Biobehavioral Health Building

University Park, PA

Final Report

2012-2013 AE Senior Thesis

Rendering provided by BCJ

Daniel Bodde

Structural Option

Advisor: Heather Sustersic

4/3/2013

Biobehavioral Health Building

University Park, PA

Building Statistics

Size:
93,500 SF

Number of Stories:
5 Stories above grade + Full Basement (100% below grade)

Dates of Construction:
November 2010 - November 2012

Cost:
Building Cost = $40,000,000

Delivery Method:

Project Team

Owner:
Penn State College of Health and Human Development

Architect:
Bohlin Cywinski Jackson

Structural Engineer:
Robert Silman Associates

MEP Engineer:
Bruce E. Brookes & Associates

Civil Engineer:
Gannett Fleming, Inc

Landscape Architect:
Michael Vergason Landscape Architects, LTD

Construction Manager:
Massaro CM Services

General Contractor:

Architecture

- A majority of the façade was designed to mimic Henderson North's Georgian style architecture with the use of large amounts of brick and natural limestone.
- North East portion of the building designed to replicate the more modern design of the HUB.
- HUB lawn terrace with stone seat walls for student hangout and can also be used as a stage for concerts.
- Large lecture hall directly under terrace

Mechanical

- 6 air handling units
- VAV boxes for multiple rooms
- Building hooked into PSU campus's chilled water and low pressure steam loop.

Lighting/Electrical

- Mixture of florescent and LED lighting.
- Dimmable lights.
- Occupancy sensors in all rooms.
- Building hooks into PSU campus power supply.

Structure

- Spread footing foundation.
- Slab on composite metal deck.
- Composite beam action.
- Lateral loads resisted by both steel moment frames and a braced frame.
- Reinforced CMU infill walls anchor

Table of Contents

Executive Summary

For this thesis, the goal was to redesign the structure of the BBH Building with reinforced concrete. This was determined to be successful. One way slabs with beams were designed to support the design loads that were applied to them. Three typical beam sections were analyzed and designed to resist flexure, shear, and torsion. The sections were also designed to satisfy immediate and long term deflection limits stated in the IBC. The girders were the deepest sections with a total depth of 28”.

The computer program ETABS was used to aid in the analysis and design of the concrete moment frames that were designed to resist lateral loads by the controlling wind cases in both directions. Excel spread sheets were used to determine how the lateral loads were distributed between the moment frames. Reinforcement values for both beams and columns were cross checked with the hand calculations to verify that adequate amounts of reinforcement were provided to resist the applied gravity and lateral loads. A brief cost analysis was done between the existing steel structure and redesigned concrete structure and it was determined that the redesign would cause a 12% increase in cost.

In order to possibly help alleviate the cost and possible schedule increase, a thin brick precast façade was researched as an alternative to the existing traditional hand laid brick façade. Information from the Precast/Prestressed Concrete Institute (PCI) helped in the determination of the heat and moisture resistance requirements needed to match that of the existing façade. A brief cost analysis was done between the existing and precast facades and it was determined that the precast would cause a 40% decrease in cost. It was determined the precast façade would be a good alternative to the existing, although the owner and architect would have to accept the more manufactured look of the panels compared to the traditional look that the hand placed brick would provide.

Finally a schedule was produced to determine what kind of impact the above changes would cause to the project. It was determined that even though the construction of the concrete redesign would take 3 months longer, the precast façade would allow for the entire building to be enclosed 6 months earlier than what was originally scheduled. This would allow for interior finishes to start earlier, which could provide a positive impact on the project’s completion.

Acknowledgements

I would like to say thank you to the following people who have in some way or the other helped me get to where I am now:

Thank you to Massaro CM Services for providing me with the construction documents for my thesis and for taking the time to answer my numerous questions, especially,

Kevin Nestor
Tim Jones
Jim Kephart
Keith Smith

Thank you to the entire AE Faculilty for sharing your knowledge and passion for this industry with me. A special thanks to the following for the guidance they gave me throughout this process,

Heather Sustersic
Kevin Parfitt

Thank you to all of my friends and family that have supported me unconditionally throughout my college career. The past couple of years have been some of the most exciting and exhausting years of my life and without your support I don't think it would have been possible to make it. Your steady stream of encouraging texts, phone calls, and facebook messages were just the pick me up I needed when things got tough.

Building Introduction

Located on the campus of the Pennsylvania State University in University Park, Pennsylvania is the Biobehavioral Health Building (Figure 1). 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.

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.

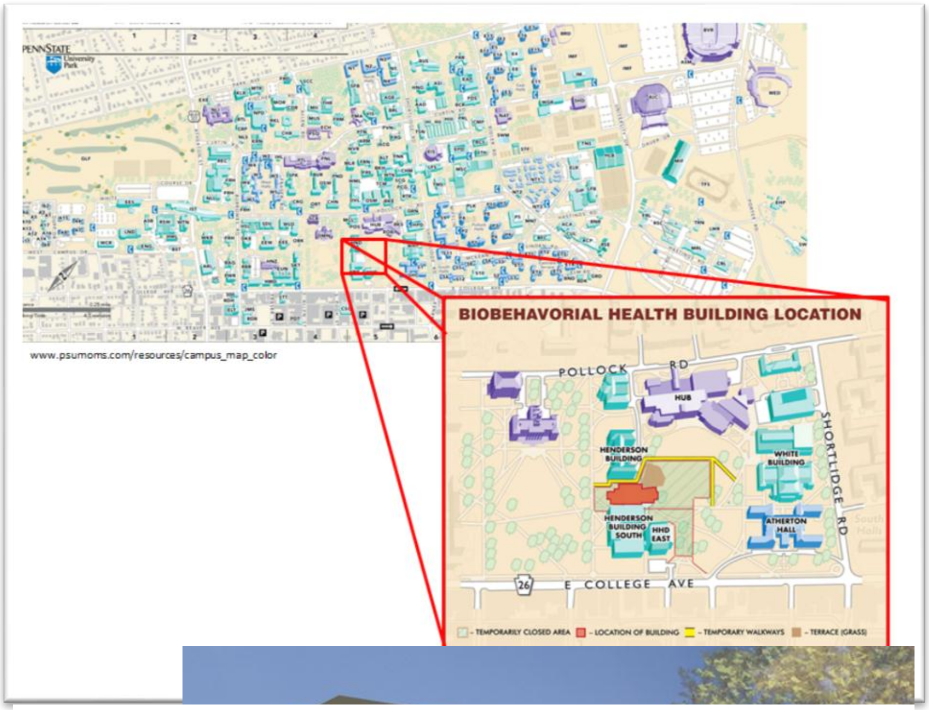


Figure 1: PSU Campus Map

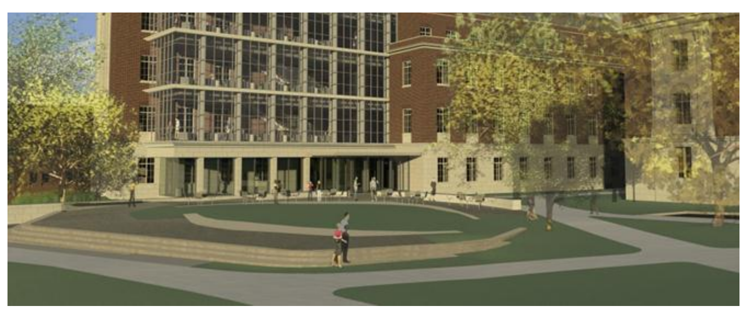


Figure 2: Rendered View from HUB Lawn

Existing Structural Overview

Foundation

CMT Laboratories, Inc. was the geotechnical engineers hired to investigate the soil conditions on which the BBH building was to be placed. In order to better understand the soil located on the site, CMT Laboratories took six test boring samples. 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. According 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 isencountered 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 be evaluated to ensure a bearing capacity of 15 ksf is provided.

The BBH Building uses a shallowstrip 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.

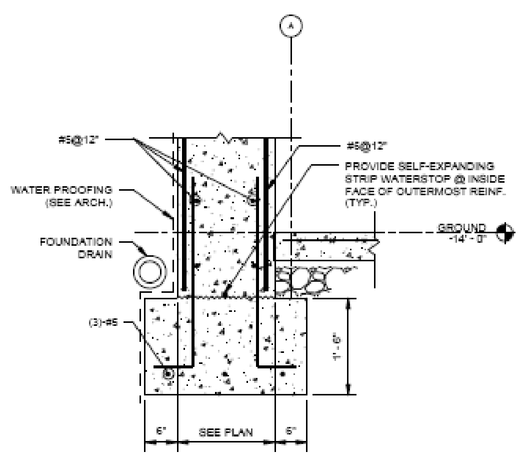


Figure 3: Typical Strip Footing

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” W2.0xW2.0welded 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.

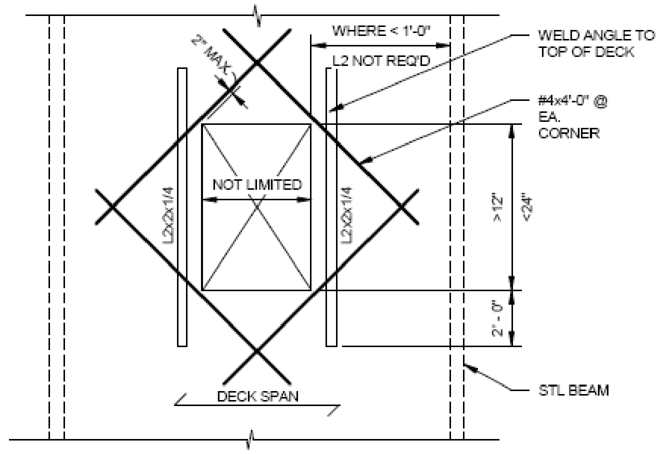


Figure 4: Openings in Slab on Steel Deck

In order to decrease beam depth the BBH building was designed as a composite steel system. Figure 5 shows a typicalsection 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.

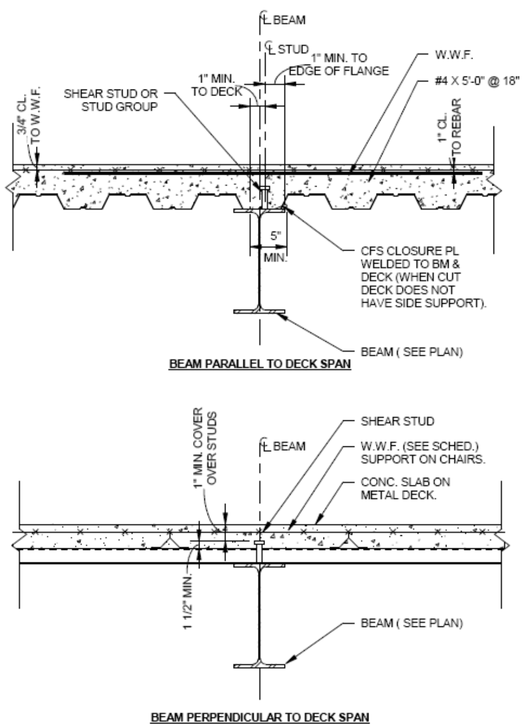


Figure 5: Typical Section Through Composite System

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.

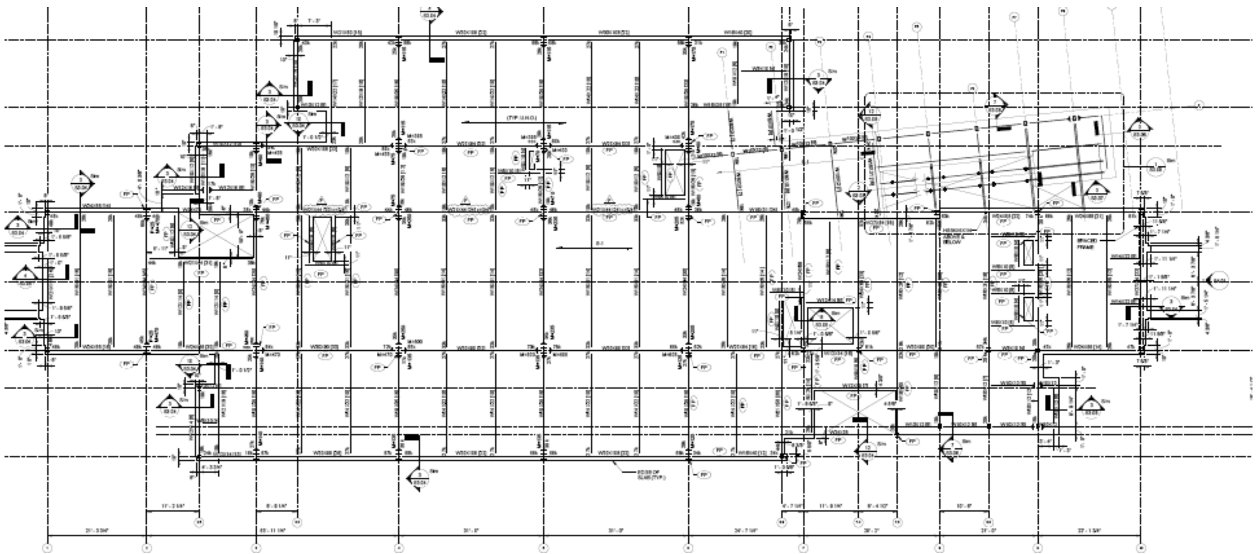


Figure 6: Typical Floor Framing Plan

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 loadsare 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).

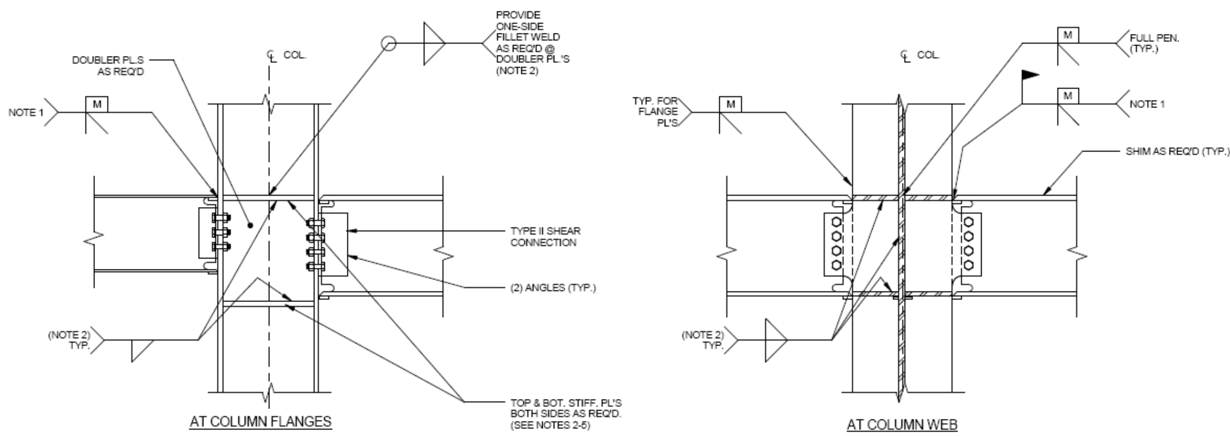


Figure 7: Typical Beam to Column Moment Connection

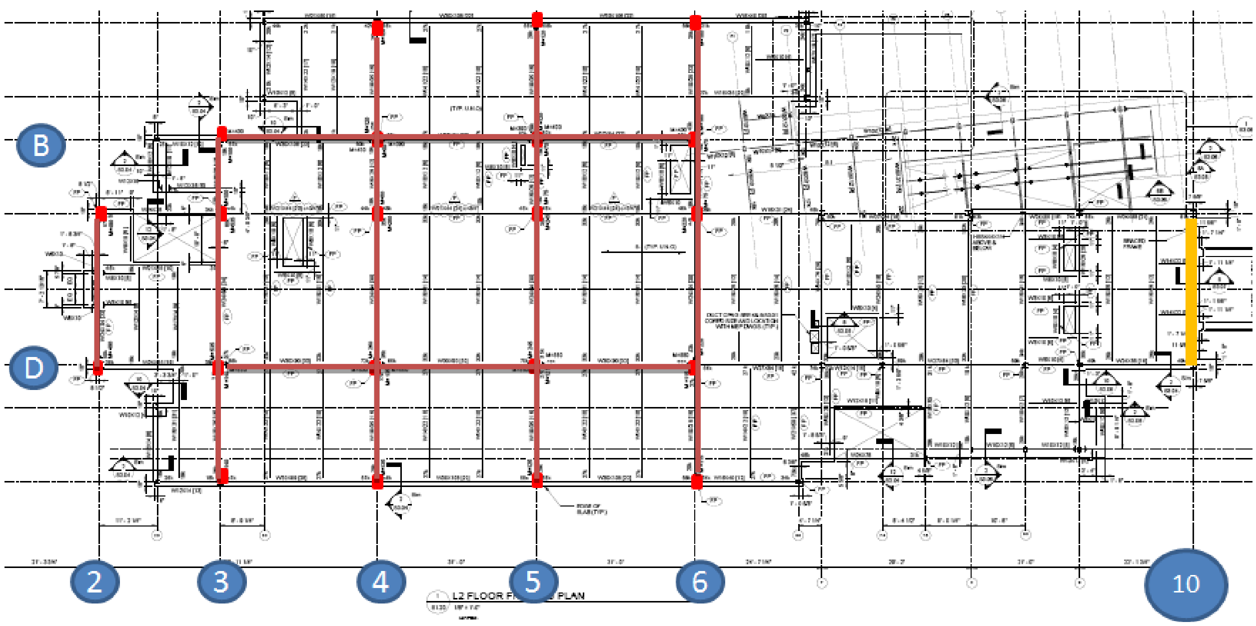


Figure 8: Location of Moment Connections (Red) and Braced Frame (Orange)

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.

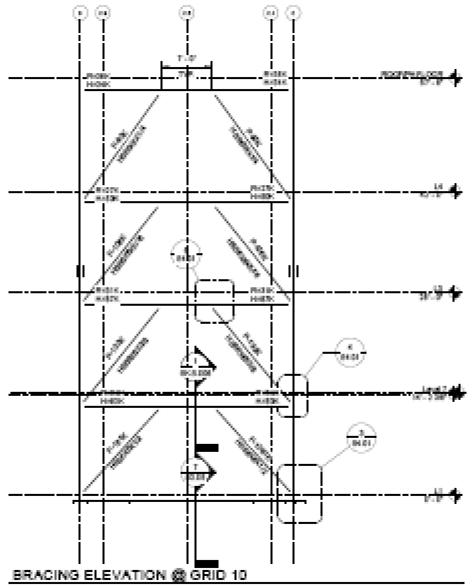


Figure 9: Eccentric Braced Frame

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:

- ASCE 7-05
- ACI 11-08
- IBC 2006

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

Problem Statement

A situation has arisen where the use of structural steel has become an unfeasible option for the structural system of the BBH Building. It is to be assumed that this change was made after the structural system had already been designed in steel. The owner has requested that the design professionals keep the original layout and look of the building as close as possible to the original design.

This change will obviously cause structural impacts that will require the floor system and lateral system to be redesigned with a different material. In order to cause the least amount of change to the original layout of the building special attention will need to be taken in certain areas. The new floor system will need to be sensitive to the floor to ceiling height impact as an increased floor system depth is undesirable. The new lateral system must be designed to not affect the layout of the floors and also not disturb the open public areas of the BBH building.

These changes in structure will result in modifications needed to be made to the schedule for construction. Using a different material will require the coordination during construction to be adjusted to minimize the schedule impact. Changes in the method of construction of certain aspects of the building might need to be adjusted in order to save time and money during construction.

Proposed Solution

Structural Depth

The alternative structural material selected for the BBH Building will be reinforced concrete. The floor system will consist of flat slabs with drop panels or one way slabs with interior beams. These systems were proven to have a total system depth less than that of the original steel design. This will allow for a greater amount of area above the ceiling for MEP equipment to run which can reduce the number of conflicts that are bound to arise during construction. Using reinforced concrete will increase the weight of the building which will cause an increase in the foundation.

