Carter G. Woodson School Rehabilitation Design

Undergraduate Engineering and Preservation
Ali Almutairi, Frances Boyd, Gracie Fink, Justin Hoppe
Tullahassee, OK | Carter G. Woodson School

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1. Team Information

1.1 Team Name

Our team name is Undergraduate Engineering & Preservation (U.E.P.). The team name focuses on who we are and the type of work we provide.

1.2 Team Members

Ali Almutairi is a senior at Oklahoma State University in civil engineering and graduating in May of 2022. He will be working for Kuwait Ministry of Electricity and Water. His previous experience as an undergraduate student are in steel design, structural analysis and concrete.

Frances Boyd is a senior civil engineering student at Oklahoma State University. After graduation, she will begin her career working remotely for Olsson, Inc. She has five years of work experience from two previous internships - one as a general civil intern and one as a landscape architect technician. Both internships have contributed to proficiency in AutoCAD Civil 3D, grading design, and other general land development design.

Gracie Fink is a senior at Oklahoma State University. After graduation she will start her career working for POWER Engineers in their Fort Worth office. She completed two internships with POWER Engineers where she helped design multiple overhead transmission lines. She is proficient in PLS-CADD and has professional experience working in a collaborative team setting.

Justin Hoppe is a senior in civil engineering at Oklahoma State University graduating in May of 2022, and will be working with Tanner Consulting in Tulsa. He has previous experience as an undergraduate researcher working with steel connections, which has given more insight into structural design. On top of structural design, he has other interests in geotechnics. Some of the skills he has are being concise while thinking and not stopping until the task is done.

1.3 Team Contact Information

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Gracie Fink
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1.4 Vision and Management Philosophy

Our team operated in a manner that was inclusive and encouraging to all team members. Through continuous communication with the Oklahoma State history department, we have provided our client with an optimal design while maintaining the historical importance of the site. As a team, we were open to all ideas and supported one another in achieving deadlines throughout the project. All team members filled their designated role while also contributing to brainstorming and technical work. In preparation for the team being affected by the ongoing pandemic, we established a system to create a video call with any team member who was unable to attend class in person to keep all members informed and involved in the daily work taking place.

2. Project Proposal

2.1 Project Description

The Carter G. Woodson school in Tullahassee, Oklahoma is a historical building with a cultural significance to the community. After an unfortunate fire in 2012, a large portion of the school was burned to a point beyond immediate repair. Despite the damage, the school has potential for preservation or even restoration on the remaining structure to protect the significance of the building to the community.

2.2 Team Project Proposal

The proposal focused on preserving the historical aspects of the remaining structure of the Carter G. Woodson school in Tullahassee, Oklahoma. The team met with history faculty member Dr. Laura Arata and the Mayor of Tullahassee, Ms. Keisha Currin, to get a preliminary understanding of the project design goals and constraints. A variety of ideas were discussed with the client during the meeting. After considering benefit vs. cost, the team put together four alternatives varying in levels of design and cost. Each of the alternatives included a base steel structure and some restoration of the existing structure. The client was able to review and compare the four alternatives to choose the one that best fit her needs. The direction of this project has ultimately been up to the client and the community of Tullahassee in collaboration with the OSU History Department.

2.3 Client Contact Information

Keisha Currin | Mayor
Email: tullahasseeok@gmail.com

Dr. Laura Arata
Email: larata@okstate.edu

3. Applicable Codes & Standards

- **Wagoner County Building Permit Requirements**
  - **https://www.ok.gov/wagonercounty/documents/Building%20Permit%20Requirements.pdf**
- **Oklahoma Historic Preservation Standards and Guidelines**
  - **https://omes.ok.gov/sites/g/files/gmc316/f/HPStandardsGuidelines.pdf**
- **American Society of Civil Engineers 7-16 (ASCE)**
  - **https://ascelibrary.org/doi/book/10.1061/9780784414248**
- **International Building Code (IBC)**
  - **https://www.ok.gov/oubcc/documents/2021%2009%2014%20IBC%202018%20Permanent%20Rule.pdf**
- **International Existing Building Code (IEBC)**
  - **https://www.ok.gov/oubcc/documents/2021%2009%2014%20IEBC%202018%20Permanent%20Rule.pdf**
- **International Fire Code (IFC)**
  - **https://www.ok.gov/oubcc/documents/2021%2009%2014%20Permanent%20Rule%20IFC%202018.pdf**
- **Americans with Disabilities Act (ADA)**
  - **https://www.ok.gov/odc/documents/SmallTownADA.pd**

4. Project Constraints

After meeting with the client and visiting the project site, six constraints for the project design were identified and have been detailed below. The constraints kept the project within its physical means while still considering the desired results of the community.

