will be able to perform without change. This is due to the metal panels that enclose the insulated middle portion of the wall.

Building Introduction

This new 9 story 161,000 square foot building will be a great addition to the university's campus. It is being built to house leaders in the public and private health policy sectors. The building is a mesh between office space and student classrooms nestled around a central sky lit atrium. The architect hopes that this mesh will help to bridge the gap between faculty and students. The classroom area appears as if the classrooms are floating on clouds in a glass enclosure. The concrete structure is enclosed by a curtain wall which is the building's main architectural feature. The curved saw blade-like curtain wall system encompasses one quarter of the building's façade and gives the building an edgy appearance.

The building façade is constructed of many different types of materials, ranging from stone to metal. The building's first floor is covered by a stone veneer giving the building a very stereotomic base. The rest of the building is clad in a mixture of glazing, metal panels, and terracotta. The West and Southeast facades are relatively similar to one another. They both have a pattern of terracotta, metal paneling, and glazing above the first floor with the majority material being covered with the terracotta. The south and north facades are also very similar except the south facade has an aluminum sunscreen system in place. Otherwise, these ends of the building are almost fully glazed. Lastly, the curved curtain wall with reveals located on the northeast side of the building is composed of mainly glazing with the reveals clad in terracotta. Some of these features can be seen in Figure 1.

Figure 1: Photo of Northwest corner of building showing façade materials. Rendering by Payette Architecture.

The majority of the roof is a garden roofing system. The system used on this project is the Sika Sarnafil Extensive Greenroof system. It uses 3in. of growing medium as well as pavers for maintenance. The rooftop penthouse will be covered with a fully adhered white, 60mm thick PVC membrane with a layer of 8in. thick tapered polyisocyanurate insulation boards underneath.

Lastly, the University Health Building is registered as a LEED – NC 2.2 Silver building. This rating includes many different LEED credits involving the façade, roof, and internal systems. The main points came from the heat island effect roof system, the building's proximity to transit, and use of efficient plumbing and lighting fixtures.

Structural Overview

Foundation

The foundation of University Health Building (UHB) consists of spread footings at the base of each column. On the western block of the building, the engineers utilized a grade beam and spread footing combination to help with the bracing of the basement wall shown in the Figure 2 below. This was not used on the east side of the building due to the absence of any underground levels. The spread footings are to be set on bedrock suitable to hold about 30,000psf according to the Geotechnical report.

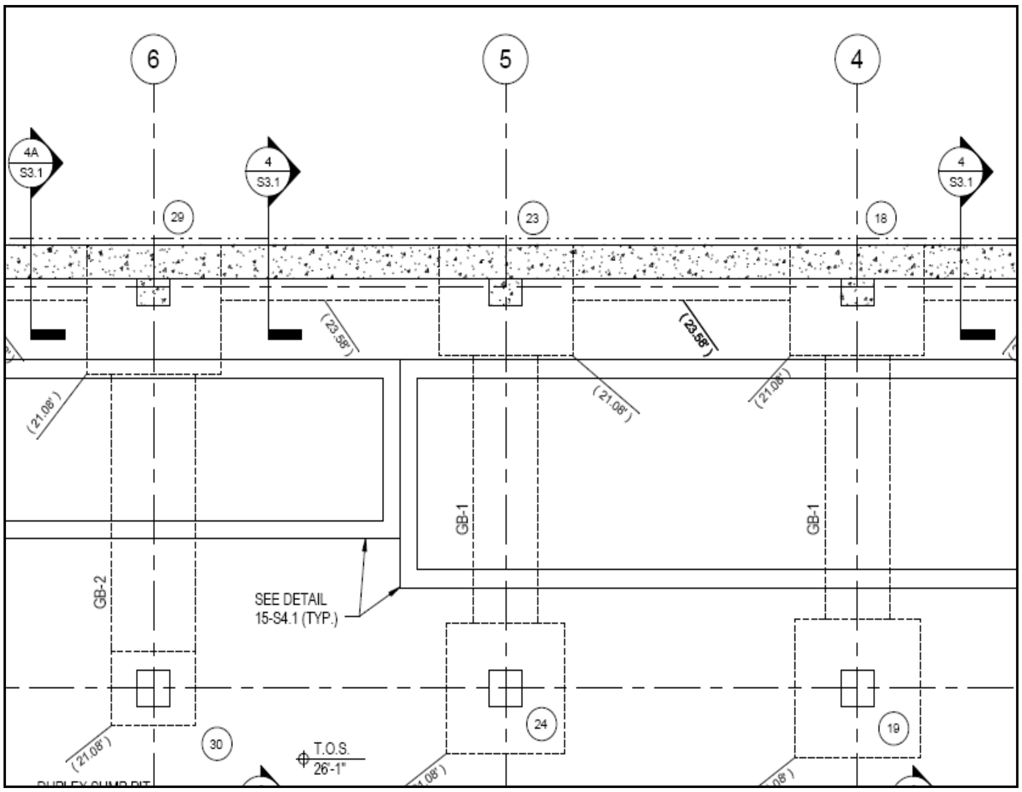


Figure 2: Grade beam and spread footing combination, taken from drawing S1.1

Floor Slabs

The basement level and ground level floor slabs are similar in the fact that they both have a relatively thick floor slab and drop panels comprised of high strength concrete in order to minimize the amount of beams necessary to handle the 21 ft. spans. Once you leave the ground floor, you will find that the slabs change from what was mentioned above to a post tensioned slab system. Also, above the ground floor on the east half of the building, the slabs have large continuous drop panels running between select columns. This type of system extends all the way to the penthouse slab with variations in slab and drop panel thicknesses.

Lateral System

Since the walls of the UHB building are non-load bearing, the lateral loads, due to wind and seismic, must be resolved by the columns and slabs of the building. The dominant lateral system of the UHB is concrete moment frames consisting of the post-tensioned slab and interior/exterior column system. In the case of wind, the load is transferred from the cladding to the exterior columns and slab edge. Then, it is distributed to the interior columns through the slab, and finally, it's transferred to the foundation through the columns. The lateral system also utilizes one shear wall located beside the elevator shaft. The shear wall is called out in Figure 3.

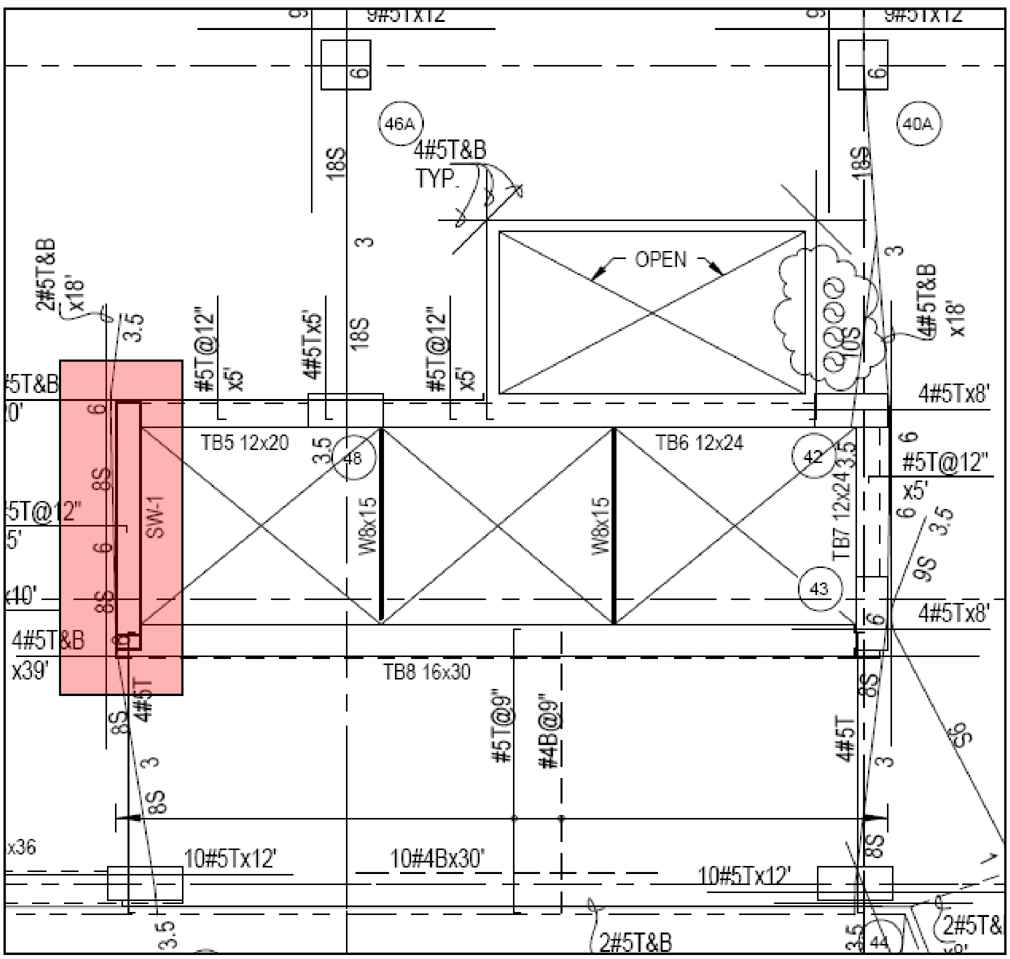


Figure 3: Location of shear wall, taken from S1.8

Roof System

The roof system is comprised of two different levels. The first being the lower roof where the green roof is located, and the second is the upper roof that covers the penthouse. The lower roof is a 12-14in. thick post tensioned slab and topped with a green roof system where exposed to the outside. The upper roof is supported by an 8in. post tensioned slab. Also, a portion of the penthouse roof is spanned with steel beams with a glazing system overtop to serve are the skylight for the central stair tower. Figure 4 below shows a partial roof plan showing the integration of the post tensioned concrete slab and central skylight area.

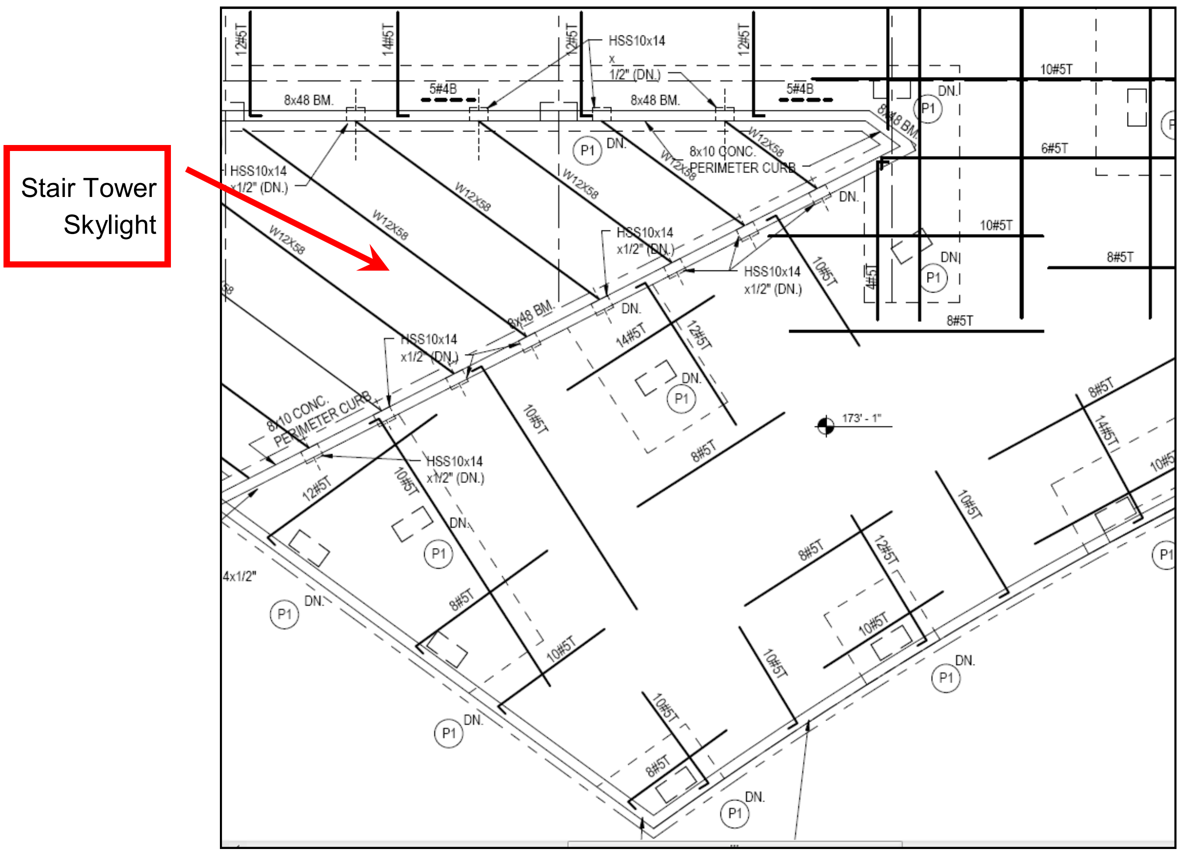


Figure 4: Integrations of both steel and concrete systems on roof, taken from drawing S1.11

Codes & References

Design Codes

Building Code

International Building Code - IBC 2006 system

Reference Codes

American Society of Civil Engineers - ASCE 7-05

American Concrete Institute Building Code - ACI 318-05, ACI 530-05, ACI 530.1-05

American Institute of Steel Construction - AISC 360-05

Thesis Codes

Building Code

International Building Code - IBC 2009

Reference Codes

American Society of Civil Engineers - ASCE 7-05

American Concrete Institute Building Code - ACI 318-08

American Institute of Steel Construction - AISC 14th Edition

American Society of Heating, Refrigeration, and Air-Conditioning Engineers - ASHRAE 2005-2008 Handbook

Handbooks

Concrete Reinforcing Steel Institute Handbook 2008

RSMeans Building Construction Cost Data 2013

ASHRAE 90.1 Energy Standard for Buildings Except Low-Rise Residential Buildings 2010

Material Strengths

General material strengths were found on S4.9 and are displayed in Figure 5. The general types and strengths can be overridden per special callouts on the floor plans. On many floors, slab strengths are a combination of 6000psi and 8000psi. See Figure 6 and 7 for good examples of the drawings superseding the general strengths. The figures show variations in concrete strength as the building elevation increases and slab thickness increases.

Item	Type	Strength
Steel Beams	ASTM-A992	Fy= 50
Post tensioning Tendons	ASTM A-416	Fu= 270
Reinforcement	ASTM-A615	Fy= 60
Masonry	ASTM C-90	f'c=1.5
Grade Beams	NW Conc.	f'c= 4
Column Footings	NW Conc.	f'c= 5
Slab on grade	NW Conc.	f'c= 5
Floor slabs	NW Conc.	f'c= 6
Columns	NW Conc.	See Fig.

Figure 5: Material strength table

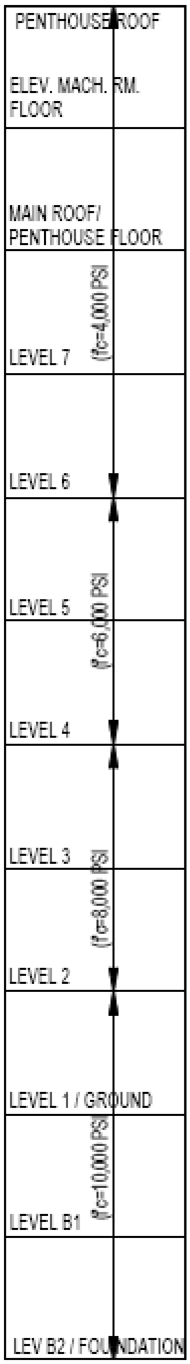


Figure 6: Variations in column concrete strengths per level

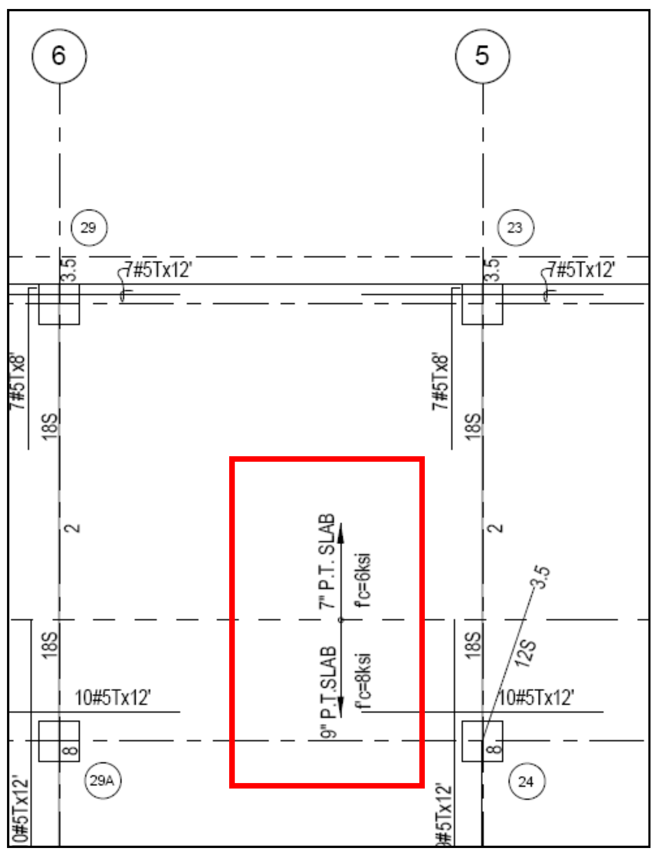


Figure 7: Variations in slab concrete strength

Proposed Structural Depth

Problem Statement

As concluded from Technical Reports I, II, and III, the UHB adequately meets structural strength and serviceably requirements for its current location in the Mid-Atlantic, but what if the building were no longer located in the Mid-Atlantic? Many companies and institutions have trademark building architecture that helps them to distinguish their brand, as a form of advertising. On the outside these buildings may appear very similar but on the inside they may need be very different to meet the structural and serviceably requirements of the building's location. The building's location can drastically change its lateral system due to it being located in either a wind or seismic controlled region.