In order to resist lateral loads a concrete moment frame will be designed. This type of system will cause the least amount of impact on the existing layout of the building considering the original designed was predominantly a steel moment frame. A cost analysis will be done to compare the existing steel structure to the propose reinforced concrete structure. The effects on the schedule will be studied later in one of the breadth topics.

Breadth 1 - Façade Study

The existing façade, consisting largely of brick and limestone, was specified by the architect to be handmade. This method of construction is very expensive due to the amount of physical labor that is involved in laying each piece one by one. An alternate façade system will be used to alleviate the assumed cost and time impact that the concrete redesign will have on the building.

A precast masonry system will replace the existing façade design. This change will affect the detailing and constructability of the façade. Therefore a study will be done to understand how the new façade will change the way the building will need to be insulated, waterproofed, and connected to the structure. A cost analysis will be done to compare the existing façade system to the new precast façade system. The effects on the schedule will be studied in the next breadth topic.

Breadth 2 - Construction Management

The purpose of this breadth is to create a schedule based on the changes that were made in the above depth and breadth. Both of these changes will affect the critical path and will need to be sequenced in a way to better control the flow of the project. Tools such as RS Means and Microsoft Project will be helpful in assembling the schedule. In order to develop a realistic schedule, critical site and construction information may need to be requested from the construction manager. The proposed adjusted schedule will be compared to the existing schedule. It is there we will be able to understand how each of the changes affected the overall project.

Design Loads

The following design loads given by the designer.

Dead

Dead Load	Uniform (psf)
Floor Slab	62.5
Roof Slab	62.5
Green Roof	25
Superimposed	5
Façade	100
Interior brick walls	4.666666667
Interior stone floors	20
Slate Roof	10

Live

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

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

Snow

The 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

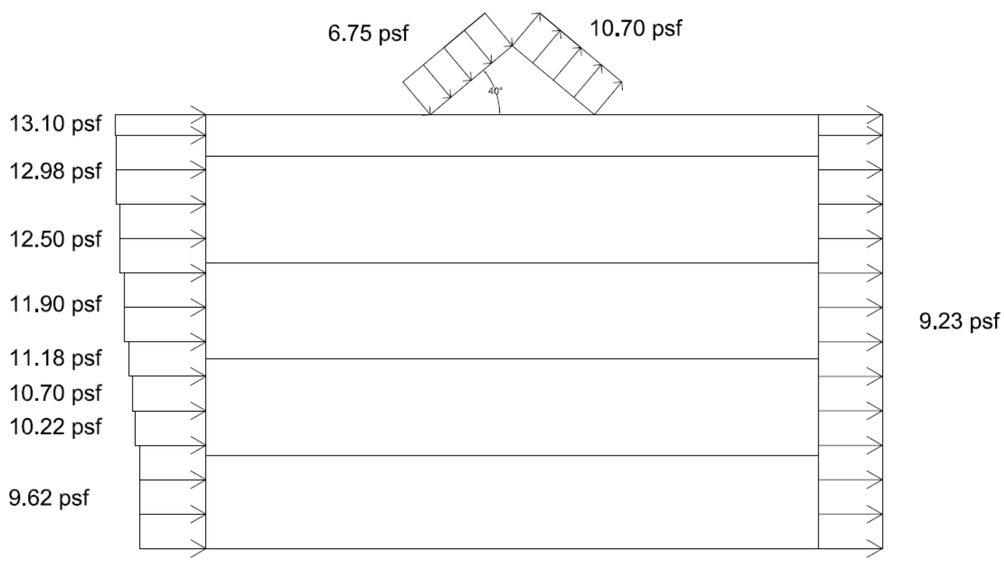


Figure 10: N-S Wind Pressure Diagram

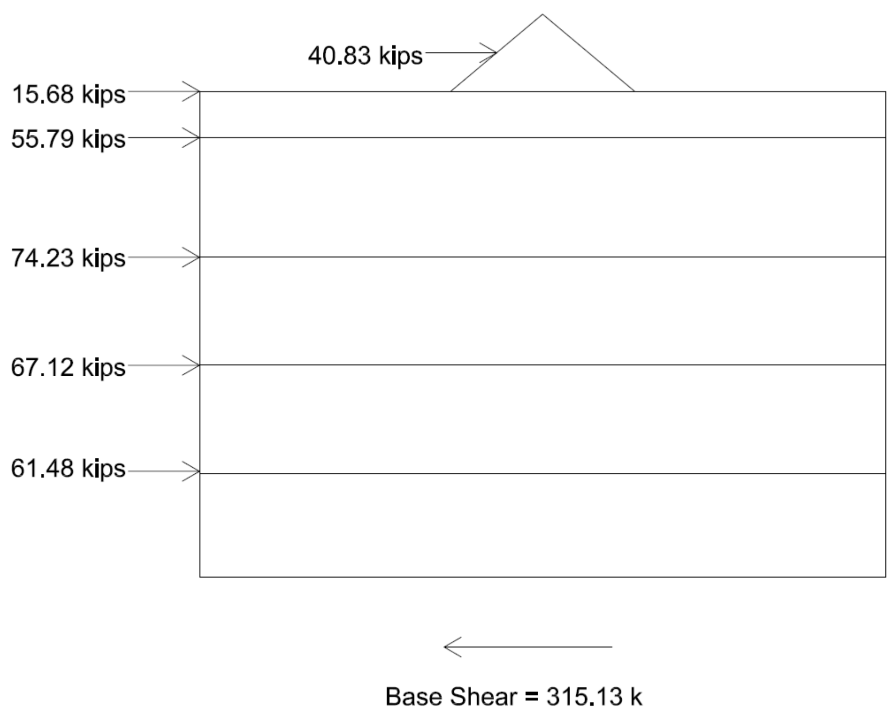


Figure 11: N-S Wind Story Force Diagram

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

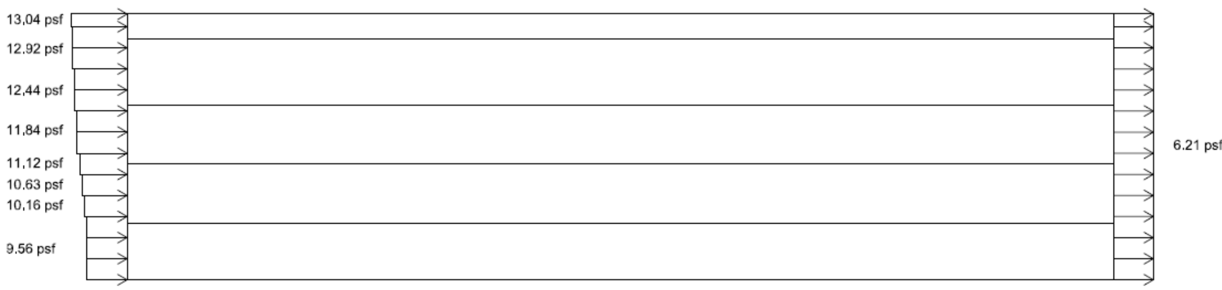


Figure 12: E-W Wind Pressure Diagram

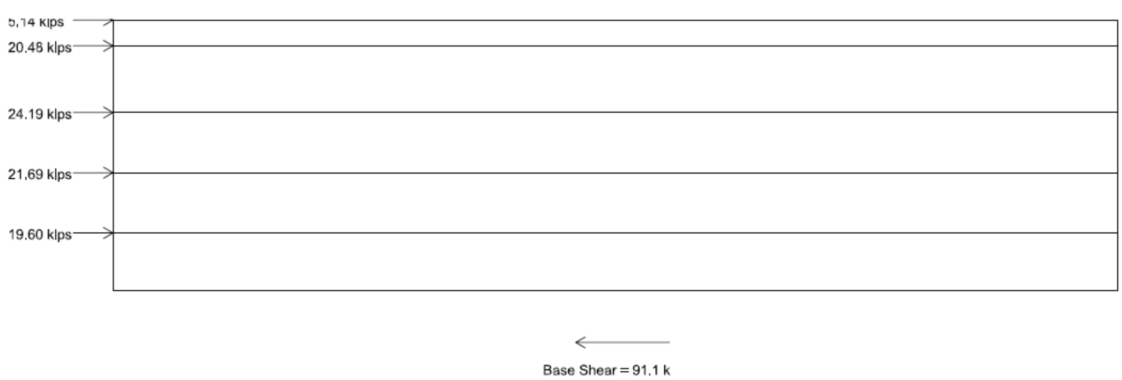


Figure 13: N-S Wind Story Force Diagram

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 B). 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 B for hand calculations. Vertical distribution of the seismic forces is shown below in Figure14

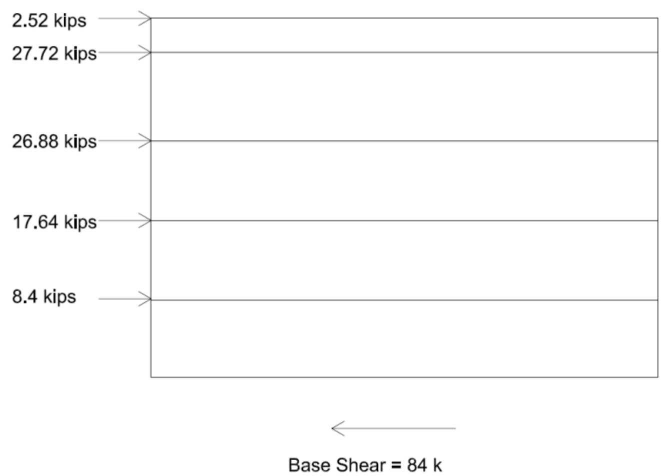


Figure 14: Vertical Distribution of seismic forces

Gravity System Redesign

In Tech Report 2 a preliminary analysis and design was done to determine 3 possible alternative floor systems. For the redesign of the BBH Building a one-way concrete slab with interior concrete beams will be used. This system was chosen over a two-way slab for the following reasons. It would be lighter weight than the two-way, it was believe the same floor to floor height could be sustained vs. the existing, and beams would need to be designed anyways because a concrete moment frame was chosen as the lateral system.The one-way system was reinvestigated from Tech 2 to provide a better design.

Typical sections were designed to allow for a simplified building design. These sections were chosen based on their span length and how many times they showed up in the building. This would allow for the designed sections to be replicated throughout the building. In order to help with the constructability of the redesign, the rebar sizes used were limited to certain sizes. The following concrete members were designed using ACI 318-11.

One-way Slab

In a typical bay the one-way slab will span 10.33 feet. The slab was designed to resist dead and live loads. Using table 9.5a in ACI, a one-way slab with continuous ends can have a minimum thickness of 5”. It was determined that the use of #5 sized rebar spaced at 12” on center for a 5” slab would be adequate in carrying the applied load. Reinforcement in the opposite direction was added to limit cracking due to the effects of shrinkage and temperature. #5 bars spaced at 10” were used.

Beams

It was determine from the reinvestigation that two interior beams would be used in place of the original design with one interior beam. This would decrease the beam depth and would put less moment on the girder supporting it. This layout chosen will replicate the existing steel layout which will provide for a very simple comparison between the two materials.

These interior beams will span in the N-S direction and are continuous from column line A to E.The beams are supported at each end by girders or edge beams running perpendicular to it. Figure 15 shows a plan view of the Interior beam layout for a typical bay.

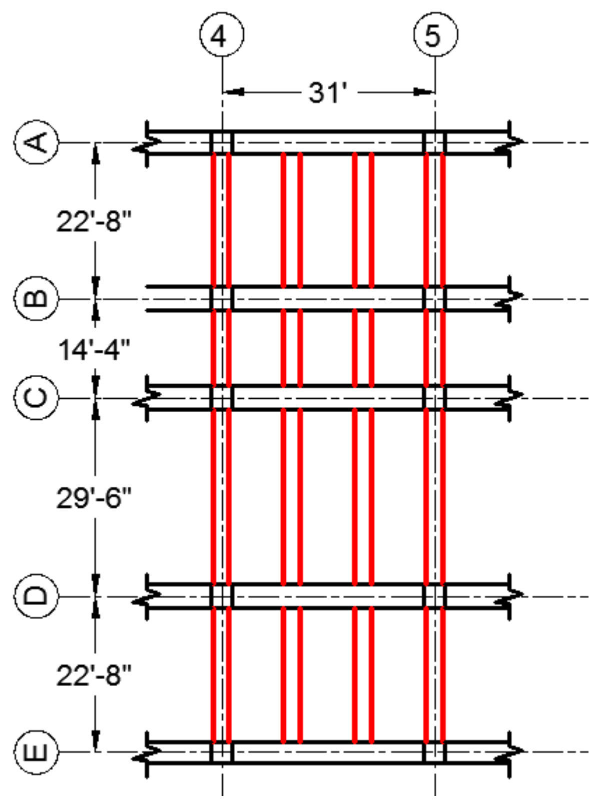


Figure 15: Typical Int Beam Layout

Through analysis it was determined that the negative moments at the supports would be the controlling factor in the size estimation of the beam section. A beam size of 14”X20” was chosen. The beams were then designed to resist flexure and shear through placement of grade 60 reinforcing bars. Figure 16 shows the layout of this rebar. It was also determined from analysis that the same beam design for the interior beams could be used for the beams that span N-S between the columns. In order to expedite some of the repetitive calculations, excel was used. See Appendix C for hand calculations of the beams.

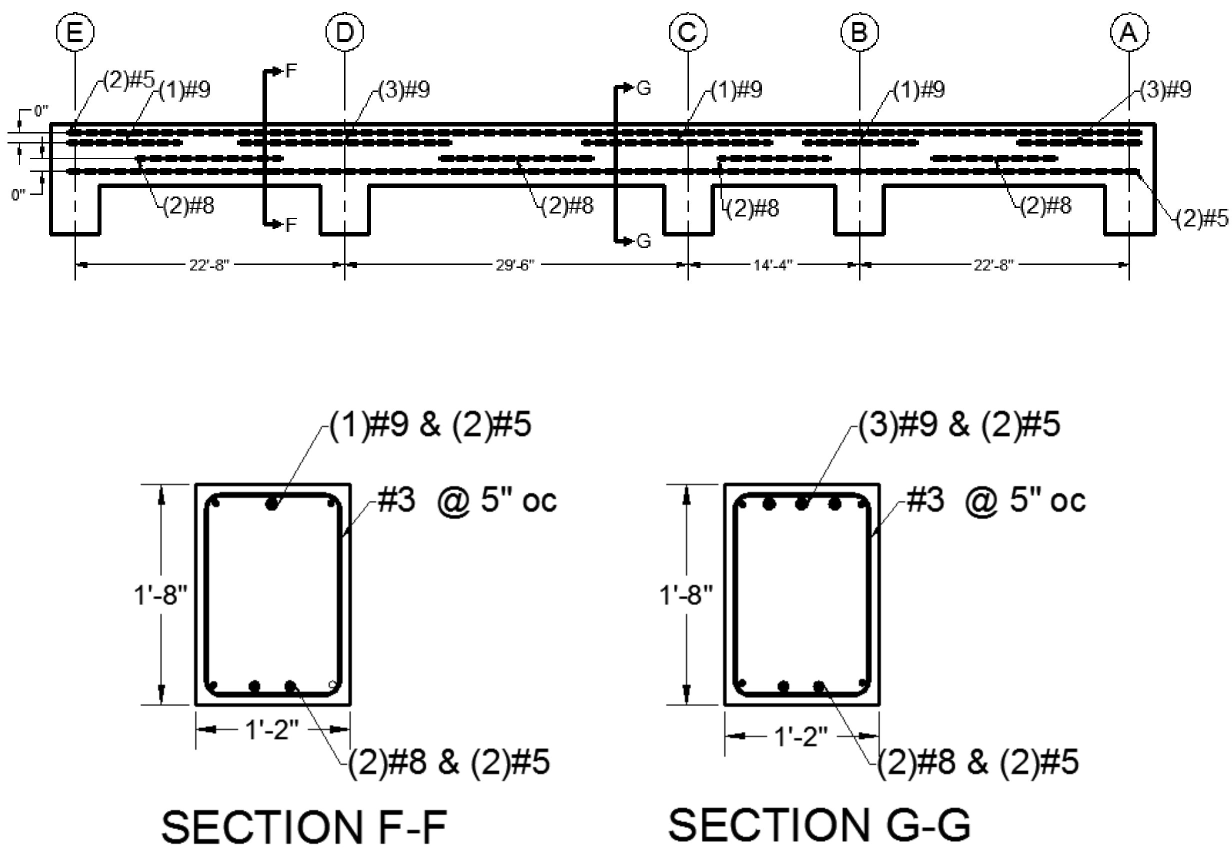


Figure 16: Typical Int Beam Reinforcement

Girders

Girders were designed to run E-W along column lines B, C, and D. They support the two interior beams that run N-S and it's self-weight. The beams produce to large concentrated loads that are applied L/3 away from the supports. In order to simplify the design of the BBH building, the girders spanning 31' were designed and replicated for the rest of the girders in the building. Figure 17 shows a plan view of the girder layout for a typical bay.

Due to the large concentrated loads, the girders are the deepest member in the building. Figure 18 shows a section and layout of the rebar that is needed to resist the moments and shear forces due to the applied load. Calculations for the design of the girder can be found in Appendix C.