A. Existing structure must remain intact as requested by the client to maintain exterior appearance. The final design of the exterior should look as close to possible as the original exterior of the structure. Residents hope to pass the school on to future generations and would like to see the school as they once did when they were children.

B. Provide protection for the existing structure by using a steel structure to support the existing walls. The steel structure will be designed from the interior of the school to support the roof load without depending on the existing school walls. Furthermore, the design of this steel structure will meet all codes and standards.

C. Develop a range of projects that meet different budget requirements, and offer the client several alternatives at different cost levels to fit the best option for their budget. The remains of the school have been reviewed by an engineer recently who concluded that the existing walls are sound; therefore, each alternative includes the existing structure as a base.

D. Design window frames to fit inside existing walls. Each of the window openings in the sandstone walls have differing dimensions and no support on the top. The team must determine the most effective way to place windows in the existing openings without affecting the existing structure and ensuring the roof has proper support. See Appendix 13.4 for a model of the window openings.

E. Use existing utility connections. The original school had utilities leading to the kitchen and restrooms. The location of the existing utilities shall be utilized for all alternatives in the design process. This limited the variety of the alternative designs.

F. Design structure with consideration of community input.

5. Summary of Data Gathered and Analyzed

Throughout the design process, information was gathered pertaining to the remaining existing structure and its geographical location. All related information has been compiled below, including a floor map study, soil report, topography report, building dimensions, and the overall condition of the remaining structure.

5.1 Original Building Condition

During the February group site visit to Tullahassee, the existing walls were determined to be stable. The structure had a concrete slab foundation that could be seen under one-third the length of the school. The remaining length of the school was covered in ash, brush, and other debris, and at the time of the site visit, it was uncertain if the concrete foundation extended to the perimeter of the building. After further research and communication with the mayor, it was determined that the foundation supported all remaining structures within the building and was in good condition. A Tullahassee community clean-up was planned to clear the foundation to get a better idea of the existing conditions. There were no remaining floors. See pictures below for reference.

Figure 1 - Interior view of the west wall within the north section of the building.

Figure 2 - Interior view of the east wall within the north section of the building.

Figure 3 - Interior view of existing south section of building, facing south.

5.2 Flood Map

The FEMA flood map for the site shows that the school is located in Zone X - an area of minimal flood hazard. There will be no concern for flooding in the design process. See Figure 4 below for the FEMA flood map.