This is an issue that designers face on a regular basis. To the public, the building will appear the same as its similar counterparts, but the building's internal components are designed to meet the requirements of the building's location.

Proposed Solution

For the depth of my senior thesis, I employ that the university is opening a new branch campus in Orlando, Florida and would like its building to be the same as the University Health building in the Mid-Atlantic. This will have an impact on the building's lateral system as it moves from its current location, where seismic was found to control lateral loading, to Orlando where wind is the controlling lateral load. This will be an interesting academic experiment as the lateral system will need to be revamped to account for the hurricane force winds. This will be done by the addition of more concrete shear walls to the UHB, which currently has one shear wall. The shear walls will help to make the structure more rigid allowing it to withstand the greater lateral loads.

The first challenge is that the new shear walls will have to be incorporated into the building's architecture. This may require small alterations to the floor plan, depending on the amount of shear walls necessary.

Also, the foundation will then be analyzed for the new loading and altered as necessary. Due to the increased wind loads, the possibility of having uplift forces on the foundation is increased. Critical spread footings will also be analyzed for the new soil type at the building's new location.

Structural Redesign

ETABS Model

A model of the UHB was constructed in ETABS in order to analyze the effects of the lateral force due to wind on the building. All columns, floor slabs, drop panels, and beams were modeled with their correct material properties so that the dead load of the building could be accurately determined by the program. The modulus of elasticity for all concrete strengths were halved to account for cracked section properties in the model. This is required by ACI 318-08 section 8.8.2 to calculate lateral deflections due to the inelastic response of the concrete members. The floor slabs were modeled as rigid diaphragms and the shear walls were modeled as shell elements so that they would only have in-plane stiffness. The shear walls modeled as shell elements will be important as the shear walls will only be designed to resist flexure in one direction. ETABS was then used to generate the wind loads and dead loads to be used in the load combinations required by the 2006 International Building Code. It was concluded in Technical Report 3 that the wind loads generated by ETABs to be very similar to those calculated by hand. Live, superimposed dead, roof live, and green roof live were placed in their respective areas as well, so that all load combinations could be utilized. The varying wind load cases from ASCE7-05 were also used. The ETABS model is depicted in Figure 8 (including the new shear walls). The load combinations are shown in Figure 9. Figure 9 also points out the controlling load cases that were used throughout the analysis process as all combinations were tried. Figure 10 lists the live loads that were used in the load combinations. The superimposed dead load used was 10psf.

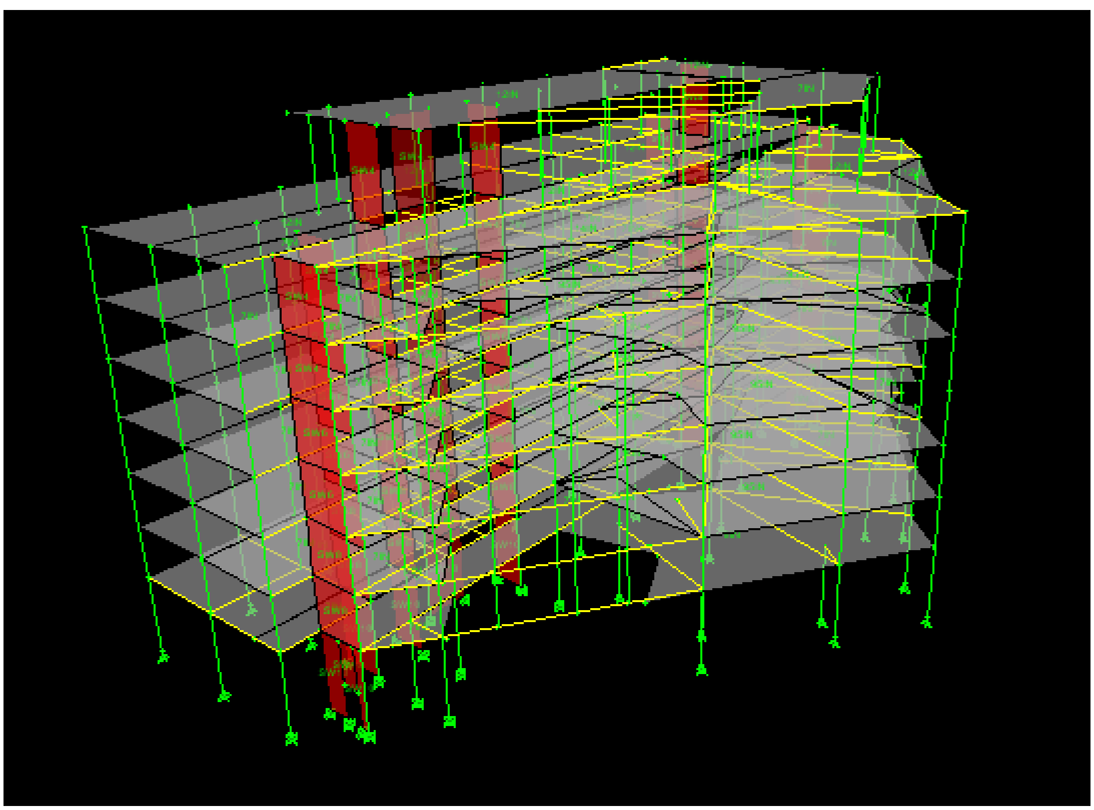


Figure 8: ETABS Model

Structural Redesign

1.4D

1.2D+1.6L+0.5(Lr or S or R) ← Controlling load combination for axial

1.2D+1.6(Lr or S or R)+(0.5L or 0.5W)

1.2D+1.6W+1.0L+0.5+0.5(Lr or S or R)

1.2D+1.0E+0.5L+0.4S

0.9D+1.0W ← Controlling load combination for lateral

0.9D+1.0E

Wind Load Cases
1.0WX
1.0WY
.75WX + .75WMX
.75WX - .75WMX
.75WY + .75WMY
.75WY - .75WMY
.75WX + .75WY
.563WX + .563WY + .563WMX + .563WMY
.563WX + .563WY + .563WMX - .563WMY
.563WX + .563WY - .563WMX + .563WMY
.563WX + .563WY - .563WMX - .563WMY

Controlling wind cases

Figure 9: Load cases used in design. Arrows indicate the controlling load cases.

Live Loads	Design (psf)
Roof	30
Mechanical Penthouse	150
Green Roof	35
Stairways	100
Corridors	100
Loading Dock	450
Light Storage	125
Retail	100
Office	80
Partitions	20

Figure 10: Live Loads used for design.

Structural Redesign

Shear Wall Design

The number of shear walls used for the new design was an iterative process which included the lengthening and the addition of shear walls as necessary in the before mentioned ETABs model. This guess and check process was used until the standard practice requirement of h/400 for story drifts was met. The building's story drifts will be discussed in detail later in the report.

In order to keep with the pre-established program of the building, the walls were constructed of 10ksi concrete at the base and decreased to 4ksi at the penthouse. This transformation is demonstrated in Figure 6. The shear walls were designed 12in. thick with boundary elements located in each end within the wall's 12in. thickness.

The wind velocity that was used for the design in Orlando was found to be 145mph. Orlando is a hurricane prone region, and the 145mph design wind velocity is set by their local code thus overwriting the velocity given by ASCE7-05. This velocity increased greatly from the 90mph that was used for design in the Mid-Atlantic. Figure 11 shows the excerpt from the Orlando Building Code stating that for Risk Category III 145mph shall be used. Figures 12-14 show the story forces and overturning moment that were calculated using ASCE7-05. Max moments and shears were then calculated by entering all load cases into the ETABS model. It was found that the load case 0.6D+1.6W controlled the design. These forces were then used for the design of the 8 new shear walls. The walls were designed for the forces at ground level and then checked at each story where a change in f'c occurred to assure that they met strength requirements. The calculations for the story forces can be found in Appendix A. Detailed calculations for the shear walls can be found in Appendices B-E.

Sec. 13.3. - Ultimate design wind speeds.

(a) Pursuant to "Note 2," *Figure 1609A*, of the *Building* volume of the building code, the ultimate design wind speeds for Risk Category II buildings and other structures within the City is hereby interpolated as 135 miles per hour.

(b) Pursuant to "Note 2," *Figure 1609B*, of the *Building* volume of the building code, the ultimate design wind speeds for Risk Category III and IV buildings and other structures within the City is hereby interpolated as 145 miles per hour.

(c) Pursuant to "Note 2," *Figure 1609C*, of the *Building* volume of the building code, the ultimate design wind speeds for Risk Category I buildings and other structures within the City is hereby interpolated as 125 miles per hour.

Ord. No. 2012-10, § 1, 3-26-2012, Doc. #1203261201)

Figure 11: Design wind speeds from the Orlando Building Code.

Structural Redesign

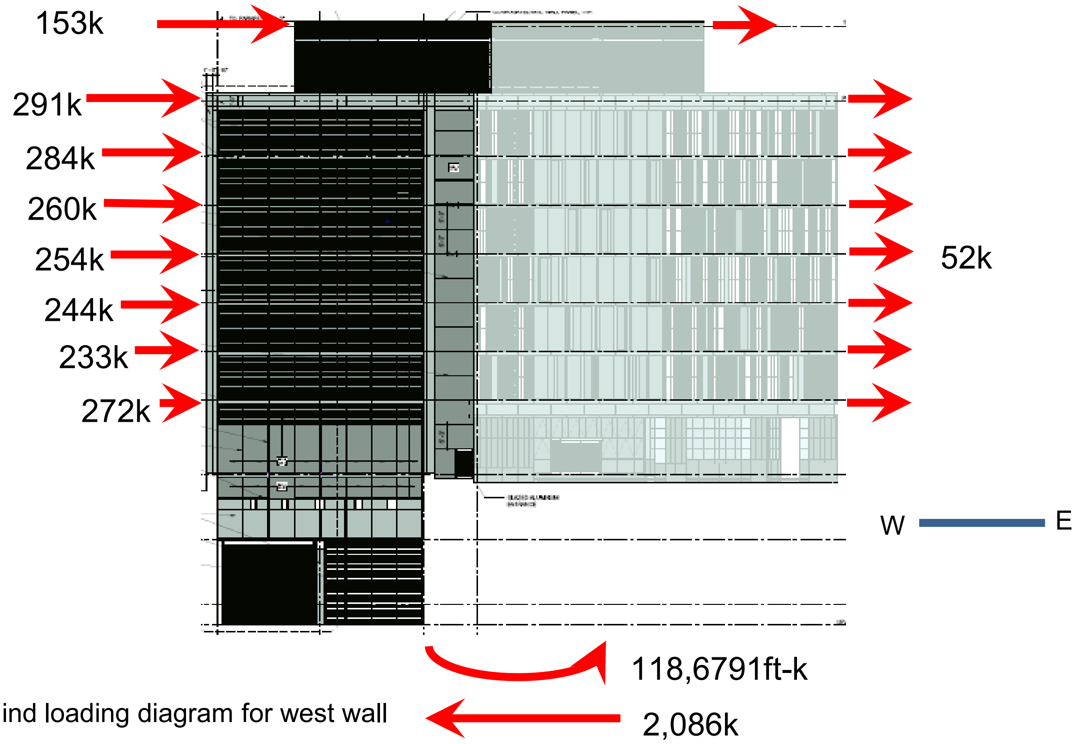


Figure 12: Wind loading diagram for west wall

West Wall												
Story	Story height	Elevation	Trib. Height (ft.)	Trib. Length (ft)	K_z	q_z	q_h	P_w (psf)	P_l (psf)	Trib. Area	Total Story Force (kip)	Overturning Moment (ft-k)
1	0	0.0	9.00	200	0	0	0	0	-52	1800	93.60	0.00
2	18	18.0	15.00	200	0.6	31.57	31.5678	38.83	-52	3000	272.49	4904.73
3	12	30.0	12.00	200	0.7	36.83	36.8291	45.30	-52	2400	233.52	7005.58
4	12	42.0	12.00	200	0.77	40.51	40.512	49.83	-52	2400	244.39	10264.44
5	12	54.0	12.00	200	0.83	43.67	43.6688	53.71	-52	2400	253.71	13700.35
6	12	66.0	12.00	200	0.87	45.77	45.7733	56.30	-52	2400	259.92	17154.90
7	12	78.0	12.75	200	0.92	48.40	48.4039	59.54	-52	2550	284.42	22184.67
Penthouse	13.5	91.5	16.00	140	0.96	50.51	50.5084	62.13	-52	2553	291.31	26654.41
T.O.C. Roof	18.5	110.0	9.25	140	1.02	53.67	53.6652	66.01	-52	1295	152.82	16810.27
									∑		2,086	118,679

Perimeters	
I=	1.15
G=	0.85
C_p Windward=	0.80
C_p Leeward=	-0.50
K_{zt}=	1.00
K_d=	0.85
Velocity=	145.00
GC_{pi}=	0.55

Figure 13: West wall story forces

South Wall												
Story	Story height	Height	Trib. Height (ft.)	Trib. Length (ft)	K_z	q_z	q_h	P_w (psf)	P_l (psf)	Trib. Area	Total Story Force (kip)	Overturning Moment (ft-k)
1	0	0	9.00	130	0	0	53.67	29.51587	-43	1170	85.08	0.00
2	18	18	15.00	130	0.6	31.57	53.67	50.98196	-43	1950	183.26	3298.77
3	12	30	12.00	130	0.7	36.83	53.67	54.55964	-43	1560	152.19	4565.79
4	12	42	12.00	130	0.77	40.51	53.67	57.06402	-43	1560	156.10	6556.19
5	12	54	12.00	130	0.83	43.67	53.67	59.21063	-43	1560	159.45	8610.22
6	12	66	12.00	130	0.87	45.77	53.67	60.6417	-43	1560	161.68	10670.95
7	12	78	12.75	130	0.92	48.40	53.67	62.43054	-43	1657.5	174.75	13630.59
Penthouse	13.5	91	16.00	90	0.96	50.51	53.67	63.86161	-43	1710	182.73	16628.74
T.O.C. Roof	18.5	110	9.25	90	1.02	53.67	53.67	66.00822	-43	832.5	90.75	9982.43
									∑		1,346	73,944

Perimeters	
I=	1.15
G=	0.85
C_p Windward=	0.80
C_p Leeward=	-0.30
K_{zt}=	1.00
K_d=	0.85
Velocity=	145.00
GC_{pi}=	0.55

Figure 14: South wall story forces

Structural Redesign

Shear Wall Design (cont.)

For ease of construction, walls of the same length were given the same rebar configuration even though calculations show that they will be over designed. This will help to alleviate confusion while the shear walls are being constructed. Figure 15 displays the rebar necessary for each shear wall. In most cases, shear reinforcement was not needed. This was due to the reinforcement needed for temperature and shrinkage being the controlling factor for design (ρ_{min}=.0025). The typical layout of reinforcing can be seen in Figure 16. The different detail for the rebar on the ground floor of walls 1 and 7 due to a door opening can be found in Appendix C.

Shear Wall Reinforcing

Wall	Length (ft)	Boundary Element*	Shear	Temperature/Shrinkage**	
				Vertical	Horizontal
1	11	(8) #9's	None	#5's @ 12"	#5's @ 12"
2	10	(8) #10's	None	#5's @ 12"	#5's @ 12"
3	8	(6) #9's	None	#5's @ 12"	#5's @ 12"
4	8	(6) #9's	None	#5's @ 12"	#5's @ 12"
5	10	(8) #10's	None	#5's @ 12"	#5's @ 12"
6	11	(8) #9's	None	#5's @ 12"	#5's @ 12"
7	11	(8) #9's	None	#5's @ 12"	#5's @ 12"
8	10	(8) #10's	None	#5's @ 12"	#5's @ 12"
		*Amount of rebar given per boundary element			
		**Walls 1 and 7 have special conditions @ ground level for door opening			

Figure 15: Rebar required for each shear wall

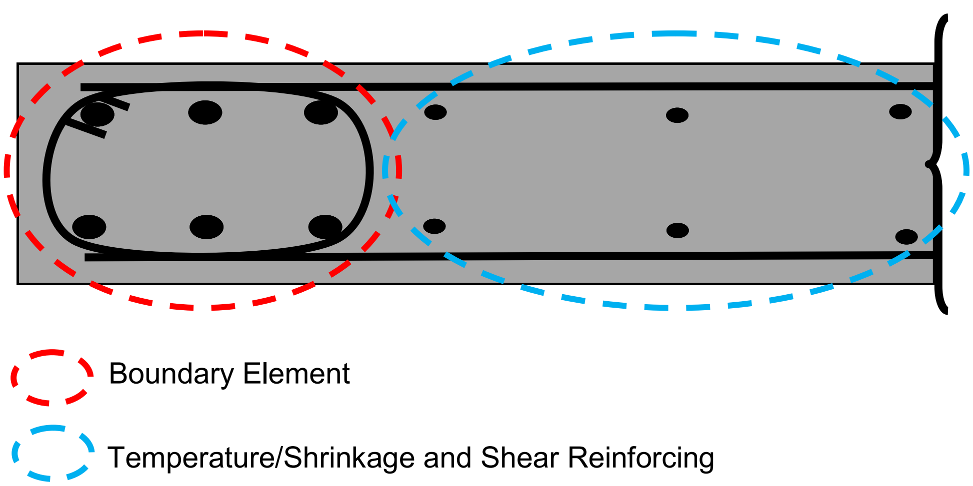


Figure 16: Typical shear wall rebar layout (not to scale)

Structural Redesign

Shear Wall Placement

The solution for the placement of the shear walls was restricted by the building's existing architecture. One had to be sure not to interrupt the flow of the architecture with rouge shear walls. The final shear wall placement was able to be completed with only minimal changes to the building's architecture. The placement of all the shear walls can be seen in Figure 17. These walls were used in conjunction with the concrete moment frames shown in Figure 18 to resist the lateral forces. All of the areas that required slight changes to the architecture are shown below in Figures 19-22 with suggestions or explanations of the changes that will be required for their new location. The remainder of the shear wall locations were able to be placed without any alterations to the floor plan, other than some walls will have to be built slightly thicker. This occurs in some office space as well as the restrooms.