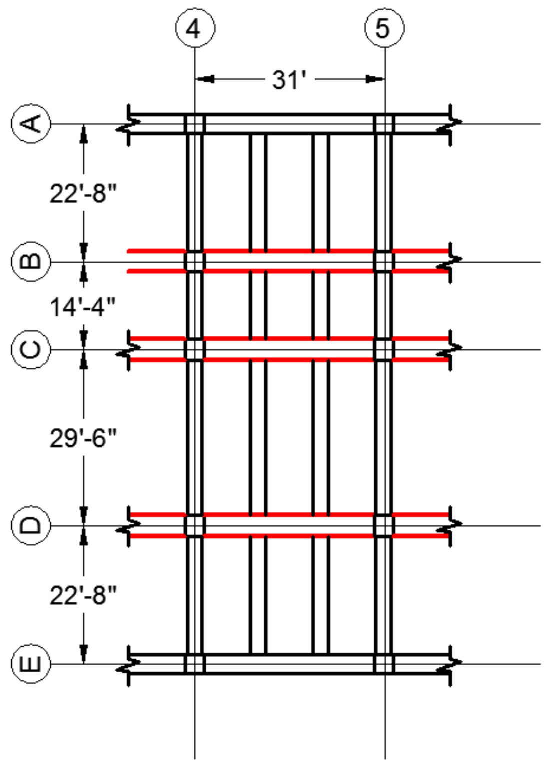


Figure 17: Typical Girder Layout

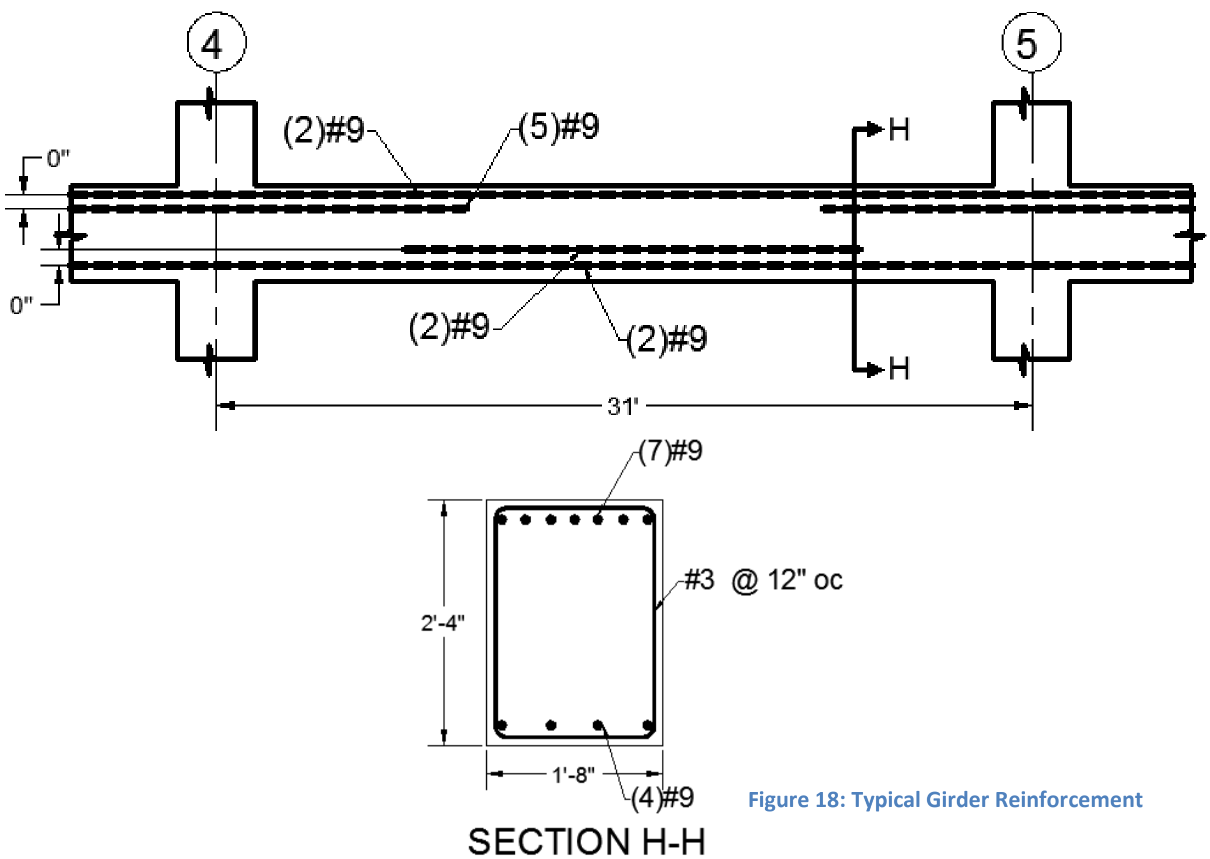


Figure 18: Typical Girder Reinforcement

Through inspection it was determined that the girders would be the controlling member to be checked for deflection. Both immediatedeflections due to live loads and dead plus live loads were check to make sure that they were under the deflection limit stated in Table 9.5(b) from ACI 318-11. It was assumed that the girders do not support nonstructural elements likely to be damaged by large deflection. Therefore the limitation for immediate deflection due to live load is L/360 and the limitation for immediate dead and live load is L/240. Unlike steel, concrete needs to also be checked for long term deflections caused by shrinkage and creep from sustained loads. In the calculation of the long term deflections it was assumed that duration of the load was greater than 60 months and that the sustained live load would be 50% of the actual live load. The limitation for long term deflections due to shrinkage, creep, and 50% sustained live load is L/240.Deflection calculations can be tedious so a spreadsheet was developed to calculate these deflections and can be found in Appendix D. Below is a summary of the deflection output compared to the deflection limits.

Girder Deflections			
	Computed	Allowable	Pass?
Δi(d+l)	0.11 in	1.55 in	Yes
Δi(live)	0.06 in	1.03 in	Yes
Δ long	0.2 in	1.55 in	Yes

Edge Beams

The edge beams are located around the exterior perimeter of the building. On one side it supports the exterior precast façade, to be discussed later in the report, and on the other side it supports two interior beams that support the slab. Figure 19 shows a plan view of the edge beam layout for a typical bay.

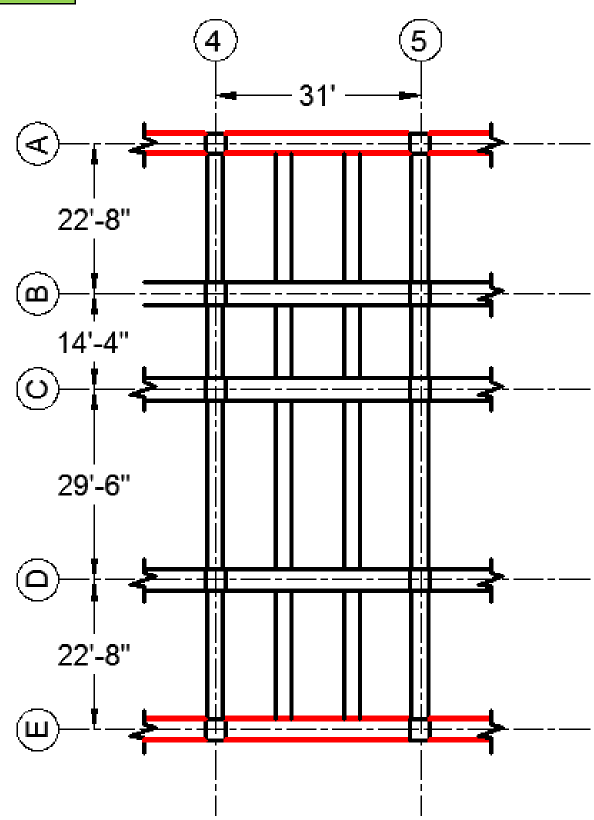
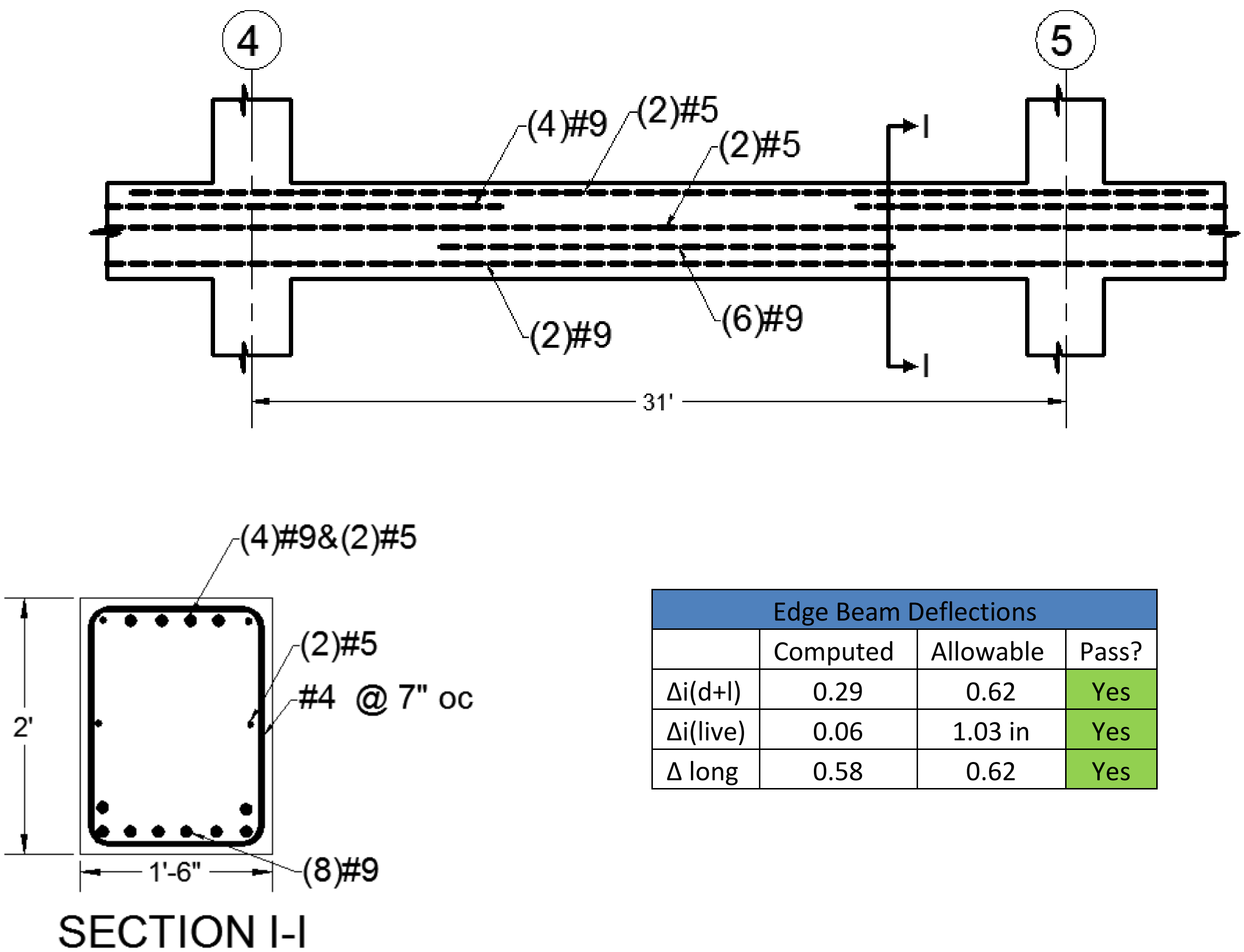


Figure 19: Typical Edge Beam Layout

The edge beams for the BBH Building were the most complex as far as design goes. Unlike the girders, the beams only support the floor on one side, which produces a large torsion in the edge beam and will need to be taken into account in the design. Once the torsion has cracked the beam the torsional resistance is provided by closed stirrups and longitudinal reinforcement. The torsional design of the beam was done using section 11.5 of ACI 318-11. The structure of the BBH Building is a statically indeterminate structure which means the torsional moment can be reduced because moment redistribution occurs after the concrete cracks. The factored torsional moment can now be calculated using the equation located in section 11.5.2(a) in ACI. From there the reinforcement needed to resist the torsion was designed. These calculations can be found in Appendix C.

Unlike the other flexural members, the cross section and longitudinal reinforcement were controlled by deflection and not bending. The edge beams support a thin brick façade that is sensitive to deflection. The industry standard for deflection limitations for members supporting masonry is L/600. After the beam was designed for bending, calculations were done to determine the immediate and long term deflections. Adjustments to the design were then made in order to decrease the deflections to be under the limit. Figure 20 shows the final cross section and reinforcement layout for a typical edge beam along with a summary of the deflection output compared to the deflection limits.



Edge Beam Deflections			
	Computed	Allowable	Pass?
Δi(d+l)	0.29	0.62	Yes
Δi(live)	0.06	1.03 in	Yes
Δ long	0.58	0.62	Yes

Figure 20: Typical Edge Beam Reinforcement

Columns

For this concrete redesign, the column layout of the existing structure was used. This was done in order to not impact the floor layout and architecture. Preliminary sizes of the columns were determined by making the column large enough to accept the beam/girder that came into it.The sizes chosen are 18”x18” and 18”x20”. Also, it was determined that the columns would be prismatic from the whole way up through the building. Although the columns do support gravity loads, they also are a major part of the lateral design and therefore were primarily designed when the lateral system was. This will be shown in the next section.

Lateral System Redesign

Moment Frame

The next step in the completion of the concrete redesign is to take a look at how the structure is going to resist lateral loads. Two main lateral systems used in concrete design are shear walls and concrete moment frames. Due to the architecture of the building concrete moment frames where chosen over shear walls. Because of inconstancies in the room layout between some floors, there was not a reasonable place to place a shear wall that would extend the full height of the building. Therefore the design of the lateral system would fully consist of concrete moment frames. This system causes no impact on the architecture of the building because the beams and columns of a concrete structure already act as a moment frame.

Even though the monolithic construction of the concrete structure naturally creates a lateral resisting frame, only certain frames were treated as such. The layout of the lateral system is similar to that of the steel structure. The only difference is that more moment frames had to be added on the east side of the building unlike the steel design, which used an eccentric braced frame at column line 10. Figure 21 shows the layout of the moment frames for the BBH Building.

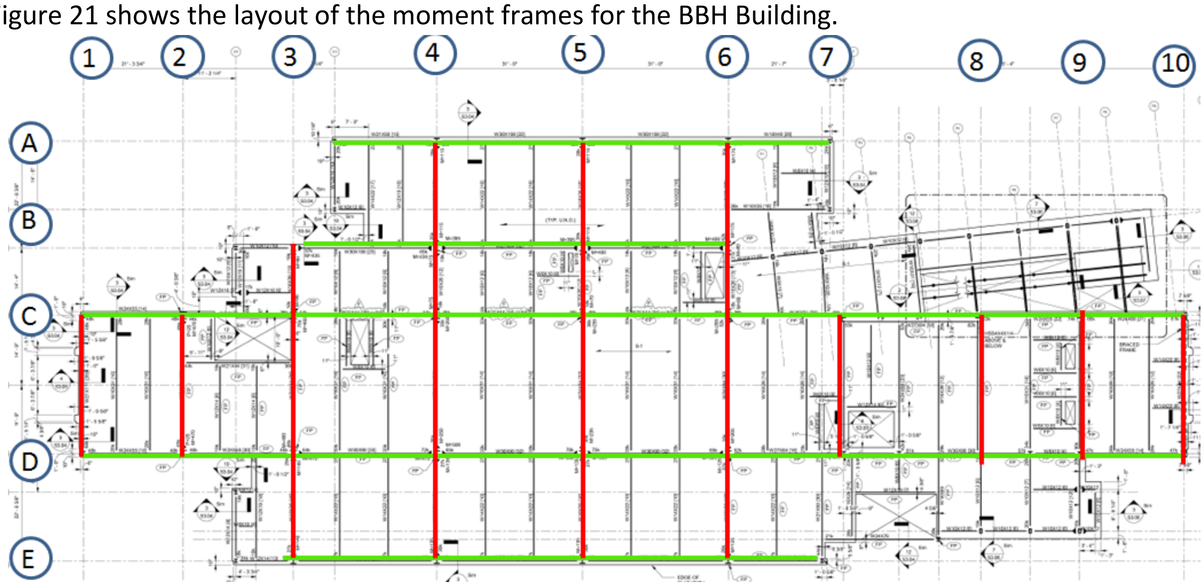


Figure 21: Moment Frame Layout

ETABS

In order to analyze the lateral system more efficiently, the structural program ETABS was used. Several concrete frame sections were created in the program. All of the frame members, such as columns, and beams were modeled using line elements. These line elements were each defined with the correct frame sections and material properties as designed. It was assumed that all base connections would be fixed. When drawing 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. But connections between beams and columns that were not considered for the moment frames needed to be released from bending resistance. 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 frames at each level so the lateral forces can be distributed to the frames based on their respective stiffness. Figure 22 and Figure 23 show 3D views of the model.

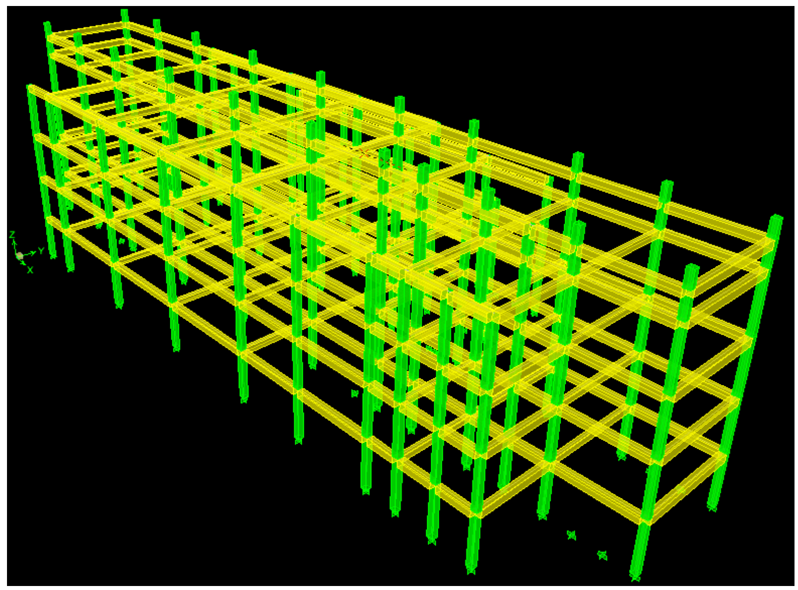


Figure 22: ETABS Lateral Structure

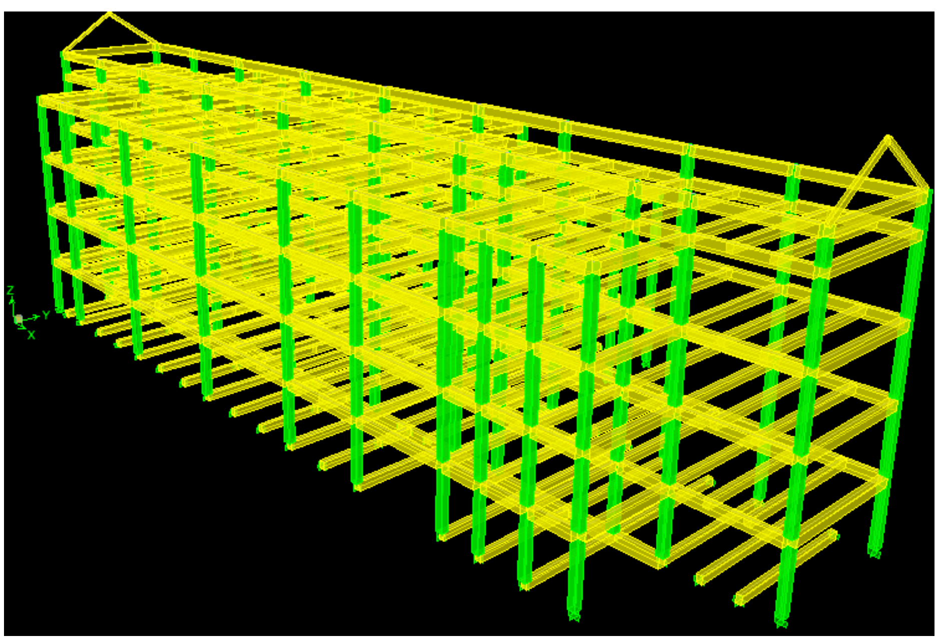


Figure 23: ETABS Gravity Structure

From the wind and seismic load calculations it was determined that wind loads control in both directions. These wind loads were placed in the computer program. Due to the eccentricity between the center of rigidity and the location of the applied wind load, a torsional moment was developed and needed to be taken into account. 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.

Manual calculations were done using excel to verify that the values from ETABS were correct. This was done by determining the relative stiffness of each frame. 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 each individual frame using separate load cases. After running the analysis in ETABS, deflection measurements were taken at the PH story of each frame using their respective load case. To find the relative stiffness, each frame's calculated stiffness was divided by the largest stiffness value. These values give us a better sense of how the lateral forces get distributed to the frames at each level. Using the calculated stiffness along with the wind loads, excel was used to determine the distribution of forces between the frames at the PH level. Table1 shows a sample of the excel spread sheet and Fig 24 shows a snapshot of the model with the wind loads applied. Comparing the two it was determined that the ETABS model is correct. Appendix E shows the complete excel output.