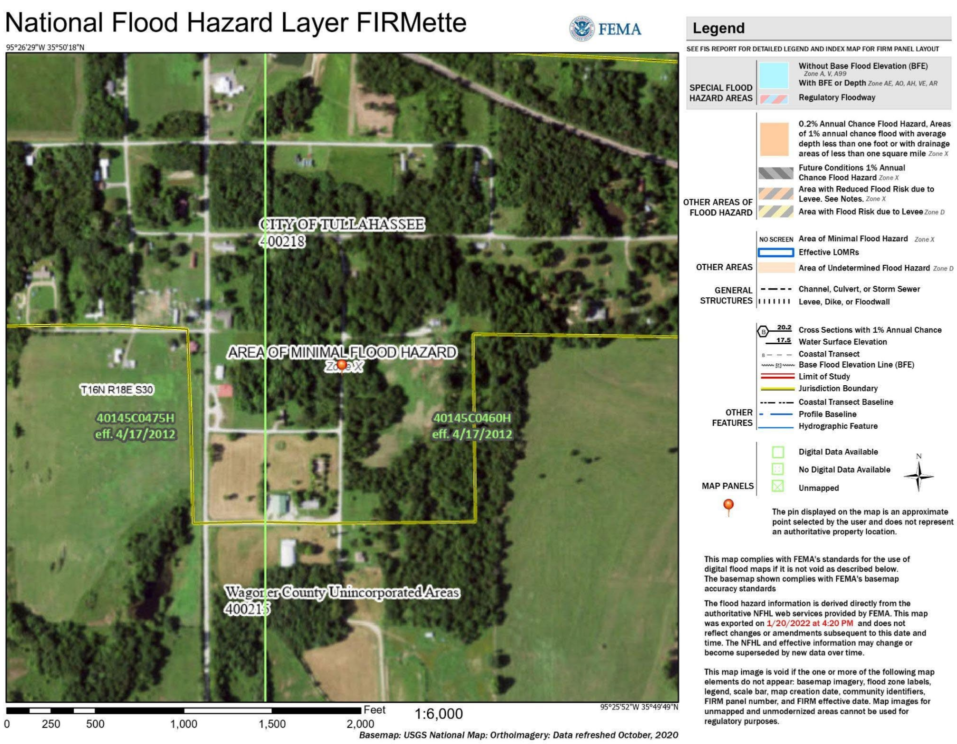


Figure 4 - FEMA National Flood Hazard Layer FIRMette

5.3 Soil Map

The soil map for the site is made up of two-thirds Okay Loam (1-3 percent slopes) and one-third Taloka Silt Loam (1-3 percent slopes). On the USDA web soil survey map, the Okay Loam is labeled OaB and the Taloka Silt Loam is labeled TaB. There are no foreseeable issues with the existing soil conditions. See Figure 5 below for the USDA web soil survey map, and refer to Table 1 for existing soil types.

Figure 5 - Soil map

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
OaB	Okay loam, 1 to 3 percent slopes	0.4	65.7%
TaB	Taloka silt loam, 1 to 3 percent slopes	0.2	34.3%
Totals for Area of Interest		**0.7**	**100.0%**

Table 1 - Soil map table

5.4 Topographic Map

The topographic map displays contours for the site and areas surrounding the site. The site likely drains north and north-east into one of the various branches of Big Creek. See Figure 6 below for the USGS topographic map and project location.