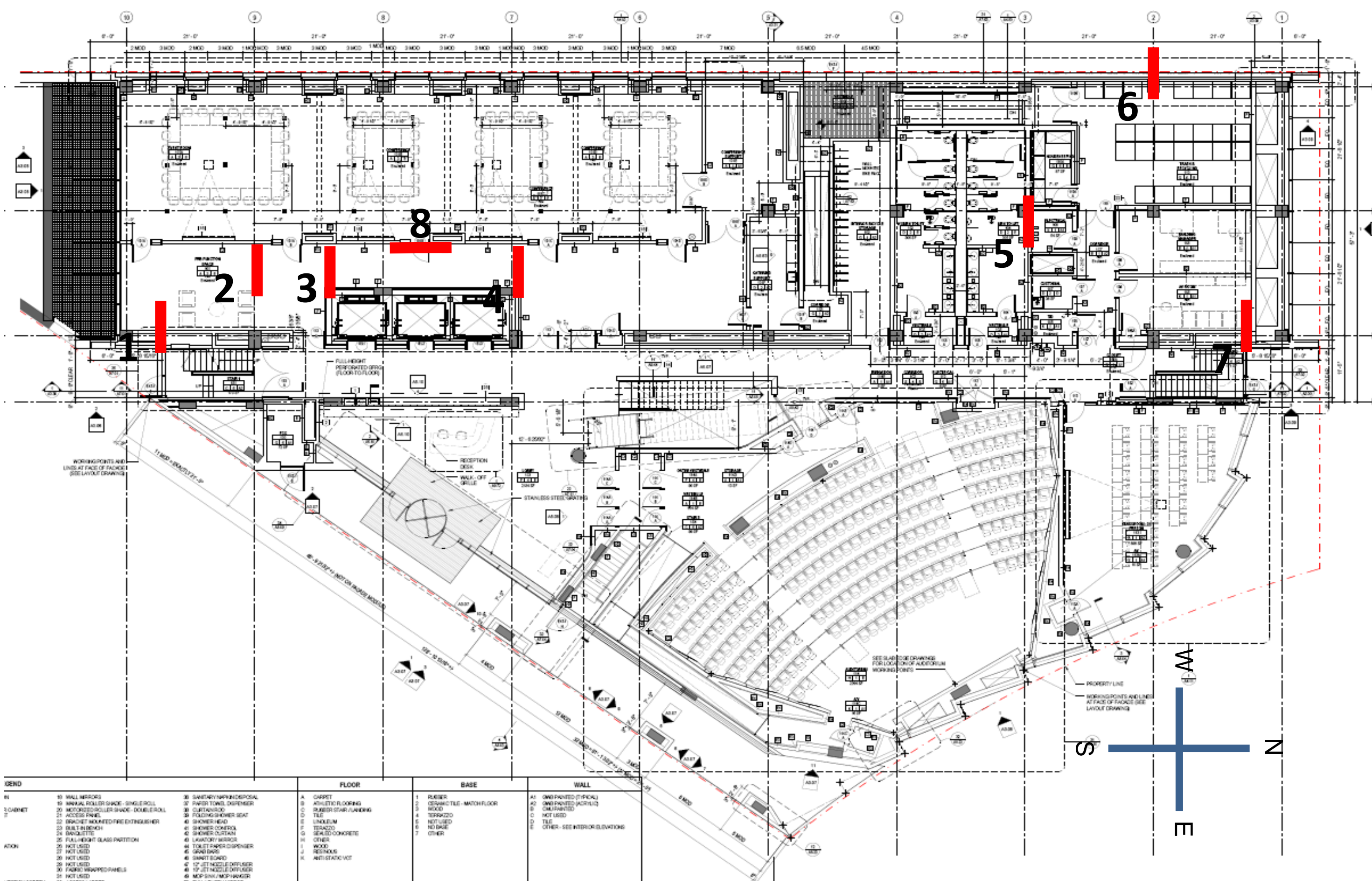


Figure 17: Shear Wall Layout

Structural Redesign

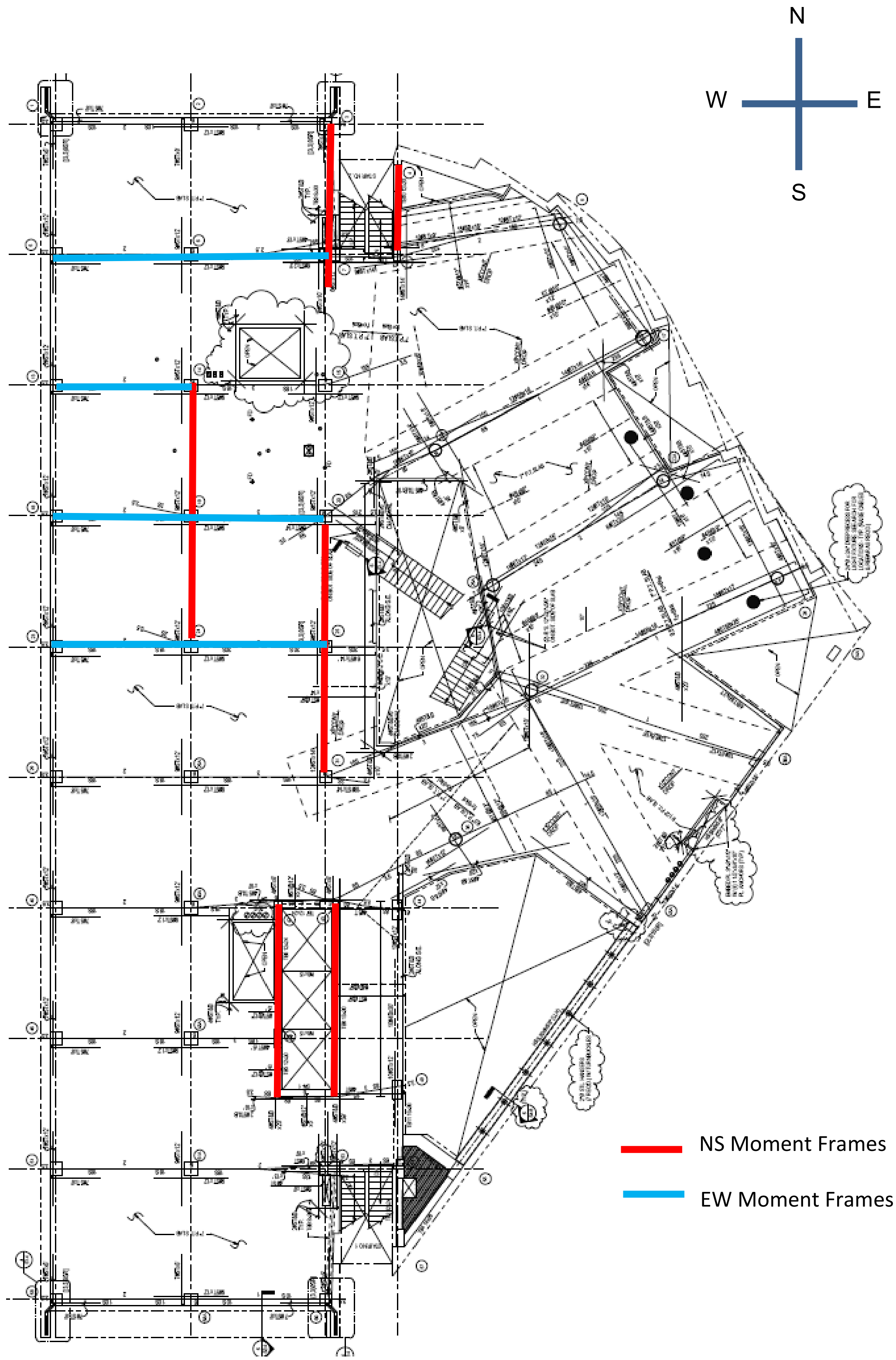


Figure 18: Location of lateral moment frames

Structural Redesign

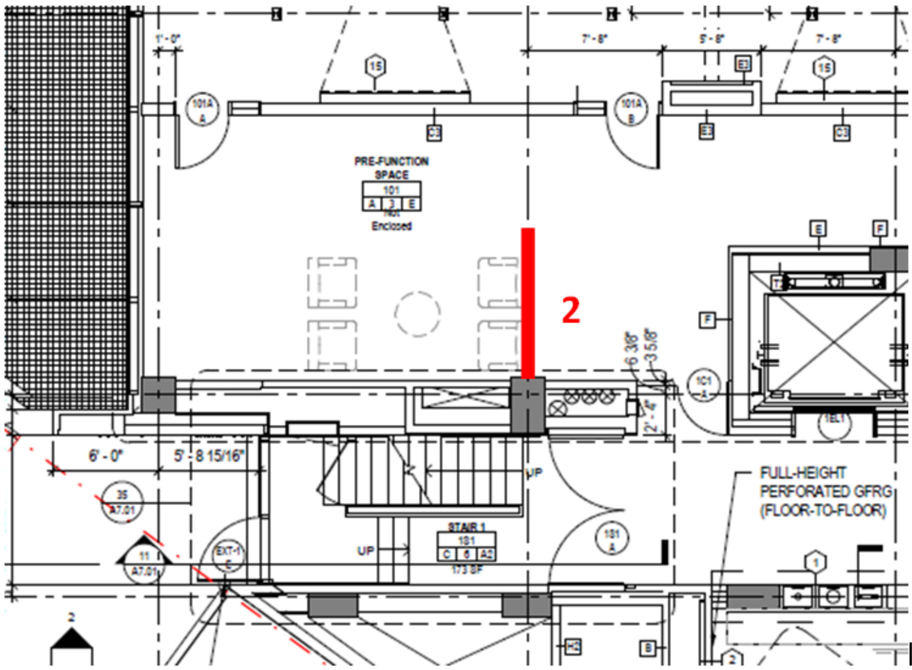


Figure 19: Shear wall 2 is located in the Pre-function Space. The shear wall helps to render this seating area into its own space rather than having it as part of the hallway. The function's patrons may like this privacy while they are waiting for the beginning of their event.

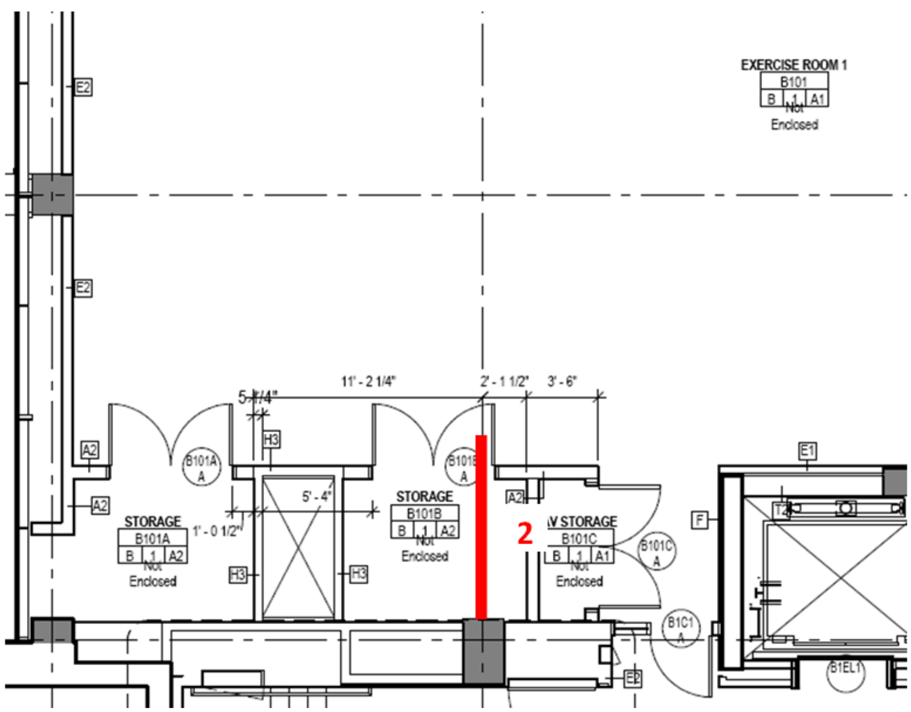


Figure 20: Shear wall 2 intersects with the exercise room located on basement level B1. This will require some shifting of the closet spaces and an increase in their depth.

Structural Redesign

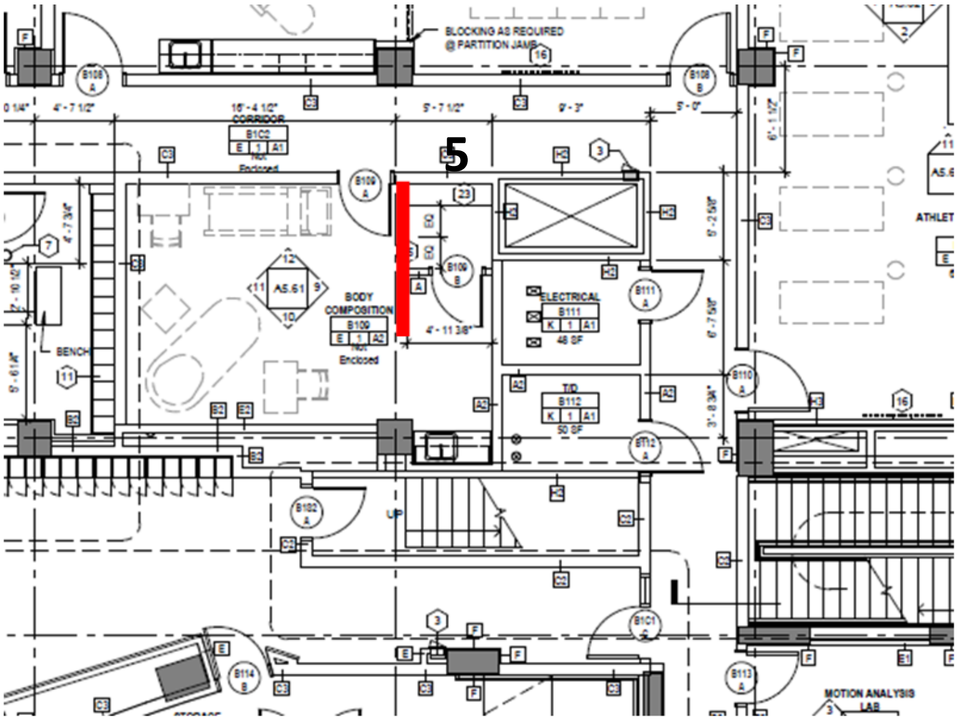


Figure 21: Shear wall 5 will require an extension to an existing closet wall. This will give the Body Composition Room more closet space, though it does reduce its amount of floor area. The new layout of the room does not interfere with equipment layout.

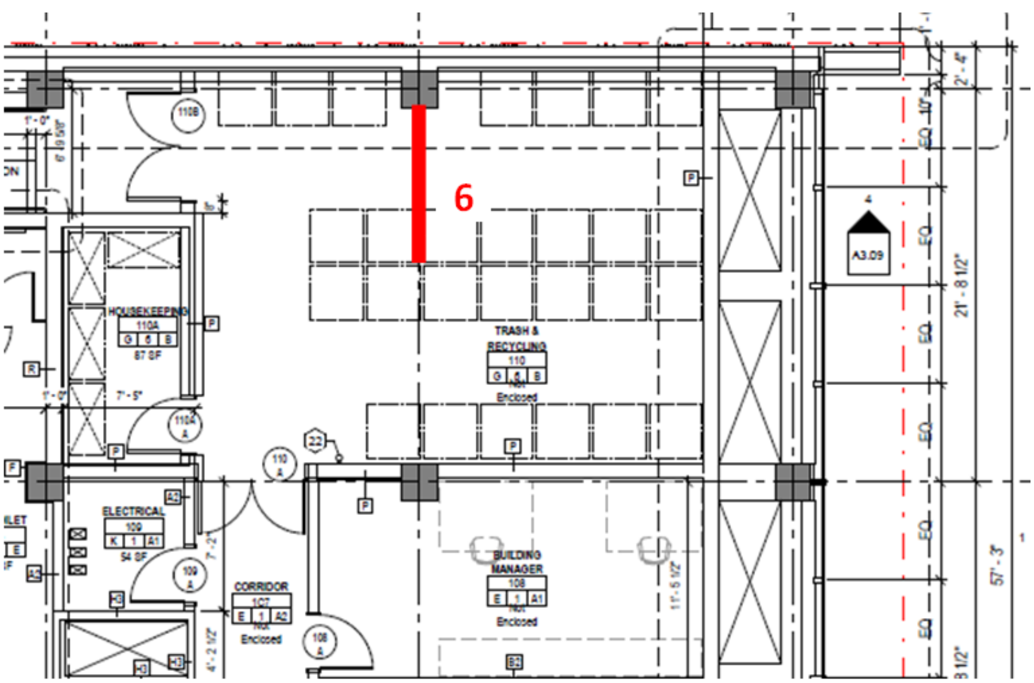


Figure 22: Shear wall 6 will intersect with the Trash and Recycling Room. This should not create any problems due to the utilitarian nature of this space.

