Office of Outreach and Engagement

FINAL DELIVERABLE

Title	Historic Rehabilitation: Schmidt Building
Completed By	Yingie Ma, Nawaf Sultan, Tanner Wynn
Date Completed	May 2019
UI Department	Department of Civil & Environmental Engineering
Course Name	CEE:4850:0001 Senior Design
Instructor	Paul Hanley, Christopher Stoakes
Community Partners	City of Manning

This project was supported by the Provost's Office of Outreach and Engagement at the University of Iowa. The Office of Outreach and Engagement partners with rural and urban communities across the state to develop projects that university students and faculty complete through research and coursework. Through supporting these projects, the Office of Outreach and Engagement pursues a dual mission of enhancing quality of life in Iowa while transforming teaching and learning at the University of Iowa.

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I: Executive Summary

The following document is a complete report prepared by NYT Engineering for the preliminary design of the historic rehabilitation of the Schmidt Building requested by the City of Manning. NYT Engineering is located at 103 South Capitol Street in Iowa City, IA. Our team is comprised of three senior civil engineering students, Nawaf Sultan, Yinjie Ma, and Tanner Wynn, pursuing bachelor's degrees in the University of Iowa's ABET accredited Department of Civil Engineering. Nawaf Sultan is the project manager and was responsible for the structural stabilization of the eastern facade. Yinjie Ma is the technical support and was responsible for the roof repair and architectural design. Tanner Wynn is the editor was responsible for structural design of the floor systems and load calculations

Upon speaking with the client, the overall goal of the project is to stabilize the building and allow it to be reoccupied with a commercial first-floor and a second-level apartment. From our initial inspection, we were able to determine four main areas of concern throughout the building structure.

The first immediate area of concern is the building's front façade. Over the years, the brick veneer has pulled away from the clay tile layer because it is resting on the sidewalk which is experiencing frost heave. We have designed a steel bracing system to push the veneer back into a vertical position and provided steps to secure it to the structural clay tile. The steel bracing system is comprised of W8x13 columns, W4x13 diagonal bracing, and MC 8x20 C-Channels. After the wall has been stabilized, the sidewalk and soil beneath the veneer will need to be excavated so the veneer can rest on a concrete ledge and prevent this issue from occurring again in the future. The roof is the next area of immediate concern, as it has deteriorated and caused water damage throughout the building. The roof's entire exterior layer will need to be replaced with a new insulation and waterproof layer. During our initial inspection, much of the roof framing was not visible, however, we expect that approximately 25 percent of the roof framing will need to be replaced due to water damage. This will be confirmed after removal of the roof's exterior layer.

After the building's façade and roof are repaired, the next step in stabilizing the building will be repairing the building's foundation and floor framing systems. Due to the water damage, much of the building's first floor column supports and framing have rotted and need to be replaced. The new first floor framing system is comprised of 6x6 DFL No. 2 columns, 6x8 DFL select structural girders, and 4x8 DFL No. 2 joists spaced at 16" O.C. The columns will be equipped with post bases to prevent future water damage from occurring. The girders will be connected to the column with CC7-½-6 column caps, and the joists will be connected to the girders with U46 joist hangers. A majority of the second floor framing was not visible during our initial inspection, however, there was an extensive amount of creep that was causing the floor to slope towards the center. To level out the floor, we have extended the existing steel framing to the eastern wall of the building. The new steel framing section includes two W14x30 beams that connect the existing steel to a W18x71 girder that sits on top of two new 16x16 brick columns. The new brick columns follow the same concept as the existing ones, except larger to account for the increased load of the second story and upper roof.

After all four of the major stability concerns have been addressed, the building is ready to be remodeled. From the request of our client, we have designed an architectural plan that lays out the first story as a bakery, and the second story as single apartment with roof access.

Due to the building being in the heart of Manning's downtown, we expect construction to cause possible traffic congestion. The location of this building may also cause neighboring businesses to be impacted while excavation of the sidewalk and roadway are taking place. The biggest challenge this project contains is stabilizing the building's façade.

Below is a summary of the cost estimation for the entirety of the project. It includes ten percent for contingencies as well as administration costs. The phases do not all need to be completed at the same time, but they do need to be completed in this order. This table provides reasonable estimates for moving this project forward on a step by step basis.

Phase Number	Description	Cost
Phase 1	Façade Bracing	45,000
Phase 2	Roof Repairs	35,000
Phase 3	First Floor Framing Removal and Replacement	87,500
Phase 4	Second Floor Framing Repair	16,500
	Contingencies	18,500
	Administration	37,000
	Project Total	240,000

II: Organization Qualifications and Experience

2.1 Name of Organization
NYT Engineering

2.2 Organization Location:
103 South Capitol Street
Iowa City, Iowa 52242

2.3 Contact Information:
Nawaf Sultan
Project Manager
(319) 430-1900
nawaf-sultan@uiowa.edu

2.4 Organization Description and Experience
We are a team of three senior students pursuing Bachelor of Science in Engineering degrees in the University of Iowa's ABET accredited Department of Civil Engineering. NYT Engineering is composed of Nawaf Sultan,Yinjie Ma, and Tanner Wynn who are all currently enrolled in the capstone design course. Over the last four years, our team has worked with experienced faculty and staff on projects that have helped to developed our skills in the field of civil engineering.

These projects have included the use of structural analysis and modeling softwares to assist in the determination of design loads and deflections, designs comprised of different structural materials, such as steel, concrete, and wood, and design of foundations and roadways. With our wide range of experience throughout the last four years, each of our team members has taken an administrative role to provide our clients with the best design possible. Nawaf Sultan, who has a structural engineering focus area, is the project manager, and was responsible for the structural stabilization of the eastern facade. Yinjie Ma, who has a structural engineering focus area, is the technical support, and was responsible for the roof repair and architectural design. Tanner Wynn, who has a civil practice focus area, is the editor, was responsible for structural design of the floor systems and load calculations.

III: Design Services

3.1 Project Scope

The proposed project consists of the historic rehabilitation of the Schmidt building located on the corner of Main Street and 6th Street in Manning, Iowa. The goal of the project was to ensure the building's stability and allow it to be reoccupied while maintaining its historic features. Upon inspection, it was apparent that the building is in critical condition and is likely to fail in the future if measures are not taken to stabilize it.

There were four main areas of the building that needed to be repaired to ensure the building remained safe. The first was the building's eastern facade. It was determined that the facade was bowing outwards due to the deterioration of the ties to the clay structural wall. That damage was caused by weathering where the ties rusted away or disconnected from the anchoring points. The age of this building and the thermal expansion and contraction are the two major players that caused the type of damage that this building underwent. There are other causes, such as the material, type of the anchors, and poor water-proofing. Buildings in Iowa generally have freeze-thaw cycles, which speed up the process of deteriorations of materials. Figure 1 illustrates a side section of the building where the veneer wall is displaced from the original vertical location.

Figure 1: View of bowed facade.

Second is the building's roof. Large portions of the roof are leaking due to weathering and have caused water damage throughout the building (Figure 2).

Figure 2: View of exposed insulation and damage on lower roof.

The entirety of the exterior of the roof will need to be replaced and we expect approximately 50% of the roof framing will also need to be replaced. Third is the building's first floor framing. Most of the wooden members have experienced severe water damage and need to be replaced (Figure 3 and 4).

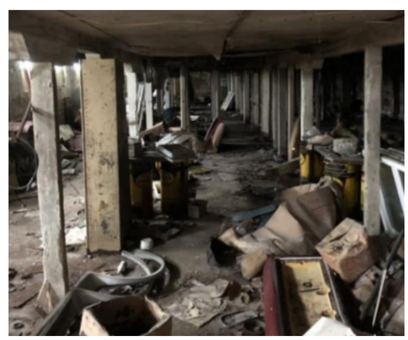

Figure 3: View of main floor framing.

Figure 4: View of floor framing posts.

The fourth area of concern is the second floor framing. Over the years, the floor has experienced creep, which has made it slope downward in the center significantly and needs to be made level.

Figure 5: View of Sloped Door Frame

Items incorporated with the design of the building rehabilitation included but are not limited to:

- Steel Bracing Design
- Concrete Ledge Design
- Roof Repair Design
- First Floor Framing Design
- Second Floor Framing Design
- Door and Window Locations and Sizes
- Material Lists
- Cost Estimates

3.2 Work Plan

The work plan chart shown in Figure 6 lays out the major tasks NYT Engineering completed in order and the number of days each task took. A graphical view of this information can be found in Figure 7. The conceptual design phase was the first key component of our work. It included selecting which design alternatives we would move forward with and creating pre-design plans for them. The next was completion of the final design. The final design included calculations, drawings, and renderings for every major component of the project. The last key component was writing the final report and giving presentations. This included laying out the work we completed in a way that easy to understand for the client.

Task	Start Date	End Date	Duration	Person Responsible
Project Introduction	1/14/2019	1/15/2019	1	Project Manager
Conference Call	1/24/2019	1/25/2019	1	Project Manager
Solution Research	1/25/2019	2/5/2019	11	All
Site Visit	2/1/2019	2/2/2019	1	Project Manager
Organize Information	2/2/2019	2/3/2019	1	Tech support
Proposal Report	2/2/2019	2/8/2019	6	Editor
Proposal Slides	2/2/2019	2/8/2019	6	Tech support
Proposal Presentation	2/11/2019	2/12/2019	1	All
Concept Design	2/12/2019	2/16/2019	4	Project Manager
Develop Conceptual Drawin	2/16/2019	2/22/2019	6	Project Manager
Complete Final Design	2/22/2019	4/7/2019	44	Project Manager
Final Design Report Draft	4/1/2019	4/12/2019	11	Editor
Final Drawing Set Draft	4/1/2019	4/12/2019	11	Tech support
Final Presentation Draft	4/1/2019	4/12/2019	11	Tech support
Final Poster Draft	4/1/2019	4/12/2019	11	Editor
Revise Final Design Report	4/13/2019	5/3/2019	20	Editor
Revise Final Drawing Set	4/13/2019	5/3/2019	20	Tech support
Revise Final Presentation	4/13/2019	4/22/2019	9	Tech support
Revise Final Poster	4/13/2019	5/3/2019	20	Editor
Give On-Campus Presenatio	4/22/2019	4/23/2019	1	All
Give Client Presenation	4/29/2019	5/10/2019	11	All
Submit Final Design Report	5/2/2019	5/3/2019	1	Project Manager
Submit Final Drawing	5/2/2019	5/3/2019	1	Project Manager
Submit Final Presentation	5/2/2019	5/3/2019	1	Project Manager
Submit Final Poster	5/2/2019	5/3/2019	1	Project Manager