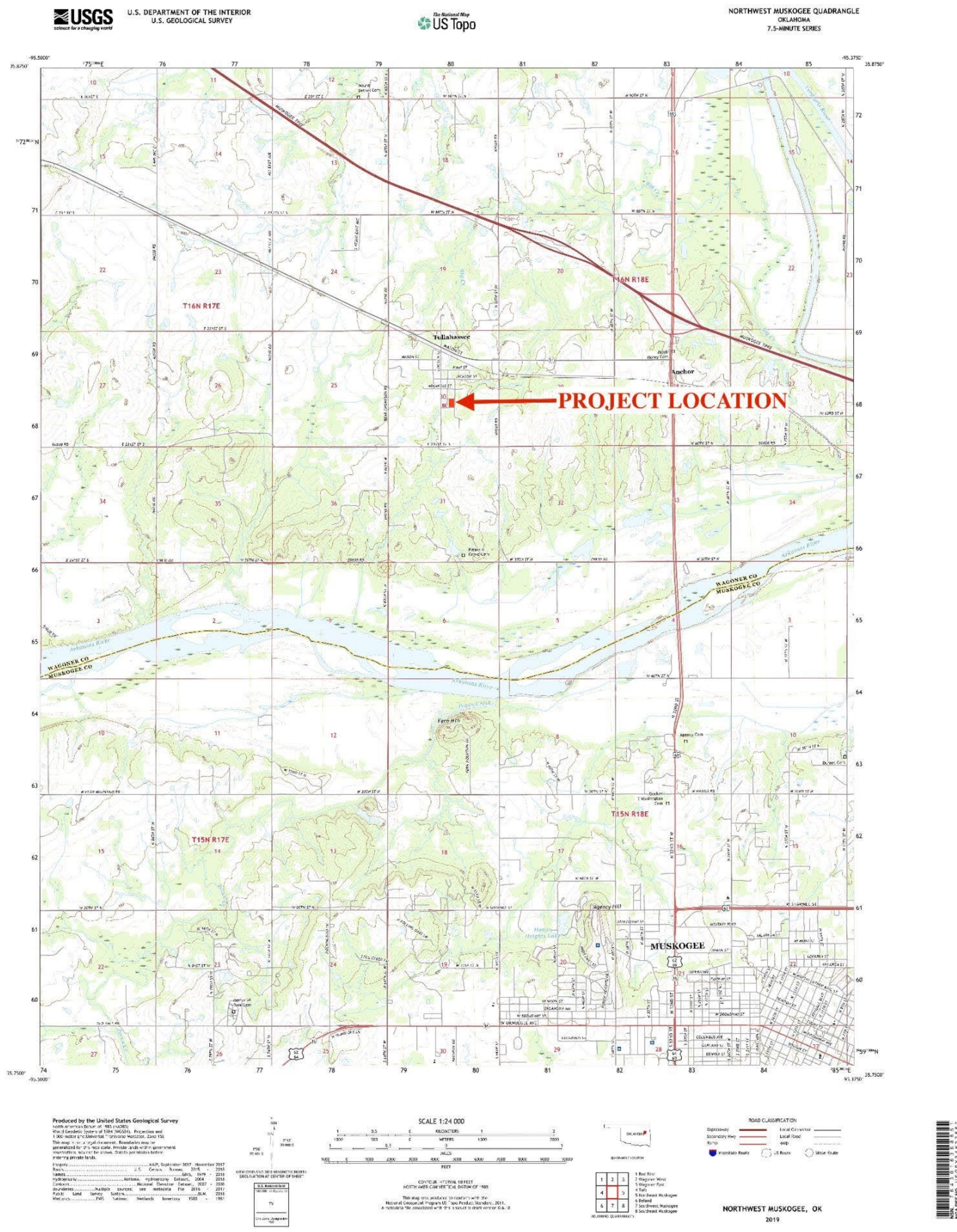


Figure 6 - Topographic Map

5.5 Building Dimensions

The building dimensions are shown in a sketch of the design model for the building. The exterior has dimensions of 160’ x 49’, but the interior is more detailed. Vertical dimensions for the building are not included because they are not accurate and were simply used to make the model. See Figure 7-9 below for detailed dimensions of the building.

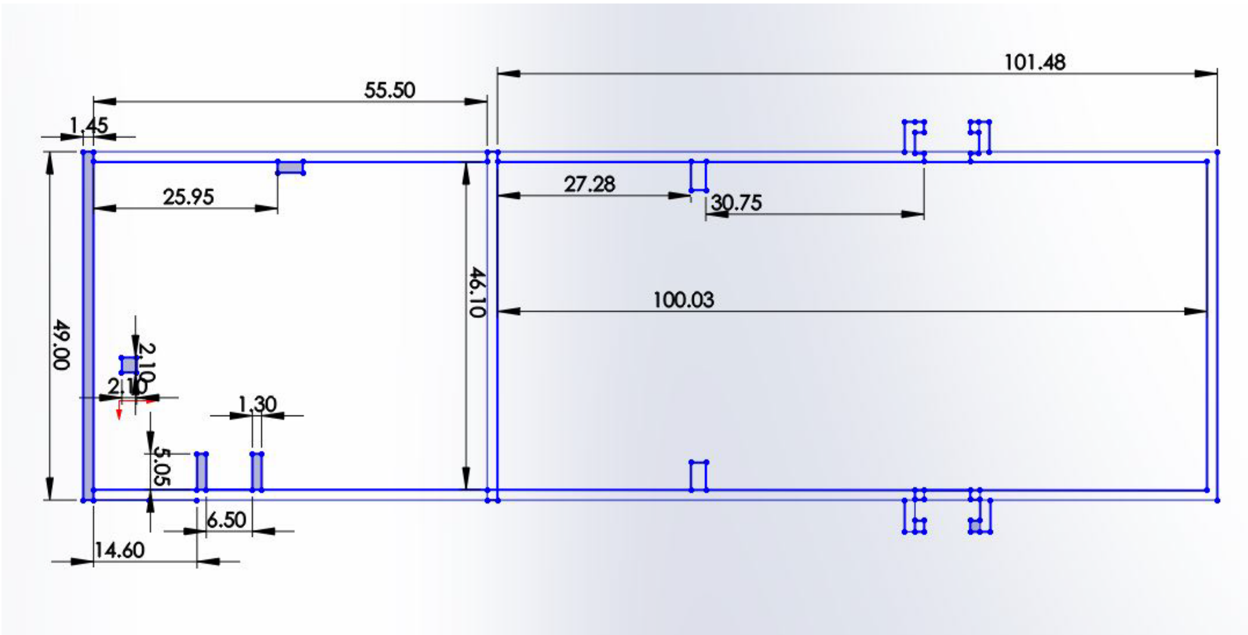


Figure 7 - Plan view of the existing building.

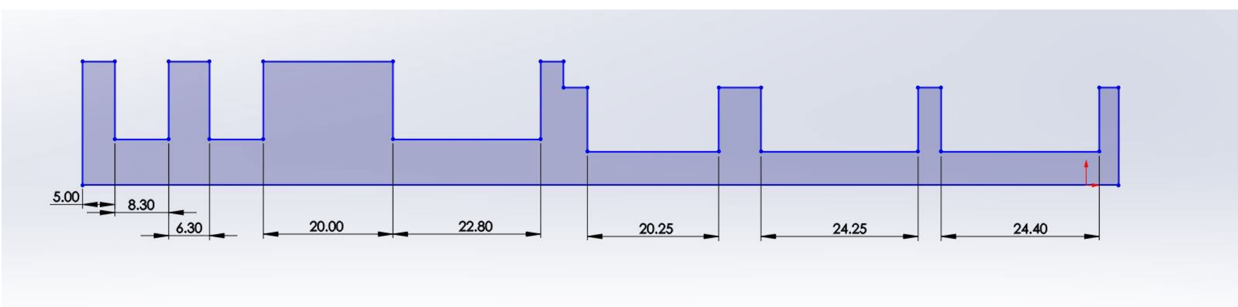


Figure 8 - Elevation view on the west side of the existing building.