Structural Redesign

Story Drifts

This analysis only considers wind forces on the building to calculate the story drift. Story drifts were analyzed twice with results taken from ETABS. The first analysis was for the original building to determine if the building's initial lateral system was capable of resisting the new loading. The building did not meet industry standards for story drift using its original system. See Figure 23. Thus, more shear walls were required to increase the strength of the lateral system. A second analysis was done after the addition of the new shear walls. Figure 24 shows that the new design falls just under the industry standard.

Original Lateral System

Wind Drift: North-South

Floor	Story Height (ft)	Story Drift Ratio X (in/in)	Story Drift X (in/in)	Story Drift Ratio Y (in/in)	Story Drift Y (in)	Total Drift X	Total Drift Y	Allowable Total Drift	Acceptable X	Acceptable Y
8	18.5	0.002661	0.590742	0.000941	0.20890	3.62898	2.05345	3.30	No	Yes
7	13.5	0.002575	0.41715	0.001531	0.24802	3.038238	1.84455	2.75	No	Yes
6	12	0.003081	0.443664	0.001914	0.27562	2.621088	1.596528	2.34	No	Yes
5	12	0.003426	0.493344	0.002184	0.31450	2.177424	1.320912	1.98	No	Yes
4	12	0.003522	0.507168	0.002205	0.31752	1.68408	1.006416	1.62	No	Yes
3	12	0.003359	0.483696	0.002049	0.29506	1.176912	0.688896	1.26	Yes	Yes
2	12	0.002825	0.4068	0.001595	0.22968	0.693216	0.39384	0.90	Yes	Yes
1	18	0.001326	0.286416	0.000760	0.16416	0.286416	0.16416	0.54	Yes	Yes

Wind Drift: East-West

Floor	Story Height (ft)	Story Drift Ratio X (in/in)	Story Drift X (in)	Story Drift Ratio Y (in/in)	Story Drift Y (in)	Total Drift X	Total Drift Y	Allowable Total Drift	Acceptable X	Acceptable y
8	18.5	0.000298	0.066156	0.005853	1.29937	0.088	13.55312	3.30	Yes	No
7	13.5	0.001152	0.001152	0.007756	1.25647	0.022	12.25375	2.75	Yes	No
6	12	0.002769	0.002769	0.01137	1.63728	0.020	10.99728	2.34	Yes	No
5	12	0.003811	0.003811	0.01441	2.07504	0.018	9.36000	1.98	Yes	No
4	12	0.004416	0.004416	0.015385	2.21544	0.014	7.28496	1.62	Yes	No
3	12	0.004302	0.004302	0.014637	2.10773	0.009	5.06952	1.26	Yes	No
2	12	0.003408	0.003408	0.011622	1.67357	0.005	2.96179	0.90	Yes	No
1	18	0.001758	0.001758	0.005964	1.28822	0.002	1.28822	0.54	Yes	No

Figure 23: Original lateral system drift results

Structural Redesign

New Lateral System										
Wind Drift: North-South										
Floor	Story Height (ft)	Story Drift Ratio X (in/in)	Story Drift X (in)	Story Drift Ratio Y (in/in)	Story Drift Y (in)	Total Drift X	Total Drift Y	Allowable Total Drift	Acceptable X	Acceptable y
8	18.5	0.002492	0.553224	0.000596	0.13231	3.06885	1.51646	3.30	Yes	Yes
7	13.5	0.002553	0.413586	0.000629	0.10190	2.515626	1.384146	2.75	Yes	Yes
6	12	0.002766	0.398304	0.006390	0.92016	2.10204	1.282248	2.34	Yes	Yes
5	12	0.003032	0.436608	0.000621	0.08942	1.703736	0.362088	1.98	Yes	Yes
4	12	0.003007	0.433008	0.000604	0.08698	1.267128	0.272664	1.62	Yes	Yes
3	12	0.003063	0.441072	0.000559	0.08050	0.83412	0.185688	1.26	Yes	Yes
2	12	0.002443	0.351792	0.000444	0.06394	0.393048	0.105192	0.90	Yes	Yes
1	18	0.001159	0.250344	0.000191	0.04126	0.04126	0.041256	0.54	Yes	Yes
Wind Drift: East-West										
Floor	Story Height (ft)	Story Drift Ratio X (in/in)	Story Drift X (in)	Story Drift Ratio Y (in/in)	Story Drift Y (in)	Total Drift X	Total Drift Y	Allowable Total Drift	Acceptable X	Acceptable y
8	18.5	0.000174	0.038628	0.00313	0.69486	0.219	3.24650	3.30	Yes	Yes
7	13.5	0.000169	0.027378	0.00307	0.49734	0.180	2.55164	2.75	Yes	Yes
6	12	0.000337	0.048528	0.003043	0.43819	0.153	2.05430	2.34	Yes	Yes
5	12	0.000134	0.019296	0.003038	0.43747	0.104	1.61611	1.98	Yes	Yes
4	12	0.000174	0.025056	0.002455	0.35352	0.085	1.17864	1.62	Yes	Yes
3	12	0.000194	0.027936	0.002557	0.36821	0.060	0.82512	1.26	Yes	Yes
2	12	0.000134	0.019296	0.00197	0.28368	0.032	0.45691	0.90	Yes	Yes
1	18	0.000059	0.012744	0.000802	0.17323	0.013	0.17323	0.54	Yes	Yes

Figure 24: New lateral system drift results

Structural Redesign

Foundation

When increasing the lateral loads of a building, one must check the impact that increase has on the foundations supporting the lateral elements. Also, the move from the Mid-Atlantic to Orlando changed the soil type drastically. The building went from spread footings on bedrock to the sandy soil found in Florida. In order to stay with the program of the building, spread footings were tried first. It was determined that the bearing capacity of the soil in Orlando was between 3-4ksi. This was not enough to carry the loading of the building. In order to fix this problem, the use of structural fill was assumed to increase the bearing capacity to 8ksi. Calculations were completed based on the 8ksi bearing capacity, and it was found that 8ksi was enough to design a spread footing for a typical column in the moment frame, but could not handle the loads of the trial shear wall. The trial shear wall and column are depicted in Figure 25. The shear wall's spread footing would have to be much too large in plan and depth to handle the overturning moment. The shear wall spread footing could not be enlarged due to conflict with other spread footings surrounding it. See Appendices F-G for detailed calculations for the column and shear wall's spread footings. The single shear wall from the existing lateral system was used as the trial shear wall in the new lateral system with hopes that the new spread footing could be compared to the old to determine a percent increase but this was not possible. Allowable Stress Design load cases were used for determining the load for the bearing on the soil and Load and Resistance Factored Design load cases were used when determining the loading for the spread footing.

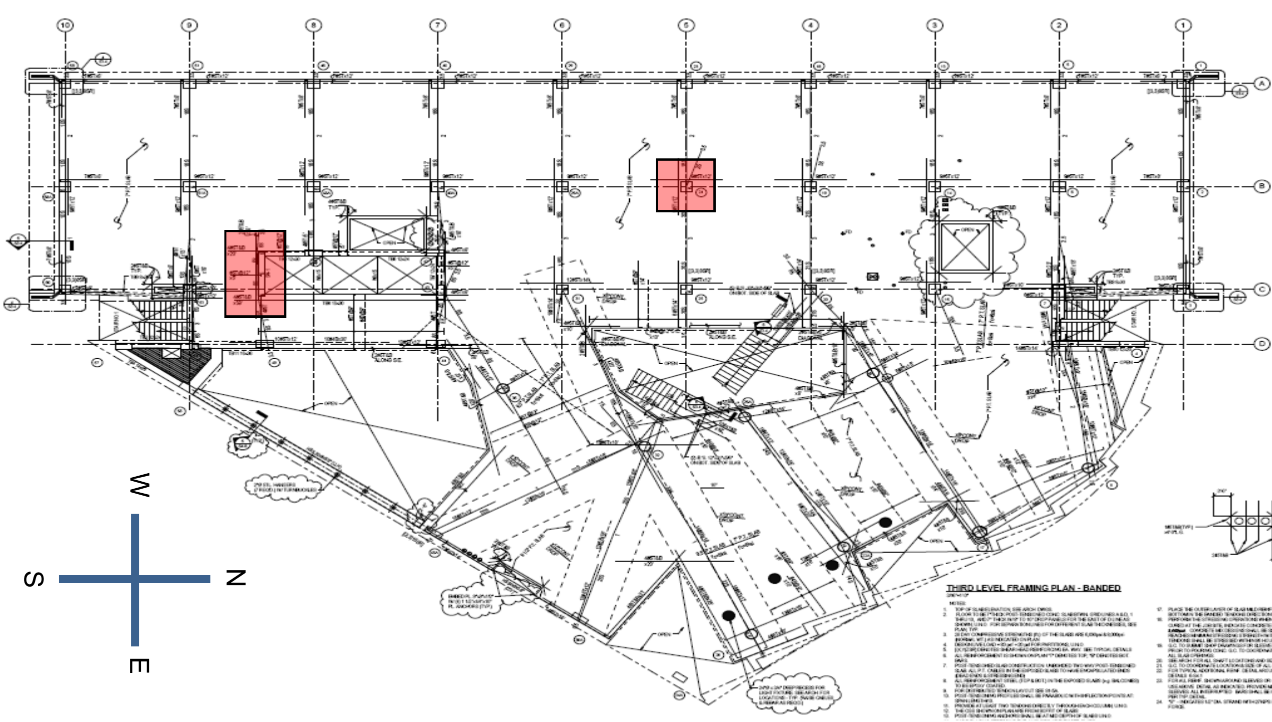


Figure 25: Floor plan with trial shear wall and column highlighted

Structural Redesign

Foundation (cont.)

The building needed to be changed to either a deep or a mat foundation. The author of this thesis chose to use a foundation consisting of caissons. See Appendices H-I for calculations of the caissons and caisson caps for the shear wall and column. The caissons were designed using a shaft and bell configuration. A bedrock bearing strength of 20ksf and a depth to bedrock of 50ft. were assumed due to the author not having geotechnical data. Tables from the CRSI Handbook 2008 were used to determine the size of the caissons and can be found in Appendix J. Figure 26 shows the results of these calculations.

Caisson Foundation							
	Shaft Dia. (ft)	Bell Dia. (ft)	Amount	Cap (ft)	Cap Reinforcement	Caisson Reinforcement	
						Verticle	Ties
Column	3.5	8.5	1	5x5	#4's @ 10' O.C. top and bottom each way	(7) #9's	#3's @ 18" from top to 10ft.
Shear Wall	3	7	2	5x13	#6's @ 10" O.C. top and bottom each way	(7) #8's	#3's @ 16" from top to 10.5ft.

Figure 26: Caisson results for trial locations

Breadth I: Cost/Foundation Schedule Analysis

Proposed Breadth

The upgrades to the UHB will cause a cost increase to the owner. This breadth study will determine the amount of this increased cost for both the shear walls and foundation. Also, the foundation schedule will be analyzed to determine the time increase due to the foundation needing to be changed from spread footings to caissons. The shear wall schedule will not be analyzed due to minimal schedule increase as the shear walls will be formed along with the rest of the building. The cost analysis will be done using RSMeans Building Construction Cost Data 2013.

Caisson Cost vs. Original Spread Footing Cost

A cost comparison was conducted between the building's original spread footing system and the new caisson and caisson cap foundation. The associated costs were determined using multiple sections of RSMeans Building Construction Cost Data 2013. A depth of 50ft. was assumed for the length necessary for each caisson. The final cost comparison can be found in Figure 27. It was estimated that the proposed caisson system will cost $570,954 more than the original system. Detailed calculations can be found in Appendices L-M and cost data can be found in Appendix N.

Proposed System	
Caissons	$ 595,832
Caisson Caps	$ 38,991
Original System	
Spread Footings	$ 63,869
Cost Difference	**$ 570,954**

Figure 27: Foundation cost breakdown

Shear Wall Cost

The shear wall cost was determined using section 03 30 53.40 of RSMeans Building Construction Cost Data 2013. This section includes forming, placement, grade 60 rebar, 3ksi concrete, labor, and equipment. This section if subdivided for 12in. concrete walls with heights of 8ft. and 14ft. so interpolation and extrapolation had to be used for the wall heights found in the UBH. Also, due to the concrete material used in the UBH not equaling 3ksi as intended by section 03 30 53.40, an addition had to be made to the material cost to account for alternate concrete strengths. Sample calculations for this estimation can be found in Appendix K. Figure 28 tallies the total cost of the shear walls. The total cost was found to be $118,694. The cost of the existing shear wall (shear wall 3) was not included due to this calculation being used to determine an increase to the cost of the building. Cost data used for calculations can be found in Appendix N.

Breadth I: Cost/Foundation Schedule Analysis

Shear Wall Cost								
Wall	Floor	Story Height (ft)	Length (ft)	Thickness (ft)	f'c (ksi)	C.Y.	Cost/C.Y.	Total
1,6,7	7	13.5	11	1	4	5.50	$ 397	$ 2,182
	6	12	11	1	4	4.89	$ 363	$ 1,776
	5	12	11	1	6	4.89	$ 384	$ 1,879
	4	12	11	1	6	4.89	$ 384	$ 1,879
	3	12	11	1	8	4.89	$ 462	$ 2,260
	2	12	11	1	8	4.89	$ 462	$ 2,260
	1	18	11	1	10	7.33	$ 650	$ 4,765
2	7	13.5	10	1	4	5.00	$ 397	$ 1,984
	6	12	10	1	4	4.44	$ 363	$ 1,615
	5	12	10	1	6	4.44	$ 384	$ 1,708
	4	12	10	1	6	4.44	$ 384	$ 1,708
	3	12	10	1	8	4.44	$ 462	$ 2,055
	2	12	10	1	8	4.44	$ 462	$ 2,055
	1	18	10	1	10	6.67	$ 650	$ 4,332
4	8	18.5	8	1	4	5.48	$ 467	$ 2,558
	7	13.5	8	1	4	4.00	$ 397	$ 1,587
	6	12	8	1	4	3.56	$ 363	$ 1,292
	5	12	8	1	6	3.56	$ 384	$ 1,367
	4	12	8	1	6	3.56	$ 384	$ 1,367
	3	12	8	1	8	3.56	$ 462	$ 1,644
	2	12	8	1	8	3.56	$ 462	$ 1,644
	1	18	8	1	10	5.33	$ 650	$ 3,465
8,5	8	18.5	10	1	4	6.85	$ 467	$ 3,198
	7	13.5	10	1	4	5.00	$ 397	$ 1,984
	6	12	10	1	4	4.44	$ 363	$ 1,615
	5	12	10	1	6	4.44	$ 384	$ 1,708
	4	12	10	1	6	4.44	$ 384	$ 1,708
	3	12	10	1	8	4.44	$ 462	$ 2,055
	2	12	10	1	8	4.44	$ 462	$ 2,055
	1	18	10	1	10	6.67	$ 650	$ 4,332
							Total	$ 118,694

Figure 28: Shear wall cost breakdown

Breadth I: Cost/Foundation Schedule Analysis

Caisson Schedule vs. Original Spread Footing Schedule

A schedule comparison was conducted between the building's original spread footing system and the new caisson and caisson cap foundation. Through discussions with an industry professional it was determined that 2-3 caissons can be completed in one day. It was also determined that 6-7 spread footings could be completed in one day. Allowing for possible incidentals, the numbers used for this analysis were as follows: 2 caissons per day and 5 spread footings per day. It was then determined that the new foundation system will take an estimated 22 days longer. See Figure 29 for the schedule comparison.

Foundation Schedule			
Type	Amount	No. Completed per Day	Days Needed
Caisson	63	2	32
Spread Footing	48	5	10

Figure 29: Foundation schedule breakdown

Breadth II: Condensation Analysis of Building Envelope

Proposed Breadth

The UHB will be moving from a mixed climate to a primarily cooling climate. The summer design parameters in Orlando will need to be checked against the current wall system to determine if the configuration will need to be updated for its new location due to the high humidity found in Orlando. This will be done by analyzing the wall and determining where condensation will form and position the vapor barrier to correct this issue if necessary. The software The Heat, Air, and Moisture Building Science Toolbox (H.A.M.) will be used to assist in this analysis.

Condensation Point

The condensation point is determined by finding the location in the wall when the wall temperature equals or is below the dew-point temperature. At this point water vapor will condense and could cause problems in a wall system if drainage is not supplied at this point. A typical section of the wall was chosen, modeled in H.A.M., and analyzed for the design parameters of Orlando, Florida. This section can be seen in Figure 30. Upon further investigation after the analysis, it was determined that the point of condensation would not be a problem due to no water vapor being allowed to enter the wall system. This is due to the insulated metal panel and aluminum composite material on either side of the wall. These materials are impermeable to vapor. Therefore, no vapor will ever reach the condensation point. The condensation will form on the metal alloys and drain out of the wall without any chance for mold growth or decay. This wall assembly will not only work in Orlando but will work in almost any environment because no matter which direction the moisture is moving through the wall it will be blocked. Cut sheets from the H.A.M. analysis can be found in Appendix O.