Figure 6: Work Plan Chart

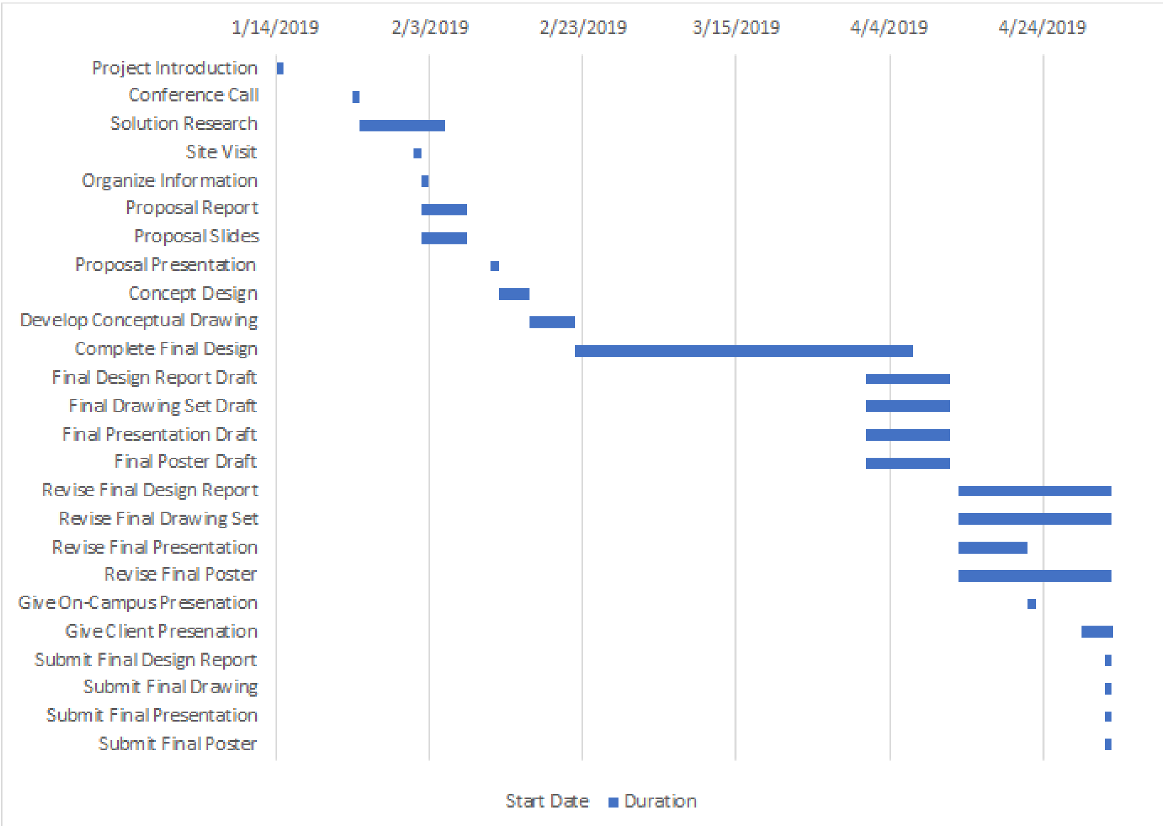

Figure 7: Work Plan Gantt Chart

IV: Constraints, Challenges and Impacts

4.1 Constraints

Some of the constraints we found for the site were that the building is at the heart of downtown. This means traffic congestion is a possibility when the construction begins. Contractors must find feasible locations for parking and material storage. Another challenge is the topography of the land which the building sits on. Any outside work on the building that required scaffolding had to be designed in a way that provided level flooring for the workers to be on, safety is our number one goal.

4.2 Challenges

Some of the challenges that this project brought were stabilizing some of the external walls, specifically the facade wall and the south side wall during construction. Not knowing the structural design of the facade wall, particularly the internal clay tile layer without beginning construction, was difficult. Finding one suitable way to maintain the stability of the structure during the construction was a major challenge we encountered. Another challenge we faced is determining how to make the cost as low as possible, due to low funding combined with poor initial building quality.

4.3 Societal Impact within the Community and/or State of Iowa
The neighboring business could be impacted when excavation of the building's front sidewalk and roadway are under construction. The building was designed as a new attraction for the city of Manning. Ideally, it will attract people in town, as well as towns in the surrounding area. The first floor was designed for commercial use and the second floor was designed as an apartment, which bring profits to the owners. One side of the building is located on 6th street, which is also known as Highway 141. This highway is a major artery that connects Manning with neighboring towns. The final outcome of this project will be beneficial to the local community and the surrounding areas.

V: Alternative Solutions that were Considered

5.1 Facade Rehabilitation Alternatives
The building's front facade is the most immediate item that needs to be addressed. Throughout the years, the brick veneer has bowed outwards due weathering and to frost heave of the sidewalk that supports it.

We considered jacking the eastern facade back into place with a steel bracing system, as well as removing and replacing the entire brick veneer. We ended up selecting the bracing system because it would be cheaper and faster to execute than replacing all the bricks. It also allows us to keep the original bricks and keep the exterior as original as possible.

We need to excavate the sidewalk and soil beneath the building's front brick veneer and place a concrete shelf beneath it to support the weight of the bricks. This will require the wall to be temporarily laterally braced with the use of either a steel framing system or drilled lateral tie backs into the wood framing. After the shelf is in place and the excavated items are replaced, the veneer can be permanently fixed to the clay tile layer and repaired as needed.

5.2 Roof Replacement
There is evident problems with the exterior of the lower roof and water damage throughout the building. During our initial inspection, not much of the roof framing was visible, but is expected to have experienced water damage.

Our only option was to remove the entire exterior layer of the lower and upper roofs because it is severely damaged. After it is removed, we will be able to further inspect the structural elements of the roof framing and determine if we will need to either replace or repair it. After the framing is fixed, new insulation will need to be installed and the exterior roofing material will need to be completely replaced with a new waterproofing layer to prevent future water damage from occurring.

5.3 Foundation and First Floor Framing Alternatives
The first floor framing has experienced extensive water damage and a majority of it is rotted. The bottom of the floor framing posts have also degraded due to water exposure.

The framing posts will need to be replaced with either larger wooden or metal columns with material around the base to prevent water damage. The floor framing will either be repaired using sister-joists (Figure 8) or completely replaced if needed. With sister-joists, we would keep the entire existing wooden members and brace them with sister-joists in the members that were unstable. Completely replacing the entire floor system would include a new floor layout with new columns, joists, and beams.

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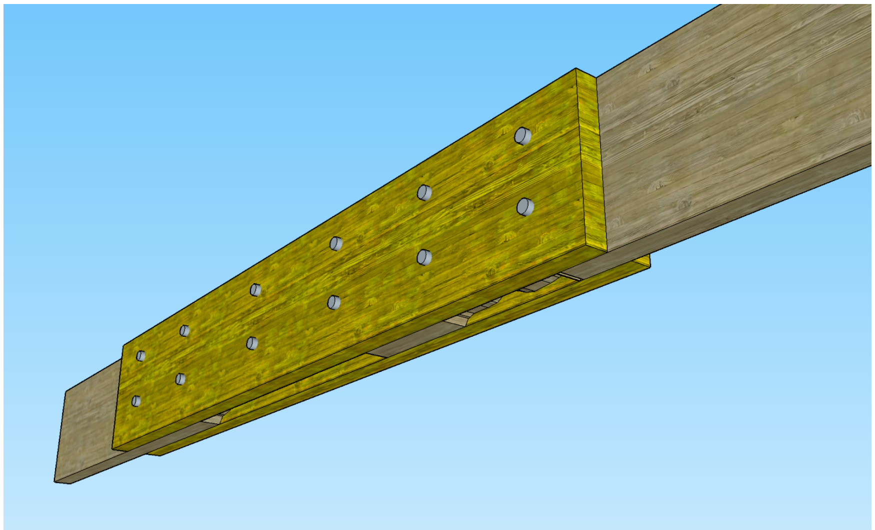


Figure 8: View of potential sister-joist configuration.

Ultimately, we decided to completely replace it with a new first floor framing system because too many of the wood members were damaged for sister-joists to be economical and the use of larger wooden posts were less expensive than using steel. If we had chosen steel columns, we would have had to cut into the foundation slab for their bases, which we did not want to do. Replacing the entire floor system also allowed us to design interior stairs for basement access.

5.4 Second Floor Framing Alternatives

The middle section of the second floor framing is sloped downward. We believe this is due to the wooden girders experiencing creep over time.

There are a few options we have for the second floor framing that depend on which option will be more effective. The first option is to repair or replace the second floor framing system with new wood due to the wood experiencing creep and water damage. The second option is to continue the brick column and steel framing from the back of the building to include the second floor. The steel framing option would also allow for the front facade to be connected to it.

We decided to move forward with the new steel framing option. We chose this because it would be less expensive and take less time than replacing the entire floor system. If we had chosen to replace the wood framing, we would have had to brace the walls from collapsing inwards and that would have been an additional cost. The steel framing design also allows for the first floor to look consistent from the from to the back

VI: Final Design Details

6.1 Facade Bracing

We have designed a steel bracing system to push the brick veneer back into a vertical position and connect it to the underlying clay tile layer. Prior to the process of re-aligning and pushing the front facade to the proper vertical position, the interior of the east wall must be removed for a detailed inspection of the clay structural wall by a fully qualified structural engineer.

After it is officially determined that the ties are the cause, the steel structure can be installed on the outside for the remodeling. Wall ties have been used in construction since the early 19th century. They are the standard for any construction that has an outer brick wall with a cavity between the structural wall and the brick wall. Usually, the brick wall acts as weather barrier, shielding the structural wall. The ties connection between the two walls improves the wall's stability, but does not act as a load bearing wall.

Phase 1

The best design for this building would be to build a steel frame that is about 7 feet away from the facade wall to allow the installation of scaffolding between the wall and the steel framing, which is usually 5 feet wide. The frame is built from (6) W8x18 steel columns that are about 15 feet tall, spaced 8'-0" on center. The columns will have diagonal bracing that is located at about 6 feet vertically (on the column) and 9 feet diagonally (to the sidewalk concrete slab). The columns and bracing are made from W4x13 steel I-beam that are about 8 feet in length. The columns and the bracing will have different lengths for each location due to the slope of the sidewalk. The sidewalk elevation decreases about 2'-6" from north-to-south on the east side of the building. Each location must have the correct length of column due to the uneven sidewalk. The columns and the bracing must have base plates added to them The plates are 12"x 12"x 1/2"thick steel, with (4) ¾" in diameter pre-drilled holes that are 1" from each edge of the plate.

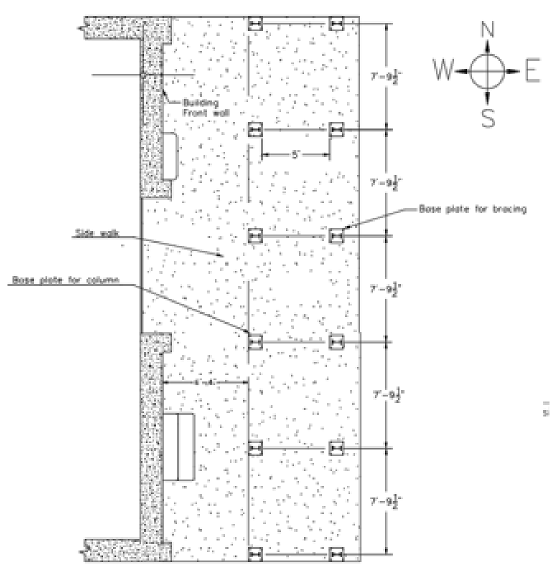


Figure 9: Location of Base Plates on Sidewalk

Figure 10: Elevation View of Bracing

After the columns have been installed, five C-Channel beams can be installed on the flange of the I-beams that are facing the building where the web of the C-Channel is flush on the flange. The connection must be made on site by using 2-1/2” long, 3/4” diameter bolts. Two bolts will be needed for each side of the C-Channel. Bolt locations and measurements can found in the plans and specification sheets.