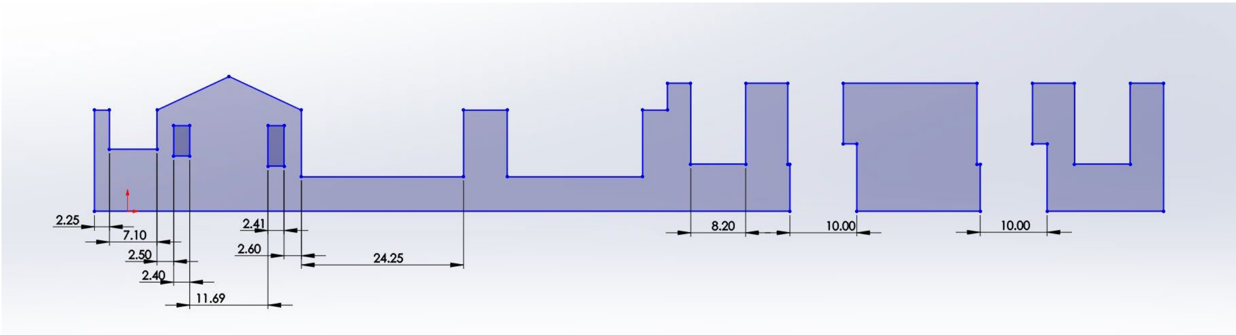


Figure 9 - Elevation view on the east side of the existing building.

6. Alternatives Analysis

To adequately serve the communities wants and needs for the final design, four alternatives were drafted and presented to the mayor. The first alternative was designed to meet the basic structural needs of the community. Each design thereafter included additional building amenities with increased costs. Each alternative and its components are detailed below in Sections 6.1-6.4. Ultimately, the first alternative was the most economical option while the fourth alternative was the least economical option. A decision matrix was created to accurately analyze and compare the alternatives. See Section 6.5 for the results of the decision matrix.

6.1 Alternative 1

Alternative 1 covered only the basic needs of the client. The client needed a functional building to be used for community events. The requirements to achieve a functional building include the design of internal support (steel structure), a roof, a kitchen, restrooms, a large open room for gathering, windows, new flooring, and sidewalks around the building to accommodate ADA standards (see Figure 10). The roof would be designed to sit on the steel structure. The kitchen and gathering room would be designed with interior walls and joists will also be designed for the floors in the southern section. The gathering room would include a history display created by the OSU History Department.