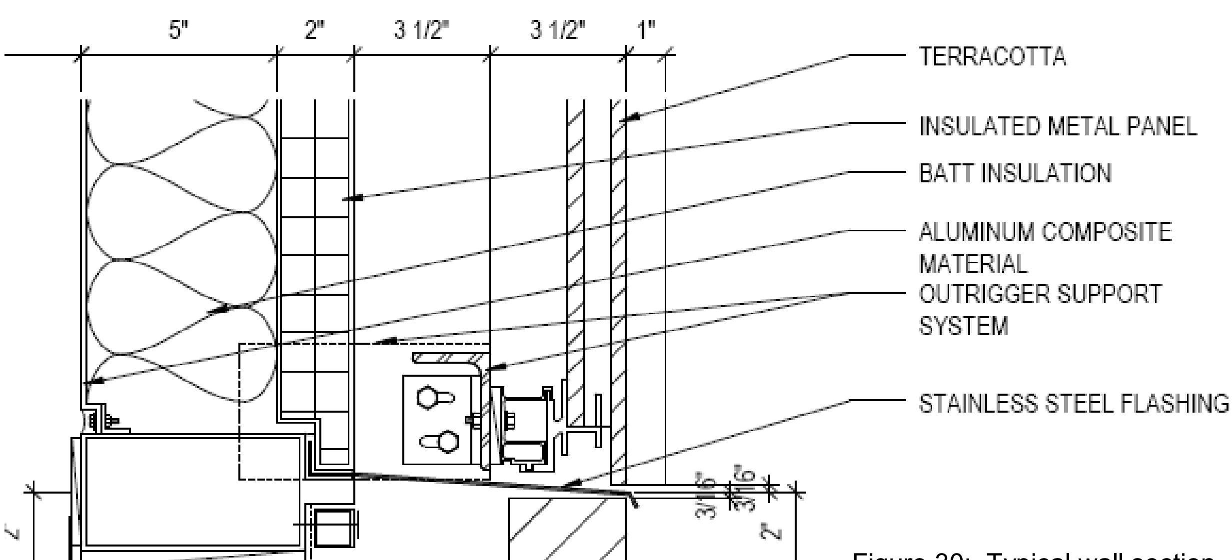


Figure 30: Typical wall section

Breadth II: Condensation Analysis of Building Envelope

R-Value Determination

The R-Value of the wall must be determined to check if it still meets code requirements in its new location. This was done by first determining the building's Climate Zone in the ASHRAE 2005-2008 Handbook. After the Climate zone was determined it was used in conjunction with ASHRAE 90.1to determine if the wall meets Orlando, Florida's energy requirements. The terra cotta was not used in the determination of the R-Value due to it only serving as a rainscreen and not effecting the walls thermal performance. Figure 31 bellows shows the required values in comparison with value of the wall. The wall is more than capable of performing to ASHRAE standards in both locations. The cut sheet from H.A.M. for the R-Value analysis can be found in Appendix O.

R-Values			
Location	Climate Zone	Required R-value (h*ft^2*F/BTU)	R-value of wall assembly (h*ft^2*F/BTU)
Washington, D.C.	4	9.5	25.4
Orlando, Florida	2	5.7	25.4

Figure 31: R-Value requirements

MAE Requirements

ETABS will be used in order to design and analyze the new lateral elements of the UHB. This will incorporate knowledge that was obtained in the AE 530 Computer Modeling of Building Structures coursework. Secondly, the knowledge obtained from AE 542 Building Enclosure Science and Design will be used when determining wind loads for specialized regions such as Orlando, Florida as well as the analysis of the building envelope.

Conclusion

In conclusion, the relocation of the UHB is feasible based on the scope of elements that were analyzed in this report. Other items such as availability of materials needed, mechanical system, and the cost associated with these changes should also be analyzed.

It was determined that the relocation of the building would require the addition of 7 new shear walls and modifications to the existing shear wall. The addition of the shear walls resulted in an increased cost to the building of $118,694. The foundation of the building was also analyzed and determined to be inadequate. The foundation system would not only have to be changed in size but also in type. Caissons were used to carry the new load. It was determined that each shear wall would need two caissons and the columns would each need one. Resulting in a total of 63 new caissons. The cost of the foundations change from spread footing to caissons including caisson caps was determined to be $570,954. This resulted in a total increase to the building's cost of $689,648.

Lastly, a typical section of the building's envelope was checked to determine whether or not it would be able to perform in Orlando, Florida. After analyzing both the walls condensation point and R-value, it was determined that it will be able to perform without change. This is due to the metal panels that enclose the insulated middle portion of the wall.

Appendix A: Wind Force Calculations

EJL | FINAL REPORT | WIND FORCES

ORLANDO, FLORIDA WIND CALC.

OCCUPANCY : III

$V = 145 mph$

IMPORTANCE : 1.15

$GC_{pi} = \pm 0.55$ (PARTIALLY ENCLOSED)

$K_z = K_h = 1.02$

$G = 0.85$

$C_{p\ \text{WINDWARD}} = 0.8$

LEEWARD = $\frac{130}{200} = .64 \quad \therefore C_{pLW} = -0.5$

$K_{zt} = 1.0$

$K_d = 0.85$

$q_z = 0.00256 V^2 K_z K_{zt} K_d I = .00256(145)^2(1.02)(1.0)(0.85)(1.15)$

$= 53.7 psf$

DESIGN WIND PRESSURE (MWFRS)

$p = q_z GC_p - q_z(GC_{pi})$

$p_{WW} = 53.7(0.85)(0.8) - 53.7(\pm .55) = 66 psf$

WEST WALL

$p_{LW} = 53.7(0.85)(-0.5) - 53.7(\pm .55) = -52 psf$

SOUTH WALL

$C_p = -0.3 \qquad \frac{L}{d} = \frac{204}{130} = 1.5$ LEEWARD

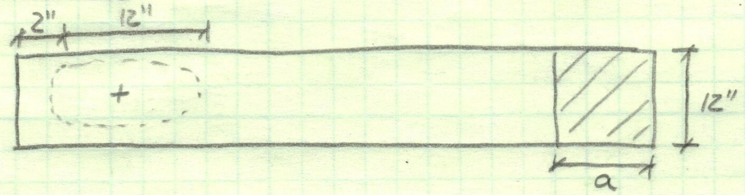
$C_p = 0.8$ WINDWARD

USE q_h

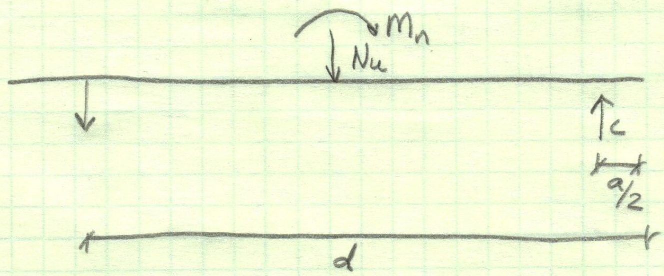
Appendix B: Sample Calculations for Shear Wall Flexure Design

EJL | FINAL REPORT | SHEAR WALL FLEXURE

WALL 2



$N_u = 1310^k$
$M_n = 4900^k$
$d = 9.33 ft$
$l_w = 10 ft$



ASSUME (4) #4's IN BOUNDARY ELEMENT (NEGLECT WEB REINFORCE)

$T = A_s f_y = (4)(.2)(60) = 48^k$

$a = \frac{T + N_u}{.85(f'_c)(b)} = \frac{48 + 1310}{.85(10)(12)} = 13.31 in^2 \leq 0.375d \therefore \phi = 0.9$

$M_n = T(d - a/2) + N_u\left(\frac{l_w - a}{2}\right) = 48\left(111.96 - \frac{13.31}{2}\right) + 1310\left(\frac{120 - 13.31}{2}\right)$

$= 74934^{k \cdot in}$

$\phi M_n = 0.9(74934)\frac{1}{12} = 5620^{k \cdot ft} \geq 4900 k \cdot ft$ ✓OK

USING EXCEL

WALL 3 = $l_w = 8 ft$
$d = 7.33 ft$
$N_u = 282^k$
$M_n = 3000^{k \cdot ft}$

(6) #9's
$\phi M_n = 3238$

WALL 4 = SAME AS 3 $M_n = 2709^{k \cdot ft}$
$N_u = 430^k$

(6) #9's
$\phi M_n = 3700^{k \cdot ft}$

WALL 5 = $l_w = 10^{ft}$
$d = 9.33 ft$
$N_u = 250^k$
$M_n = 4304^{k \cdot ft}$

(6) #10's

$\phi M_n = 4777 k ft$

EJL FINAL REPORT SHEAR WALL FLEXURE

WALL 6 = $l_w = 11\ ft$
$d = 10.33\ ft$
$N_u = 254^k$
$M_n = 5900^{k \cdot ft}$

(8) #10's
$\phi M_n = 6646^{k \cdot ft}$

WALL 8 = $l_w = 10\ ft$
$d = 9.33\ ft$
$N_u = 700^k$
$M_n = 7200^{k \cdot ft}$

(8) #10's

$\phi M_n = 7638^{k \cdot ft}$

Appendix C: Sample Calculations for Shear Wall Shear Design

WALL 2 SHEAR DESIGN $V_u = 176^k$

$$\frac{h_w}{l_w} = \frac{91.5}{10} = 9.15 \rightarrow \text{SLENDER}$$

$$V_c = 3.3\lambda\sqrt{f'_c}\,hd + \frac{N_u d}{4 l_w} = 3.3\sqrt{10{,}000}(12)(9.33)(12) + \frac{1310000(9.33)(12)}{4(120)}$$

$$= 753^k$$

$$V_c = \left[0.6\lambda\sqrt{f'_c} + \frac{l_w\left(1.25\sqrt{f'_c} + 0.2\frac{N_u}{l_w h}\right)}{\left(\frac{M_u}{V_u} - \frac{l_w}{2}\right)}\right]hd$$

$$= \left[0.6\sqrt{10000} + \frac{120\left(1.25\sqrt{10000} + \frac{0.2(1310000)}{120(12)}\right)}{\frac{58808}{176} - \frac{120}{2}}\right](12)(9.33)(12) = 260^k$$

CONTROLS

$\phi V_c = 0.75(260) = 194^k > 176^k$ ∴ NO SHEAR REIN. REQ'D

* SAMPLE CALCULATIONS FOR V_S USE $V_c = 216$

$$V_s \geq \left(\frac{V_u}{\phi} - V_c\right) = \frac{176}{0.75} - 216 = 18.7$$

$$V_s = \frac{A_v f_y d}{S} \Rightarrow A_v = \frac{18.7(12)}{60(9.33)(12)} = 0.033\text{ in}^2$$

$$S_{min} = \begin{cases} 3h = 3(10) = 30 \\ 18'' \\ \frac{l_w}{5} = \frac{120}{5} = 14 \end{cases}$$

USE 12" SPACING

USE #3'S @ 12" O.C. EACH SIDE $A = .22\text{ in}^2 \geq .033$

$\phi(V_s + V_c) \geq V_u$ ✓OK

$$\rho_t = \frac{.22}{(12)(12)} = .0015 \leq .0025 \quad \text{N.G.}$$

HORIZONTAL SHEAR REQ.

$.0025(h)(s) = A_{SH} = 0.0025(12)(12) = 0.54\ in^2/ft$

USE #5's EACH SIDE @ 12" O.C.

$\rho_t = \frac{0.62}{(12)(12)} = 0.0043 \geq .0025$

VERTICLE SHEAR REQ.

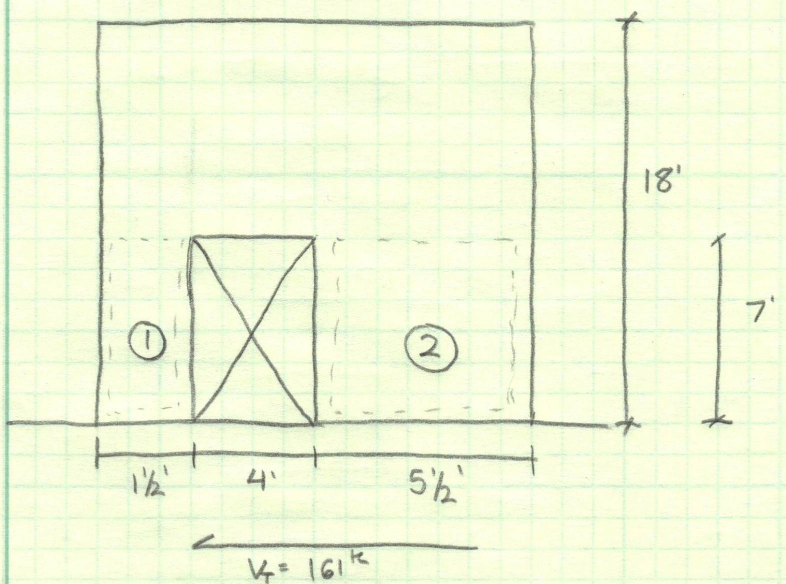
$\rho_v = 0.0025 + 0.5\left(2.5 - \frac{91.5(12)}{120}\right)(0.0031 - 0.0025) = (-)$ USE 0.0025

USE #5's EACH SIDE @ 12" O.C

BETWEEN BOUNDARY ELEMENTS

EJL FINAL REPORT SHEAR WALL OPENING

SHEAR WALL 1 @ OPENING



$V_1 = 55^k$ $V_2 = 106^k$ (FROM ETABS MODEL)

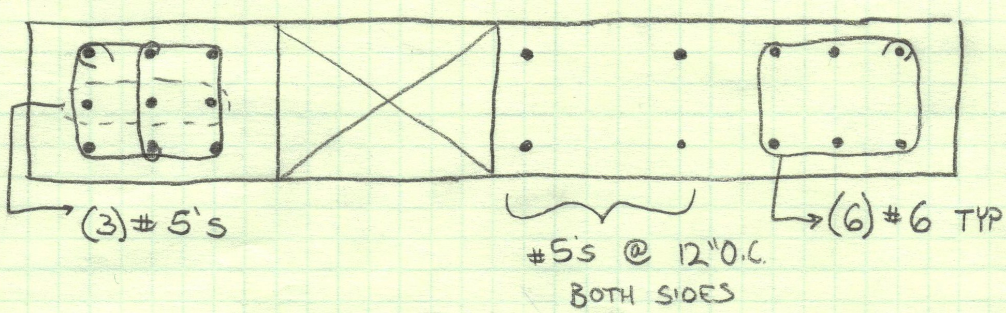
FROM EXCEL WALL 1 CONTROLS OVER WALL 7

$A_{sREQ_1} = .74\text{ in}^2$ A_{sREQ_2} = #5's @ 12in EACH SIDE ✓ OK

(3) #5's

$A_s = .93\text{ in}^2$

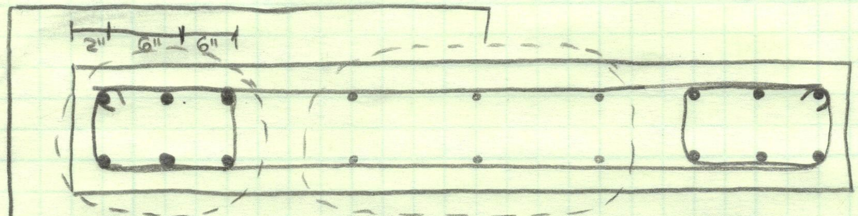
PLACED AS SHOWN BELOW



Appendix D: Typical Shear Wall Detail

EJL FINAL REPORT SW DETAIL

TYPICAL SHEAR WALL DETAIL



BOUNDARY ELEMENT REINFORCING

#5's @ 12" BOTH SIDES

#5's @ 12" HORIZ. BOTH SIDES

#3 TIES FOR BOUNDARY ELEMENTS

Appendix E: Tabulated Calculations for Shear Wall Flexure and Shear

Wall 1 Level 1					
F'c	10	ksi	t=	12	in
Lw=	132	in	Mn=	34681	k*in
Nu=	397.5	k	Vu=	161	k
d=	123.96	in			
(#)	No. Bar	Area bar	Grade		
6	6	0.44	60		
Flexure					
T=	158.4				
a=	5.45	<	46.485		
.9M=	39919.89	>	34681	k*in	

Wall 1 Level 2					
F'c	8	ksi	t=	12	in
Lw=	132	in	Mn=	22109	k*in
Nu=	204	k	Vu=	135	k
d=	123.96	in			
(#)	No. Bar	Area bar	Grade		
6	6	0.44	60		
Flexure					
T=	158.4				
a=	4.441176	<	46.485		
.9M=	29065.07	>	22109	k*in	
Shear					
Vc=	486.9515				
	356.093				
.75Vc=	267.0697				
Vs=	-176.093				
Areq=	-0.28411				
rho=	-0.00197	<	0.0025		

Wall 1 Level 4					
F'c	6	ksi	t=	12	in
Lw=	132	in	Mn=	12581	k*in
Nu=	62	k	Vu=	105	k
d=	123.96	in			
(#)	No. Bar	Area bar	Grade		
6	6	0.44	60		
Flexure					
T=	158.4				
a=	3.601307	<	46.485		
.9M=	20997.36	>	12581	k*in	
Shear					
Vc=	394.7912				
	450.9477				
.75Vc=	296.0934				
Vs=	-254.791				
Areq=	-0.41109				
rho=	-0.00285	<	0.0025		

Wall 1 Level 6					
F'c	4	ksi	t=	12	in
Lw=	132	in	Mn=	5850	k*in
Nu=	16	k	Vu=	61	k
d=	123.96	in			
(#)	No. Bar	Area bar	Grade		
6	6	0.44	60		
Flexure					
T=	158.4				
a=	4.27451	<	46.485		
.9M=	18286.67	>	5850	k*in	
Shear					
Vc=	314.2171				
	588.85				
.75Vc=	235.6629				
Vs=	-232.884				
Areq=	-0.37574				
rho=	-0.00261	<	0.0025		