The C-Channels are MC 8x20 that are about 7’-10” in length. The exact length only can be determined after the columns are installed and plumbed, therefore, the C-Channels must be delivered at 8 feet in length and cut onsite for the exact length. The C-Channel beams will be installed on the columns at the point of maximum outward bowing on the face of the building, approximately 14 feet from the bottom of the garage door. The C-Channels are installed to support manual 1-ton hydraulic jacks that can be mounted at different locations along the C-Channel beams as needed. No more than two jacks can be installed per C-Channel. The jacks will have about 5 feet extension rods. At the end of the rods, there will be a plate for distributing the load on the facade. That plates can vary in sizes depending on the location and as needed. After the facade is pushed to the correct vertical location, the facade will be supported temporarily.

Phase 2

Removal of some of the brick veneer in specific locations must be done to properly preserve the bricks for reinstallation after. The location and number of bricks removed per location can be found in Figure 11. It is important to clean the surfaces of older grout to provide better anchor to grout bonding. The brick removal be done one at a time to minimize the likelihood of structural failure during the removal process.

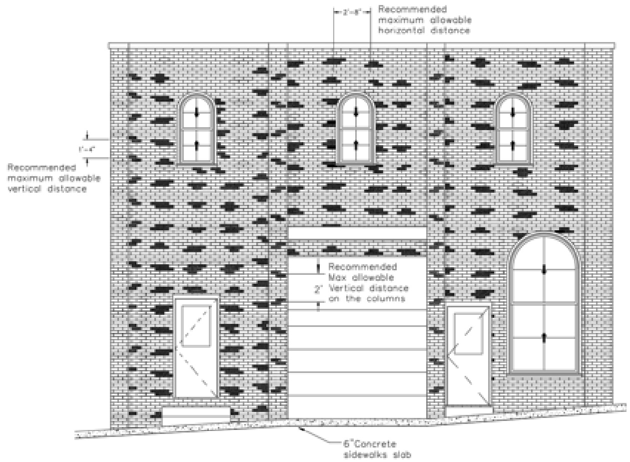


Figure 11: Brick Removal Locations

After all of the specified locations are removed, veneer anchors will be installed. For this project, we will be specifying the use of 315 Flexible Dovetail Triangular Anchors for the flat surface of the veneer wall as shown in Figure 12. The wire is carbon steel prefabricated from cold-drawn steel wire conforming to ASTM A1064/A1064M with a tensile strength of 80,000 psi and a yield point of 70,000 psi.

It is crucial that the wall ties are spaced the correct distance apart to ensure that the brick layer is stable without the possibility of it failing during or after the remodeling process. The recommended spacing for the anchor straps on the brick wall (one layer) is about 35 inches horizontally and 17 inches vertically. It is a requirement to have additional vertical ties around openings line doors and windows.

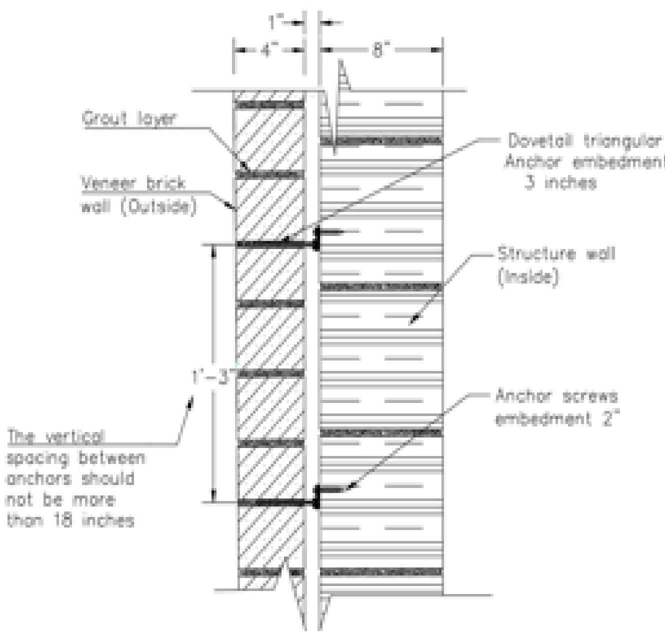


Figure 12: Anchor Installation Details

The Dovetail Triangular Anchors must be screwed into the structural wall by using 2” Hex head concrete anchor screws. To mount the anchors in the proper location, follow the guides and specification on page C-46 and 47 in the Building Code Requirements for Masonry Structures.

The other type of anchor that is recommended, specifically for the two thicker sections of the wall (the columns on each side of the garage door), are the stitch-tie type. This is where a hole would be drilled through the veneer wall the locations and the distances specified in the appendix. For better anchoring, it is recommended to drill the pilot hole through the brick, unlike the installation of the Dovetail Triangular Anchors. The contractors must follow the guides and specification for proper installation.

Based on the Building Code Requirements for Masonry Structures, the epoxy must have a shear strength of at least 50 psi (345 kPa) based on gross unit surface area when tested in accordance with ASTM C 482 on page C-48.

Another option would be to use the same the idea of drilling a pilot hole through the veneer, and instead of using the is CTP Stitch-ties, using threaded stainless bolts that will be anchored to the clay tile. These would have a decorative look and act as a supporting plate for larger area as shown in Figure 13. By having large anchored area, it would reduce the number of Dovetail Triangular Anchors, meaning less materials and installation time. That would lead to cutting cost, but with this type of installation, the spacing would need to be re-designed. There are no plans or specifications provided within this report for this type of anchoring.

Figure 13: Decorative supporting plate

After anchoring the brick veneer to the clay tile wall, the bricks that were removed can be reinstalled. Prior to installation, the bricks must be cleaned from any old grout. This will improve anchor to grout bonding.

Phase 3

Touch up the veneer by grinding off some of the grout that is damaged throughout the whole face of the building and other locations that require re-anchoring. Some of the steps to re-grouting are choosing a similar grout color for the area to make it blend in with the existing grout and remove the old grout with a grout saw. The grout mix is as important as choosing the correct color. The mix instructions should be provided by the manufacture. Use a float to place and spread the grout into the areas where the old grout was removed. The final step is use the float again to remove any grout left on the surfaces. A wet sponge can be used to remove the excess grout.

Phase 4

The concrete brick ledge was designed using the LRFD method included in ASCE-7. With the density of concrete masonry being 120 lb/ft^3, it gave a load of load of approximately 30 psi acting on the concrete ledge. We specify to use 3500 psi normal weight concrete. The location of the concrete ledge is beneath the brick veneer on the eastern side of the building. The soil and sidewalk in this area will need to be excavated as shown in Figure 14. During the removal of the sidewalk, the bottom of the brick veneer must be supported with typical construction practices. After excavation of the soil, the wood framework can be placed and filled with approximately three yards of the specified concrete. Once it has been allowed to cure for at least seven days, the framing can be removed and the load of the veneer can be allowed to rest on the concrete ledge. Tapcon concrete screws with a length of 7-½" will then be installed through the ledge and into the clay structural wall. These screws will need to be spaced at a vertical and horizontal distance of two feet. After installation of the screws, the exterior concrete ledge will need to be covered with waterproofing membrane.

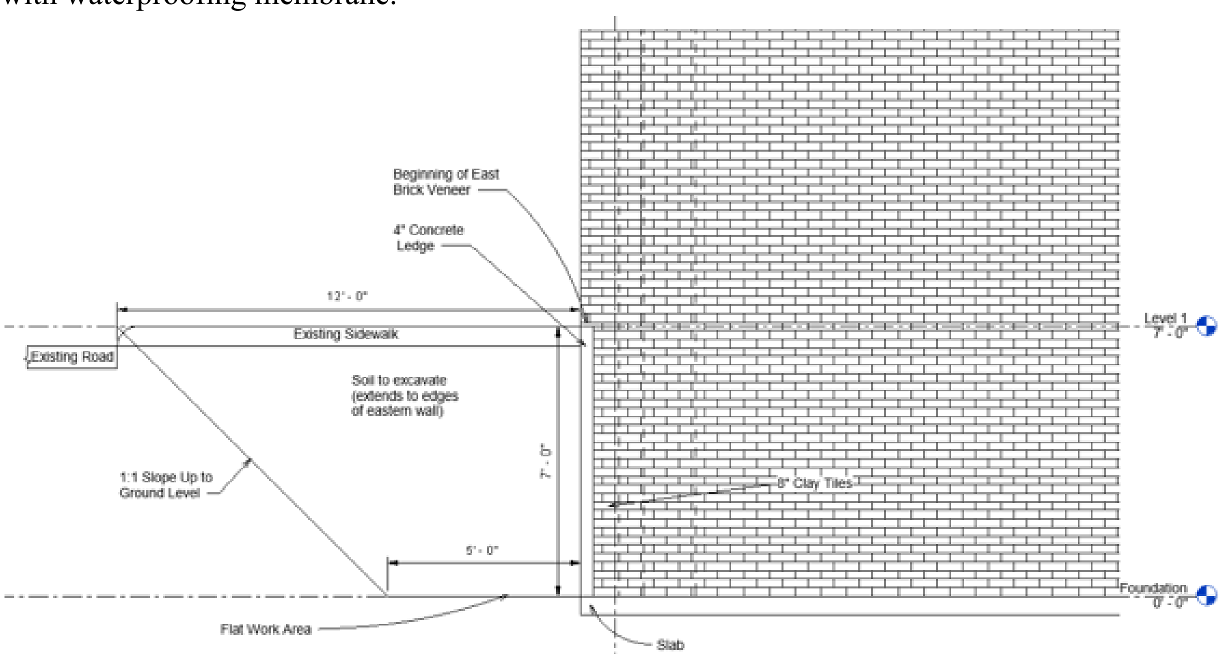


Figure 14: Concrete brick ledge and excavation plan.

6.2 Roof Repair

The roof was partially damaged over the west end of the building because of the rainwater seepage that accumulated for years. The current roofing system consists of lightweight EPDM membrane that is also seriously damaged for a large area. The damaged roof framing will be removed and rebuilt, and the surface system will be completely replaced. An EPDM roof system was chosen for this building as it was the one that was used previously. Typically, EPDM roofing assemblies, when properly-installed, properly-maintained and protected are capable of providing a service life of thirty to forty years. Wide rolls (10' +/-) of roofing membrane will be adhered over fastened roof insulation materials, with approximately 3-5'' laps. Fully-adhered assemblies include mechanically-fastened insulation components, with the EPDM membrane glued to the insulation based on EPDM bonding cement, and contact adhesive. (Figure 16)

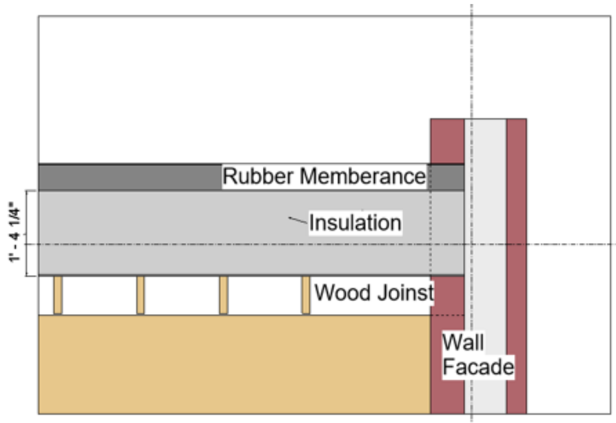


Figure 15: Roof cross-section diagram

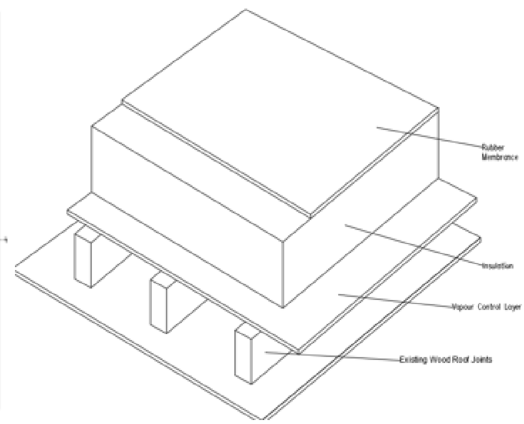


Figure 16: Roof layer system

6.3 First Floor Framing Removal and Replacement

The use of typical construction practices is suggested to support the load of the first floor while replacement of the floor framing system is in progress. Calculation of the loads acting on the floor framing system are based on the allowable strength design (ASD) specified in ASCE-7. The calculated stresses experienced in the wood members were compared with the allowable stresses specified in the National Design Specifications for Wood Construction. The ratio between these two is called the Design-Capacity (DCR). The optimum DCR for design falls between 0.9 and 1.0. All governing factors in the design of the wood members fell between this range, meaning they are being used efficiently.