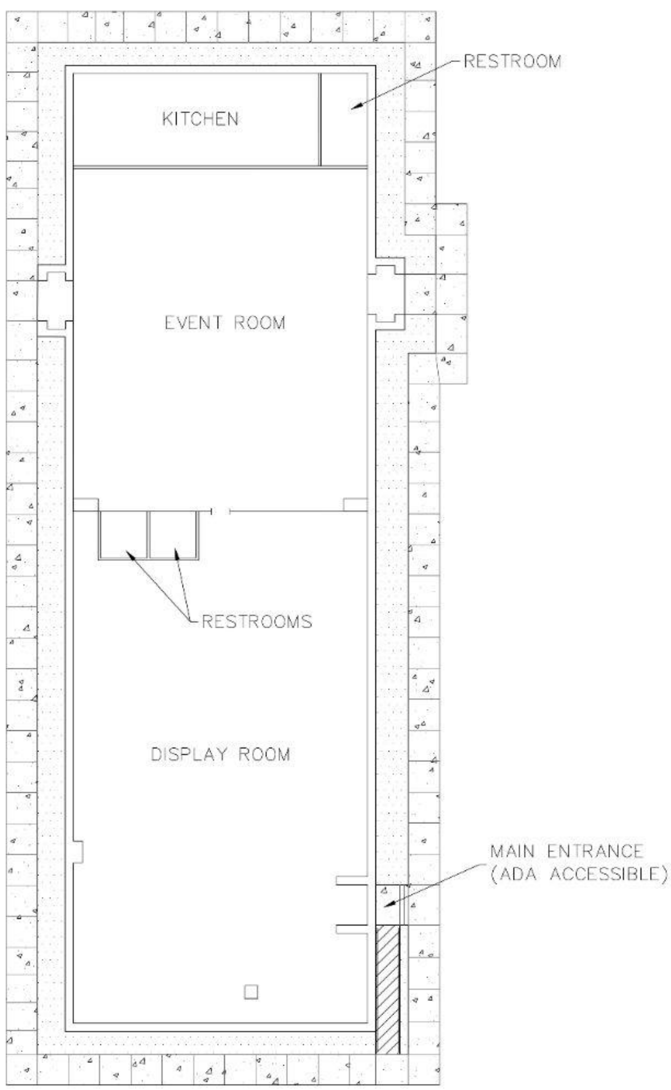


Figure 10 - Floor plan of Alternative 1.

6.2 Alternative 2

Alternative 2 included the basic needs of the client as well as supplemental wants for the project. In addition to the components listed in Alternative 1, this alternative would include a stage in the gathering room and restrooms in the southern section of the building (see Figure 11). The stage would be designed in the same location as the original building; however, the restrooms would be located directly south of the stage - not in their original location.

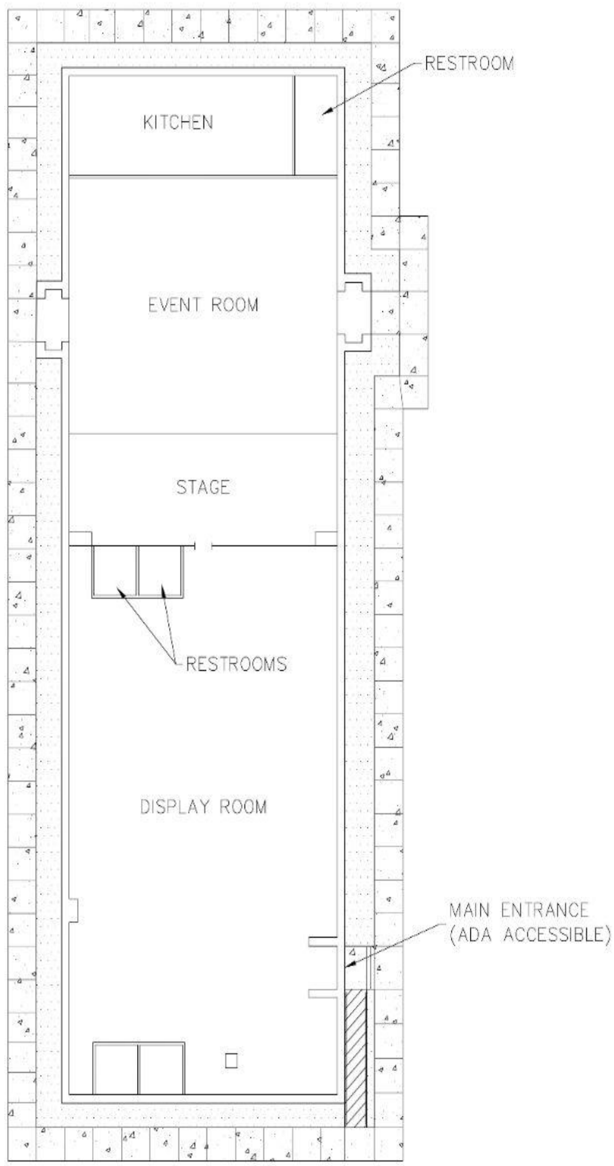


Figure 11 - Floor plan of Alternative 2.

6.3 Alternative 3

Alternative 3 built on Alternative 2 with additional wants of the client. These additional wants included an outdoor concrete pad extending from the kitchen, a fence enclosing the concrete pad, and two small classrooms behind the stage in the southern portion of the building (see Figure 12). The concrete pad would be a usable space for community events, and the fence would help keep stray dogs out while keeping children close to the building. The restrooms would be designed in the same location as the original building for this alternative.