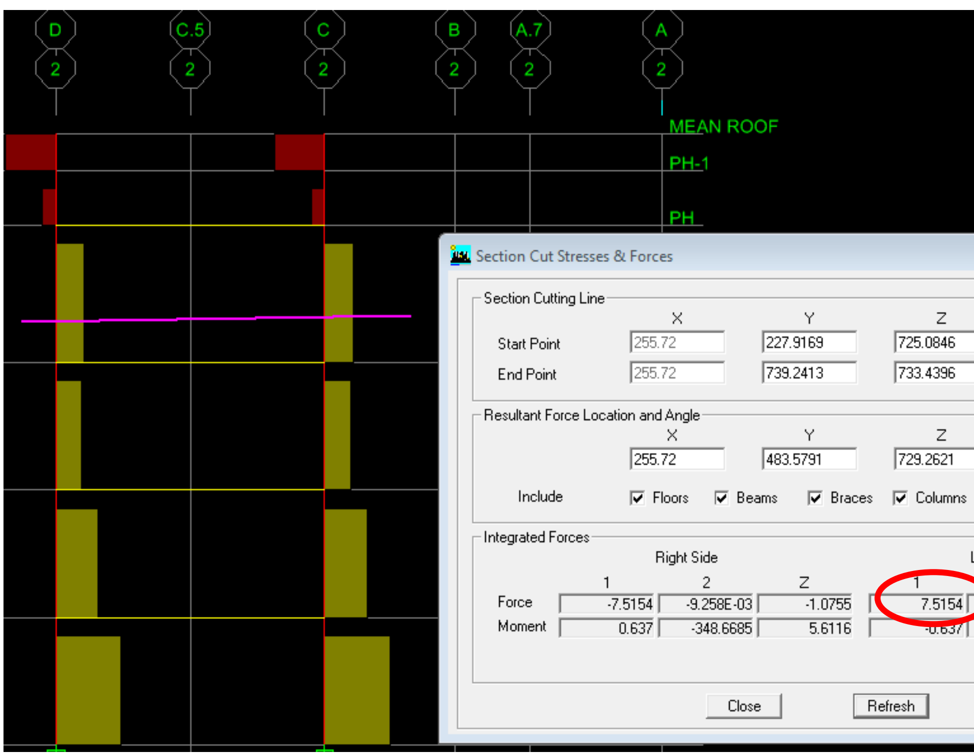


Figure 24: ETABS Output Snapshot

PH Level					
N-S	ft	E-W	ft		ft
CR=	34	CR=	106	.15By=	36
CM=	32	CM=	115	.15Bx=	13
CR-CP=	2.00	CR-CP=	-9.00		

W1: Case 1 NS			
F_{NS} (kip)	112.3	e_{NS} (ft)	9
F_{EW} (kip)	0	e_{EW} (ft)	0
M_{NS} (k-ft)	1010.7		
M_{EW} (k-ft)	0		

Frame	K (k/in)	ΣK_{NS} (k/in)	ΣK_{EW} (k/in)	Direct shear (kip)	d (in)	Kd^2	∑Kd^2	Torsional Moment Shear (kip)	Total shear (kip)
B	0.5502	0.00	4.23	0.00	32.50	581.15	17444.38	1.04	1.04
C	1.2902	0.00	4.23	0.00	18.20	427.38	17444.38	1.36	1.36
D	1.3881	0.00	4.23	0.00	-11.30	177.25	17444.38	-0.91	-0.91
E	1.0000	0.00	4.23	0.00	-34.00	1156.00	17444.38	-1.97	-1.97
1	0.1972	3.51	0.00	6.32	-106.00	2215.32	17444.38	-1.21	5.10
2	0.2560	3.51	0.00	8.20	-84.70	1836.50	17444.38	-1.26	6.94
3	0.3437	3.51	0.00	11.01	-61.30	1291.35	17444.38	-1.22	9.79
4	0.4860	3.51	0.00	15.57	-30.70	458.02	17444.38	-0.86	14.70
5	0.5848	3.51	0.00	18.73	0.30	0.05	17444.38	0.01	18.74
6	0.5309	3.51	0.00	17.00	31.30	520.08	17444.38	0.96	17.97
7	0.4180	3.51	0.00	13.39	55.90	1306.04	17444.38	1.35	14.74
8	0.2922	3.51	0.00	9.36	85.00	2111.43	17444.38	1.44	10.80
9	0.2248	3.51	0.00	7.20	106.00	2525.70	17444.38	1.38	8.58
10	0.1727	3.51	0.00	5.53	128.20	2838.10	17444.38	1.28	6.81

Table 1: Manual Wind Shear Calculations

Lateral Design

Once the ETABS model was proven to be correct it could now be used to aid in the design of the columns that are part of the moment frames. The model was used to determine the moment and axial acting on a column due to the controlling load case. Three columns were checked and designed using these loads. Figure 25 shows the cross sections of the columns after they were designed.

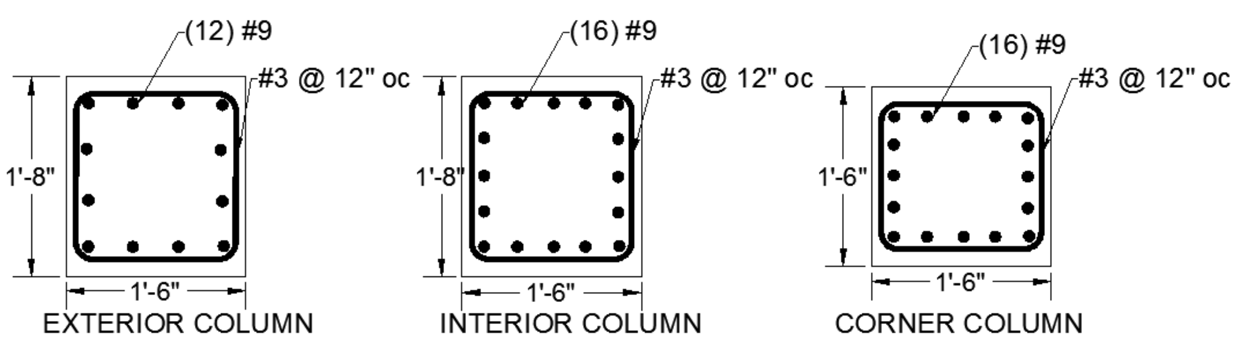


Figure 25: Column Cross Sections

Appendix E shows the hand calculations done to determine the amount of reinforcement required to handle the loads. Charts from Wight and Macgregor (see reference page) were used to estimate the reinforcement ratio for the columns. The results of the hand calculations were checked with the design results of ETABS and were determined to be sufficient. Since the beams that are part of the lateral system were already designed for gravity loads, all that had to be done was to check to see if the design was sufficient for lateral loads as well. Looking at the design output from ETABS it was determined that the design was also sufficient to carry the applied lateral loads as well as gravity.

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. The load factor of .9D+L+W was 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.

Displacement - X Direction					
Story	Height	Story Height	Displacement	H/400	Pass?
	(ft)	(ft)	(in)	(in)	Yes
PH	57	15	0.18	1.71	Yes
4	42	14	0.15	1.26	Yes
3	28	14	0.11	0.84	Yes
2	14	14	0.05	0.42	Yes

Displacement - Y Direction					
Story	Height	Story Height	Displacement	H/400	Pass?
	(ft)	(ft)	(in)	(in)	Yes
PH	57	15	1.6	1.71	Yes
4	42	14	1.25	1.26	Yes
3	28	14	0.82	0.84	Yes
2	14	14	0.31	0.42	Yes

Overturning and Foundation Impact

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	42	74.23	3118
3	25	67.12	1678
2	14	61.48	861
		Overturning Moment=	12,560 k-ft

Resisting Moment
M_{resist}=8,473k x 89'/2 x .67 = 252,622 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.).

In order to check if the foundations needed to be change the weight of the new structure was compared to the existing. Naturally a concrete structure would weigh more than a steel structure. But due to the change in the façade system, it was determined that the total weight of new structure was similar to the existing (the new façade system will be explained more in the next section). Therefore it was concluded by inspection, of the weight of the new and existing, the foundations would not need to be redesigned. Appendix B shows the estimation of the building's weight.

Cost Impact

A cost impact was done between the existing and redesigned structures. These costs were found by using the information provided in RS Means: Building Construction Cost Data. The table below shows a quick comparison of the costs. The new concrete structure proves to be much more expensive than the existing steel structure. Appendix F shows a more detailed breakdown of the costs.

Steel		Concrete	
Structural steel	$297,025	Beams	$220,000
Metal Deck	$49,345	Columns	$94,504
1" Spray on Fireproofing	$12,968	Slab	$141,000
Concrete fill	$48,183	Total	$455,504
Total	$407,521		
% increses	12%		
Cost Diff	$47,984		

Façade Study Breadth

Existing Façade

The existing façade of the BBH Building consists of 8" modular brick supported by steel angles welded to the exterior structural steel beams. Behind the brick is 8" fully grouted CMU infill wall that support the brick laterally. This type of façade is very expensive and time consuming to install and also has a considerably large weight to it. In order to offset the time, cost, and weight increase of the concrete construction an alternative façade would be looked into. The specifics of the actual architectural look of the precast will not be covered in this breadth. It is important to note that the textural look of the precast will be the most noticeable difference between the traditional hand placed brick and the precast panels.

Thin Brick Precast

For the new façade, a thin brick precast design would be used. This type of system would come in precast/pre-engineered panels. These panels will be cast off site at a certified precast plant where they can be made under precise conditions and not be in the way of onsite construction. These ideal conditions allow for the precasting plant to be precise in the size, pattern, shape, and quality of the concrete mixing. Since the precast panels are made off site they can be made ahead of time and shipped to the site whenever it is time to place them on the building. According to the Precast Concrete Institute (PCI), erection crews can put up approximately 3000-4500 square feet of precast panels per day. This will greatly decrease the construction time that is involved in getting the building closed in.

The steps of constructing a thin brick precast panel are as follows:

1. Prepare and construct the precast forms.
2. Place a form liner in the casing bed for the placement of thin bricks.
3. Place Thin brick in form liner
4. Place reinforcement
5. Place Concrete over thin brick and reinforcement
6. Once concrete is to full strength the panel can be lifted and stripped of the form liner and cleaned

From here the panels are ready to be sent to the site and put into place when the contractor requests.

The thin bricks come in multiple shapes, textures, colors, and sizes. The architect and/or owner should consult with the manufacturer and precaster to determine these things early in the design process. Figure 26 shows some of the shapes that the thin brick comes in.

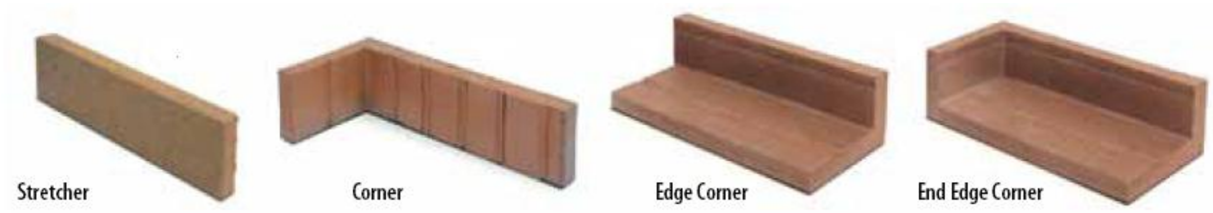


Figure 26: Typical Thin Brick Shapes

Thin Brick Proposal for BBH

The first step in design of the precast panels is to determine its size. Larger panels are desired in order to limit the number of panels that needed to be place and to limit the number of construction joints. The transportation of the panel was determined to be the major controlling factor in how big the panel could be. The motor carrier height restrictions from PennDot state that the overall height of a transport vehicle cannot exceed 13.5 feet and can be a maximum of 65 feet long. Using these restrictions a panel size of 30 feet long and 14' high will be used. In order to be transported on a truck it will have to be tilted on the bed. Figure 27 shows a sketch of the typical panel and Figure 28 shows the layout of these panels on the north elevation.

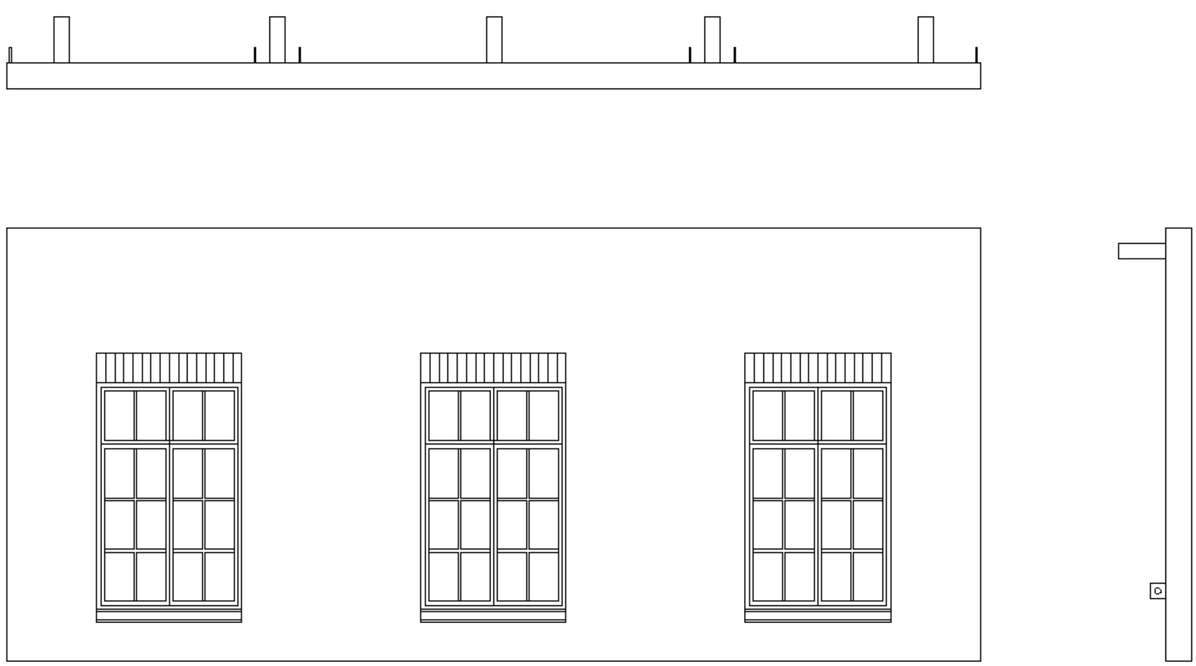


Figure 27: Sketch of Typical Panel

Figure 28: Typical Panel Layout

These panels will be lifted off the truck when they arrive and hung on the edge beams that were designed in the structural depth of this report. The method in which they will be hung will be determined by the engineer who designed the panel. It will be assumed that the panel will be hung by HSS tubes that are attached to them. Kickbacks will also be used to support the panels from swaying and will be placed so that they are concealed in office partitions so that they do not interfere with the interior layout. Figure29 a typical connection detail provided by the Precast Concrete Institute (PCI).

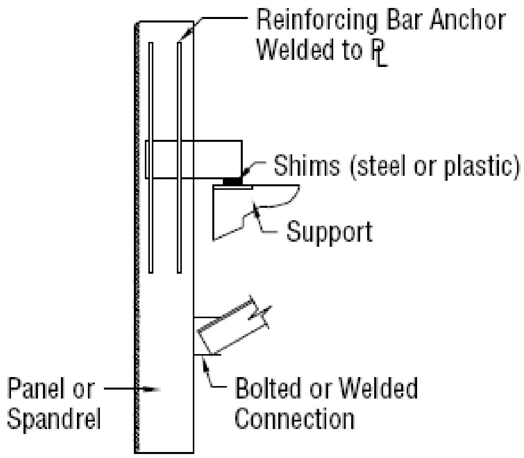


Figure 29: Typical Connection Detail

Heat and Moisture Resistance

Like any façade it is important to make sure that it can limit the heat transfer through the wall and keep the outside air moisture from leaking into the building. It is important to limit the infiltration of these things because they can cause major damage to the façade, interior finishes, and the buildings mechanical system. In order to make sure this does not happen, a comparison was made between the already capable existing façade and the new precast façade. The table below shows a comparison of the heat resistance values.

Existing Wall			Precast Wall		
Material	Thickness (in)	R-Value	Material	Thickness (in)	R-Value
Brick	4	0.4	Thin Brick	1	0.1
Air Cavity	2.375	1	Sandwich Panel Walls	4	4.4
Rigid Insulation	2	4.35	Air Cavity	18.5	1
Fully Grouted CMU	8	2.3	Wall Insulation Between Studs(Total R=13)	4	5.1
Air Cavity	10.5	1	GWB	0.625	0.45
GWB	0.625	0.45	Total Resistance		11.05
Total Resistance		9.5			

The precast was able to achieve at least the same R value as the existing by using a "sandwich" wall design along with adding insulation in the stud wall cavity. Figure 30 shows a typical section of a precast sandwich panel wall taken from PCI compared to an existing section of the BBH building.

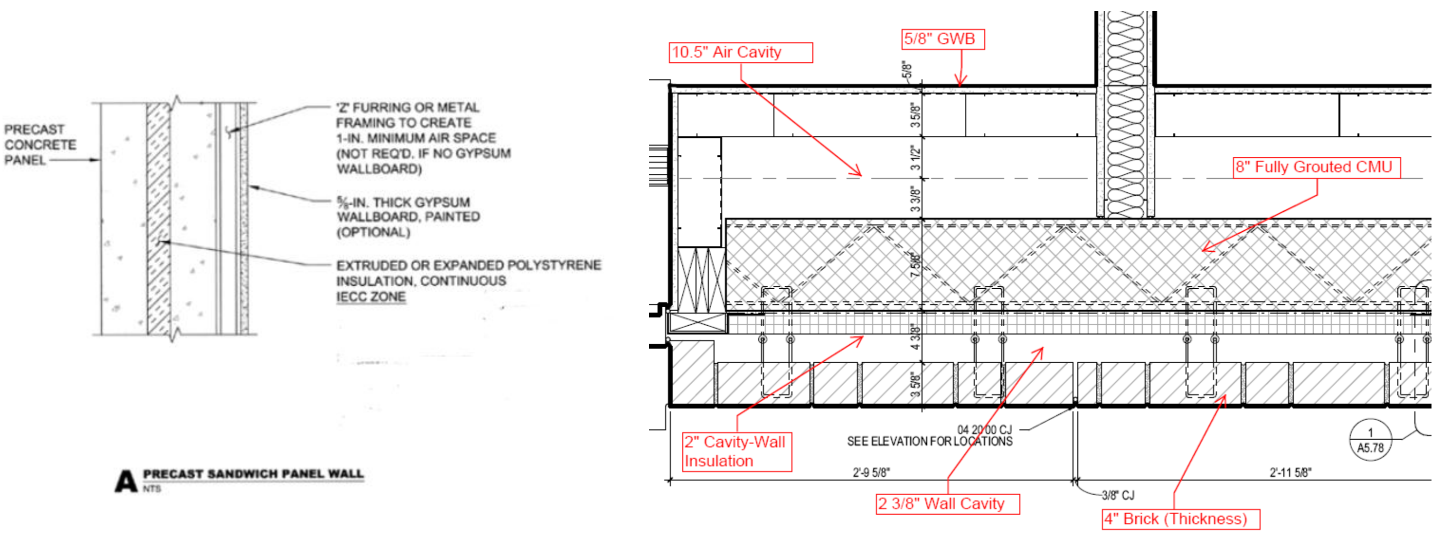


Figure 30: Typical Sandwich Wall Section (Left) & Typical Existing Wall Section (Right)

Fortunately this type of façade is dense enough that it allows minimal air leakage through it. Therefore no air barrier of flashing is required. Lastly the joints between the panels and windows need to be sealed with at silicone sealant. This will prevent any leakages between the joints of the system and will also keep the panels from pushing into each other due to temperature and wind effects.

Cost Impact

A cost impact was done between the existing and precast façade design. These costs were found by using the information provided in RS Means: Building Construction Cost Data. The table below shows a quick comparison of the costs. The new façade proves to be much less expensive than the existing façade. Appendix F shows a more detailed breakdown of the costs.