Each column is a 6x6 No. 2 DFL, all of which are equipped with a 6x6 12-gauge galvanized adjustable post base (Figure 17) to prevent them from experiencing water damage in the future. The girders are all 6x8 DFL select structural and are connected to the columns with CC66 column caps (Figure 18). The joists are all 4x8 DFL No. 2 spaced at 16" O.C. and connected to the girders with U46 joist hangers (Figure 19). The joists that connect to the northern and southern walls will rest on top of the existing ledge and be braced with 6" of mortar around their perimeter (Figure 20). All connectors were selected with the use of the Simpson Wood Construction Connectors Manual.

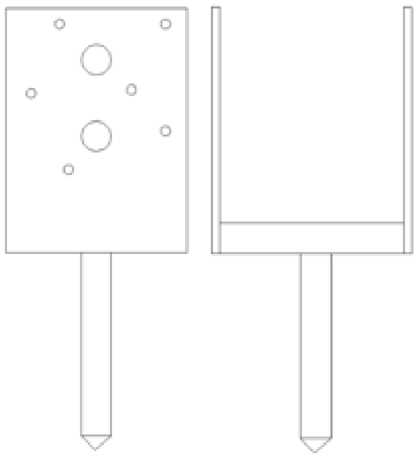


Figure 17: 6x6 Post Base

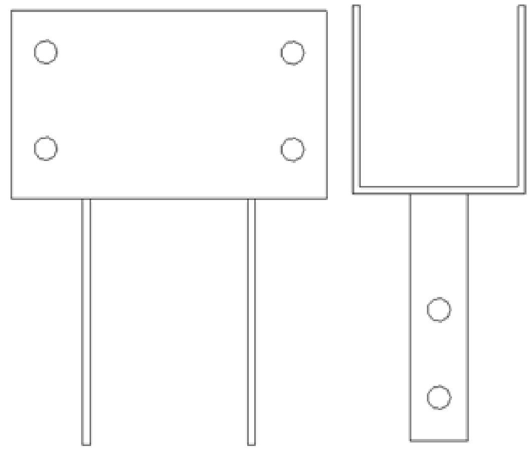


Figure 18: CC66 column cap

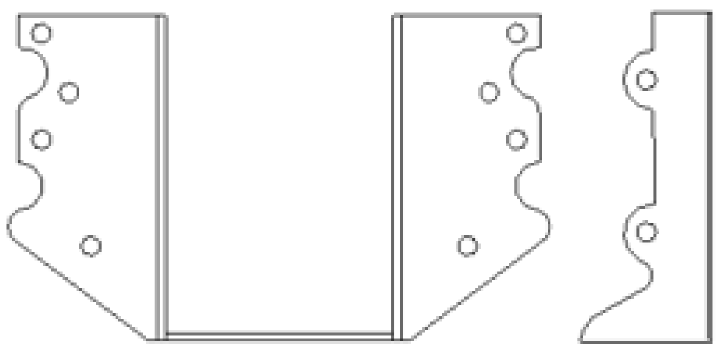


Figure 19: U46 Joist Hanger

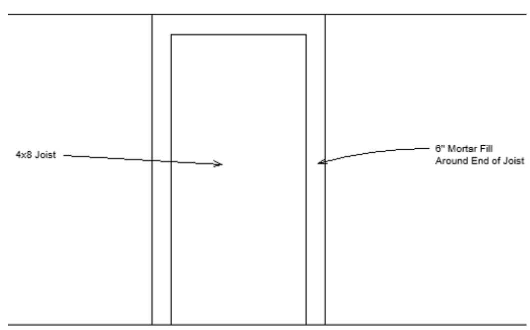


Figure 20: Joist to Wall Connection

6.4 Second Floor Framing Repair

The location of the brick columns must be measured by following the plans and specifications that are provided in the appendix. The brick columns are located on the interior two eastern corners of the building. They extend from the foundation slab, through the main floor, and up to the bottom of the second story. The length of the brick columns are about 26 feet long, starting from the foundation slab. The exact measurements can be found in the drawing plans.

The column installation must be done by following the guidelines found in the Building Code Requirements for Masonry Structures. Some of the requirements include the distance between each anchor to the existing wall, and how many layers of brick are allowed before the grout will be too compressed.

The brick columns will be used to support a W18x71 steel beam. The steel beam will hold half of the loads from the second floor and the upper roof. They must be leveled at their maximum height, as shown in Figure 21.

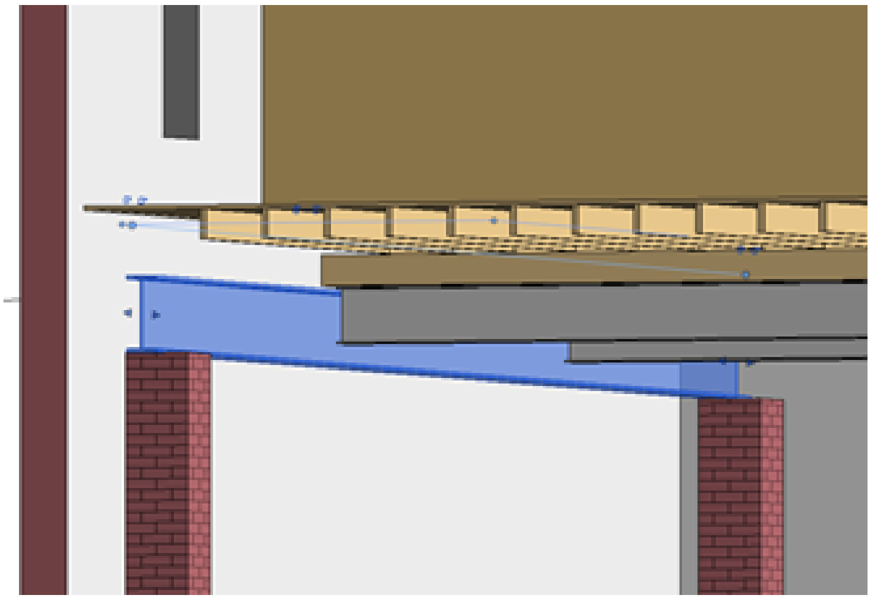


Figure 21: 3D View W18x71 location.

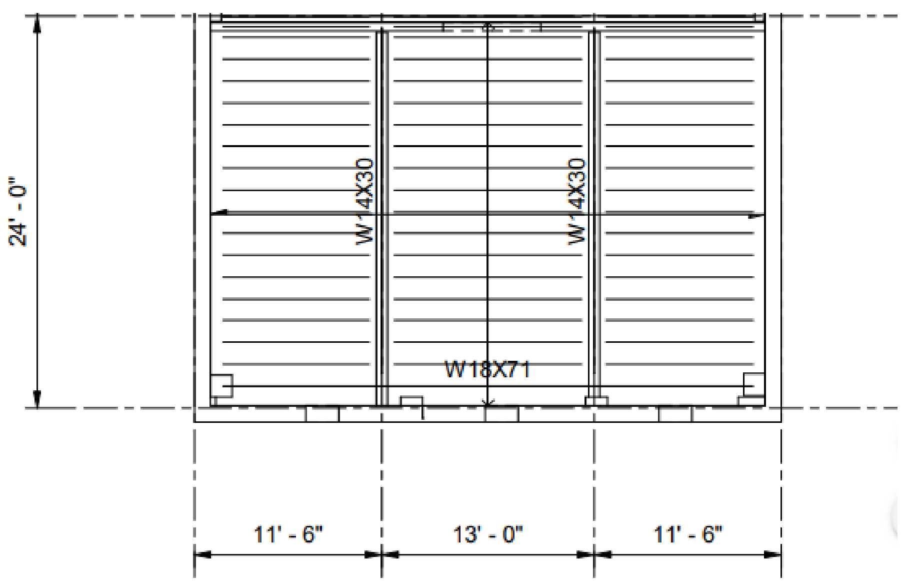


Figure 22: Plan View of New Steel Locations

The connection between the brick columns and the I-beam will consist of two 3/4” diameter bolts that go through the I-beam and the base plate, into the brick column (Figure 23). Specifications for these bolts and holes can be found in the drawing plans located in the appendix.