Wall 1.1 Shear					
F'c	10	ksi	t=	12	in
Lw=	18	in	Mn=	34681.73	k*in
Nu=	397.5	k	Vu=	55	k
d=	16	in			
Shear					
Vc=	151.6933				
	14.26141				
.75Vc=	10.69606				
Vs=	59.07192				
Areq=	0.738399				
rho=	0.005128	<	0.0025		

Wall 1.2 Shear					
F'c	10	ksi	t=	12	in
Lw=	66	in	Mn=	34681.73	k*in
Nu=	397.5	k	Vu=	106	k
d=	64	in			
Shear					
Vc=	349.8036				
	84.91256				
.75Vc=	63.68442				
Vs=	56.42078				
Areq=	0.176315				
rho=	0.001224	<	0.0025		

Wall 2 Level 1					
F'c	10	ksi	t=	12	in
Lw=	120	in	Mn=	58808.99	k*in
Nu=	1310	k	Vu=	175	k
d=	111.96	in			
(#)	No. Bar	Area bar	Grade		
6	9	1	60		
Flexure					
T=	360				
a=	16.37255	<	41.985		
.9M=	94711.07	>	58808	k*in	
Shear					
Vc=	748.9191				
	259.8761				
.75Vc=	194.9071				
Vs=	-26.5428				
Areq=	-0.04741				
rho=	-0.00033	<	0.0025		

Wall 2 Level 2					
F'c	8	ksi	t=	12	in
Lw=	120	in	Mn=	30089.9	k*in
Nu=	782.65	k	Vu=	108	k
d=	111.96	in			
(#)	No. Bar	Area bar	Grade		
6	9	1	60		
Flexure					
T=	360				
a=	14.00306	<	41.985		
.9M=	71337.87	>	30089.9	k*in	
Shear					
Vc=	579.1078				
	234.7205				
.75Vc=	176.0404				
Vs=	-90.7205				
Areq=	-0.16206				
rho=	-0.00113	<	0.0025		

Wall 2 Level 4					
F'c	6	ksi	t=	12	in
Lw=	120	in	Mn=	14967.8	k*in
Nu=	421.75	k	Vu=	78	k
d=	111.96	in			
(#)	No. Bar	Area bar	Grade		
6	9	1	60		
Flexure					
T=	360				
a=	12.77369	<	41.985		
.9M=	54555.91	>	14967.8	k*in	
Shear					
Vc=	441.7996				
	252.3964				
.75Vc=	189.2973				
Vs=	-148.396				
Areq=	-0.26509				
rho=	-0.00184	<	0.0025		

Wall 2 Level 6					
F'c	4	ksi	t=	12	in
Lw=	120	in	Mn=	5076.73	k*in
Nu=	170.97	k	Vu=	33	k
d=	111.96	in			
(#)	No. Bar	Area bar	Grade		
6	9	1	60		
Flexure					
T=	360				
a=	13.01397	<	41.985		
.9M=	42397.91	>	5076.73	k*in	
Shear					
Vc=	320.2852				
	227.6034				
.75Vc=	170.7025				
Vs=	-183.603				
Areq=	-0.32798				
rho=	-0.00228	<	0.0025		

Wall 3 Level 1					
F'c	10	ksi	t=	12	in
Lw=	96	in	Mn=	35747.4	k*in
Nu=	282	k	Vu=	95	k
d=	87.96	in			
(#)	No. Bar	Area bar	Grade		
6	9	1	60		
Flexure					
T=	360				
a=	6.294118	<	32.985		
.9M=	38863.07	>	35747.4	k*in	
Shear					
Vc=	412.9172				
	117.0254				
.75Vc=	87.76905				
Vs=	9.641267				
Areq=	0.021922				
rho=	0.000152	<	0.0025		

Wall 3 Level 2					
F'c	8	ksi	t=	12	in
Lw=	96	in	Mn=	12170	k*in
Nu=	193	k	Vu=	42	k
d=	87.96	in			
(#)	No. Bar	Area bar	Grade		
6	9	1	60		
Flexure					
T=	360				
a=	6.776961	<	32.985		
.9M=	35150.19	>	12170	k*in	
Shear					
Vc=	355.7574				
	117.5492				
.75Vc=	88.16192				
Vs=	-61.5492				
Areq=	-0.13995				
rho=	-0.00097	<	0.0025		

Wall 3 Level 4					
F'c	6	ksi	t=	12	in
Lw=	96	in	Mn=	6739	k*in
Nu=	161	k	Vu=	21	k
d=	87.96	in			
(#)	No. Bar	Area bar	Grade		
6	9	1	60		
Flexure					
T=	360				
a=	8.513072	<	32.985		
.9M=	33458.35	>	6739	k*in	
Shear					
Vc=	306.6878				
	95.38563				
.75Vc=	71.53922				
Vs=	-67.3856				
Areq=	-0.15322				
rho=	-0.00106	<	0.0025		

Wall 3 Level 6					
F'c	4	ksi	t=	12	in
Lw=	96	in	Mn=	3051	k*in
Nu=	101	k	Vu=	16	k
d=	87.96	in			
(#)	No. Bar	Area bar	Grade		
6	9	1	60		
Flexure					
T=	360				
a=	11.29902	<	32.985		
.9M=	30518.26	>	3051	k*in	
Shear					
Vc=	243.4332				
	108.649				
.75Vc=	81.48675				
Vs=	-87.3157				
Areq=	-0.19853				
rho=	-0.00138	<	0.0025		

Wall 4 Level 1					
F'c	10	ksi	t=	12	in
Lw=	96	in	Mn=	32513	k*in
Nu=	431	k	Vu=	66	k
d=	87.96	in			
(#)	No. Bar	Area bar	Grade		
6	9	1	60		
Flexure					
T=	360				
a=	7.754902	<	32.985		
.9M=	44357.88	<	32513	k*in	
Shear					
Vc=	447.0475				
	108.872				
.75Vc=	81.65398				
Vs=	-20.872				
Areq=	-0.07119				
rho=	-0.00049	<	0.0025		

Wall 4 Level 2					
F'c	8	ksi	t=	12	in
Lw=	96	in	Mn=	14534	k*in
Nu=	212	k	Vu=	37	k
d=	87.96	in			
(#)	No. Bar	Area bar	Grade		
6	9	1	60		
Flexure					
T=	360				
a=	7.005515	<	32.985		
.9M=	35840.2	<	14534	k*in	
Shear					
Vc=	360.0294				
	100.2992				
.75Vc=	75.22437				
Vs=	-50.9658				
Areq=	-0.17383				
rho=	-0.00121	<	0.0025		

Wall 4 Level 4					
F'c	6	ksi	t=	12	in
Lw=	96	in	Mn=	11070	k*in
Nu=	175	k	Vu=	77	k
d=	87.96	in			
(#)	No. Bar	Area bar	Grade		
6	9	1	60		
Flexure					
T=	360				
a=	8.74183	<	32.985		
.9M=	33954.44	<	11070	k*in	
Shear					
Vc=	309.8947				
	183.6529				
.75Vc=	137.7397				
Vs=	-80.9863				
Areq=	-0.27622				
rho=	-0.00192	<	0.0025		

Wall 4 Level 6					
F'c	4	ksi	t=	12	in
Lw=	96	in	Mn=	3684	k*in
Nu=	127	k	Vu=	29	k
d=	87.96	in			
(#)	No. Bar	Area bar	Grade		
6	9	1	60		
Flexure					
T=	360				
a=	11.93627	<	32.985		
.9M=	31369.61	<	3684	k*in	
Shear					
Vc=	249.3889				
	169.6814				
.75Vc=	127.261				
Vs=	-131.015				
Areq=	-0.44684				
rho=	-0.0031	<	0.0025		

Wall 5 Level 1					
F'c	10	ksi	t=	12	in
Lw=	120	in	Mn=	51654	k*in
Nu=	249	k	Vu=	121	k
d=	111.96	in			
(#)	No. Bar	Area bar	Grade		
6	10	1.27	60		
Flexure					
T=	457.2				
a=	6.925784	<	41.985		
.9M=	57326.06	>	51654		
Shear					
Vc=	501.4945				
	150.7504				
.75Vc=	113.0628				
Vs=	10.58291				
Areq=	0.028357				
rho=	0.000197	<	0.0025		

Wall 5 Level 2					
F'c	8	ksi	t=	12	in
Lw=	120	in	Mn=	26026	k*in
Nu=	207	k	Vu=	57	k
d=	111.96	in			
(#)	No. Bar	Area bar	Grade		
6	10	1.27	60		
Flexure					
T=	457.2				
a=	8.139706	<	41.985		
.9M=	54814.42	>	26026		
Shear					
Vc=	444.8374				
	129.2379				
.75Vc=	96.92843				
Vs=	-53.2379				
Areq=	-0.14265				
rho=	-0.00099	<	0.0025		

Wall 5 Level 4					
F'c	6	ksi	t=	12	in
Lw=	120	in	Mn=	9566	k*in
Nu=	165	k	Vu=	33	k
d=	111.96	in			
(#)	No. Bar	Area bar	Grade		
6	10	1.27	60		
Flexure					
T=	457.2				
a=	10.16667	<	41.985		
.9M=	52132.74	>	9566		
Shear					
Vc=	381.9127				
	146.4201				
.75Vc=	109.8151				
Vs=	-102.42				
Areq=	-0.27444				
rho=	-0.00191	<	0.0025		

Wall 5 Level 6					
F'c	4	ksi	t=	12	in
Lw=	120	in	Mn=	3594	k*in
Nu=	120	k	Vu=	26	k
d=	111.96	in			
(#)	No. Bar	Area bar	Grade		
6	10	1.27	60		
Flexure					
T=	457.2				
a=	14.14706	<	41.985		
.9M=	48874.74	>	3594		
Shear					
Vc=	308.3965				
	248.2556				
.75Vc=	186.1917				
Vs=	-213.589				
Areq=	-0.57232				
rho=	-0.00397	<	0.0025		

Wall 6 Level 1					
F'c	10	ksi	t=	12	in
Lw=	132	in	Mn=	71057	k*in
Nu=	394	k	Vu=	210	k
d=	123.96	in			
(#)	No. Bar	Area bar	Grade		
8	9	1	60		
Flexure					
T=	480				
a=	8.568627	<	46.485		
.9M=	73584.28	>	71057	k*in	
Shear					
Vc=	583.3821				
	215.2289				
.75Vc=	161.4217				
Vs=	64.77109				
Areq=	0.104503				
rho=	0.000726	<	0.0025		

Wall 6 Level 2					
F'c	8	ksi	t=	12	in
Lw=	132	in	Mn=	41790	k*in
Nu=	253	k	Vu=	166	k
d=	123.96	in			
(#)	No. Bar	Area bar	Grade		
8	9	1	60		
Flexure					
T=	480				
a=	8.982843	<	46.485		
.9M=	65615.93	>	41790	k*in	
Shear					
Vc=	498.4554				
	231.7842				
.75Vc=	173.8381				
Vs=	-10.4508				
Areq=	-0.01686				
rho=	-0.00012	<	0.0025		

Wall 6 Level 4					
F'c	6	ksi	t=	12	in
Lw=	132	in	Mn=	28149	k*in
Nu=	193	k	Vu=	102	k
d=	123.96	in			
(#)	No. Bar	Area bar	Grade		
8	9	1	60		
Flexure					
T=	480				
a=	10.99673	<	46.485		
.9M=	61684.56	>	28149	k*in	
Shear					
Vc=	425.5464				
	182.4668				
.75Vc=	136.8501				
Vs=	-46.4668				
Areq=	-0.07497				
rho=	-0.00052	<	0.0025		

Wall 6 Level 6					
F'c	4	ksi	t=	12	in
Lw=	132	in	Mn=	5286	k*in
Nu=	105	k	Vu=	35	k
d=	123.96	in			
(#)	No. Bar	Area bar	Grade		
8	9	1	60		
Flexure					
T=	480				
a=	14.33824	<	46.485		
.9M=	56013.18	>	5286	k*in	
Shear					
Vc=	335.1119				
	269.6252				
.75Vc=	202.2189				
Vs=	-222.958				
Areq=	-0.35973				
rho=	-0.0025	<	0.0025		

Wall 7 Level 1					
F'c	10	ksi	t=	12	in
Lw=	132	in	Mn=	34599	k*in
Nu=	762	k	Vu=	133	k
d=	123.96	in			
(#)	No. Bar	Area bar	Grade		
6	6	0.44	60		
Flexure					
T=	158.4				
a=	9.023529	<	46.485		
.9M=	59197.17	>	34599	k*in	

Wall 7 Level 2					
F'c	8	ksi	t=	12	in
Lw=	132	in	Mn=	26398	k*in
Nu=	290	k	Vu=	125	k
d=	123.96	in			
(#)	No. Bar	Area bar	Grade		
6	6	0.44	60		
Flexure					
T=	158.4				
a=	5.495098	<	46.485		
.9M=	33788.94	>	26398	k*in	
Shear					
Vc=	507.1419				
	280.5572				
.75Vc=	210.4179				
Vs=	-113.891				
Areq=	-0.18375				
rho=	-0.00128	<	0.0025		

Wall 7 Level 4					
F'c	6	ksi	t=	12	in
Lw=	132	in	Mn=	21141	k*in
Nu=	217	k	Vu=	102	k
d=	123.96	in			
(#)	No. Bar	Area bar	Grade		
6	6	0.44	60		
Flexure					
T=	158.4				
a=	6.133987	<	46.485		
.9M=	29525.32	>	21141	k*in	
Shear					
Vc=	431.1809				
	241.7998				
.75Vc=	181.3498				
Vs=	-105.8				
Areq=	-0.1707				
rho=	-0.00119	<	0.0025		

Wall 7 Level 6					
F'c	4	ksi	t=	12	in
Lw=	132	in	Mn=	10442	k*in
Nu=	78	k	Vu=	91	k
d=	123.96	in			
(#)	No. Bar	Area bar	Grade		
6	6	0.44	60		
Flexure					
T=	158.4				
a=	5.794118	<	46.485		
.9M=	21688.56	>	10442	k*in	
Shear					
Vc=	328.7731				
	414.5561				
.75Vc=	246.5798				
Vs=	-207.44				
Areq=	-0.33469				
rho=	-0.00232	<	0.0025		

Wall 7.1 Shear					
F'c	10	ksi	t=	12	in
Lw=	18	in	Mn=	34599.51	k*in
Nu=	762.77	k	Vu=	45	k
d=	16	in			
Shear					
Vc=	232.8644				
	15.30069				
.75Vc=	11.47552				
Vs=	44.69931				
Areq=	0.558741				
rho=	0.00388	<	0.0025		

Wall 7.2 Shear					
F'c	10	ksi	t=	12	in
Lw=	66	in	Mn=	34599.51	k*in
Nu=	762.77	k	Vu=	88	k
d=	64	in			
Shear					
Vc=	438.3539				
	90.77883				
.75Vc=	68.08412				
Vs=	26.55451				
Areq=	0.082983				
rho=	0.000576	<	0.0025		

Wall 8 Level 1					
F'c	10	ksi	t=	12	in
Lw=	120	in	Mn=	85751	k*in
Nu=	697.1	k	Vu=	360	k
d=	111.96	in			
(#)	No. Bar	Area bar	Grade		
8	10	1.27	60		
Flexure					
T=	609.6				
a=	12.81078	<	41.985		
.9M=	91536.2	>	85751	k*in	
Shear					
Vc=	605.9602				
	281.3004				
.75Vc=	210.9753				
Vs=	198.6996				
Areq=	0.354947				
rho=	0.002465	<	0.0025		

Wall 8 Level 2					
F'c	8	ksi	t=	12	in
Lw=	120	in	Mn=	49872	k*in
Nu=	216	k	Vu=	263	k
d=	111.96	in			
(#)	No. Bar	Area bar	Grade		
8	10	1.27	60		
Flexure					
T=	609.6				
a=	10.11765	<	41.985		
.9M=	69330.83	>	49872	k*in	
Shear					
Vc=	446.9367				
	248.467				
.75Vc=	186.3503				
Vs=	102.1996				
Areq=	0.182565				
rho=	0.001268	<	0.0025		

Wall 8 Level 4					
F'c	6	ksi	t=	12	in
Lw=	120	in	Mn=	13381	k*in
Nu=	154	k	Vu=	171	k
d=	111.96	in			
(#)	No. Bar	Area bar	Grade		
8	10	1.27	60		
Flexure					
T=	609.6				
a=	12.47712	<	41.985		
.9M=	65454.34	>	13381	k*in	
Shear					
Vc=	379.3469				
	1106.668				
.75Vc=	284.5102				
Vs=	-151.347				
Areq=	-0.27036				
rho=	-0.00188	<	0.0025		

Wall 8 Level 6					
F'c	4	ksi	t=	12	in
Lw=	120	in	Mn=	11329	k*in
Nu=	61	k	Vu=	44	k
d=	111.96	in			
(#)	No. Bar	Area bar	Grade		
8	10	1.27	60		
Flexure					
T=	609.6				
a=	16.43627	<	41.985		
.9M=	59759.76	>	11329	k*in	
Shear					
Vc=	294.6347				
	122.4427				
.75Vc=	91.83201				
Vs=	-63.776				
Areq=	-0.11393				
rho=	-0.00079	<	0.0025		

Appendix F: Spread Footing Calculation for Column

SQUARE FOOTING DESIGN FOR COLUMN

UNFACTORED: $P = 415^k$ $M = 300^{k \cdot ft}$ $(.6D + W)$ $q_a = 8\,ksf$

$$q_a = \frac{P}{B^2} \pm \frac{M(6)}{B^3} = \frac{415}{B^2} + \frac{300(6)}{B^3} = 8 \qquad B = 9\,ft$$