Traditional Masonry		Precast Panels	
Brick	$156,142	Brick	$336,938
CMU	$86,289	Cornice	$81,250
Cleaning & Repointing	$57,526	Total	$418,188
Grouting	$152,185		
Reinf	$8,000		
Scaffolding	$12,500		
Total	$472,642		
% decrease	40%		
Cost Diff	$54,454		

Schedule Analysis Breadth

A schedule analysis was done to determine how efficient the new design of the BBH Building is compared to the existing design. Parts of the schedule that are not impacted by the redesign would be assumed to be the same. The impacted parts of the schedule were assumed to be from when the foundations were finished to when the exterior façade was complete. Massaro CM Services provided an existing schedule that was used to determine key dates and durations. From their schedule it was determined that the foundations from column line 1to 5 would be completed at the end of April 2011. This would be the starting date of this schedule analysis. Key dates were pulled from the existing schedule and are summarized below.

-Steel Skeleton and Deck Detailing completed November 1, 2011

-CMU infill walls completed January 5, 2012

-Brick & Limestone façade completed October 26, 2012

The construction of the new concrete structure would be split up into two sequences. The first sequence will be to construct from the ground level to the penthouse from column lines 1 to 5. Sequence two will follow the same schedule for column lines 5 to 10 after the first sequence is complete. Splitting the construction up like this will allow for the MEP contractors to start their work in column lines 1-5 while the rest of the building is being constructed. The completion of the precast panels were scheduled in a way that they will be completed 28 days after the concrete at the PH level had time to cure. Figure 31 is the revised schedule, done in Microsoft Project, for this construction. A summary of the notes for the durations can be found in Appendix F.

ID		Task Mode	Task Name	Duration	Start	Finish
1			1-5 Ground: Column Reinf.	3 days	Mon 5/2/11	Wed 5/4/11
2			1-5 Ground: Column Forming	5 days	Tue 5/3/11	Mon 5/9/11
3			1-5 Ground: Column Pour	1 day	Tue 5/10/11	Tue 5/10/11
4			1-5 Level 1: Beam and Slab Forming	15 days	Thu 5/5/11	Wed 5/25/11
5			1-5 Level 1: Beam and Slab Reinf	4 days	Fri 5/20/11	Wed 5/25/11
6			1-5 Level 1: Beam and Slab Pour	2 days	Thu 5/26/11	Fri 5/27/11
7			1-5 Level 1: Column Reinf.	3 days	Thu 6/2/11	Mon 6/6/11
8			1-5 Level 1: Column Forming	5 days	Fri 6/3/11	Thu 6/9/11
9			1-5 Level 1: Column Pour	1 day	Fri 6/10/11	Fri 6/10/11
10			1-5 Level 2: Beam and Slab Forming	15 days	Tue 6/7/11	Mon 6/27/11
11			1-5 Level 2: Beam and Slab Reinf.	4 days	Wed 6/22/11	Mon 6/27/11
12			1-5 Level 2: Beam and Slab Pour	2 days	Mon 6/27/11	Tue 6/28/11
13			1-5 Level 2: Column Reinf.	3 days	Mon 7/4/11	Wed 7/6/11
14			1-5 Level 2: Column Forming	5 days	Tue 7/5/11	Mon 7/11/11
15			1-5 Level 2: Column Pour	1 day	Tue 7/12/11	Tue 7/12/11
16			1-5 Level 3: Beam and Slab Forming	15 days	Thu 7/7/11	Wed 7/27/11
17			1-5 Level 3: Beam and Slab Reinf.	4 days	Fri 7/22/11	Wed 7/27/11
18			1-5 Level 3: Beam and Slab Pour	2 days	Thu 7/28/11	Fri 7/29/11
19			1-5 Level 3: Column Reinf.	3 days	Thu 8/4/11	Mon 8/8/11
20			1-5 Level 3: Column Forming	5 days	Fri 8/5/11	Thu 8/11/11
21			1-5 Level 3: Column Pour	1 day	Fri 8/12/11	Fri 8/12/11
22			1-5 Level 4: Beam and Slab Forming	15 days	Mon 8/8/11	Fri 8/26/11
23			1-5 Level 4: Beam and Slab Reinf.	4 days	Tue 8/23/11	Fri 8/26/11
24			1-5 Level 4: Beam and Slab Pour	2 days	Mon 8/29/11	Tue 8/30/11
25			1-5 Level 4: Column Reinf.	2 days	Mon 9/5/11	Tue 9/6/11
26			1-5 Level 4: Column Forming	3 days	Tue 9/6/11	Thu 9/8/11
27			1-5 Level 4: Column Pour	1 day	Fri 9/9/11	Fri 9/9/11
28			1-5 PH: Beam and Slab Forming	7 days	Wed 9/7/11	Thu 9/15/11
29			1-5 PH: Beam and Slab Reif	2 days	Wed 9/14/11	Thu 9/15/11
30			1-5 PH: Beam and Slab Pour	1 day	Fri 9/16/11	Fri 9/16/11
31			1-5: Precast Panel Placement	15 days	Wed 10/12/11	Tue 11/1/11
32			5-10 Ground: Column Reinf.	3 days	Fri 9/9/11	Tue 9/13/11
33			5-10 Ground: Column Forming	5 days	Mon 9/12/11	Fri 9/16/11
34			5-10 Ground: Column Pour	1 day	Mon 9/19/11	Mon 9/19/11
35			5-10 Level 1: Beam and Slab Forming	15 days	Wed 9/14/11	Tue 10/4/11
36			5-10 Level 1: Beam and Slab Reinf	4 days	Thu 9/29/11	Tue 10/4/11
37			5-10 Level 1: Beam and Slab Pour	2 days	Wed 10/5/11	Thu 10/6/11
38			5-10 Level 1: Column Reinf.	3 days	Wed 10/12/11	Fri 10/14/11
39			5-10 Level 1: Column Forming	5 days	Thu 10/13/11	Wed 10/19/11
40			5-10 Level 1: Column Pour	1 day	Thu 10/20/11	Thu 10/20/11
41			5-10 Level 2: Beam and Slab Forming	15 days	Mon 10/17/11	Fri 11/4/11
42			5-10 Level 2: Beam and Slab Reinf.	4 days	Tue 11/1/11	Fri 11/4/11
43			5-10 Level 2: Beam and Slab Pour	2 days	Mon 11/7/11	Tue 11/8/11
44			5-10 Level 2: Column Reinf.	3 days	Mon 11/14/11	Wed 11/16/11
45			5-10 Level 2: Column Forming	5 days	Tue 11/15/11	Mon 11/21/11
46			5-10 Level 2: Column Pour	1 day	Tue 11/22/11	Tue 11/22/11
47			5-10 Level 3: Beam and Slab Forming	15 days	Thu 11/17/11	Wed 12/7/11
48			5-10 Level 3: Beam and Slab Reinf.	4 days	Fri 12/2/11	Wed 12/7/11
49			5-10 Level 3: Beam and Slab Pour	2 days	Tue 12/13/11	Wed 12/14/11
50			5-10 Level 3: Column Reinf.	3 days	Tue 12/20/11	Thu 12/22/11
51			5-10 Level 3: Column Forming	5 days	Wed 12/21/11	Tue 12/27/11
52			5-10 Level 3: Column Pour	1 day	Wed 12/28/11	Wed 12/28/11
53			5-10 Level 4: Beam and Slab Forming	15 days	Fri 12/23/11	Thu 1/12/12
54			5-10 Level 4: Beam and Slab Reinf.	4 days	Mon 1/9/12	Thu 1/12/12
55			5-10 Level 4: Beam and Slab Pour	2 days	Fri 1/13/12	Mon 1/16/12
56			5-10 Level 4: Column Reinf.	2 days	Mon 1/23/12	Tue 1/24/12
57			5-10 Level 4: Column Forming	3 days	Tue 1/24/12	Thu 1/26/12
58			5-10 Level 4: Column Pour	1 day	Fri 1/27/12	Fri 1/27/12
59			5-10 PH: Beam and Slab Forming	7 days	Wed 1/25/12	Thu 2/2/12
60			5-10 PH: Beam and Slab Reif	2 days	Wed 2/1/12	Thu 2/2/12
61			5-10 PH: Beam and Slab Pour	1 day	Fri 2/3/12	Fri 2/3/12
62			5-10: Precast Panel Placement	15 days	Fri 2/24/12	Thu 3/15/12

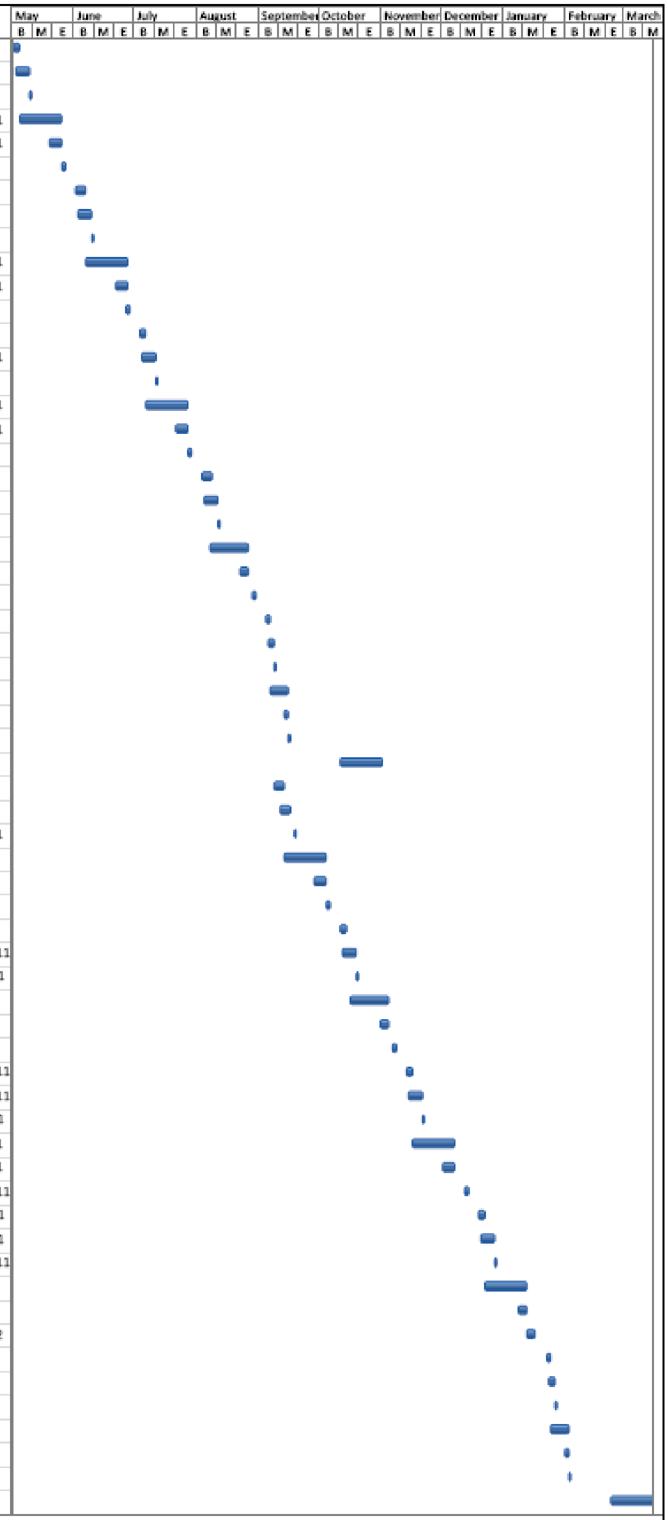


Figure 31: Adjusted Schedule

From the constructed schedule it was determined that the full concrete structure would be complete February 3, 2012. This is three months longer than what the existing steel structure took. While this may have taken longer it was determined that the new precast façade would be completed 6 months earlier than the existing façade construction. Therefore it can be concluded that while the construction of the concrete took longer, the precast façade made up for the loss of time by enclosing the building sooner than the existing façade construction. This will allow for the finishes of the BBH building to start sooner, which could possible lead to a much sooner completion date.

Conclusion

In summary, the goal of redesigning the structure of the BBH Building with reinforced concrete was successful. One way slabs with beams were designed to support the design loads that were applied to them. Three typical beam sections were analyzed and designed to resist flexure, shear, and torsion. The sections were also designed to satisfy immediate and long term deflection limits stated in the IBC. The girders were the deepest sections with a total depth of 28”.

ETABS was used to aid in the analysis and design of the concrete moment frames that were designed to resist lateral loads by the controlling wind cases in both directions. Excel spread sheets were used to determine how the lateral loads were distributed between the moment frames. The output from excel was then compared to that output of ETABS and it was determined that the ETABS model was correct. The output from ETABS provided axial and bending values for the columns. These values were used to design three typical columns for the building. Reinforcement values for both beams and columns were cross checked with the hand calculations to verify that adequate amounts of reinforcement were provided to resist the applied gravity and lateral loads. Once the structure was fully designed and check for strength, deflections for lateral displacement were done and it was determined that all displacements were under the H/400 limit stated in ASCE 7-05. A brief cost analysis was done between the existing steel structure and redesigned concrete structure and it was determined that the redesign would cause a 12% increase in cost.

In order to possibly help alleviate the cost and possible schedule increase, a thin brick precast façade was researched as an alternative to the existing traditional hand laid brick façade. Information from the Precast/Prestressed Concrete Institute (PCI) helped in the determination of the heat and moisture resistance requirements needed to match that of the existing façade. A brief cost analysis was done between the existing and precast facades and it was determined that eh precast would cause a 40% decrease in cost. This would alleviate the cost impact from the concrete redesign. Therefore it was determined the precast façade would be a good alternative to the existing, although the owner and architect would have to accept the more manufactured look of the panels compared to the traditional look that the hand laid brick would provide.

Finally a schedule was produced to determine what kind of impact the above changes would cause to the project. It was determined that even though the constructing of the concrete redesign would take 3 months longer, the precast façade would allow for the entire building to be enclosed 6 months earlier than the what was originally scheduled. This would allow for interior finishes to start earlier, which could provide a positive impact on the project’s completion.

References

Wight, James K., and James G. Macgregor. *Reinforced Concrete: Mechanics & Design*. 5th ed. Upper Saddle River: Pearson Prentice Hall, 2009.

"Energy Conservation and Condensation Control." *Designer's Notebook*. Precast/Prestressed Concrete Insitute, n.d. Web. 28 March. 2013. <http://www.pci.org>.

"Clay Product-Faced Precast Concrete." *Designer's Notebook*. Precast/Prestressed Concrete Insitute, n.d. Web. 28 Jan. 2013. <http://www.pci.org>.

Waier, Phillip R. *RSMeans Building Construction Cost Data 2012*. Norwell, MA: RSMeans, 2011.

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

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

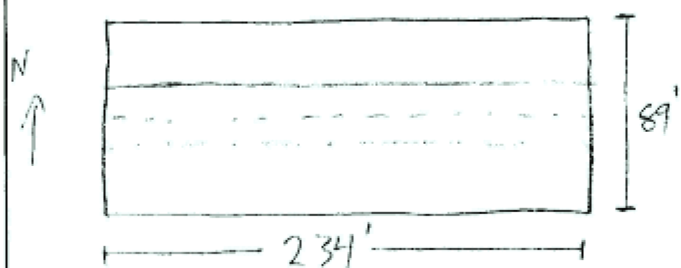
Daniel Bodde	Tech 1	Wind Calc	2

6.5.11.1 Internal Pressure Coefficient, GC_{pi}

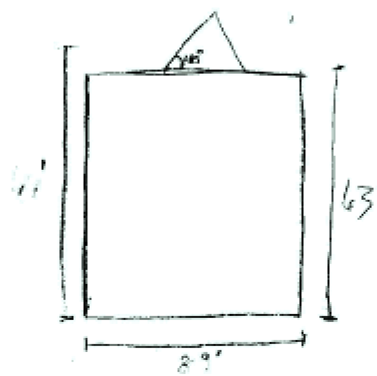
$GC_{pi} = \pm 0.18$ for enclosed buildings (Fig 6-5)

6.5.12.2.1 Design wind pressures for the MWFRS

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



plan



elevation

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

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

N-S Wall

Windward Wall $C_p = 0.8$

Leeward Wall :

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

$C_p = -0.5$

E-W Wall

WW Wall $C_p = 0.8$

LW Wall :

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

Interpolate to find C_p

$C_p = -0.27$

Daniel Bodde	Tech 1	Wind Calc	3

Roof (N-S only)

$\theta = 40° \geq 10°$

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

Interpolate to find Cp

WW Cp = -.1 & .3

LW Cp: -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
	63	15.14	13.10
roof	67	15.51	6.75

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	11.63	10.63
30	12.34	11.12
40	13.40	11.84
50	14.28	12.44
60	14.98	12.92
63	15.14	13.04

LW

$P = -6.21$ psf

Daniel Bodde	Tech 1	Wind Calc	4

Wind Force at each story (N-S)

2nd

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

3rd

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

4th

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

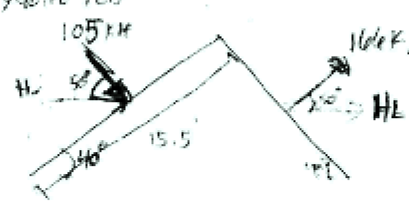
PH

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

Bottom of roof

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

Horiz Force On gabled roof



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

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

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

Daniel Bodde	Tech 1	Wind calc	5

Wind Force at each Story (E-W)

2nd

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

$$= \boxed{19.60K}$$

3rd

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

$$= \boxed{21.69K}$$

4th

$$\left[(11.84)(5.5) + (12.44)(9.75') + (6.21)(14.75)\right]\frac{89}{1000}$$

$$= \boxed{24.19K}$$

PH

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

$$= \boxed{20.42K}$$

Bot of roof

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

Base Shear

N-S

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

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

E-W

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

Daniel Bodde	Tech 1	Wind Calc	6

Overturning Moment

N-S

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

E-W

$$M_{E-W} = (19.6)(13.5) + (21.69)(22.5) + (24.19)(41.5)$$
$$+ (20.48)(37) + (3.14)(63)$$
$$= 3354.1\ K\text{-}ft$$

PH	PH Level					
	N-S	ft	E-W	ft		ft
	CR=	34	CR=	106	.15By=	36
	CM=	32	CM=	115	.15Bx=	13
	CR-CP=	2.00	CR-CP=	-9.00		

W1	W1: Case 1 NS			
	F_{NS} (kip)	112.3	e_{NS} (ft)	9
	F_{EW} (kip)	0	e_{EW} (ft)	0
	M_{NS} (k-ft)	1010.7		
	M_{EW} (k-ft)	0		

Frame	K (k/in)	ΣK_{NS} (k/in)	ΣK_{EW} (k/in)	Direct shear (kip)	d (in)	Kd^2	∑Kd^2	Torsional Moment Shear (kip)	Total shear (kip)
B	0.5502	0.00	4.23	0.00	32.50	581.15	17444.38	1.04	1.04
C	1.2902	0.00	4.23	0.00	18.20	427.38	17444.38	1.36	1.36
D	1.3881	0.00	4.23	0.00	-11.30	177.25	17444.38	-0.91	-0.91
E	1.0000	0.00	4.23	0.00	-34.00	1156.00	17444.38	-1.97	-1.97
1	0.1972	3.51	0.00	6.32	-106.00	2215.32	17444.38	-1.21	5.10
2	0.2560	3.51	0.00	8.20	-84.70	1836.50	17444.38	-1.26	6.94
3	0.3437	3.51	0.00	11.01	-61.30	1291.35	17444.38	-1.22	9.79
4	0.4860	3.51	0.00	15.57	-30.70	458.02	17444.38	-0.86	14.70
5	0.5848	3.51	0.00	18.73	0.30	0.05	17444.38	0.01	18.74
6	0.5309	3.51	0.00	17.00	31.30	520.08	17444.38	0.96	17.97
7	0.4180	3.51	0.00	13.39	55.90	1306.04	17444.38	1.35	14.74
8	0.2922	3.51	0.00	9.36	85.00	2111.43	17444.38	1.44	10.80
9	0.2248	3.51	0.00	7.20	106.00	2525.70	17444.38	1.38	8.58
10	0.1727	3.51	0.00	5.53	128.20	2838.10	17444.38	1.28	6.81