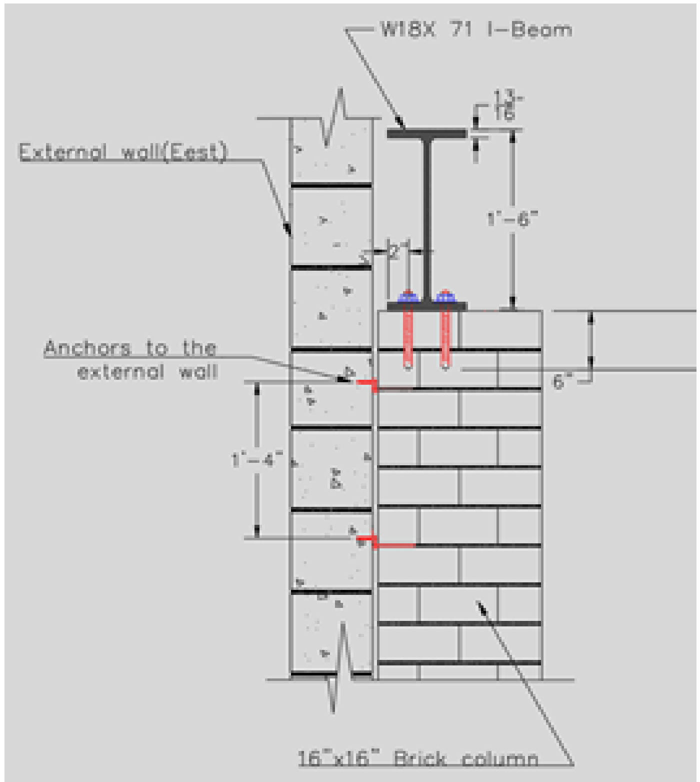


Figure 23: I-Beam location on the brick columns

VII: Engineers Cost Estimate

Phase 1					
Veneer wall Work					
Renting equmpments	Craft @ Hrs	Quantity	Unit	cost /month	
Scaffolding Rent (4 High @ 7 ft) 14' with base plate for a month		4	EA	230	920
	Craft @ Hrs	Quantity	Unit	cost /day	
Welding equmpment rent(201-300 amp)		1	EA	110	110
Venner Wall brick removal and Installation					
	Craft @ Hrs	Quantity	units	cost /unit	
Pre fabrecated steel brasing (W 8 X 18)		0.585	Ton	6564.2	3840.057
Pre fabrecated steel brasing (W 4 X 13)		0.312	Ton	6564.2	2048.0304
Pre fabrecated steel brasing (MC 8X 20)		0.4	Ton	4391.85	1756.74
wall Leveling (pushing)	avg 900-1200 $	1	lump sum	1500	1500
Concrete Screws		2	box = 50	31.45	62.9
Brick removal (Only In Set locations)@ 100%		150	EA	16.04	2406
Installing Stitch-tie anchors @ 100%	150 total	3	box = 50	140	420
Install dovetail triangular anchors @ 100%		127	EA	12.96	1645.92
Re-Installing bricks @ 100%	m1@.211	900	SF	16.04	14436
Re-grout Joints @ 50 %		450	SF	20	9000
Cleaning grout		1088	SF	0.66	718.08
Veneer Wall Footing work					
	Craft @ Hrs	Quantity	Unit	cost /unit	
Side walk demo (6 ")	3@.078	340	CY	6.33	2152.2
Dirt Excavation		45	CY	6.3	283.5
Under ground electric work (if needed)					
Under grount water/ sewier lines (if need)					
Reinforcing work site space for footing		24	HR	30	720
Form work for concrret footing		12	LF	30	360
Concrete ready mix placing (for footing)		1.677	CY 500 (min)	500	838.5
Forms removal		6	LF	30	180
Work site space Reinforcment removal		6	HR	30	180
Dirt filling and compacting	S1@.47	45	CY	4.25	191.25
Gravil filling and compacting		3.14	CY	4.25	13.345
Concrete ready mix placing (for Side walk)	p8 @ .016	340	SF	3.04	1033.6
				Total	44816.12

Phase 2					
First Floor (Roof Work)					
	Craft @ Hrs	Quantity	units	t/ sq=(100 SF)/ 4 man team	
Roof demo	CL@2.91	1	lump sum	7500	7500
Roof installation @ 100		1	lump sum	17000	17000
Scond Floor (Roof work)					
Roof demo		1	lump sum	2500	2500
Roof installation @ 100		1	lump sum	8029	8029
				Total	35029

Phase 3

Inspection	Craft @ Hrs	Quantity	Units	Cost/unit	Total
Asbestos hazard survys (4 hrs min)		4	HR	90.4	361.6
sample anaylsys (10 sample min)		10	EA	55.3	553
report Writing ($200 min)		1	report	200	200

Asbestos removal	Craft @ Hrs	Quantity	Units	Cost/unit	
ceilling insulation (500-5,000)sf		952	SF	20	19040
pip insulation (1000-3000) sf		216	LF	28.6	6177.6
wall insulation (500-5,000)sf		300	LF	45.7	13710

First floor work (Removal)	Craft @ Hrs	Quantity	units	Cost /unit	
Firs floor concrete slab demo (light weight on deck)	cl @0.029	898	SF	7.3	6555.4

First floor work (Installation)	Craft @ Hrs	Quantity	units	cost/ unit	
Column base		16	EA	28.23	451.68
Floor columns (12 X 12 in timber)		115	LF	10.5	1207.5
6X8 (in) Timber girders		420	LF	10.5	4410
Nails		1	box	63.2	63.2
Floor Joists (trusses)		2695	LF	5.31	14310.45
Floor joist hangers (First floor)	1 box = 100 eh	5	468	171.05	855.25
Floor shetting (3/4" OSB)	C8 @0.011	2788	SF	1.45	4042.6
Floor finsh(hardboard wallcovering3/16"pegboard)	C8 @ .025	2788	SF	1.73	4823.24
Brick columns instalation	M1@.696	52	LF	50.8	2641.6
Brick columns anchors		60	EA	12.96	777.6
Steel girder instalation (W 14X 30)		0.72	Ton	4391.85	3162.132
Steel girder instalation (W 18X 71)		0.035	Ton	4095.15	143.33025
Steel girder Connections to beam (plates)		8	EA	45	360
Steel girder Connections to brick colums (bolts)		4	EA	7.25	29
fireproofing the steel beams (spry)	A1@0.026	116	LF	32.25	3741
				Total	**87616.18**

Phase 4

Second floor (Removal)					
wall demo, ceiling demo	cl @0.022	Quantity			
Debris removal	$3000 min charge	1	CY	1500	1500
Wast Disposal 40 cy (lumber, drywall ,roofing)		20	40 CY	45	900
dump fees ($)/ time		4	EA	120	480
hauling cost/40cy		4	40 CY	320	1280

Second floor (Installation)	Craft @ Hrs	Quantity	units	cost /unit	
Floor joist installation		48	EA	8.9	427.2
Roof joist hangers and Second floor @ 50%	1 box = 100 eh	3	234	171.05	513.15
Floor sheets installation (3/4" OSB)		690	SF	5.31	3663.9
Nails		1	Box	45	45
Walls installation 2x4x8		320	SF	10.84	3468.8
Wall insulation (R-21)	BC@.008	320	SF	1.85	592
Drywall, mudding ,taping , sanding		320	SF	7.8	2496
Drywall nails and screws		1	Box	80	80
Painting		1	5 GL	400	400
Finshing		24	Hr	30	720
				Total	**16566.05**

Phases total	**184027.35**
Contingencies (10 %)	**18402.74**
Engineering and Administration (20 %)	**36805.47**
Project Total ($)	**239235.56**

VIII: Appendix

DEAD LOADS

Roof (Second Floor / First Floor)

Upper Roof Dead Load Components	Material	Load (psf)
Roof Covering	waterproofing membrane, bituminous smooth surface	1.00
Roof Insulation (per inch)	Rigid insulation	1.50
Roof insulation Thickness (in)	4	6.00
Roof Underlayment	waterproofing membrane, self adhering/liquid applied	0.35
Roof Sheathing	Plywood/OSB, 5/8 in	3.60
Roof Framing		
Roof Framing Roof Slope	flat	12.00
Horizontal Plane Upper Roof Dead Load		10.95

(Plus 6 psf for Mechanical, Plumbing, and Lighting and
5 psf for steel framing, 2 psf for wood framing)

$D_{FirstRoof} := 10.95\ psf + 6\ psf + 5\ psf + 2\ psf = 23.95\ psf$ (load onto small brick columns)

$D_{SecondRoof} := 10.95\ psf + 6\ psf + 2\ psf = 18.95\ psf$

Second Level Floor (Need to determine actual second floor framing upon further inspection)
(To get an estimated load)

Floor Dead Load Components	Material	Load (psf)
Floor Finish	hardwood flooring, 3/4 in	3
Concrete Topping (per inch)	Normal weight	12.08333
Concrete Thickness (in)	0	0
Floor Underlayment	Wood panel underlayment, 1/4 in	0.8
Subfloor	Plywood/OSB, 3/8 in	1.2
Floor Framing	Type in	Calc
Ceiling Insulation (per inch)	Batt, Fiberglass	0.04
Ceiling Insulation Thickness (in)	0.5	0.02
Ceiling	Gypsum board, 1/2 in	2.2
Lighting	Lighting and Conduit	1
Mechanical	Duct Allowance	4
Plumbing	Piping Allowance	1
Floor Dead Load		13.22

(Plus 5 psf for steel framing+2 psf for wood framing)

$D_{2wood} := 13.22\ psf + 5\ psf + 2\ psf = 20.22\ psf$ (kitchen/living room)

Floor Dead Load Components	Material	Load (psf)
Floor Finish	Carpet with pad	2

$D_{2carpet} := 12.22\ psf + 5\ psf + 2\ psf = 19.22\ psf$ (bedrooms)

Floor Dead Load Components	Material	Load (psf)
Floor Finish	ceramic or porcelain tile, 3/8 in	5

$D_{2tile} := 15.22\ psf + 5\ psf + 2\ psf = 22.22\ psf$ (bathrooms)

(All loads act on larger brick columns)

First Level Floor

Floor Dead Load Components	Material	Load (psf)
Floor Finish	hardwood flooring, 3/4 in	3
Concrete Topping (per inch)	Normal weight	12.08333
Concrete Thickness (in)	0	0
Floor Underlayment	Wood panel underlayment, 1/4 in	0.8
Subfloor	Plywood/OSB, 3/8 in	1.2
Floor Framing	Type in	Calc
Ceiling Insulation (per inch)	Batt, Fiberglass	0.04
Ceiling Insulation Thickness (in)	0.5	0.02
Ceiling	Gypsum board, 1/2 in	2.2
Lighting	Lighting and Conduit	0
Mechanical	Duct Allowance	0
Plumbing	Piping Allowance	1
Floor Dead Load		8.22

$D_{1wood} := 8.22\ psf + 2\ psf = 10.22\ psf$ (all area besides kitchen)

$D_{1tile} := 10.22\ psf + 2\ psf = 12.22\ psf$ (kitchen)

(All loads act on wood foundation columns)

Foundation Slab

$D_{slab} := 150\ pcf \cdot 6\ in = 75\ psf$ (Assuming 6" thick slab)

LIVE LOADS

Roof (Second Floor / First Floor)

(If first level roof is intended to be accessible)

$L_{1r} := 40\ psf$ (Less than 50 people at a time) (First Level Roof Only)

$L_r := 20\ psf$ (Second Level Roof is 20 psf Regardless)

Second Level Floor

$L_2 := 40\ psf$ (All areas except for stairs)

$L_{2stairs} := 40\ psf$ (Stairs for 1-2 family dwellings)

First Level Floor

$L_{1kitchen} := 40\ psf$ (Private Room)

$L_{1other} := 100\ psf$ (Restaurant / Public Area / Stairs to Basement)

Foundation Slab

$L_{slab} := 40\ psf$ (Storage space, heavier than ceiling but lighter than storage warehouse)

Snow Loads

$p_g := 25\ psf$ Ground Snow Load for Manning, Iowa

Balanced Snow Load

Thermal Factor $C_t := 1.0$ Table 7.3-2

Exposure Factor $C_e := 1.0$ Table 7.3-1 (B, Partially Exposed)

Risk Category II $I_s := 1.0$

Flat Roof Snow Load $p_f := 0.7 \cdot C_t \cdot C_e \cdot I_s \cdot p_g = 17.5\ psf$

Minimum Snow Load for Low-Sloped Roof

Roof slope downward from East to West $\theta := \tan\left(\frac{(8\ in)}{20.5\ ft}\right) = 1.864\ deg$

Minimum Snow Load for Low-Sloped Roof $p_m := 20\ psf \cdot I_s = 20\ psf$

$(20.5\ ft \cdot 4) \cdot \sin(\theta) = 2.667\ ft$

Roof Slope Factor $C_s := 1.0$

Sloped Roof Snow Load $p_s := C_s \cdot p_f = 17.5\ psf$

Minimum snow load to design for: $p_m = 20\ psf$

$p_f := p_m = 20\ psf$

Unbalanced snow loads need not be considered for slopes >30.2 deg or slopes < 2.38 deg

Snow Drift Loads

Snow Unit Weight $\gamma_s := \frac{0.13}{ft} \cdot p_g + 14\ pcf = 17.25\ pcf$

Height of balanced snow load $h_b := \frac{p_s}{\gamma_s} = 1.014\ ft$

Clear height of $h_{cp} := 2\ ft - h_b = 0.986\ ft$

Clear height of adjacent building wall and 2nd story wall $h_{ab} := 11\ ft - h_b = 9.986\ ft$