$P = 1100^k$ $(D + L + L_R)$

$$A = 1100^k / 8\,ksf = 137\,ft^2 \qquad \sqrt{137} = \boxed{12\,ft}$$

KERN CHECK 12x12

$$\frac{B}{6} = 2 \qquad e = 300^{k \cdot ft} / 415^k = .722 < 2 \checkmark \text{ IN KERN}$$

OVERTURNING

$M_{OVER} = 300^{k \cdot ft}$

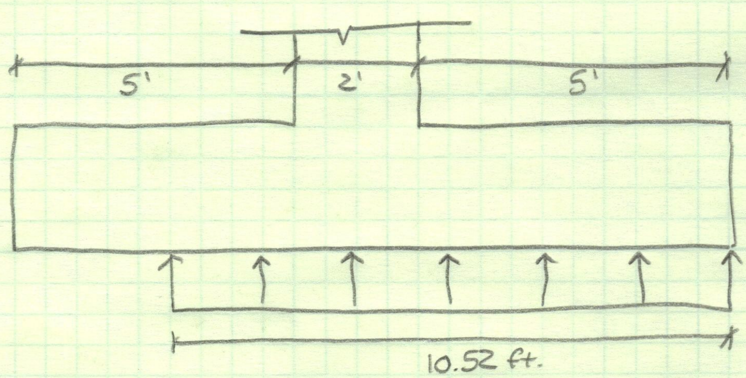
$M_{RESIST} = 415^k (12/2) = 2490^{k \cdot ft}$ $\quad F.O.S = \frac{2490}{300} = 8$

FACTORED = $P = 624^k$ $M = 460^{k \cdot ft}$ $(0.9D + 1.6W)$

$P = 1481^k$ $(1.2D + 1.6L + 0.5L_R)$

$B' = 12 - 2(0.74) = 10.52\,ft$

$A' = 10.52(12) = 126\,ft^2$ $\quad q = \frac{624^k}{126\,ft^2} = 5\,ksf$



PUNCHING SHEAR

$$4d^2 + 2d(b+c) = \frac{P_u}{V_c}$$

$$V_c = \phi 4\sqrt{f'c} = 0.75(4)\sqrt{5000} = 212\ psi$$

$$4d^2 + 2d(12'' + 24'') = \frac{148000}{212}$$

$$d = 34''$$

$$h = 34 + 3 - {}^{1.128}/_2 = 38'' \rightarrow d = 34.4$$

MOMENT

$$l = \frac{12 - {}^{24}/_{12}}{2} = 5 \qquad M = 5ksf\left(12ft \times {}^{5^2}/_2\right) = 750^{k\,ft}$$

ASSUME $\phi = 0.9$ $j = 0.95$

$$A_s = \frac{750 \times 12}{0.9(60)(0.95)(34.4)} = 5.1\ in^2/12 = .425\ in^2/ft$$

#6's @ 10" O.C. $A_s = 0.528\ in^2/ft$ $\beta_1 = 0.80$

$$a = \frac{(0.528)(12)(60)}{0.85(5)(12)(12)} = 0.621 \qquad c = {}^{0.621}/_{0.80} = 0.77$$

$$\varepsilon_s = \frac{.003}{0.77}(34.4 - 0.77) = 0.131 \geq 0.005 \therefore \phi = 0.9$$

$$A_{s_{MIN}} = 0.0018(12)(34.4) = 0.74\ in^2/ft$$

$\boxed{\text{\#6's @ 10" O.C TOP AND BOTTOM EACH WAY}}$

Appendix G: Spread Footing Calculation for Shear Wall

EJL | FINAL REPORT | SHEAR WALL FOOTING

SHEAR WALL FOOTING DESIGN $L = 8ft$ WALL 2

UNFACTORED $P = 557^k$ L+D

$P = 190^k$ $M = 1240^{k \cdot ft}$ .6D+W

TRY 6×? FOOTING

$8 = \frac{557^k}{6B}$ $B = 12^{ft}$ $8 = \frac{190}{6B} \pm \frac{1240(6)}{6B^2}$

$\boxed{B = 15ft}$

KURN

$1240^{k \cdot ft} / 190^k = 6.5'$ ∴ NOT IN KURN

FACTORED $P = 746^k$ 1.2D + 1.6L + .5L_R

$P = 282^k$ $M = 2505^{k \cdot ft}$ 0.6D + 1.6W

TRY 6×18 FOOTING (STILL NOT IN CURN)

$e = 2505/282 = 8.8\ ft$

$B' = 18 - 2(8.8) = .4'$ $q = \frac{282}{2.4} = 117.5$ WILL CONTINUE USING $q = 8^{ksf}$ AND RESIST THE REMAINING MOMENT WITH FOOTING WEIGHT

$A' = 6(.4) = 2.4\ ft^2$

ASSUME $d = 30$ IN

MOMENT

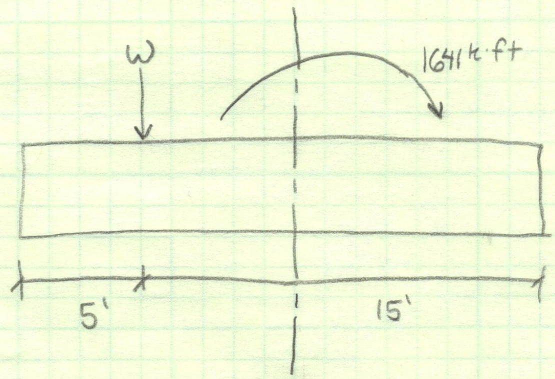
$l = \frac{18-8}{2} = 5\ ft$ $M = 8^{ksf}\left(6 \times \frac{5^2}{2}\right) = 120^{k \cdot ft}$

MAKE FOOTING 6×20

$l = 6ft$ $M = 864\ k \cdot ft$

EJL FINAL REPORT SHEAR WALL FOOTING

WEIGHT OF FOOTING MUST RESIST $M = 2505 - 864 = 1641^{k \cdot ft}$



MOMENT ABOUT CENTER

½ OF FOUNDATION WEIGHT ASSUMED TO RESIST

$$M_c = 1641 - \frac{(5\,ft)(150\,{}^{lb}/_{ft^3})(10ft)(6ft)(h)}{1000} = 0$$

$$\boxed{h = 36\,ft}$$

SHEAR WILL NOT CONTROL BY INSPECTION

REINFORCEMENT

ASSUME $\phi = 0.9$ $j = .95$

$$A_s = \frac{864^{k \cdot ft} \times 6}{(0.9)(60)(0.95)(428)} = .23\ in^2$$

TEMP & SHRINK WILL CONTROL

$$d = 36(12) - 3\tfrac{1}{2}'' = 428.5''$$

$$A_{s\,min} = .0018(12'')(428.5) = 9.25\ in^2/ft$$

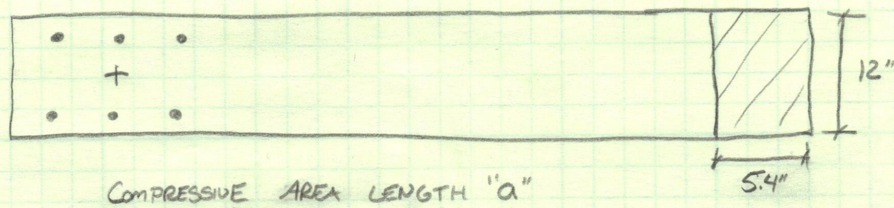
21 LAYERS OR #6'S SPACED @ 12" EACH WAY

Appendix H: Caisson/Caisson Cap Calculations for Shear Wall

ESL FINAL REPORT SHEAR WALL CAISSONS

SHEAR WALL CAISSON

BED ROCK BEARING = 20^{ksf}



COMPRESSIVE AREA LENGTH "a"
FROM SHEAR WALL DESIGN TABLE
SHEAR WALL #3 ASD

.6D+W CONTROLLED

AXIAL = $190^k/2 = 95^k$

AXIAL ENDUCED BY MOMENT = $(10\,ksi)(5.4)(12) = 648^k$
↳ CONCRETE STR

TOTAL COMP. FORCE = 743^k

USING $3^{ksi} = f'_c$

USING DRILLED CAISSONS ∴ $f_c = 0.30 f'_c$

USING TABLE IN CRSI HANDBOOK 2008 Pg 13-48

SHAFT DIAMETER = 3'0" → GOOD FOR CONSTRUCTABILITY INSPECTION

BELL DIAMETER = 7'0"

EJL FINAL REPORT SHEAR WALL CAISSON

CAISSON CAP SHEAR WALL 3 TRY 5 × 13

PUNCHING SHEAR

$V_c = \left(2+\frac{4}{\beta_c}\right)\lambda\sqrt{f'_c}\, b_o d$

CRITICAL SECTION WILL BE TAKEN AS PERIMETER OFF WALL FOR CONSERVATY

CONTROLS

$b_o = 2(12) + 2(8)(12) = 216_{IN}$ (WALL) $b_o = (2)\pi(36) = 226_{IN}$

LOAD CASE D+L = 560^k
TO CREATE MAX AXIAL

$f'_c = 5^{ksi}$ $\beta = 0.80$ $\lambda = 1.0$

$560^k = \left(2 + 4/.80\right)\sqrt{5000}\,(216)(d)$

$d = 5.46''$

ONE WAY SHEAR

$V_c = 2\lambda\sqrt{f'_c}\, b_w d$

$b_w = 60_{IN}$

$560^k = 2\sqrt{5000}\,(60)(d)$

$d = 26.4''$

USING #6'S

$h = 27 + 3 + .75/2 = 31''$

EJL | FINAL REPORT | SHEAR WALL CAISSON

FLEXURE (TREATING CAP AS A BM SPANING B/W CAISSONS FIXED-FIXED)

$560^k / 8ft = 70^{klf}$

$M = \frac{wl^2}{12} = \frac{70(8-2(1.5))^2}{12} = 146^{k \cdot ft}$

$A_s = \frac{M}{4d} = \frac{146}{4(26.4)} = 1.38$ $\quad \rho = \frac{1.38}{(60)(26.4)} = .00087 \leq .0018$

$1.38 / 5ft = .276 in^2/ft$ MIN A_s CONTROLS

$A_{s\,mn} = .0018(12)(31'') = .66\,in^2/ft$

#6's @ 10' O.C TOP & BOTTOM BOTH WAYS

$A_s = .73 in^2/ft$

$M_n = \phi A_s f_y (d - a/2)$

$A_s = 6(.44) = 2.64\,in^2$

$a = \frac{2.64(60)}{.85(5)(26.4)} = 1.41$

$M_n = 2.64(60)(26.4 - 1.41/2) = 4070\,k \cdot in = 339^{k \cdot ft}$

$\phi M_n = 0.9(339) = 305\,k \cdot ft > 146^{k \cdot ft}$ ✓ OK

$c = a/B_1 = 1.41/.80 = 1.76$

$\varepsilon_s = \frac{.003}{1.76}(26.4 - 1.76) = .042 > .005 \therefore \phi = 0.9$

Appendix I: Caisson/Caisson Cap Calculations for Column

EJL | FINAL REPORT | COLUMN CAISSON

COLUMN CAISSON

$P = 1100^k$ D+L (MAX AXIAL LOAD)

CAISSON

3^{ksi} BRG = 20 ksf

$\varnothing = 3'\text{-}6''$ BELL = $8'\text{-}6''$

CAP

PUNCHING SHEAR

$$V_c = \left(2 + \frac{4}{\beta_c}\right) \lambda \sqrt{f'_c}\, b_o d$$

$\beta_1 = 0.80$ $b_o = \pi(42'') = 132''$

$$1100^k = \left(2 + \frac{4}{0.80}\right)\sqrt{5000}\,(132)d$$

$d = 16.8''$

$h = 20''$

$A_{s\,min} = 0.0018(12)(20'') = .43\ in^2/ft$

#4's @ 10" O.C TOP & BOTTOM

Appendix J: CRSI Design Tables (2008 Handbook)

Appendix K: Shear Wall Cost Analysis

EJL | FINAL REPORT | SHEAR WALL COST

SHEAR WALL COST

RS MEANS BUILDING CONSTRUCTION COST DATA 2013 : 03 30 53.40

PER C.Y

12" THK 3ksi		MAT.	LABOR	EQUIP
h = 8ft		140	139	11.65
14ft		150	223	18.70
18ft		157	279	25.75
12ft		147	195	16.35
13.5ft	USE	14ft		
18.5 ft	USE	18ft		

03 30 53.40 : INCLUDES, GR 60 STL, FORMWORK, PLACEMENT, CONC., FINISHING

03 31 05.35

PER C.Y.

	COST
3ksi	97
10 ksi	285
8 ksi	201
6ksi	123
4 ksi	102

Ex/ 18ft WALL 10ksi

TOTAL COST = (157 - 97 + 285) + 279 + 25.75 = \$ 649.75/C.Y

Appendix L: Caisson/Caisson Cap Cost Analysis

EJL FINAL REPORT CAISSON COST

CAISSON COST

USING RSMEANS 2013 BUILDING CONSTRUCTION COST DATA

SECTION 31. 63. 26.13

SHEAR WALLS → 2 CAISSONS EACH × 8 WALLS = 16

COLUMNS → 1 CAISSON EACH × 47 COLUMNS = 47

63

LOOKING AT AVERAGE COLUMN/SHEAR WALL LOADINGS

25% → 3'0" Ø SHAFT 7'0" Ø BELL

75% → 3'6" Ø SHAFT 8'6" Ø BELL

SITES AROUND ORLANDO INDICATE PUMPING WILL BE NECESSARY ACCORDING TO WATERDATA.USGS.GOV

	16 → 3'0" Ø	47 → 3'6" Ø
SHAFT	\$ 116.50 × 50 × 16	\$ 152.25 × 50 × 16
BELL	\$ 1756 × 16	\$ 2540 × 16
	\$ 121,296	\$ 162,440

TOTAL = \$ 283,736 FOR 50ft LENGTH

INCLUDES EXCAVATION, 50lb REINFORCING, CASING, PUMPING

INSPECTION = \$ 310 × 63 = \$19,530

MOBILIZATION = \$ 2165 × 63 = \$136,269

LOAD HAUL EXCESS = \$ 5.53 × 28263 = \$156,297

= \$ 312,096

5654

CAISSON TOTAL = \$ 595,832

EJL | FINAL REPORT | CAISSON COST

$ft^3 = .037 yd^3$

CAISSON CAP

30 53.40 SPREAD FOOTINGS

SW

(2) 5x13 (3) 5x16 (3) 5x15 ESTIMATED 35" THK

ALL ≥ 5 C.Y

COLUMNS

(47) 5x5 ESTIMATED 20" THK

1CY < ALL < 5CY

3^{ksi} CONC = 97/CY

5^{ksi} = 105/CY

INCREASE MAT. BY 8/CY

$$\text{VOLUME} = \left[(2)(65) + 3(80) + 3(75)\right]\frac{35}{12} + \left[47(25)\right]\frac{20}{12}$$

1735 ft^3 + 1958 ft^3

64 yd^3 + 72 yd^3

COST = 64(251.76) + 72(317.76) = $38,991

Appendix M: Existing Spread Footing Cost

EJL FINAL REPORT SPREAD FOOTING COST

CURRENT FOUNDATION

		ft^2
5.5 × 5.5	𝍸	151
6 × 8	𝍸 |	288
6 × 7	|||	126
7×7	𝍸 ||	343
3×6	𝍸 |||	144
6×6	||||	144
9+9	||||	324
8×7	||||	224
5×4	𝍸 |	120
		1864

AVG = 35" THK → 5436 ft^3 (.037)

201 yd^3

USE 1 CY < ALL < 5CY SOME ARE MORE BUT THIS WILL BE A CONSERVATIVE EST.