W2	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)	ΣK_{NS} (k/in)	ΣK_{EW} (k/in)	Direct shear (kip)	d (ft)	Kd^2	∑Kd^2	Torsional Moment Shear (kip)	Total shear (kip)
B	0.5502	0.00	4.23	3.33	32.50	581.15	17444.38	0.05	3.38
C	1.2902	0.00	4.23	7.81	18.20	427.38	17444.38	0.07	7.88
D	1.3881	0.00	4.23	8.40	-11.30	177.25	17444.38	-0.05	8.36
E	1.0000	0.00	4.23	6.05	-34.00	1156.00	17444.38	-0.10	5.95
1	0.1972	3.51	0.00	0.00	-106.00	2215.32	17444.38	-0.06	-0.06
2	0.2560	3.51	0.00	0.00	-84.70	1836.50	17444.38	-0.06	-0.06
3	0.3437	3.51	0.00	0.00	-61.30	1291.35	17444.38	-0.06	-0.06
4	0.4860	3.51	0.00	0.00	-30.70	458.02	17444.38	-0.04	-0.04
5	0.5848	3.51	0.00	0.00	0.30	0.05	17444.38	0.00	0.00
6	0.5309	3.51	0.00	0.00	31.30	520.08	17444.38	0.05	0.05
7	0.4180	3.51	0.00	0.00	55.90	1306.04	17444.38	0.07	0.07
8	0.2922	3.51	0.00	0.00	85.00	2111.43	17444.38	0.07	0.07
9	0.2248	3.51	0.00	0.00	106.00	2525.70	17444.38	0.07	0.07
10	0.1727	3.51	0.00	0.00	128.20	2838.10	17444.38	0.06	0.06

W3	Case 2 NS + .15By									
	$.75F_{NS}$ (kip)	84.225			e_{NS} (ft)	45				
	$.75F_{EW}$ (kip)	0			e_{EW} (ft)	0				
	M_{NS} (k-ft)	3790.125								
	M_{EW} (k-ft)	0								
	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)
	B	0.5502	0.00	4.23	0.00	32.50	581.15	17444.38	3.89	3.89
	C	1.2902	0.00	4.23	0.00	18.20	427.38	17444.38	5.10	5.10
	D	1.3881	0.00	4.23	0.00	-11.30	177.25	17444.38	-3.41	-3.41
	E	1.0000	0.00	4.23	0.00	-34.00	1156.00	17444.38	-7.39	-7.39
	1	0.1972	3.51	0.00	4.74	-106.00	2215.32	17444.38	-4.54	0.20
	2	0.2560	3.51	0.00	6.15	-84.70	1836.50	17444.38	-4.71	1.44
	3	0.3437	3.51	0.00	8.26	-61.30	1291.35	17444.38	-4.58	3.68
	4	0.4860	3.51	0.00	11.67	-30.70	458.02	17444.38	-3.24	8.43
	5	0.5848	3.51	0.00	14.05	0.30	0.05	17444.38	0.04	14.09
	6	0.5309	3.51	0.00	12.75	31.30	520.08	17444.38	3.61	16.36
	7	0.4180	3.51	0.00	10.04	55.90	1306.04	17444.38	5.08	15.12
	8	0.2922	3.51	0.00	7.02	85.00	2111.43	17444.38	5.40	12.42
	9	0.2248	3.51	0.00	5.40	106.00	2525.70	17444.38	5.18	10.58
	10	0.1727	3.51	0.00	4.15	128.20	2838.10	17444.38	4.81	8.96
W4	Case 2 NS - .15By									
	$.75F_{NS}$ (kip)	84.225			e_{NS} (ft)	-27				
	$.75F_{EW}$ (kip)	0			e_{EW} (ft)	0				
	M_{NS} (k-ft)	-2274.075								
	M_{EW} (k-ft)	0								
	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)
	B	0.5502	0.00	4.23	0.00	32.50	581.15	17444.38	-2.33	-2.33
	C	1.2902	0.00	4.23	0.00	18.20	427.38	17444.38	-3.06	-3.06
	D	1.3881	0.00	4.23	0.00	-11.30	177.25	17444.38	2.04	2.04
	E	1.0000	0.00	4.23	0.00	-34.00	1156.00	17444.38	4.43	4.43
	1	0.1972	3.51	0.00	4.74	-106.00	2215.32	17444.38	2.72	7.46
	2	0.2560	3.51	0.00	6.15	-84.70	1836.50	17444.38	2.83	8.98
	3	0.3437	3.51	0.00	8.26	-61.30	1291.35	17444.38	2.75	11.00
	4	0.4860	3.51	0.00	11.67	-30.70	458.02	17444.38	1.94	13.62
	5	0.5848	3.51	0.00	14.05	0.30	0.05	17444.38	-0.02	14.03
	6	0.5309	3.51	0.00	12.75	31.30	520.08	17444.38	-2.17	10.59
	7	0.4180	3.51	0.00	10.04	55.90	1306.04	17444.38	-3.05	6.99
	8	0.2922	3.51	0.00	7.02	85.00	2111.43	17444.38	-3.24	3.78
	9	0.2248	3.51	0.00	5.40	106.00	2525.70	17444.38	-3.11	2.29
	10	0.1727	3.51	0.00	4.15	128.20	2838.10	17444.38	-2.89	1.26
W5	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)	ΣK_{NS} (k/in)	ΣK_{EW} (k/in)	Direct shear (kip)	d (ft)	Kd^2	ΣKd^2	Torsional Moment Shear (kip)	Total shear (kip)
B	0.5502	0.00	4.23	2.50	32.50	581.15	17444.38	-0.22	2.28
C	1.2902	0.00	4.23	5.86	18.20	427.38	17444.38	-0.28	5.57
D	1.3881	0.00	4.23	6.30	-11.30	177.25	17444.38	0.19	6.49
E	1.0000	0.00	4.23	4.54	-34.00	1156.00	17444.38	0.41	4.95
1	0.1972	3.51	0.00	0.00	-106.00	2215.32	17444.38	0.25	0.25
2	0.2560	3.51	0.00	0.00	-84.70	1836.50	17444.38	0.26	0.26
3	0.3437	3.51	0.00	0.00	-61.30	1291.35	17444.38	0.26	0.26
4	0.4860	3.51	0.00	0.00	-30.70	458.02	17444.38	0.18	0.18
5	0.5848	3.51	0.00	0.00	0.30	0.05	17444.38	0.00	0.00
6	0.5309	3.51	0.00	0.00	31.30	520.08	17444.38	-0.20	-0.20
7	0.4180	3.51	0.00	0.00	55.90	1306.04	17444.38	-0.28	-0.28
8	0.2922	3.51	0.00	0.00	85.00	2111.43	17444.38	-0.30	-0.30
9	0.2248	3.51	0.00	0.00	106.00	2525.70	17444.38	-0.29	-0.29
10	0.1727	3.51	0.00	0.00	128.20	2838.10	17444.38	-0.27	-0.27

W6	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)	ΣK_{NS} (k/in)	ΣK_{EW} (k/in)	Direct shear (kip)	d (ft)	Kd^2	ΣKd^2	Torsional Moment Shear (kip)	Total shear (kip)
B	0.5502	0.00	4.23	2.50	32.50	581.15	17444.38	0.30	2.79
C	1.2902	0.00	4.23	5.86	18.20	427.38	17444.38	0.39	6.25
D	1.3881	0.00	4.23	6.30	-11.30	177.25	17444.38	-0.26	6.04
E	1.0000	0.00	4.23	4.54	-34.00	1156.00	17444.38	-0.56	3.98
1	0.1972	3.51	0.00	0.00	-106.00	2215.32	17444.38	-0.35	-0.35
2	0.2560	3.51	0.00	0.00	-84.70	1836.50	17444.38	-0.36	-0.36
3	0.3437	3.51	0.00	0.00	-61.30	1291.35	17444.38	-0.35	-0.35
4	0.4860	3.51	0.00	0.00	-30.70	458.02	17444.38	-0.25	-0.25
5	0.5848	3.51	0.00	0.00	0.30	0.05	17444.38	0.00	0.00
6	0.5309	3.51	0.00	0.00	31.30	520.08	17444.38	0.27	0.27
7	0.4180	3.51	0.00	0.00	55.90	1306.04	17444.38	0.39	0.39
8	0.2922	3.51	0.00	0.00	85.00	2111.43	17444.38	0.41	0.41
9	0.2248	3.51	0.00	0.00	106.00	2525.70	17444.38	0.39	0.39
10	0.1727	3.51	0.00	0.00	128.20	2838.10	17444.38	0.37	0.37

W7	Case 3 NS & EW			
	$.75F_{NS}$ (kip)	84.225	e_{NS} (ft)	9
	$.75F_{EW}$ (kip)	19.215	e_{EW} (ft)	2
	M_{NS} (k-ft)	758.025		
	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	ΣKd^2	Torsional Moment Shear (kip)	Total shear (kip)
B	0.5502	0.00	4.23	2.50	32.50	581.15	17444.38	0.82	3.32
C	1.2902	0.00	4.23	5.86	18.20	427.38	17444.38	1.07	6.94
D	1.3881	0.00	4.23	6.31	-11.30	177.25	17444.38	-0.72	5.59
E	1.0000	0.00	4.23	4.54	-34.00	1156.00	17444.38	-1.55	2.99
1	0.1972	3.51	0.00	4.74	-106.00	2215.32	17444.38	-0.95	3.78
2	0.2560	3.51	0.00	6.15	-84.70	1836.50	17444.38	-0.99	5.16
3	0.3437	3.51	0.00	8.26	-61.30	1291.35	17444.38	-0.96	7.29
4	0.4860	3.51	0.00	11.67	-30.70	458.02	17444.38	-0.68	10.99
5	0.5848	3.51	0.00	14.05	0.30	0.05	17444.38	0.01	14.06
6	0.5309	3.51	0.00	12.75	31.30	520.08	17444.38	0.76	13.51
7	0.4180	3.51	0.00	10.04	55.90	1306.04	17444.38	1.07	11.11
8	0.2922	3.51	0.00	7.02	85.00	2111.43	17444.38	1.13	8.15
9	0.2248	3.51	0.00	5.40	106.00	2525.70	17444.38	1.09	6.49
10	0.1727	3.51	0.00	4.15	128.20	2838.10	17444.38	1.01	5.16

W8	Case 4 NS+.15By & EW+.15Bx									
	$.563F_{NS}$ (kip)	63.2249			e_{NS} (ft)	45				
	$.563F_{EW}$ (kip)	14.42406			e_{EW} (ft)	-11				
	M_{NS} (k-ft)	2845.1205								
	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)
	B	0.5502	0.00	4.23	1.88	32.50	581.15	17444.38	2.75	4.63
	C	1.2902	0.00	4.23	4.40	18.20	427.38	17444.38	3.62	8.02
	D	1.3881	0.00	4.23	4.73	-11.30	177.25	17444.38	-2.42	2.32
	E	1.0000	0.00	4.23	3.41	-34.00	1156.00	17444.38	-5.24	-1.82
	1	0.1972	3.51	0.00	3.56	-106.00	2215.32	17444.38	-3.22	0.34
	2	0.2560	3.51	0.00	4.62	-84.70	1836.50	17444.38	-3.34	1.28
	3	0.3437	3.51	0.00	6.20	-61.30	1291.35	17444.38	-3.24	2.95
	4	0.4860	3.51	0.00	8.76	-30.70	458.02	17444.38	-2.30	6.47
	5	0.5848	3.51	0.00	10.55	0.30	0.05	17444.38	0.03	10.57
	6	0.5309	3.51	0.00	9.57	31.30	520.08	17444.38	2.56	12.13
	7	0.4180	3.51	0.00	7.54	55.90	1306.04	17444.38	3.60	11.13
	8	0.2922	3.51	0.00	5.27	85.00	2111.43	17444.38	3.83	9.10
	9	0.2248	3.51	0.00	4.05	106.00	2525.70	17444.38	3.67	7.72
	10	0.1727	3.51	0.00	3.11	128.20	2838.10	17444.38	3.41	6.52

W9	Case 4 NS-.15By & EW-.15Bx									
	$.563F_{NS}$ (kip)	63.2249			e_{NS} (ft)	-27				
	$.563F_{EW}$ (kip)	14.42406			e_{EW} (ft)	15				
	M_{NS} (k-ft)	-1707.0723								
	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	∑Kd^2	Torsional Moment Shear (kip)	Total shear (kip)
	B	0.5502	0.00	4.23	1.88	32.50	581.15	17444.38	-1.53	0.35
	C	1.2902	0.00	4.23	4.40	18.20	427.38	17444.38	3.62	8.02
	D	1.3881	0.00	4.23	4.73	-11.30	177.25	17444.38	-2.42	2.32
	E	1.0000	0.00	4.23	3.41	-34.00	1156.00	17444.38	-5.24	-1.82
	1	0.1972	3.51	0.00	3.56	-106.00	2215.32	17444.38	-3.22	0.34
	2	0.2560	3.51	0.00	4.62	-84.70	1836.50	17444.38	-3.34	1.28
	3	0.3437	3.51	0.00	6.20	-61.30	1291.35	17444.38	-3.24	2.95
	4	0.4860	3.51	0.00	8.76	-30.70	458.02	17444.38	-2.30	6.47
	5	0.5848	3.51	0.00	10.55	0.30	0.05	17444.38	0.03	10.57
	6	0.5309	3.51	0.00	9.57	31.30	520.08	17444.38	2.56	12.13
	7	0.4180	3.51	0.00	7.54	55.90	1306.04	17444.38	3.60	11.13
	8	0.2922	3.51	0.00	5.27	85.00	2111.43	17444.38	3.83	9.10
	9	0.2248	3.51	0.00	4.05	106.00	2525.70	17444.38	3.67	7.72
	10	0.1727	3.51	0.00	3.11	128.20	2838.10	17444.38	3.41	6.52

W10	Case 4 NS+.15By & EW-.15Bx									
	.563F_{NS} (kip)	63.2249			e_{NS} (ft)	45				
	.563F_{EW} (kip)	14.42406			e_{EW} (ft)	15				
	M_{NS} (k-ft)	2845.1205								
	M_{EW} (k-ft)	216.3609								
	Frame	K (k/in)	∑K_{NS} (k/in)	∑K_{EW} (k/in)	Direct shear (kip)	d (ft)	Kd^2	∑Kd^2	Torsional Moment Shear (kip)	Total shear (kip)
	B	0.5502	0.00	4.23	1.88	32.50	581.15	17444.38	3.14	5.01
	C	1.2902	0.00	4.23	4.40	18.20	427.38	17444.38	4.12	8.52
	D	1.3881	0.00	4.23	4.73	-11.30	177.25	17444.38	-2.75	1.98
	E	1.0000	0.00	4.23	3.41	-34.00	1156.00	17444.38	-5.97	-2.56
	1	0.1972	3.51	0.00	3.56	-106.00	2215.32	17444.38	-3.67	-0.11
	2	0.2560	3.51	0.00	4.62	-84.70	1836.50	17444.38	-3.81	0.81
	3	0.3437	3.51	0.00	6.20	-61.30	1291.35	17444.38	-3.70	2.50
	4	0.4860	3.51	0.00	8.76	-30.70	458.02	17444.38	-2.62	6.14
	5	0.5848	3.51	0.00	10.55	0.30	0.05	17444.38	0.03	10.58
	6	0.5309	3.51	0.00	9.57	31.30	520.08	17444.38	2.92	12.49
	7	0.4180	3.51	0.00	7.54	55.90	1306.04	17444.38	4.10	11.64
	8	0.2922	3.51	0.00	5.27	85.00	2111.43	17444.38	4.36	9.63
	9	0.2248	3.51	0.00	4.05	106.00	2525.70	17444.38	4.18	8.24
	10	0.1727	3.51	0.00	3.11	128.20	2838.10	17444.38	3.89	7.00

W11	Case 4 NS-.15By & EW+.15Bx									
	.563F_{NS} (kip)	63.2249			e_{NS} (ft)	-27				
	.563F_{EW} (kip)	14.42406			e_{EW} (ft)	-11				
	M_{NS} (k-ft)	-1707.0723								
	M_{EW} (k-ft)	-158.66466								
	Frame	K (k/in)	∑K_{NS} (k/in)	∑K_{EW} (k/in)	Direct shear (kip)	d (ft)	Kd^2	∑Kd^2	Torsional Moment Shear (kip)	Total shear (kip)
	B	0.5502	0.00	4.23	1.88	32.50	581.15	17444.38	-1.91	-0.04
	C	1.2902	0.00	4.23	4.40	18.20	427.38	17444.38	-2.51	1.89
	D	1.3881	0.00	4.23	4.73	-11.30	177.25	17444.38	1.68	6.41
	E	1.0000	0.00	4.23	3.41	-34.00	1156.00	17444.38	3.64	7.05
	1	0.1972	3.51	0.00	3.56	-106.00	2215.32	17444.38	2.24	5.79
	2	0.2560	3.51	0.00	4.62	-84.70	1836.50	17444.38	2.32	6.94
	3	0.3437	3.51	0.00	6.20	-61.30	1291.35	17444.38	2.25	8.45
	4	0.4860	3.51	0.00	8.76	-30.70	458.02	17444.38	1.60	10.36
	5	0.5848	3.51	0.00	10.55	0.30	0.05	17444.38	-0.02	10.53
	6	0.5309	3.51	0.00	9.57	31.30	520.08	17444.38	-1.78	7.80
	7	0.4180	3.51	0.00	7.54	55.90	1306.04	17444.38	-2.50	5.04
	8	0.2922	3.51	0.00	5.27	85.00	2111.43	17444.38	-2.66	2.61
	9	0.2248	3.51	0.00	4.05	106.00	2525.70	17444.38	-2.55	1.51
	10	0.1727	3.51	0.00	3.11	128.20	2838.10	17444.38	-2.37	0.75

Appendix B: 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 $T < T_L$

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

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

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

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

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

$F_{Lvl4} = (0.32)(84) = 26.88$ K

$F_{Lvl3} = (0.21)(84) = 17.64$ K

$F_{Lvl2} = (0.10)(84) = 8.4$ K

Lvl 2	Area	DL	Weight
Slab	16600	63	1037500
superimposed	16600	5	83000
Steel	16600	0	0
Façade	8663	100	866250
Int Brick	2590	4.7	12086.667
Total	16600		2,032,837
Lvl 3	Area	DL	Weight
Slab	16600	63	1037500
superimposed	16600	5	83000
Steel	16600	0	0
Façade	8820	100	882000
CMU	8820	0	0
Int Brick	1400	4.7	6533.3333
Stone Floor	1700	20	34000
			2,043,033
Lvl 4	Area	DL	Weight
Slab	16600	63	1037500
superimposed	16600	5	83000
Steel	16600	0	0
Façade	9293	100	929250
CMU	9293	0	0
Int Brick	1500	4.7	7000
Stone Floor	1700	20	34000
			2,090,750
PH	Area	DL	Weight
Slab	6000	63	375000
Roof Deck	4700	63	293750
superimposed	10700	5	53500
Steel	10700	0	0
Façade	9000	100	900000
CMU	9000	0	0
Green Roof	4700	25	117500
			1,739,750
Roof	Area	DL	Weight
conc slab	7310	63	456875
Slate	7310	10	73100
steel	7310	0	0
superimposed	7310	5	36550
			566,525
Bld weight (lbs)			8,472,895

Appendix C: Gravity Calculations

A-B & D-E int beam | Typ

Min Slab Thickness

$h_{min} = l/28 = \frac{10.33 \times 12}{28} = 4.42$ Say 5"