Drift Load Check $\frac{h_{cp}}{h_b} = 0.971$ $\frac{h_{ab}}{h_b} = 9.843$

hc/hb < 0.2 Drift loads must be considered

WIND FROM S to N

Northern Wall (adjacent building)

Calculate height of leeward drift

$l_{u\_leeward} := 50\ ft$

$$h_{d\_leeward\_theo} := \left(0.45\ ft \cdot \sqrt[3]{\frac{l_{u\_leeward}}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\ ft\right) \cdot \sqrt{I_s} = 2.532\ ft$$

$$h_{d\_leeward} := min\left(h_{d\_leeward\_theo}, 0.6 \cdot l_{u\_leeward}\right) = 2.532\ ft$$

Calculate height of windward drift

$l_{u\_windward} := 34\ ft$

$$h_{d\_windward} := 0.75 \cdot \left(0.45\ ft \cdot \sqrt[3]{\frac{l_{u\_windward}}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\ ft\right) \cdot \sqrt{I_s} = 1.534\ ft$$

Calculate design snow drift height

$h_{d\_theo} := \max\left(h_{d\_windward}, h_{d\_leeward}\right) = 2.532\ ft$

$h_d := min\left(h_{d\_theo}, h_{ab}\right) = 2.532\ ft$

Calculate design snow drift width

$w_{d1} := 4 \cdot h_d = 10.129\ ft$ $\qquad w_{d2} := \dfrac{4 \cdot {h_d}^2}{h_{ab}} = 2.569\ ft$

$w_{d\_theo} := \text{if}\left(h_d \le h_{ab}, w_{d1}, w_{d2}\right) = 10.129\ ft$

$w_{d\_max} := min\left(8 \cdot h_{ab}, 34\ ft\right) = 34\ ft$

$w_d := min\left(w_{d\_theo}, w_{d\_max}\right) = 10.129\ ft$

Calculate snow drift load on Northern wall

$p_d := h_d \cdot \gamma_s + p_f = 63.682\ psf$

Southern/Northern Parapet wall

Calculate height of leeward drift

N/A

Calculate height of windward drift

$l_{u\_windward} := 34\ ft$

$h_{d\_windward} := 0.75 \cdot \left(0.45\ ft \cdot \sqrt[3]{\dfrac{l_{u\_windward}}{ft}} \cdot \sqrt[4]{\dfrac{p_g}{psf} + 10} - 1.5\ ft\right) \cdot \sqrt{I_s} = 1.534\ ft$

Calculate design snow drift height

$h_{d\_theo} := h_{d\_windward} = 1.534\ ft$

$h_d := min\left(h_{d\_theo}, h_{cp}\right) = 0.986\ ft$

Calculate design snow drift width

$w_{d1} := 4 \cdot h_d = 3.942\ ft$ $\qquad w_{d2} := \dfrac{4 \cdot {h_d}^2}{h_{cp}} = 3.942\ ft$

$w_{d\_theo} := \text{if}\left(h_d \le h_{cp}, w_{d1}, w_{d2}\right) = 3.942\ ft$

$w_{d\_max} := min\left(8 \cdot h_{cp}, 34\ ft\right) = 7.884\ ft$

$w_d := min\left(w_{d\_theo}, w_{d\_max}\right) = 3.942\ ft$

Calculate snow drift load on Northern/Southern Parapet Walls

$p_d := h_d \cdot \gamma_s + p_f = 37\ psf$

Wind from E to W

Eastern wall extending from 1st floor roof to second floor roof

Calculate height of leeward drift

$l_{u\_leeward} := 24\ ft$

$h_{d\_leeward\_theo} := \left(0.45\ ft \cdot \sqrt[3]{\dfrac{l_{u\_leeward}}{ft}} \cdot \sqrt[4]{\dfrac{p_g}{psf} + 10} - 1.5\ ft\right) \cdot \sqrt{I_s} = 1.657\ ft$

$h_{d\_leeward} := min\left(h_{d\_leeward\_theo}, 0.6 \cdot l_{u\_leeward}\right) = 1.657\ ft$

Calculate height of windward drift

$l_{u\_windward} := 82\ ft$

$h_{d\_windward} := 0.75 \cdot \left(0.45\ ft \cdot \sqrt[3]{\frac{l_{u\_windward}}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\ ft\right) \cdot \sqrt{I_s} = 2.441\ ft$

Calculate design snow drift height

$h_{d\_theo} := \max\left(h_{d\_windward}, h_{d\_leeward}\right) = 2.441\ ft$

$h_d := min\left(h_{d\_theo}, h_{ab}\right) = 2.441\ ft$

Calculate design snow drift width

$w_{d1} := 4 \cdot h_d = 9.766\ ft$ $\qquad$ $w_{d2} := \frac{4 \cdot {h_d}^2}{h_{ab}} = 2.388\ ft$

$w_{d\_theo} := \text{if}\left(h_d \le h_{ab}, w_{d1}, w_{d2}\right) = 9.766\ ft$

$w_{d\_max} := min\left(8 \cdot h_{ab}, 82\ ft\right) = 79.884\ ft$

$w_d := min\left(w_{d\_theo}, w_{d\_max}\right) = 9.766\ ft$

Calculate snow drift load on Eastern Wall

$p_d := h_d \cdot \gamma_s + p_f = 62.114\ psf$

Eastern parapet wall on 2nd story roof

Calculate height of leeward drift

N/A

Calculate height of windward drift

$$l_{u\_windward} := 24\ ft$$

$$h_{d\_windward} := 0.75 \cdot \left(0.45\ ft \cdot \sqrt[3]{\frac{l_{u\_windward}}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\ ft\right) \cdot \sqrt{I_s} = 1.243\ ft$$

Calculate design snow drift height

$$h_{d\_theo} := h_{d\_windward} = 1.243\ ft$$

$$h_d := min\left(h_{d\_theo}, h_{cp}\right) = 0.986\ ft$$

Calculate design snow drift width

$$w_{d1} := 4 \cdot h_d = 3.942\ ft \qquad w_{d2} := \frac{4 \cdot {h_d}^2}{h_{cp}} = 3.942\ ft$$

$$w_{d\_theo} := \text{if}\left(h_d \le h_{cp}, w_{d1}, w_{d2}\right) = 3.942\ ft$$

$$w_{d\_max} := min\left(8 \cdot h_{cp}, 82\ ft\right) = 7.884\ ft$$

$$w_d := min\left(w_{d\_theo}, w_{d\_max}\right) = 3.942\ ft$$

Calculate snow drift load on Eastern parapet wall

$$p_d := h_d \cdot \gamma_s + p_f = 37\ psf$$

Wind Loads

Elevations of floors

$z_1 := 0\ ft$ (first floor at ground level) (constant below 15ft)

$z_1 := 15\ ft$

$z_2 := 28\ ft - 1\ ft - 7\ ft = 20\ ft$

$z_3 := 39\ ft - 7\ ft = 32\ ft$

$z_p := z_3 + 2\ ft = 34\ ft$

Building Dimensions

$L := 106\ ft$ $B := 36\ ft$

$\frac{L}{B} = 2.944$ $\frac{B}{L} = 0.34$

Select Risk Category and basic wind speed

Location: Manning, IA
Risk Category II

$V := 110\ mph$ ASCE 26.5-1D

$I_w := 1.0$

Determine Kd

$K_d := 0.85$

Determine Kzt

$K_{zt} := 1$

Determine exposure category

$$z_{ground} := 1358\ ft$$

Exposure B

Calculate Ke

$$K_e := \exp\left(-0.00000362 \cdot \frac{z_{ground}}{ft}\right) = 0.995$$

Calculate G

North - South

$$L_{eff} := \frac{B \cdot (z_1 + z_2 + z_3)}{(z_1 + z_2 + z_3)} = 36\ ft \qquad h := z_3 = 32\ ft$$

$$approxFreq := \text{if}\left(h \le 300\ ft \wedge h < 4 \cdot L_{eff}, \text{“Yes”}, \text{“No”}\right) = \text{“Yes”}$$

$$n_a := \frac{43.5}{\left(\frac{h}{ft}\right)^{0.9}}\ Hz = 1.922\ Hz$$

$$n_1 := \begin{array}{l} \text{if } approxFreq = \text{“Yes”} \\ \quad \| \; n_a \\ \text{else} \\ \quad \| \; 0 \end{array} = 1.922\ Hz$$

$$dynamicResponse := \begin{array}{l} \text{if } n_1 < 1\ Hz \\ \quad \| \; \text{“Flexible”} \\ \text{else} \\ \quad \| \; \text{“Rigid”} \end{array} = \text{“Rigid”}$$

$$G_{ew} := 0.85$$

$$G := 0.85$$

East - West

$$L_{eff} := \frac{L \cdot (z_1 + z_2 + z_3)}{(z_1 + z_2 + z_3)} = 106\ ft \qquad h := z_3 = 32\ ft$$

$$approxFreq := \text{if}\left(h \le 300\ ft \wedge h < 4 \cdot L_{eff}, \text{"Yes"}, \text{"No"}\right) = \text{"Yes"}$$

$$n_a := \frac{43.5}{\left(\frac{h}{ft}\right)^{0.9}}\ Hz = 1.922\ Hz$$

$$n_1 := \begin{cases} n_a & \text{if } approxFreq = \text{"Yes"} \\ 0 & \text{else} \end{cases} = 1.922\ Hz$$

$$dynamicResponse := \begin{cases} \text{"Flexible"} & \text{if } n_1 < 1\ Hz \\ \text{"Rigid"} & \text{else} \end{cases} = \text{"Rigid"}$$

$$G_{ew} := 0.85$$

Determine Enclosure Classification

Partially Enclosed

Internal Pressure Coefficient

$$GC_{pi} := 0.55$$

Calculate Kz

$a := 7.0 \qquad z_g := 1200\ ft$

$$K_1 := 2.01 \cdot \left(\frac{15\ ft}{z_g}\right)^{\frac{2}{a}} = 0.575 \qquad K_3 := 2.01 \cdot \left(\frac{32\ ft}{z_g}\right)^{\frac{2}{a}} = 0.714$$

$$K_2 := 2.01 \cdot \left(\frac{20\ ft}{z_g}\right)^{\frac{2}{a}} = 0.624 \qquad K_p := 2.01 \cdot \left(\frac{34\ ft}{z_g}\right)^{\frac{2}{a}} = 0.726$$

$$q_1 := \left(\frac{0.00256\ psf}{mph^2}\right) \cdot K_1 \cdot K_{zt} \cdot K_d \cdot K_e \cdot (V)^2 = 15.058\ psf$$

$$q_2 := \left(\frac{0.00256\ psf}{mph^2}\right) \cdot K_2 \cdot K_{zt} \cdot K_d \cdot K_e \cdot (V)^2 = 16.348\ psf$$

$$q_3 := \left(\frac{0.00256\ psf}{mph^2}\right) \cdot K_3 \cdot K_{zt} \cdot K_d \cdot K_e \cdot (V)^2 = 18.697\ psf$$

$$q_p := \left(\frac{0.00256\ psf}{mph^2}\right) \cdot K_p \cdot K_{zt} \cdot K_d \cdot K_e \cdot (V)^2 = 19.024\ psf$$