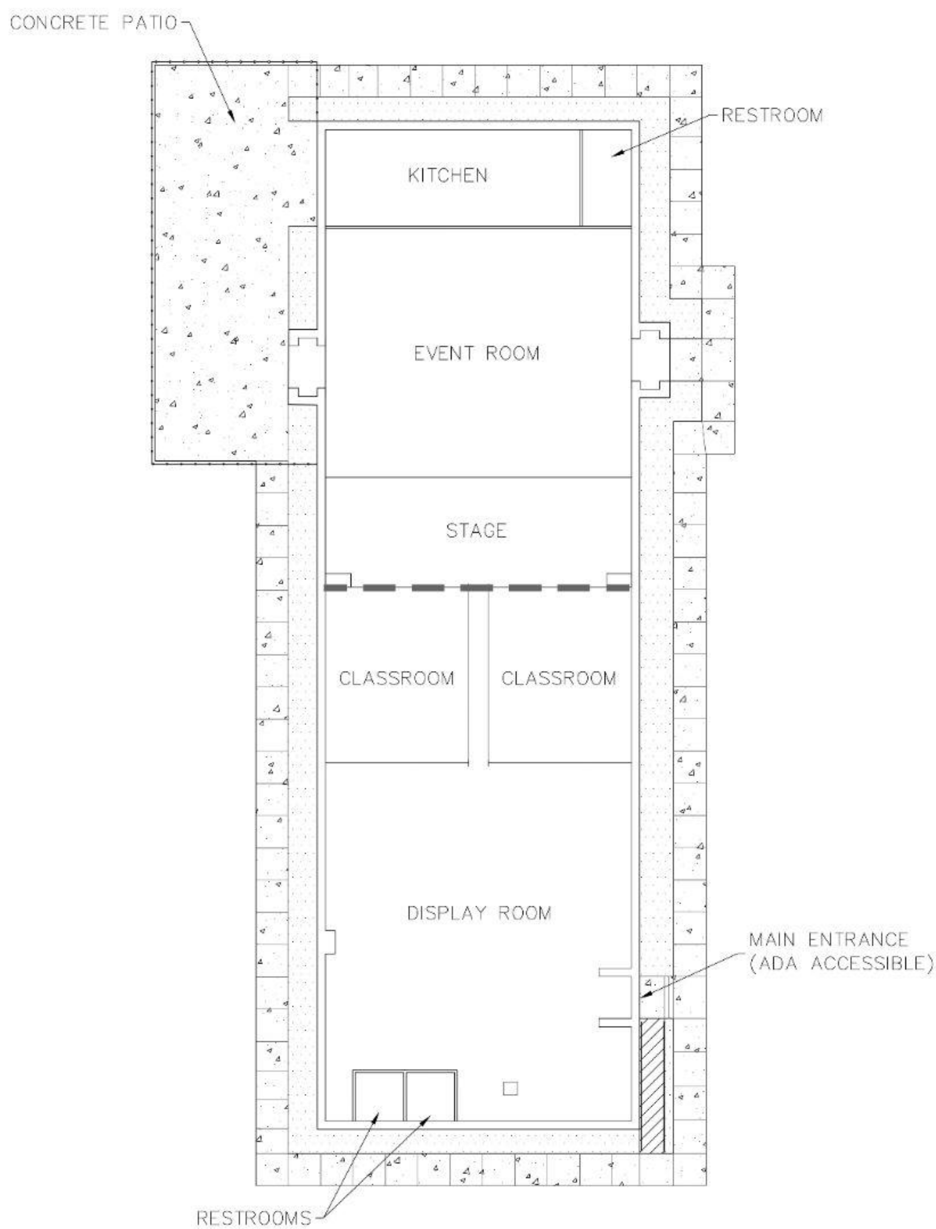


Figure 12 - Floor plan of Alternative 3.

6.4 Alternative 4

Alternative 4 built on Alternative 3 and included all the additional wants of the client. This alternative was the most expensive option. The design included an awning, an extension of the western kitchen window, and landscape design near the entrances of the school (see Figure 13). The awning would be on the west side of the building, hanging over the concrete slab. The awning would span about 35 feet across the length of the concrete patio and extend the entire width of the concrete patio (approximately 20 feet). The northwest (kitchen) window would be extended downward to include a sliding window for ease of access to the kitchen from the outside. The landscape design would mainly include small shrubs and groundcovers.