317.76 (201) = \$63,869

Appendix N: RSMeans Building Construction Cost Data 2013

Appendix O: Condensation/R-Value Analysis

TABLE 5.5-4 Building Envelope Requirements for Climate Zone 4 (A, B, C)*

Opaque Elements	Nonresidential		Residential		Semiheated	
	Assembly Maximum	Insulation Min. R-Value	Assembly Maximum	Insulation Min. R-Value	Assembly Maximum	Insulation Min. R-Value
Roofs						
Insulation Entirely above Deck	U-0.048	R-20.0 c.i.	U-0.048	R-20.0 c.i.	U-0.173	R-5.0 c.i.
Metal Building[a]	U-0.055	R-13.0 + R-13.0	U-0.055	R-13.0 + R-13.0	U-0.097	R-10.0
Attic and Other	U-0.027	R-38.0	U-0.027	R-38.0	U-0.053	R-19.0
Walls, Above-Grade						
Mass	U-0.104	R-9.5 c.i.	U-0.090	R-11.4 c.i.	U-0.580	NR
Metal Building	U-0.084	R-19.0	U-0.084	R-19.0	U-0.113	R-13.0
Steel-Framed	U-0.064	R-13.0 + R-7.5 c.i.	U-0.064	R-13.0 + R-7.5 c.i.	U-0.124	R-13.0
Wood-Framed and Other	U-0.089	R-13.0	U-0.064	R-13.0 + R-3.8 c.i.	U-0.089	R-13.0
Walls, Below-Grade						
Below-Grade Wall	C-1.140	NR	C-0.119	R-7.5 c.i.	C-1.140	NR
Floors						
Mass	U-0.087	R-8.3 c.i.	U-0.074	R-10.4 c.i.	U-0.137	R-4.2 c.i.
Steel-Joist	U-0.038	R-30.0	U-0.038	R-30.0	U-0.069	R-13.0
Wood-Framed and Other	U-0.033	R-30.0	U-0.033	R-30.0	U-0.066	R-13.0
Slab-On-Grade Floors						
Unheated	F-0.730	NR	F-0.540	R-10 for 24 in.	F-0.730	NR
Heated	F-0.860	R-15 for 24 in.	F-0.860	R-15 for 24 in.	F-1.020	R-7.5 for 12 in.
Opaque Doors						
Swinging	U-0.700		U-0.700		U-0.700	
Nonswinging	U-1.500		U-0.500		U-1.450	
Fenestration	**Assembly Max. U**	**Assembly Max. SHGC**	**Assembly Max. U**	**Assembly Max. SHGC**	**Assembly Max. U**	**Assembly Max. SHGC**
Vertical Glazing, 0%–40% of Wall						
Nonmetal framing (all)[c]	U-0.40	SHGC-0.40 all	U-0.40	SHGC-0.40 all	U-1.20	SHGC-NR all
Metal framing (curtainwall/storefront)[d]	U-0.50		U-0.50		U-1.20	
Metal framing (entrance door)[d]	U-0.85		U-0.85		U-1.20	
Metal framing (all other)[d]	U-0.55		U-0.55		U-1.20	
Skylight with Curb, Glass, % of Roof						
0%–2.0%	U_{all}–1.17	$SHGC_{all}$–0.49	U_{all}–0.98	$SHGC_{all}$–0.36	U_{all}–1.98	$SHGC_{all}$–NR
2.1%–5.0%	U_{all}–1.17	$SHGC_{all}$–0.39	U_{all}–0.98	$SHGC_{all}$–0.19	U_{all}–1.98	$SHGC_{all}$–NR
Skylight with Curb, Plastic, % of Roof						
0%–2.0%	U_{all}–1.30	$SHGC_{all}$–0.65	U_{all}–1.30	$SHGC_{all}$–0.62	U_{all}–1.90	$SHGC_{all}$–NR
2.1%–5.0%	U_{all}–1.30	$SHGC_{all}$–0.34	U_{all}–1.30	$SHGC_{all}$–0.27	U_{all}–1.90	$SHGC_{all}$–NR
Skylight without Curb, All, % of Roof						
0%–2.0%	U_{all}–0.69	$SHGC_{all}$–0.49	U_{all}–0.58	$SHGC_{all}$–0.36	U_{all}–1.36	$SHGC_{all}$–NR
2.1%–5.0%	U_{all}–0.69	$SHGC_{all}$–0.39	U_{all}–0.58	$SHGC_{all}$–0.19	U_{all}–1.36	$SHGC_{all}$–NR

*The following definitions apply: c.i. = *continuous insulation* (see Section 3.2), NR = no (insulation) requirement.
[a]When using R-value compliance method, a thermal spacer block is required; otherwise use the *U-factor* compliance method. See Table A2.3.
[b]Exception to Section A3.1.3.1 applies.
[c]Nonmetal framing includes framing materials other than metal with or without metal reinforcing or cladding.
[d]Metal framing includes metal framing with or without thermal break. The "all other" subcategory includes operable windows, fixed windows, and non-entrance *doors*.

Table From ASHRAE 90.1 (2010)

TABLE 5.5-2 Building Envelope Requirements for Climate Zone 2 (A, B)*

Opaque Elements	Nonresidential Assembly Maximum	Nonresidential Insulation Min. R-Value	Residential Assembly Maximum	Residential Insulation Min. R-Value	Semiheated Assembly Maximum	Semiheated Insulation Min. R-Value
Roofs						
Insulation Entirely above Deck	U-0.048	R-20.0 c.i.	U-0.048	R-20.0 c.i.	U-0.218	R-3.8 c.i.
Metal Building[a]	U-0.055	R-13.0 + R-13.0	U-0.055	R-13.0 + R-13.0	U-0.097	R-10.0
Attic and Other	U-0.027	R-38.0	U-0.027	R-38.0	U-0.081	R-13.0
Walls, Above-Grade						
Mass	U-0.151[b]	R-5.7 c.i.[b]	U-0.123	R-7.6 c.i.	U-0.580	NR
Metal Building	U-0.093	R-16.0	U-0.093	R-16.0	U-0.113	R-13.0
Steel-Framed	U-0.124	R-13.0	U-0.064	R-13.0 + R-7.5 c.i.	U-0.124	R-13.0
Wood-Framed and Other	U-0.089	R-13.0	U-0.089	R-13.0	U-0.089	R-13.0
Walls, Below-Grade						
Below-Grade Wall	C-1.140	NR	C-1.140	NR	C-1.140	NR
Floors						
Mass	U-0.107	R-6.3 c.i.	U-0.087	R-8.3 c.i.	U-0.322	NR
Steel-Joist	U-0.052	R-19.0	U-0.052	R-19.0	U-0.069	R-13.0
Wood-Framed and Other	U-0.051	R-19.0	U-0.033	R-30.0	U-0.066	R-13.0
Slab-On-Grade Floors						
Unheated	F-0.730	NR	F-0.730	NR	F-0.730	NR
Heated	F-1.020	R-7.5 for 12 in.	F-1.020	R-7.5 for 12 in.	F-1.020	R-7.5 for 12 in.
Opaque Doors						
Swinging	U-0.700		U-0.700		U-0.700	
Nonswinging	U-1.450		U-0.500		U-1.450	
Fenestration	**Assembly Max. U**	**Assembly Max. SHGC**	**Assembly Max. U**	**Assembly Max. SHGC**	**Assembly Max. U**	**Assembly Max. SHGC**
Vertical Glazing, 0%–40% of Wall						
Nonmetal framing (all)[c]	U-0.75	SHGC-0.25 all	U-0.75	SHGC-0.25 all	U-1.20	SHGC-NR all
Metal framing (curtainwall/storefront)[d]	U-0.70		U-0.70		U-1.20	
Metal framing (entrance door)[d]	U-1.10		U-1.10		U-1.20	
Metal framing (all other)[d]	U-0.75		U-0.75		U-1.20	
Skylight with Curb, Glass, % of Roof						
0%–2.0%	U_{all}-1.98	$SHGC_{all}$-0.36	U_{all}-1.98	$SHGC_{all}$-0.19	U_{all}-1.98	$SHGC_{all}$-NR
2.1%–5.0%	U_{all}-1.98	$SHGC_{all}$-0.19	U_{all}-1.98	$SHGC_{all}$-0.19	U_{all}-1.98	$SHGC_{all}$-NR
Skylight with Curb, Plastic, % of Roof						
0%–2.0%	U_{all}-1.90	$SHGC_{all}$-0.39	U_{all}-1.90	$SHGC_{all}$-0.27	U_{all}-1.90	$SHGC_{all}$-NR
2.1%–5.0%	U_{all}-1.90	$SHGC_{all}$-0.34	U_{all}-1.90	$SHGC_{all}$-0.27	U_{all}-1.90	$SHGC_{all}$-NR
Skylight without Curb, All, % of Roof						
0%–2.0%	U_{all}-1.36	$SHGC_{all}$-0.36	U_{all}-1.36	$SHGC_{all}$-0.19	U_{all}-1.36	$SHGC_{all}$-NR
2.1%–5.0%	U_{all}-1.36	$SHGC_{all}$-0.19	U_{all}-1.36	$SHGC_{all}$-0.19	U_{all}-1.36	$SHGC_{all}$-NR

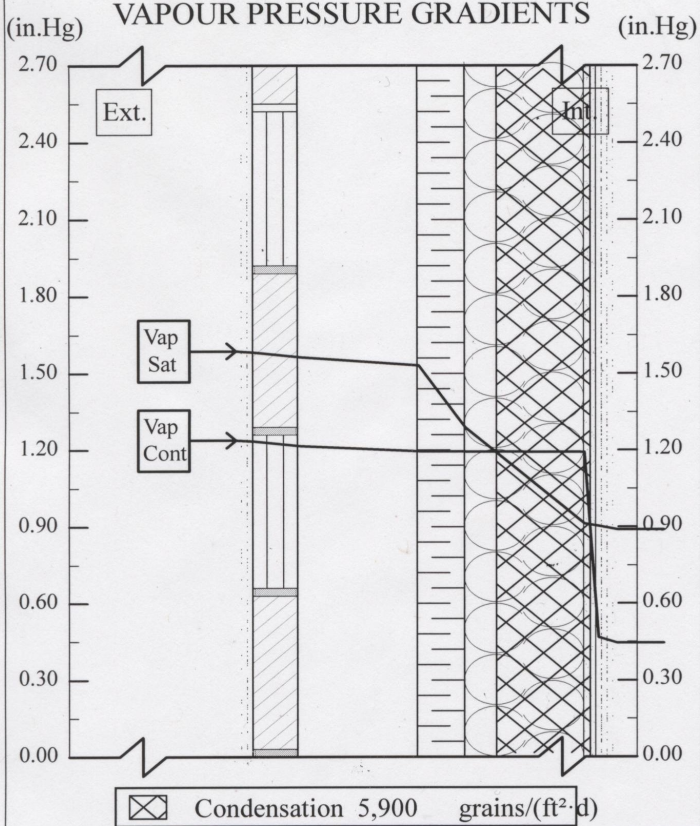
*The following definitions apply: c.i. = *continuous insulation* (see Section 3.2), NR = no (insulation) requirement.
[a]When using R-value compliance method, a thermal spacer block is required; otherwise use the *U-factor* compliance method. See Table A2.3.
[b]Exception to Section A3.1.3.1 applies.
[c]Nonmetal framing includes framing materials other than metal with or without metal reinforcing or cladding.
[d]Metal framing includes metal framing with or without thermal break. The "all other" subcategory includes operable windows, fixed windows, and non-entrance *doors*.

Table From ASHRAE 90.1 (2010)

CONDENSATION ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11a)

WALL SECTION AND VAPOUR PRESSURE GRADIENTS



PROJECT

Name	Thesis
Number	
City	Orlando, FL
Date	3/19/2013
Analysis by:	Evan Landis
Wall Type	
Option	

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext
Temp (°F)	—	—	75	93
RH (%)	—	—	50	78
DPT (°F)	—	—	56	86

PENNSYLVANIA STATE UNIVERSITY

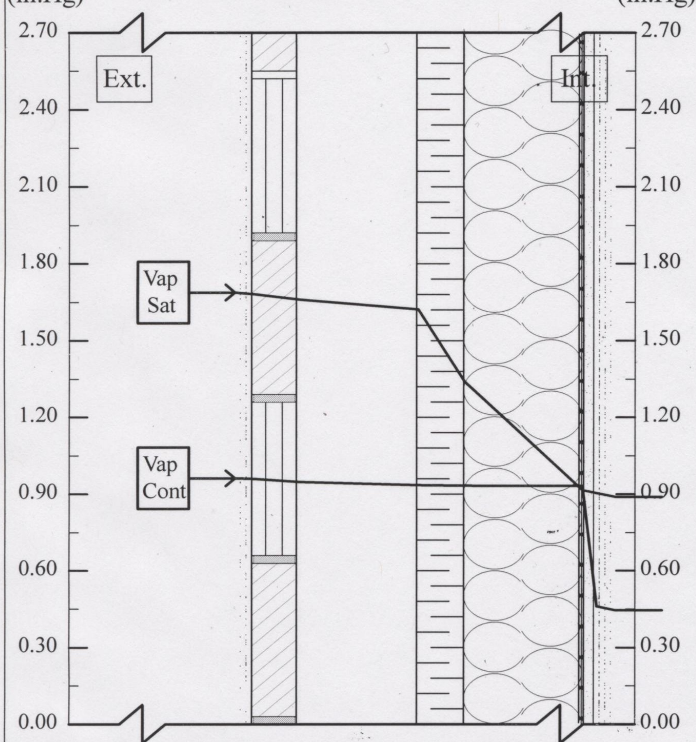
104 ENGINEERING, UNIT A
UNIVERSITY PARK, PA, USA, 1680

	Material	Manufacturer	Model No.	Rvap (1/M)	Temp (°F)	VapSat (in.Hg)	VapCo (in.Hg
1	air film (ext), 3/4 in.	No Recor...	Generic...	0.001	92.9	1.557	1.215
2	Terra Cotta blk., 2 in.	No Recor...	Generic...	0.005	92.5	1.539	1.197
3	cavity, 5-1/2 in.	No Recor...	Generic...	0.006	91.8	1.507	1.177
4	rigid ins.,(expand.), 2 in.	No Recor...	Generic...	0.001	86.3	1.268	1.175
5	batt ins., 5 in.	No Recor...	Generic...	0.001	75.8	0.898	1.173
6	gypsum bd., 1/2 in., (#1)	No Recor...	Generic...	0.197	75.4	0.889	0.460
7	air film (int), 3/4 in.	No Recor...	Generic...	0.006	75.0	0.876	0.438
8							
9							
10							
11							
12							
	TOTAL or (Layer 0)			0.217	(93.0)	(1.563)	(1.219

CONDENSATION ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11a)

WALL SECTION AND VAPOUR PRESSURE GRADIENTS



Condensation 350 grains/($ft^2 \cdot d$)

PROJECT

Name: Thesis

Number:

City: Washington, DC

Date: 3/19/2013

Analysis by: Evan Landis

Wall Type: | Option:

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext
Temp (°F)	–	–	75	95
RH (%)	–	–	50	57
DPT (°F)	–	–	56	78

PENNSYLVANIA STATE UNIVERSITY

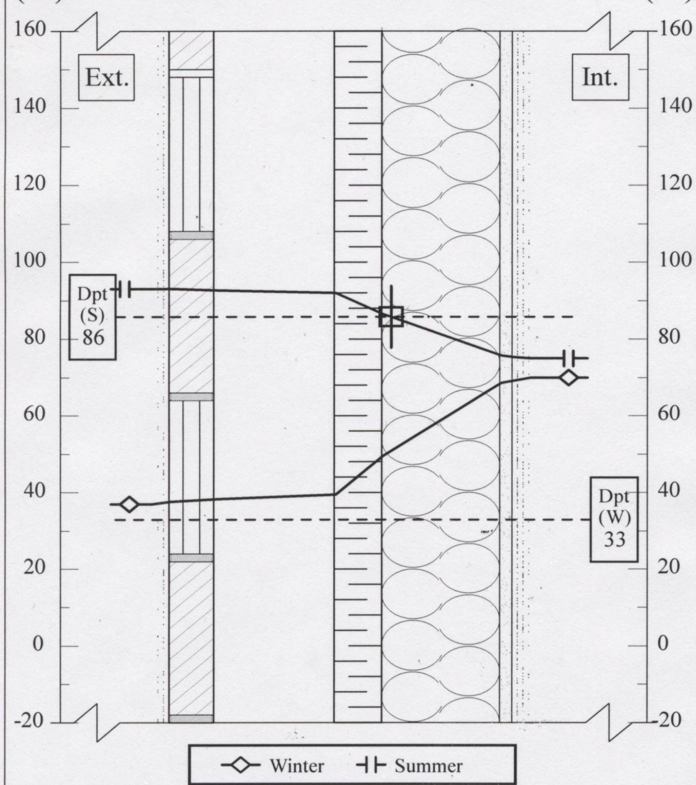
104 ENGINEERING, UNIT A
UNIVERSITY PARK, PA, USA, 1680

	Material	Manufacturer	Model No.	Rvap (1/M)	Temp (°F)	VapSat (in.Hg)	VapCo (in.Hg
1	air film (ext), 3/4 in.	No Recor...	Generic...	0.001	94.9	1.655	0.945
2	Terra Cotta blk., 2 in.	No Recor...	Generic...	0.005	94.4	1.634	0.933
3	cavity, 5-1/2 in.	No Recor...	Generic...	0.006	93.7	1.597	0.920
4	rigid ins.,(expand.), 2 in.	No Recor...	Generic...	0.001	87.6	1.319	0.919
5	batt ins., 5 in.	No Recor...	Generic...	0.001	75.8	0.901	0.917
6	gypsum bd., 1/2 in., (#1)	No Recor...	Generic...	0.197	75.5	0.890	0.453
7	air film (int), 3/4 in.	No Recor...	Generic...	0.006	75.0	0.876	0.438
8							
9							
10							
11							
12							
	TOTAL or (Layer 0)			0.217	(95.0)	(1.662)	(0.947

R VALUE ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11)

WALL SECTION AND TEMPERATURE GRADIENTS



PROJECT

Name	Thesis
Number	
City	Orlando, FL
Date	3/19/2013
Analysis by:	Evan Landis
Wall Type	
Option	

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext
Temp (°F)	70	37	75	93
RH (%)	25	75	50	78
DPT (°F)	33	31	56	86

PENNSYLVANIA STATE UNIVERSITY

104 ENGINEERING, UNIT A
UNIVERSITY PARK, PA, USA, 1680

	Generic Material	Manufacturer	Model No.	Thick (in.)	RVal (R)	W.Temp. (°F)	S.Tem (°F)
1	air film (ext), 3/4 in.	No Recor...	Generic...	0.75	0.17	37.6	93.1
2	Terra Cotta blk., 2 in.	No Recor...	Generic...	2.00	0.54	38.3	92.7
3	cavity, 5-1/2 in.	No Recor...	Generic...	5.00	0.98	39.5	92.0
4	rigid ins.,(expand.), 2 in.	No Recor...	Generic...	2.00	7.90	49.5	86.5
5	batt ins., 5 in.	No Recor...	Generic...	5.00	15.24	68.6	75.8
6	gypsum bd., 1/2 in., (#1)	No Recor...	Generic...	0.50	0.46	69.2	75.4
7	air film (int), 3/4 in.	No Recor...	Generic...	0.75	0.64	70.0	75.0
	Total or (Layer 0)			14.50	25.94	(37.4)	(93.2)