Determine External Pressure Coefficients

$$EW := \frac{B}{L} = 0.34 \qquad NS := \frac{L}{B} = 2.944$$

Windward Wall: $C_{p\_windward} := 0.8$

Leeward Wall: $C_{p\_leewardEW} := -0.5$ $C_{p\_leewardNS} := -0.377$

Windward Parepet: $GC_{pn\_windward} := 1.5$

Leeward Parapet: $GC_{pn\_leeward} := -1.0$

Positive Internal EW
Windward

$$p_{1\_pos} := q_1 \cdot G \cdot C_{p\_windward} - q_3 \cdot (GC_{pi}) = -0.044\ psf$$

$$p_{2\_pos} := q_2 \cdot G \cdot C_{p\_windward} - q_3 \cdot (GC_{pi}) = 0.833\ psf$$

$$p_{3\_pos} := q_3 \cdot G \cdot C_{p\_windward} - q_3 \cdot (GC_{pi}) = 2.431\ psf$$

Leeward

$$p_{leeward\_pos} := q_3 \cdot G \cdot C_{p\_leewardEW} - q_3 \cdot GC_{pi} = -18.23\ psf$$

Net Pressure

$$p_{1\_netpos} := p_{1\_pos} + \mathrm{abs}\left(p_{leeward\_pos}\right) = 18.186\ psf$$

$$p_{2\_netpos} := p_{2\_pos} + \mathrm{abs}\left(p_{leeward\_pos}\right) = 19.063\ psf$$

$$p_{3\_netpos} := p_{3\_pos} + \mathrm{abs}\left(p_{leeward\_pos}\right) = 20.661\ psf$$

Negative Internal EW

Windward

$$p_{1\_pos} := q_1 \cdot G \cdot C_{p\_windward} - q_3 \cdot \left(-GC_{pi}\right) = 20.523\ psf$$

$$p_{2\_pos} := q_2 \cdot G \cdot C_{p\_windward} - q_3 \cdot \left(-GC_{pi}\right) = 21.4\ psf$$

$$p_{3\_pos} := q_3 \cdot G \cdot C_{p\_windward} - q_3 \cdot \left(-GC_{pi}\right) = 22.998\ psf$$

Leeward

$$p_{leeward\_pos} := q_3 \cdot G \cdot C_{p\_leewardEW} - q_3 \cdot \left(-GC_{pi}\right) = 2.337\ psf$$

Net Pressure

$$p_{1\_netpos} := p_{1\_pos} + \text{abs}\left(p_{leeward\_pos}\right) = 22.86\ psf$$

$$p_{2\_netpos} := p_{2\_pos} + \text{abs}\left(p_{leeward\_pos}\right) = 23.737\ psf$$

$$p_{3\_netpos} := p_{3\_pos} + \text{abs}\left(p_{leeward\_pos}\right) = 25.335\ psf$$

Positive Internal NS

Windward

$$p_{1\_pos} := q_1 \cdot G \cdot C_{p\_windward} - q_3 \cdot \left(GC_{pi}\right) = -0.044\ psf$$

$$p_{2\_pos} := q_2 \cdot G \cdot C_{p\_windward} - q_3 \cdot \left(GC_{pi}\right) = 0.833\ psf$$

$$p_{3\_pos} := q_3 \cdot G \cdot C_{p\_windward} - q_3 \cdot \left(GC_{pi}\right) = 2.431\ psf$$

Leeward

$$p_{leeward\_pos} := q_3 \cdot G \cdot C_{p\_leewardNS} - q_3 \cdot GC_{pi} = -16.275\ psf$$

Net Pressure

$$p_{1\_netpos} := p_{1\_pos} + \text{abs}\left(p_{leeward\_pos}\right) = 16.231\ psf$$

$$p_{2\_netpos} := p_{2\_pos} + \text{abs}\left(p_{leeward\_pos}\right) = 17.108\ psf$$

$$p_{3\_netpos} := p_{3\_pos} + \text{abs}\left(p_{leeward\_pos}\right) = 18.706\ psf$$

Negative Internal NS

Windward

$$p_{1\_pos} := q_1 \cdot G \cdot C_{p\_windward} - q_3 \cdot \left(-GC_{pi}\right) = 20.523\ psf$$

$$p_{2\_pos} := q_2 \cdot G \cdot C_{p\_windward} - q_3 \cdot \left(-GC_{pi}\right) = 21.4\ psf$$

$$p_{3\_pos} := q_3 \cdot G \cdot C_{p\_windward} - q_3 \cdot \left(-GC_{pi}\right) = 22.998\ psf$$

Leeward

$$p_{leeward\_pos} := q_3 \cdot G \cdot C_{p\_leewardNS} - q_3 \cdot \left(-GC_{pi}\right) = 4.292\ psf$$

Net Pressure

$$p_{1\_netpos} := p_{1\_pos} + \text{abs}\left(p_{leeward\_pos}\right) = 24.815\ psf$$

$$p_{2\_netpos} := p_{2\_pos} + \text{abs}\left(p_{leeward\_pos}\right) = 25.692\ psf$$

$$p_{3\_netpos} := p_{3\_pos} + \text{abs}\left(p_{leeward\_pos}\right) = 27.29\ psf$$

Parapet wall

windward

$$p_{p\_windward} := q_p \cdot GC_{pn\_windward} = 28.536\ psf$$

$$p_{p\_leeward} := q_p \cdot GC_{pn\_leeward} = -19.024\ psf$$

$$p_{p\_net} := p_{p\_windward} + \text{abs}\left(p_{p\_leeward}\right) = 47.56\ psf$$

Historic Rehabilitation
Structural Calculations

WOOD

Wood Columns

Tributary Area

$$A_t := 2 \cdot \left(\frac{1}{2} \cdot (11\ ft) \cdot \frac{1}{2} \cdot (12\ ft)\right) + 2 \cdot \left(\frac{1}{2} \cdot (13\ ft) \cdot \frac{1}{2} \cdot (12\ ft)\right) = 144\ ft^2$$

Calculate column forces

$P_{dead} := 12.2\ psf \cdot A_t = 1.757\ kip$ $\qquad P_{snow} := 0\ kip$

$P_{live} := 100\ psf \cdot A_t = 14.4\ kip$ $\qquad P_{wind} := 0\ kip$

ASD Load Combinations (use for wood)

1. D
2. $D + L$
3. $D + (L_r \text{ or } S \text{ or } R)$
4. $D + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$
5. $D + (0.6W)$
6. $D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } S \text{ or } R)$
7. $0.6D + 0.6W$

$P_1 := P_{dead} + P_{live} = 16.157\ kip$ (max load on wooden column)

Assume using No.2 DFL 6x6 Timber

DOUGLAS FIR-LARCH										
Dense Select Structural	Beams and Stringers	1,900	1,100	170	730	1,300	1,700,000	620,000	0.50	WCLIB
Select Structural		1,600	950	170	625	1,100	1,600,000	580,000		
Dense No. 1		1,550	775	170	730	1,100	1,700,000	620,000		
No. 1		1,350	675	170	625	925	1,600,000	580,000		
No. 2		875	425	170	625	600	1,300,000	470,000		
Dense Select Structural	Posts and Timbers	1,750	1,150	170	730	1,350	1,700,000	620,000		
Select Structural		1,500	1,000	170	625	1,150	1,600,000	580,000		
Dense No. 1		1,400	950	170	730	1,200	1,700,000	620,000		
No. 1		1,200	825	170	625	1,000	1,600,000	580,000		
No. 2		750	475	170	625	700	1,300,000	470,000		

Compressive strength parallel to grain $\qquad F_c := 700\ psi$

$G := 0.49$ $\qquad b := 5.5\ in$ $\qquad d := b$ $\qquad E_{min} := 470000\ psi$

$A := b \cdot d = 30.25\ in^2$ $\qquad I := 263.7\ in^4$ $\qquad E := 1300000\ psi$

Adjustment Factors

$C_D := 1$ $\quad C_F := 1$

$C_M := 1$ $\quad C_{fu} := 1$

$C_t := 1$ $\quad C_i := 1$

$C_L := 1$ $\quad C_r := 1$

Calculate Cp

$K_e := 1$ $\quad l_u := 7\ ft$

$E'_{min} := E_{min} \cdot C_i = \left(4.7 \cdot 10^5\right)\ psi$

$l_e := l_u \cdot K_e = 7\ ft$

$\frac{l_e}{d} = 15.273$ < 50 $\quad c := 0.8$

$$F_{cE} := \frac{\left(0.822 \cdot E'_{min}\right)}{\left(\frac{l_e}{d}\right)^2} = \left(1.656 \cdot 10^3\right)\ psi \qquad F_{cstar} := F_c \cdot C_D \cdot C_F \cdot C_i = 700\ psi$$

$$C_P := \frac{\left(1 + \left(\frac{F_{cE}}{F_{cstar}}\right)\right)}{2 \cdot c} - \left(\left(\frac{\left(1 + \left(\frac{F_{cE}}{F_{cstar}}\right)\right)}{2 \cdot c}\right)^2 - \frac{\left(\frac{F_{cE}}{F_{cstar}}\right)}{c}\right)^{0.5} = 0.892$$

Adjusted Design Value

$F'_c := F_c \cdot C_F \cdot C_D \cdot C_P \cdot C_i = 624.426\ psi$

$f_c := \frac{P_1}{A} = 534.109\ psi$ $\qquad DCR := \frac{f_c}{F'_c} = 0.855$

Use 6x6 No. 2 DFL wooden columns.

Wood Joists

$$w_d := 12.2\ psf \cdot 16\ in = 0.016\ \frac{kip}{ft} \qquad w_l := 100\ psf \cdot 16\ in = 0.133\ \frac{kip}{ft}$$

$$w := w_d + w_l = 0.15\ \frac{kip}{ft}$$

$$L := 13\ ft \qquad l_u := \frac{L}{2} = 6.5\ ft$$

Assume 4x8 No. 2 DFL

DOUGLAS FIR-LARCH										
Select Structural	2" & wider	1,500	1,000	180	625	1,700	1,900,000	690,000	0.50	WCLIB WWPA
No. 1 & Btr		1,200	800	180	625	1,550	1,800,000	660,000		
No. 1		1,000	675	180	625	1,500	1,700,000	620,000		
No. 2		900	575	180	625	1,350	1,600,000	580,000		
No. 3		525	325	180	625	775	1,400,000	510,000		
Stud	2" & wider	700	450	180	625	850	1,400,000	510,000		
Construction	2" - 4" wide	1,000	650	180	625	1,650	1,500,000	550,000		
Standard		575	375	180	625	1,400	1,400,000	510,000		
Utility		275	175	180	625	900	1,300,000	470,000		

$$A := 25.38\ in^2 \qquad I := 111.1\ in^4 \qquad S := 30.66\ in^3$$

$$E := 1600000\ psi \qquad E_{min} := 580000\ psi \qquad F_b := 900\ psi \qquad F_v := 180\ psi$$

Adjustment Factors

$$C_M := 1 \qquad C_F := 1.3$$
$$C_t := 1 \qquad C_{fu} := 1$$
$$C_i := 1 \qquad C_r := 1.15$$
$$C_D := 1$$