Slab slf wt = $150 \times \frac{5}{12} = 62.5$ psf

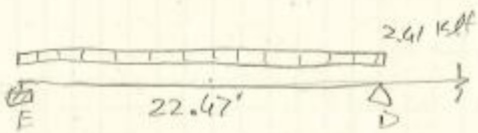
Beam Design

DL = 15 + 62.5 = 77.5

LL = 100 psf

wu = 1.2D + 1.6L = 253 psf × 10.33'/1000 = 2.61 klf

Span E-D



From staad

M_u = 44 k-ft ×1.1 for self wt
= 48.5 k-ft

Estimate size

$bd^2 = 20 M_u$ Say $b = \frac{4}{5}d$

$d^3 = (20)(48.5)\frac{5}{4} = 10.64$

$h = d + 2.5 = 13.14$" use h = 14" & b = 10"

Self wt effects:

$w_{sw} = \frac{14 \times 10}{144} \times 150 = 146$ plf

wu = 2.61 + 1.2 × .146 = 2.79 klf

M_u = 47.5 k-ft

A-B & D-E int beam

Required Pos Steel:

$As\ req = \frac{Mu}{4d} = \frac{47.5}{4(11.5)} = 1.03\ in^2$

try (4) #5 $\quad 4 \times 0.31 = 1.24\ in^2 = As$

New $d = 12''$

Nominal Moment:

$a = \frac{As\ fy}{0.85 f'c\ b} = \frac{(1.24)(60)}{0.85(4)(10)} = 2.19''$

$c = \frac{a}{\beta_1} = \frac{2.19}{0.85} = 2.57$

$\varepsilon_s = \varepsilon_u \left(\frac{d-c}{c}\right) = 0.003\left(\frac{12-2.57}{2.57}\right) = .011 > \varepsilon_y \therefore \phi = 0.9$

$\phi M_n = \phi As fy \left(d - \frac{a}{2}\right) = 0.9(1.24)(60)\left(12 - \frac{2.19}{2}\right)\frac{1}{12} = 60.8\ k\text{-}ft$

$\phi M_n > M_u$ ✓

check As_{min} & As_{max}:

$As_{min} > \begin{cases} \frac{3\sqrt{f'c}\ bd}{fy} = \frac{3\sqrt{4000}(10)(12)}{60000} = .38 \\ \frac{200\ bd}{fy} = \frac{(200)(10)(12)}{60{,}000} = 0.4 \end{cases}$ $\quad As > As_{min}$ ✓

$\rho_{max} = 0.85\beta_1 \frac{f'c}{fy} \cdot \frac{\varepsilon u}{\varepsilon u + 0.004} = 0.85^2 \frac{4}{60} \frac{0.003}{0.003 + .004}$

$= 0.21$

$As_{max} = (0.21)(10)(12) = 2.48\ in^2 > As$ ✓

Required Neg Steel @ E & A

$M_u = 90.21\ k\text{-}ft$

$As_{req} = \frac{90.21}{(4)(11.5)} = 1.96\ in^2$

use (2) #9 $\quad As = 2.0\ in^2$

$M_n = 92.12 > M_u$ ✓

As min & max check out

A-B & D-E int beam

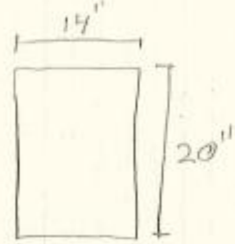
Need bigger beam b/c Neg mom @ D & A is large

$M_u = 175$ k-ft $\times 1.1 = 193$ k-ft

Estimate size

$d^2 = 20 \times 193 \times 3/4$

$d = 16.9"$



$h = 20"$ & $b = 14"$

Self wt

$w_{sw} = \frac{20 \times 14}{144} \times 150 = .292$ klf

New $M_u = 196$ k-ft

Required neg steel @ col line D & A

$A_{s\,req} = \frac{M_u}{4d} = \frac{196}{4(17.5)} = 2.8\text{ in}^2$

use $\boxed{(2)\#5 \ \& \ (2)\#10}$ $A_s = 3.14\text{ in}^2$

$\phi M_n = 220.5$ k-ft $> M_u$ ✓

meets A_s max & min req.

Recheck pos mom reinf. for beam D-E

From STAAD w/ ext pinned connections

$M_u = 93$ k-ft $< M_n = 96$ k-ft (from spreadsheet)

$\therefore$ $\boxed{(2)\#5 \ \& \ (1)\#9}$ is ok

For pos mom reinf beam A-B

From STAAD w/ ext pinned connections

$M_u = 133$ k-ft

Req. reinf

$A_{s\,req} = \frac{133}{4(17.5)} = 1.9$

use (2)#9 & (1)#11 $A_s = 2.18\text{ in}^2$ $d = 17.5$

$M_n = 150$ k-ft > 133 $\therefore$ ✓

A-B & D-E int beam

Required neg steel @ col B & E

$A_{s\,req} = \frac{89}{4(17.5)} = 1.27 \text{ in}^2$

$\boxed{\text{use (2) \#5 \& (1) \#9}}$ $A_s = 1.42$ $d = 18''$

$\phi M_n = 95.4$ k-ft

C-D int beam Typ

Design Beam

Use same size beam as beam E-D & B-A

Design Positive mom reinf.

$M_u = 134$ k-ft

$A_{s\,req} = \frac{134}{4(17.5)} = 1.91 \text{ in}^2$

use (2) #5 & (1) #11 $A_s = 2.18$ $d = 17.5$

$\phi M_n = 158$ k-ft

Beam Shear

Design Shear For Bm C-D. Use Design For The Rest

$V_c = 2\sqrt{f'_c}\, b_w d = 2\sqrt{4000}(14)(11.5) = 20{,}365\ lb = 20.4K$

$\phi V_n = 0.5\phi V_c = 0.5(0.75)(20.4) = 7.65K$

$V_s = \frac{V_u}{\phi} - V_c = \frac{29.5}{0.75} - 20.4 = 18.9K$

$V_s \le 8\sqrt{4000}(14)(11.5) = 81.5K$ ✓

Stop shear reinf. @ 8' away from supp

Max Spacing

$4\sqrt{4000}(14)(11.5) \times \frac{1}{1000} = 40.7K > 18.9K$

$S_{max} = \min \begin{cases} d/2 = 11.5/2 = 5.75" & \text{use } 5" \\ 24" \end{cases}$

Min reinf

$A_{v\,min} = \max \begin{cases} 0.75\sqrt{f'_c}\, b_w \frac{s}{f_{yt}} = (0.75)(\sqrt{4000})(14)\left(\frac{5}{60000}\right) = 0.055\ in \\ 50 b_w \frac{s}{f_{yt}} = 50(14)\left(\frac{5}{60000}\right) = \underline{0.058\ in} \end{cases}$

use #3 @ 5" as min reinf

$A_v = 2\text{legs} \times 0.11\,in^2/\text{leg} = 0.22\,in^2 > 0.06\,in^2$

Design Reinf

$s = \frac{(0.22)(60)(11.5)}{18.9} = 8.0" > 5"$ use 5" spacing

Beam Shear

Beams AB & DE

Use same reinf & spacing

$V_u = 24.5\ K$

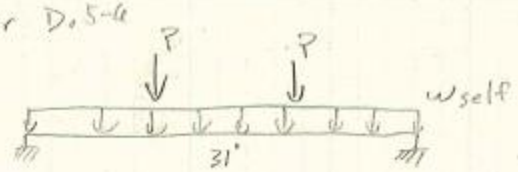
Discontinue reinf 5' away from support

Beam BC

No shear reinf required

E-W Girders ℄ D

Girder D, 5-6



$P = V_{BED} + V_{BDC} = 42.5K + 45.3K = 87.8K$

Apply P load in STAAD

Pos mom @ mid span

$M_u = 302$

Estimate size

$bd^2 = 20\,M_u$ say $b = \frac{4}{5}d$

$d = 19.6''$

$h = 19.6 + 2.5'' = 22.1''$ — use 28''

new $d = 26 - 2.5 = 25.5''$

$b = 20''$

Self weight effects

$w = 160 \times \frac{20}{12} \times \frac{28}{12} \times \frac{1}{1000} = .58$ Klf

Req pos reinf

$A_s = \frac{M_u}{4d} = \frac{324}{4(23.5)} = 3.45\ in^2$

use (4) #9

From spread sheet

$\phi M_n = 388$ K-ft $> M_u$ OK

Satifys all ductility and As requirements

Req. Neg. Reinf

$A_s = \frac{648}{4(23.5)} = 6.89 \text{ in}^2$

(2) #5 & (5) #10 $\quad A_s = 6.97 \quad$ or (7) #9

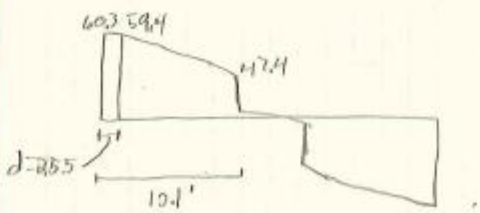
$20 - 2(1.5) - 2(.5) = 16''$

$(2 \times .625) + (5 \times \quad) + (7-1) = 13.6'' \quad \therefore$ reinf fits

$\phi M_n = 703 \text{ k-ft} \geq M_u = 648 \text{ k-ft}$

Girder Shear

For ALL Girders



$V_c = 2\sqrt{f'_c}\, b_w d = 2\sqrt{4000}\,(20)(25.5)\,\frac{1}{1000} = 64.5K$

$\phi V_n = 0.5\phi V_c = (0.5)(0.75)(64.5) = 24.2K$

$V_s = \frac{59.4}{0.75} - 64.5 = 14.7K$

$V_s < 8\sqrt{4000}\,(20)(25.5) = 258K$ ✓

Stop reinf @ 11' away from supp

Max Spacing

$4\sqrt{4000}\,(20)(25.5) \times \frac{1}{1000} = 129 > 14.7$

$S_{max} = \min \begin{cases} d/2 = \frac{25.5}{2} = 12.75 & \text{use } 12'' \\ 24'' \end{cases}$

Min. reinf

$A_{vmin} = \max \begin{cases} (0.75)(\sqrt{4000})(20)\left(\frac{12}{60000}\right) = 0.19\,in^2 \\ 50(20)\left(\frac{12}{60{,}000}\right) = \underline{0.2\,in^2} \end{cases}$

use #3 @ 12" as min

$A_v = 2$ legs $\times\ 0.11\,in^2/\text{leg} = 0.22\,in^2$

Design Reinf

$S = \frac{(0.22)(60)(25.5)}{14.7} = 22'' > 12$ use 12" spacing

Daniel Bodde | | Edge Bm Design

Edge beam design for gravity loads only.

Loads acting on beam

DL:

Slab: Estimate thickness using table 9.5c from ACI 318-11

assuming 24"x24" column

$t = \ell_n/33 = \frac{31'-2'}{33} \times 12''/ft = 10.55$ use 11"

weight $= 11.33' \times \frac{11''}{12''/ft} \times 150\,pcf = 1558$ plf

Precast Panel:

Size = 31' x 14'

thickness ≈ 10"

weight $= 14' \times \frac{10''}{12''/ft} \times 150\,pcf = 1750$ plf

Superimposed load = 15 psf x 11.33' = 170 plf

LL:

office LL = 80 psf x 11.33' = 293 plf

Beam Design

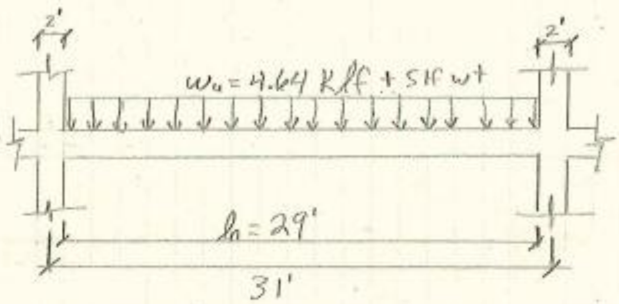
DL = 1558 + 1750 + 170 = 3478 plf

LL = 293 plf

$w_u = 1.2D + 1.6L$

$= [1.2(3478) + 1.6(293)] \times \frac{1}{1000} = 4.64$ Klf

Edge Bm Design 2



Use Moment Coeff. Method to determine max Pos moment.

$M_u^+ = \frac{w_u l_n^2}{16} = \frac{(4.64)(29)^2}{14} = 244 \times 1.1 = 268 \text{ K-ft}$

est eff of slf wt

Estimate Beam Size

$bd^2 = 20 M_u$ try $b = \frac{4}{5} d$

$\left(\frac{4}{5} d\right) d^2 = 20 M_u$

$d^3 = 25 M_u = 25 \times 268 = 6700$

$d = 18.85''$

$h = d + 2.5 = 21.35$ use $h = 22''$ $\therefore d = 19.5''$ & $b = 16''$

$bd^2 = 16 \times 19.5^2 = 6084 \text{ in}^3$

Compute Self Wt. Effects

$w_{sw} = \frac{16 \times 22}{144} \times 150 = 367 \text{ plf}$

$w_u = 4.44 \text{ Klf} + [1.2(367)] \times \frac{1}{1000} = 5.08 \text{ Klf}$

$M_u = \frac{(5.08)(29)^2}{16} = 267 \text{ K-ft}$ $20 \times 267 = 5340 \text{ in}^3 < 6084$ ✓

Edge Beam Design

Required Steel

$A_s = \frac{M_u}{4d} = \frac{247}{4(17.5)} = 3.42 \text{ in}^2$ use (4)#9 $= 4(1 \text{ in}^2) = 4 \text{ in}^2$

$d = 22 - 1.5 - \frac{1.128}{2} = 19.9''$

Nominal Moment

$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{(4)(60)}{0.85(4)(16)} = 4.41 \text{ in}$

$c = \frac{a}{\beta_1} = \frac{4.41}{0.85} = 5.19 \text{ in}$

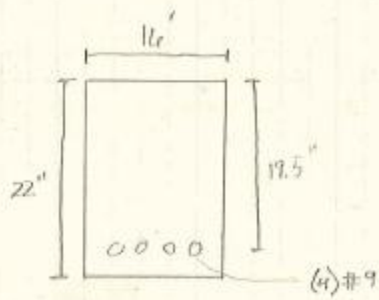
check $\varepsilon_s > \varepsilon_y$

$\varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{5.19}(19.9 - 5.19) = 0.0085 > \varepsilon_y$ ✓

$\therefore \phi = 0.9$

$M_n = \phi A_s f_y \left(d - \frac{a}{2}\right) = (0.9)(4)(60)\left(19.9 - \frac{4.41}{2}\right) \times \frac{1}{12 \text{ in/ft}}$

$= 318.5 \text{ k-ft} > 247 \text{ k-ft}$ ✓



New Edge Beam Design

$M^- = 340$ k-ft controls

$M^+ = 170$ k-ft

$d^3 = 25 M_u$ $b = \frac{4}{5} d$

$d = 20.4$

$h = 20.4 + 2.5 = 22.9$ use $h = 24$

$d = 21.5$

$b = 18''$

required neg reinf

$A_s = \frac{M_u}{4d} = \frac{340}{4(21.5)} = 3.95 \text{ in}^2$

use 2#5 & 4#9 $A_s = 4.62 \text{ in}^2$

$\phi M_n = 400$ k-ft $\geq M_u$

required pos reinf

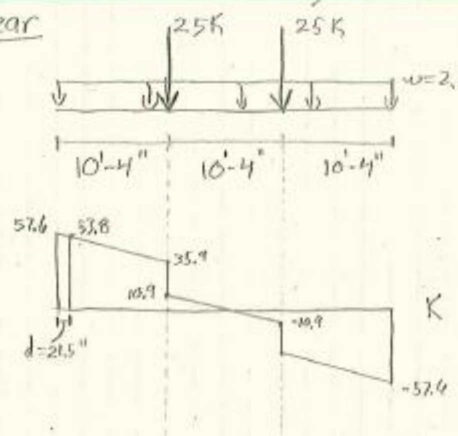
$A_s = \frac{M_u}{4d} = \frac{170}{4(21.5)} = 1.98 \text{ in}^2$

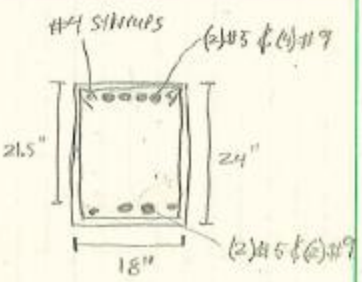
use 2#5 & 2#9 $A_s = 2.62 \text{ in}^2$

$\phi M_n = 238$ k-ft $\geq M_u$

Edge Beam Shear & Torsion

Shear





Concrete Shear strength (ACI 318-11 3.3.1)

$V_c = 2\sqrt{f'c}\, b_w d = 2\sqrt{4000}(18)(21.5) = 48{,}952 \text{ lb} = 49\text{K}$

$\phi V_n = 0.5\phi V_c = (0.5)(0.75)(49) = 18.4\text{ K}$

$V_s = \frac{V_u}{\phi} - V_c = \frac{53.8}{0.75} - 49 = 22.7\text{ K}$

$V_s \le 8\sqrt{4000}(18)(21.5) = 196\text{ K}$ ✓

No shear reinf. req. @ 10'-4" away from support

Max Spacing

$4\sqrt{4000}(18)(21.5)\times\frac{1}{1000} = 98\text{K} \ge 22.7\text{ K}$

$S_{max} = \min\begin{cases} d/2 = 21.5/2 = \underline{10"} \\ 24" \end{cases}$

Min Shear reinf

$A_{vmin} = \max\begin{cases} 0.75\sqrt{f'c}\, b_w \frac{s}{f_{yt}} = (0.75)(\sqrt{4000})(18)\left(\frac{10}{60000}\right) = 0.14\text{ in}^2 \\ 50 b_w \frac{s}{f_{yt}} = (50)(18)\left(\frac{10}{60000}\right) = 0.15\text{ in}^2 \end{cases}$

use #4 @ 10" as min reinf

$A_v = 2\text{ legs} \times 0.2\text{ in}^2/\text{leg} = 0.4\text{ in}^2 > 0.15\text{ in}^2$

Shear 2

Design Shear Reinf

$V_s = A_v f_{yt} \frac{d}{s}$

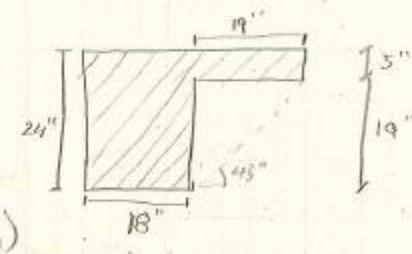
$s = \frac{A_v f_{yt} d}{V_s} = \frac{(0.44)(60)(21.5)}{22.6} = 22.8'' > 10''$ use 10'' spacing

Use (2) #3 @ 10'' starting 2'' from face of support

Check Torsion

$T_u = 154$ K-ft from STAAD

$\phi T_n = \phi 4 \sqrt{f'_c} \left(\frac{A_{cp}^2}{P_{cp}}\right)$



$A_{cp} = (18)(24) = 432$

$P_{cp} = 2(18+24) = 84$

$\phi T_n = 35$ K-ft $= T_u$ (compatibility torsion)

$x_1 = 18 - 2(1.5+.25) = 14.5$
$y_1 = 24 - 2(1.5+.25) = 20.5$

$A_{oh} = 14.5 \times 20.5 = 297.25 \text{ in}^2$
$A_o = .85 \times 297.5 = 252.7 \text{ in}^2$

$P_h = 2(14.5 + 20.5) = 70$ in

check section size

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u P_h}{1.7 A_{oh}^2}\right)^2} < \phi\left(\frac{V_c}{b_w d} + 8\sqrt{f'_c}\right)$$

$$\sqrt{\left(\frac{57,600}{(18)(21.5)}\right)^2 + \left(\frac{(420000)(70)}{1.7(297.5)^2}\right)^2} < 0.75\left(\frac{49,000}{(18)(21.5)} + 8\sqrt{4000}\right)$$