Bending

Calculate C_L

$$\frac{l_u}{7.25\ in} = 10.759$$

$$\ell_e = 1.63\ \ell_u + 3d \quad \text{where } 7 \le \ell_u/d \le 14.3$$

$$l_e := 1.63 \cdot l_u + 3 \cdot 7.25\ in = 148.89\ in$$

$$R_B := \sqrt{\frac{l_e \cdot 7.25\ in}{(3.5\ in)^2}} = 9.387$$ < 50 -> ok

$$F_{bE} := 1.2 \cdot \frac{\left(E_{min}\right)}{{R_B}^2} = \left(7.898 \cdot 10^3\right)\ psi$$

$$F_{bstar} := F_b \cdot C_D \cdot C_F \cdot C_r = \left(1.346 \cdot 10^3\right)\ psi$$

$$C_L := 1 + \frac{\left(\frac{F_{bE}}{F_{bstar}}\right)}{1.9} - \left(\left(1 + \frac{\left(\frac{F_{bE}}{F_{bstar}}\right)}{1.9}\right)^2 - \frac{\left(\frac{F_{bE}}{F_{bstar}}\right)}{0.95}\right)^{0.5} = 0.842$$

$$d := 7.25\ in \qquad b := 3.5\ in \qquad \frac{d}{b} = 2.071$$

Design Check

Bending

$$F'_b := F_{bstar} \cdot C_L = \left(1.133 \cdot 10^3\right)\ psi$$

$$f_b := \frac{2.8\ ft \cdot kip}{S} = \left(1.096 \cdot 10^3\right)\ psi$$

$$DCR := \frac{f_b}{F'_b} = 0.967$$

Shear

$$F'_v := F_v \cdot C_D = 180\ psi$$

$$f_v := \frac{3}{2} \cdot \left(\frac{0.9\ kip}{b \cdot d}\right) = 53.202\ psi$$

$$DCR := \frac{f_v}{F'_v} = 0.296$$

Deflection

$\delta_{st} := 0.5 \cdot 0.4804\ in = 0.24\ in$ $\quad < \quad$ $\frac{L}{360} = 0.433\ in$ ok

$\delta_{lt} := 0.00578\ in + 0.24\ in = 0.246\ in$

$\delta_{tot} := 1.5 \cdot \delta_{lt} + \delta_{st} = 0.609\ in$ $\quad < \quad$ $\frac{L}{240} = 0.65\ in$ ok

Use 4x8 DFL @ 16" O.C. for wooden joists.

Wood Girders

$$w_d := 12.2\ psf \cdot \left(\frac{10.5\ ft + 13\ ft}{2}\right) = 0.143\ \frac{kip}{ft} \qquad w_l := 100\ psf \cdot \left(\frac{10.5\ ft + 13\ ft}{2}\right) = 1.175\ \frac{kip}{ft}$$

$$w := w_d + w_l = 1.318\ \frac{kip}{ft}$$

$$L := 12\ ft \qquad l_u := \frac{L}{2} = 6\ ft$$

Assume 6x8 Select Structural DFL

DOUGLAS FIR-LARCH										
Dense Select Structural	Beams and Stringers	1,900	1,100	170	730	1,300	1,700,000	620,000	0.50	WCLIB
Select Structural		1,600	950	170	625	1,100	1,600,000	580,000		
Dense No. 1		1,550	775	170	730	1,100	1,700,000	620,000		
No. 1		1,350	675	170	625	925	1,600,000	580,000		
No. 2		875	425	170	625	600	1,300,000	470,000		
Dense Select Structural	Posts and Timbers	1,750	1,150	170	730	1,350	1,700,000	620,000		
Select Structural		1,500	1,000	170	625	1,150	1,600,000	580,000		
Dense No. 1		1,400	950	170	730	1,200	1,700,000	620,000		
No. 1		1,200	825	170	625	1,000	1,600,000	580,000		
No. 2		750	475	170	625	700	1,300,000	470,000		

$$A := 41.25\ in^2 \qquad I := 193.4\ in^4 \qquad S := 51.56\ in^3$$

$$E := 1600000\ psi \qquad E_{min} := 580000\ psi \qquad F_b := 1350\ psi \qquad F_v := 170\ psi$$

Adjustment Factors

$$C_M := 1 \qquad C_F := 1$$
$$C_t := 1 \qquad C_{fu} := 1$$
$$C_i := 1 \qquad C_r := 1$$
$$C_D := 1$$

Bending

Calculate C_L

$$\frac{l_u}{5.5\ in} = 13.091$$

$$\ell_e = 1.63\ \ell_u + 3d \quad \text{where } 7 \le \ell_u/d \le 14.3$$

$$l_e := 1.63 \cdot l_u + 3 \cdot 7.25\ in = 139.11\ in$$

$$R_B := \sqrt{\frac{l_e \cdot 7.25\ in}{(5.5\ in)^2}} = 5.774$$

< 50 -> ok

$$F_{bE} := 1.2 \cdot \frac{(E_{min})}{R_B^{\ 2}} = (2.088 \cdot 10^4)\ psi$$

$$F_{bstar} := F_b \cdot C_D \cdot C_F \cdot C_r = (1.35 \cdot 10^3)\ psi$$

$$C_L := 1 + \frac{\left(\frac{F_{bE}}{F_{bstar}}\right)}{1.9} - \left(\left(1 + \frac{\left(\frac{F_{bE}}{F_{bstar}}\right)}{1.9}\right)^2 - \frac{\left(\frac{F_{bE}}{F_{bstar}}\right)}{0.95}\right)^{0.5} = 0.939$$

Design Check

$$F'_b := F_{bstar} \cdot C_L = (1.267 \cdot 10^3)\ psi$$

$$f_b := \frac{5.62\ ft \cdot kip}{S} = (1.308 \cdot 10^3)\ psi$$

$$DCR := \frac{f_b}{F'_b} = 1.032$$

Shear

$$F'_v := F_v \cdot C_D = 170\ psi$$

$$f_v := \frac{3}{2} \cdot \left(\frac{2.19\ kip}{b \cdot d}\right) = 129.458\ psi$$

$$DCR := \frac{f_v}{F'_v} = 0.762$$

Deflection

$\delta_{st} := 0.5 \cdot 0.00553\ in = 0.003\ in$ < $\frac{L}{360} = 0.4\ in$ ok

$\delta_{lt} := 0.000673\ in + 0.003\ in = 0.004\ in$

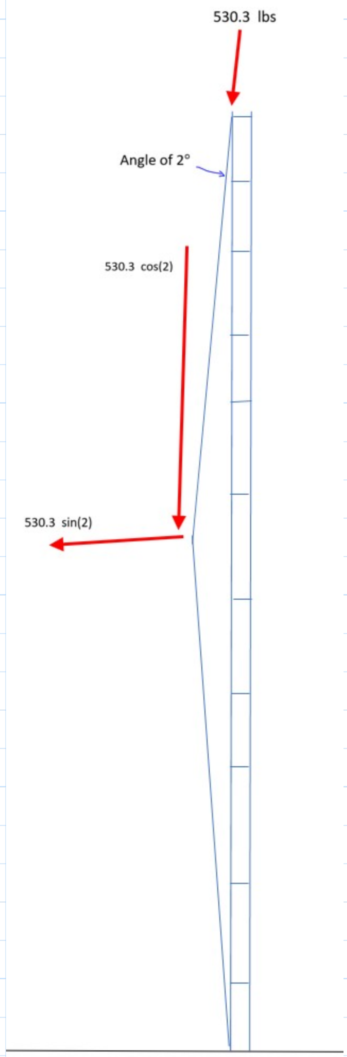
$\delta_{tot} := 1.5 \cdot \delta_{lt} + \delta_{st} = 0.008\ in$ < $\frac{L}{240} = 0.6\ in$ ok

Use 6x8 Select Structural DFL

East Wall (facade)

Veneer Wall Force Calculations

Brick weigh 33 psf
Wall area is above the max bow out is 34 ft X 12 ft = 408 ft^2



liner (horizontal) force of the brick is

$$33\ \frac{lbs}{ft^2}\ X\ 12\ ft \quad := 398.97\ \frac{lbs}{ft}$$

For each 16" wide section of the wall

$$398.97\ \frac{lbs}{ft}\left(\frac{16}{12}\right) \quad := 530.3 \quad lbs$$

After mesureing the angle of the bow out of 2 degrees

We can calculate the horizontal componant of
the force

$$530.3\ cos(2) \quad := 18.51$$

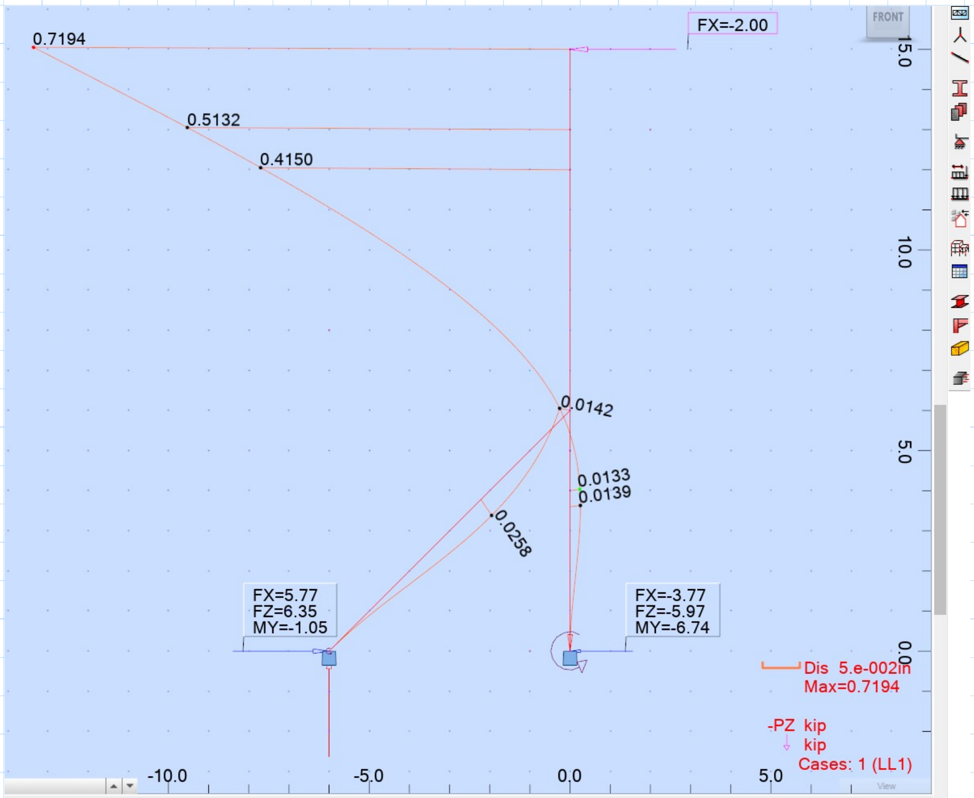
Steel Fram Calculations

1- Steel columns $15\ ft$ @ Max

Max Allowable (Δ) for AA 92 for 50 ksi

Max Allowable $(\Delta) = \dfrac{L(in)}{240}$ $\dfrac{15 \cdot 12}{240}$ $:= 0.75\ ft$

Using Robot to calculate the deflection @ Max height the diflection is 0.719 ft



Since we will be using 1 Ton hydralice jack =2000 lbs
We calculated it using the max (2000) at the top of the 15 ft W 8 x 18 column

since the calculated deflection is less than the allowable then the I-beam will work

Calculations of the hydralice jack force on the C- Channel

Since the max deflection at the middle of the channel

We can use the max force 2000 lbs

As you can see the Robot calculation for the deflection of 0.0058 ft