$246 < 474$ ∴ OK

$T_n = \frac{T_u}{\phi} = \frac{35}{0.75} = 46.7 = 560,000$ lb-in

$\frac{A_t}{s} = \frac{T_n}{2 A_o f_{yt} \cot\theta} = \frac{560,000}{(2)(252.7)(60000)(\cot(45))} = 0.018$ for 1 leg

Shear

$V_u = 33{,}800 \geq \frac{1}{2}\phi V_c = \frac{1}{2}(0.75)(49{,}000) = 18375$

$\therefore$ reinf req

$V_s = \dfrac{53{,}800 - (0.75 \times 49{,}000)}{0.75} = 22{,}733$ lb

$\dfrac{A_v}{s} = \dfrac{V_s}{f_{yt} d} = \dfrac{22733}{(60{,}000)(21.5)} = 0.0176 \text{ in}^2/\text{in}$ for 2 legs

total web reinf required for 2 legs

$\dfrac{A_{v+t}}{s} = \dfrac{2A_t}{s} + \dfrac{A_v}{s} = 2(0.018) + 0.0176 = 0.0536 \text{ in}^2/\text{in}$ for 2 legs

using #4 stirrups

$s = \dfrac{(2)(0.20)}{0.0536} = 7.46''$ use 7 in

max Spacing $\dfrac{p_h}{8} = \dfrac{70}{8} = 8.75$ in

Min area of Stirrups A_v

$A_v + 2A_t = 0.75\sqrt{f'c}\,\dfrac{b_w s}{f_{yt}} = 0.75\sqrt{4000}\,\dfrac{(18)(7)}{60000} = 0.1$ in

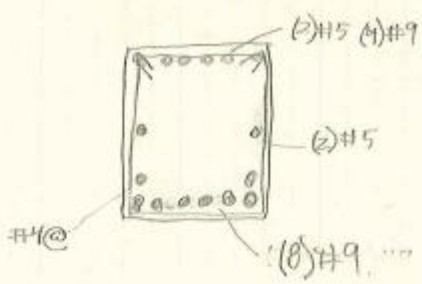
$\dfrac{50(18)(7)}{60000} = 0.105 > 0.1$ ✓

$0.1 \text{ in}^2 < (2)(0.2) = 0.4 \text{ in}^2$

use #4 @ 7 in

Additional longitudinal reinf

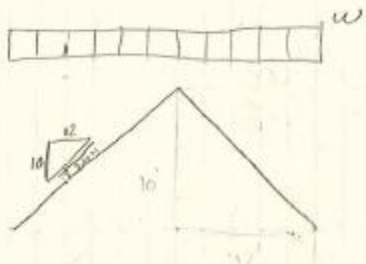
$A_\ell = \dfrac{A_t}{s} p_h \left(\dfrac{f_{yt}}{f_y}\right)\cot^2\theta = (0.018)(70)\left(\dfrac{60}{60}\right)(1.00)^2 = 1.24 \text{ in}^2$

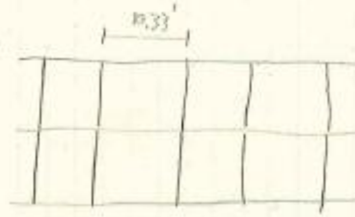


add (2)#5 at mid
& (1) #9 at bottom

can be terminated
14' away from supports

Roof Design





Slab thickness:

$$\ell/28 = \frac{10.33 \times 12}{28} = 4.4" \quad \text{use } 5"$$

DL

$$\text{slab} = \frac{5}{12} \times 150 = 62.5 \text{ psf}$$

$$\text{slate} = 10.0 \text{ psf}$$

$$72.5 \text{ psf}$$

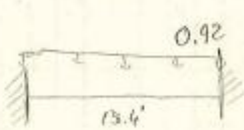
LL

Roof LL = 20 psf

$$w = [1.2(72.5) + 1.6(20)] \times 10.33' \times \frac{1}{1000} = 1.2 \text{ klf}$$

$$\frac{1.2}{15.6} = \frac{x}{12} \qquad x = 0.92 \text{ klf}$$

$$\frac{1.2}{15.6} = \frac{y}{10} \qquad y = 0.77 \text{ klf}$$



End moment = 18.7 k-ft

mid moment = 28.0 k-ft

Roof Design

Estimate Size

$d^3 = 20 \times 29.5 \times \frac{3}{4}$

$d = 9.03$

$h = 9.0 + 2.5 = 11.5"$ use $h = 12"$

$d = 10"$

$b = 8"$

Self weight

$\frac{12 \times 8}{144} \times 150 = 100$ plf

$\frac{100}{15.6} = \frac{x}{12} = 76.9$ plf $= 0.08$ klf

New moments

End = 18.9 k-ft

Mid = 28.3 k-ft

use same cross section

Required Pos Steel

A_s req $= \frac{M_u}{4d} = \frac{28.3}{4(10)}$ 0.71 in^2

use 2 #6 $A_s = 0.88$

$\phi M_n = 35.8$ k-ft $> M_u^+$ ✓

Required neg steel

A_s req $= \frac{18.9}{4(10)} = 0.47 \text{ in}^2$

use 2#5 $A_s = .62 \text{ in}^2$

$\phi M_n = 24$ k-ft $> M_u^-$ ✓

Appendix D: Deflections

Edge Beam

b	18	in						
h	24	in						
L	31	ft		Md	122.00	k-ft	Md+l	149.50
steel	2#5 & 1#9			Ml	27.5	k-ft		
As	8	in^2		Msus	135.75	k-ft		
d	21.5	in		Mo dead	367	k-ft		
f'c	4000	psi		Mo sus	408.5	k-ft		
Es	29000	ksi		Mo live	83	k-ft		
Ec	3605	ksi		Mo d+l	450	k-ft		
fr	474.3416	psi						
output								
Ig	20736							
Ie (dead)	15104							
Ie (sus)	14912							
Ie (d+l)	14785							
Mcr	58.54731							
n	8.044391							
B	0.279698							
kd	9.328974							
Icr	14404.56							
K dead	0.60							
K sus	0.60							
Δi(dead)	0.23	in						
Δi(live)	0.06	in						
Δi(sus)	0.26	in						
Δi(d+l)	0.29							
Δ long	0.58	in						
Allowable for live								
l/360	1.033333	in						
Allowable for D+L								
l/600	0.62	in						

Girder

b	20	in						
h	28	in						
L	31	ft		Md	59.00	k-ft	114.00	
steel	4#9			Ml	55	k-ft		
As	4	in^2		Msus	86.5	k-ft		
d	26.5	in		Mo dead	176	k-ft		
f'c	4000	psi		Mo sus	258	k-ft		
Es	29000	ksi		Mo live	165	k-ft		
Ec	3605	ksi			341			
fr	474.3416	psi						
output								
Ig	36587							
Ie (dead)	133415	>Ig	use Ig					
Ie (sus)	52177	>Ig	use Ig					
Ie (d+l)	30912							
Mcr	103.3011							
n	8.044391							
B	0.621551							
kd	7.764437							
Icr	14415.61							
K dead	0.60							
K sus	0.60							
Δi(dead)	0.05	in						
Δi(live)	0.06	in						
Δi(sus)	0.07	in						
Δi(d+l)	0.11							
Δ long	0.20	in						
Allowable for live								
l/360	1.033333	in						
Allowable for D+L								
l/240	1.55	in						

Appendix E: Lateral Calculations

Column Design

Check for LL Reduction at 1 ext. & 1 int. col.

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right)$$

Ext Col E.5

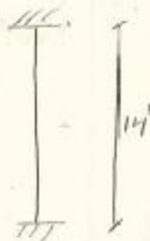
$$LL = 100\left(0.25 + \frac{15}{\sqrt{4\left(31 \times \frac{22.62}{2}\right)}}\right) = 65 \text{ psf}$$

Int Col D.5

$$LL = 100\left(0.25 + \frac{15}{\sqrt{4(31 \times 26)}}\right) = 52 \text{ psf}$$

Slab DL

$$DL_{slab} = 150 \text{ lb/ft}^3 \times \frac{5''}{12''/ft} = 62.5 \text{ psf}$$



Design Combo:

$1.2D + 1.0L + 1.6W$

$\Sigma P_u = 14528$

$V_{us} = 322$ K

$\Delta_o = 0.002$ in

$$Q = \frac{\Sigma P_u \Delta_o}{V_{us} l_c}$$

$$= \frac{(14528)(0.002)}{(322)(168)} = 5.37 \times 10^{-4} < 0.05 \therefore \text{Non Sway Frame}$$

Ext Col C-9 L1

Etabs Info:

- Design Combo 7: 1.2D + 1.0L + 1.6W-5

$P_u = 530K$

$M_u = 280 \text{ k-ft}$

Use Appendix A From W&M 5th ed.

$h = 20''$

$\gamma = 0.75$

$e = \frac{280(12)}{530} = 6.3$

$\frac{e}{h} = \frac{6.3}{20} = 0.32$

$\frac{\phi P_n}{bh} = \frac{530}{(18)(20)} = 1.47$

$\frac{\phi M_n}{bh^2} = \frac{(280)(12)}{18(20)^2} = 0.47$

$f'_c = 4000, f_y = 60$

$\gamma = 0.75$

$\rho = 0.023$

$\rho = 0.023 = 2.3\% < 8\%$ ✓

$A_{s\,req} = (0.023)(18)(20) = 8.3 \text{ in}^2$ use $\boxed{12\#9 \quad A_s = 12.00 \text{in}^2}$

Corner	Col C-16	L2

ETABS Info:

-Design Combo 7 : $1.2D + 1.0L + 1.6W\text{-}5$

ASCE 7-05 fig 6-9 case 2

$P_u = 439$ K

$M_u = 270$ k-ft

Use Appendix A From Wight & MacGregor 5th ed.

$h = 18''$

$\gamma = \frac{18-(2)(2.5)}{18} = 0.72$

$e = \frac{M_u}{P_u} = \frac{(270)(12)}{439} = 7.4$

$\frac{e}{h} = \frac{7.4}{18} = 0.41$

$\frac{\phi P_n}{bh} = \frac{439}{18^2} = 1.35$

$\frac{\phi M_n}{bh^2} = \frac{(270)(12)}{18^3} = 0.55$

$f'_c = 4000,\ f_y = 60$ $\quad \gamma = 0.60 \quad \gamma = 0.75$

$\rho = 0.049 \quad \rho = 0.035$

From Interpolation

$\rho = 0.038 = 3.8\%$ compared to 3.7% from spColumn & ETABS

$< 8\%$ OK ✓

$A_{s\,req} = \rho bh = (0.038)(18)^2 = 12.3\ in^2$ use 16 #9 As=16.0in^2

Tie Spacing

$S \leq \begin{cases} (16)(1.128) = 18.05 \\ (48)(0.375) = 18'' \\ 18'' \end{cases}$ use #3 ties @ 12" oc

Int Col D-5 L1

ETABS Info

Design Combo 4 : 1.2D +1.0L −1.6W-2

$P_u = 781$ K

$M_u = 248$ K-ft

$h = 20 \quad b = 18$

$\gamma = \dfrac{20-2(2.5)}{20} = 0.75$

$e = \dfrac{(248)(12)}{781} = 3.8$

$\dfrac{e}{h} = \dfrac{3.8}{20} = 0.19$

$\dfrac{\phi P_n}{bh} = \dfrac{781}{(18)(20)} = 2.17$

$\dfrac{\phi M_n}{bh^2} = \dfrac{248(12)}{(18)(20)^2} = 0.41$

$f'_c = 4000, \; f_y = 60$

$\gamma = 0.75$

$\rho = 0.037$

$\rho = 0.037 = 3.7\% < 8\%$ ✓

$A_{s,req} = (0.037)(18)(20) = 13.32$ use 16 #9 As=16.0in^2

Appendix F: Cost Impact & Durations

Depth:

CIP STRUCTURE							
			DECK THICKNESS	**0.41666667**			
CONCRETE DECKS - NAME				**11,000**	**SF**		
CONCRETE - BUY				178	CY	**103.00**	$18,359
CONCRETE - PLACE				178	CY	**40.00**	$7,130
MESH				11,000	SF	**0.00**	$0
REBAR		**1.5**	#/SF	16,500	LBS	**2.00**	$33,000
FINISH TOP				11,000	SF	**0.55**	$6,050
RUB BOTTOM				11,000	SF	**0.00**	$0
FORMING				10,000	SF	**7.00**	$70,000
CONCRETE INDIRECTS				64,538	%	**0.10**	$6,454
DECK	- COST PER CUBIC YARD				$791	/CY	
DECK	- COST PER SQUARE FOOT				$12.82	/SF	
CONCRETE DECKS - NAME					**-SUBTOTAL**	**$140,992**	

			W'	**D'**				
2028	**BEAM "A"**		**1.67**	**2.33**	**400**	**LF**		
	CONCRETE - BUY				61	CY	**103.00**	$6,234
	CONCRETE - PLACE				61	CY	**40.00**	$2,421
	REBAR		**242**	#/CY	14,648	LBS	**2.00**	$29,296
	FORMING - 3 SIDES				2,532	SF	**7.00**	$17,724
	FINISH 3 SIDES				2,532	SF	**0.00**	$0
	CONCRETE INDIRECTS				55,675	%	**0.10**	$5,568
	BEAM	-COST PER CUBIC YARD				$1,012	/CY	
	BEAM	-COST PER LINEAR FOOT				$153	/LF	
	BEAM "A"				**-SUBTOTAL**		**$61,243**	
			W'	**D'**				
1824	**BEAM "B"**		**1.50**	**2.00**	**650**	**LF**		
	CONCRETE - BUY				76	CY	**103.00**	$7,811
	CONCRETE - PLACE				76	CY	**40.00**	$3,033
	REBAR		**122**	#/CY	9,252	LBS	**2.00**	$18,503
	FORMING - 3 SIDES				3,575	SF	**7.00**	$25,025
	FINISH 3 SIDES				3,575	SF	**0.00**	$0
	CONCRETE INDIRECTS				54,373	%	**0.10**	$5,437
	BEAM	-COST PER CUBIC YARD				$789	/CY	
	BEAM	-COST PER LINEAR FOOT				$92	/LF	
	BEAM "B"				**-SUBTOTAL**		**$59,810**	

			W'	D'				
1420	**BEAM "C"**		1.17	1.67	**1400**	LF		
	CONCRETE - BUY				**106.379**	CY	**$103**	$10,957
	CONCRETE - PLACE				**106.379**	CY	**$40**	$4,255
	REBAR		156	#/CY	**16595.12**	LBS	**$2**	$33,190
	FORMING - 3 SIDES				**6314**	SF	**$7**	$44,198
	FINISH 3 SIDES				**6314**	SF	**$0**	$0
	CONCRETE INDIRECTS				**92600.45**	%	**$0**	$9,260
	BEAM	- COST PER CUBI C YARD				957.524413	**/CY**	
	BEAM	- COST PER LI NEAR FOOT				72.7574925	**/LF**	
	BEAM "C"				**-SUBTOTAL**		**$101,860**	
		HT	**W'**	**D'**				
2018	**COLUMNS - A1**	**14.00**	**1.50**	**1.67**	**23**	**EA**		
	CONCRETE - BUY				31	CY	**125.00**	$3,921
	CONCRETE - PLACE				31	CY	**45.00**	$1,412
	REBAR		**342**	#/CY	10,728	LBS	**2.00**	$21,456
	FORMING				1,191	SF	**7.50**	$8,930
	RUBBING				483	SF	**0.00**	$0
	CONCRETE INDIRECTS				35,719	%	**0.10**	$3,572
	COLUMN	- COST PER CUBIC YARD				$1,253	/CY	
	COLUMN	- COST PER EACH				$1,708	/EA	
	COLUMNS - A1		**-SUBTOTAL**		**$39,291**			
		HT	**W'**	**D'**				
	COLUMNS - A2	**14.00**	**2.00**	**2.00**	**20**	**EA**		
	CONCRETE - BUY				44	CY	**125.00**	$5,444
	CONCRETE - PLACE				44	CY	**45.00**	$1,960
	REBAR		**381**	#/CY	16,595	LBS	**2.00**	$33,189
	FORMING				1,280	SF	**7.50**	$9,600
	FINISHING				560	SF	**0.00**	$0
	CONCRETE INDIRECTS				50,194	%	**0.10**	$5,019
	COLUMN	- COST PER CUBIC YARD				$1,268	/CY	
	COLUMN	- COST PER EACH				$2,761	/EA	
	COLUMNS - A2		**-SUBTOTAL**		**$55,213**			

STEEL FRAME STRUCTURE					
STRUCTURAL STEEL					
COLUMNS		30	TONS	2,500.00	$75,425
BEAMS		60	TONS	2,500.00	$151,100
TUBE STEEL - HSS		5	TONS	3,000.00	$15,000
CONNECTIONS (10 - 15% OF STRUCT. ST. TOTAL)		14	TONS	2,500.00	$33,750
MOMENT CONNECTIONS		50	EA	350.00	$17,500
SHEAR STUDS		1,700	EA	2.50	$4,250
STRUCTURAL STEEL - LBS/SF OF BUILDING			XXXXX	LBS/SF	
STRUCTURAL STEEL	**-SUBTOTAL**	**$297,025**			
METAL DECK					
COMPOSITE FLOOR DECK		15,183	SF	3.25	$49,345
METAL DECK	**-SUBTOTAL**	**$49,345**			
SPRAY-ON-FIREPROOFING - 1" THICK					
BEAMS & COLUMNS		7,410	SF	1.00	$7,410
SPRAY-ON-FIREPROOFING - 1" THIC	**-SUBTOTAL**	**$7,410**			
	FACTOR				
CONCRETE FILL ON METAL DECK - 4"	**0.27**	**15,183**	**SF**		
CONCRETE - BUY		159	CY	115.00	$18,333
CONCRETE - PLACE		159	CY	55.00	$8,768
MESH		15,183	SF	0.55	$8,351
FINISH		15,183	SF	0.55	$8,351
CONCRETE INDIRECTS		43,803	%	0.10	$4,380
DECK FILL - COST PER CUBIC YARD			$302	/CY	
DECK FILL - COST PER SQUARE FOOT			$3.17	/SF	
CONCRETE FILL ON METAL DECK - 4	**-SUBTOTAL**	**$48,183**			

Precast Breadth:

PRECAST CONCRETE				
THIN BRICK PRECAST	8,218	SF	41.00	$336,938

EXTERIOR WALL					
MASONRY & STONE		**38,739**	**SF**		
FACE BRICK VENEER					
RUNNING BOND		0	SF	15.00	$0
FLEMISH BOND		8,218	SF	19.00	$156,142
CMU - 10" - GROUTED & REINFORCED		8,218	SF	10.50	$86,289
GROUTING		1,217	CY	125.00	$152,185
REINFORCING - REBAR		4,000	LBS	2.00	$8,000
CLEANING BRICK & REPOINTING		8,218	SF	7.00	$57,526
SCAFFOLDING		8,218	SF	1.50	$12,327
MASONRY & STONE	**-SUBTOTAL**	$553,719			

Durations:

Per floor

Structure	1 to 5	5 to 10	Unit	Daily Output	Days		
Column Reinforment	6.80	6.8	TONS	2.3	3.0		Starts 7 days after beam and slab pour
Column Forming	1235	1235	SF	238	5.2		Starts 1 day after Column Reinf
Pour Columns	37	37	CY	92	0.4		
Beam Forming	6200	6200	SF	650	9.5	use 2 crews	Beam and Slab form starts 2 days after column form
Beam Reinf	10.1	10.1	TONS	2.7	3.7		beam reif starts 4 days befor beam and slab form ends
Slab Forming	5500	5500	SF	1100	5.0	use 2 crews	
Slab Reinf	4.1	4.1	TONS	2.9	1.4		
Pour Slab and Beams	210	210	CY	120	1.8		
Precast Panels	4107	4107	SF	3000	15 total per sequence		