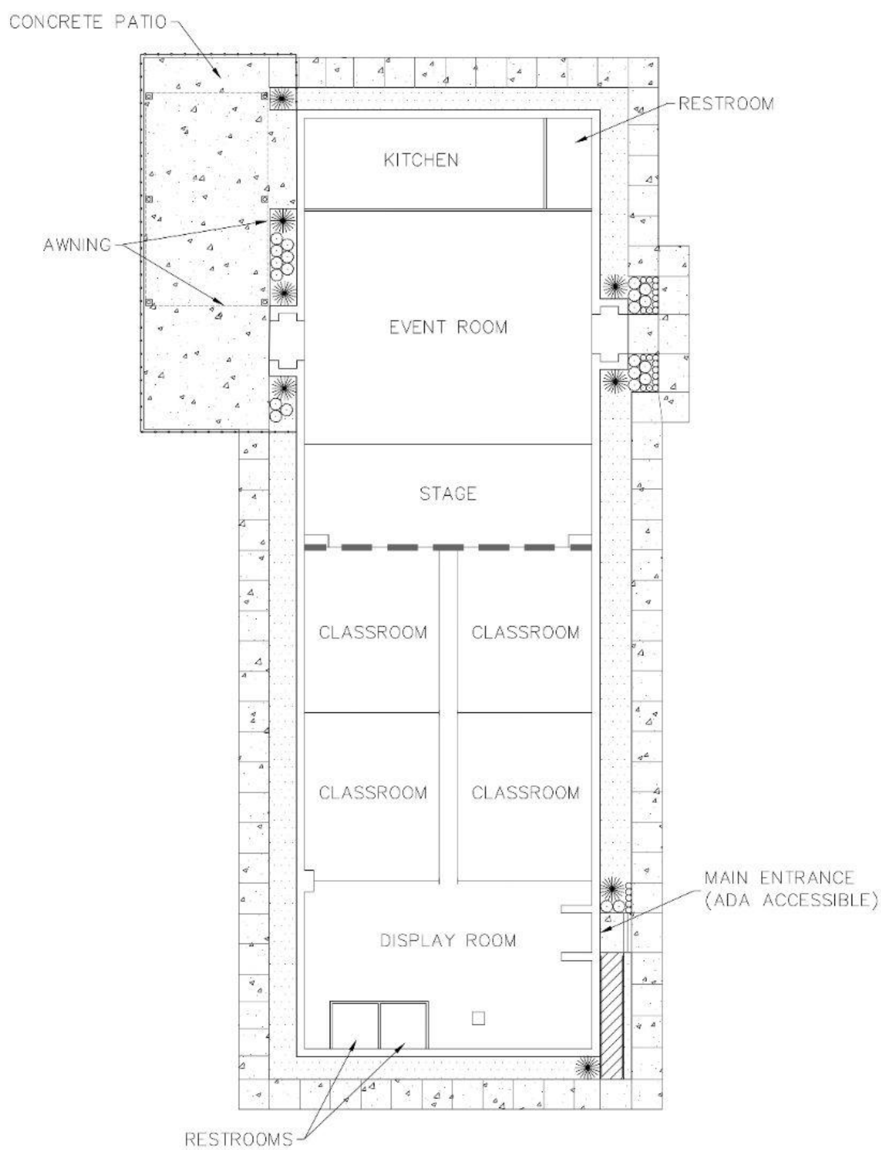


Figure 13 - Floor plan of Alternative 4.

6.5 Decision Matrix

The team created a decision matrix to weigh each alternative against each other using a list of criteria that represented the basic needs of the project. A scale from 1 to 5 was used – 5 being the best and 1 being the worst – to rank the four alternatives in each category. The decision matrix below shows the alternatives ranked by their total score, with the highest score showing the most practical alternative choice for this project. See Figure 14 to review the decision matrix scores. Alternative 1 received the highest score overall.

	Historical Importance	Constructability	Maintenance	Utility Impacts	Environmental Impacts	Affordability	Community Wants	Total
Alternative #1	4	5	5	5	5	5	1	30
Alternative #2	4	4	4	3	5	4	3	27
Alternative #3	5	4	3	5	5	3	4	29
Alternative #4	4	3	3	5	5	2	5	27

Figure 14 - Decision Matrix

7. Description of Selected Approach

The team met with the client and presented the four alternatives as well as the scores from the decision matrix. Based on the information provided, the client preferred the design of Alternative 3 and 4, but ultimately chose for the team to design Alternative 4. The chosen alternative did not have the highest score in the decision matrix due to high cost, but it would provide the client and community with a design that went above and beyond the basic components needed for the restoration of the school. Due to uncertain funding and an uncertain construction timeline, the decision to design Alternative 4 also made the most sense as the project can be broken down into individual components (steel infrastructure, concrete patio, landscape, etc.) to decide which parts would be included in the final construction plans once the budget was finalized. To preserve the historical importance of the building, the design of this alternative focused on matching the original layout of the building as closely as possible. In the final design, the bathrooms, kitchen, stage, and classrooms were placed in their respective original locations with only minor alterations or additions (see Figure 15).

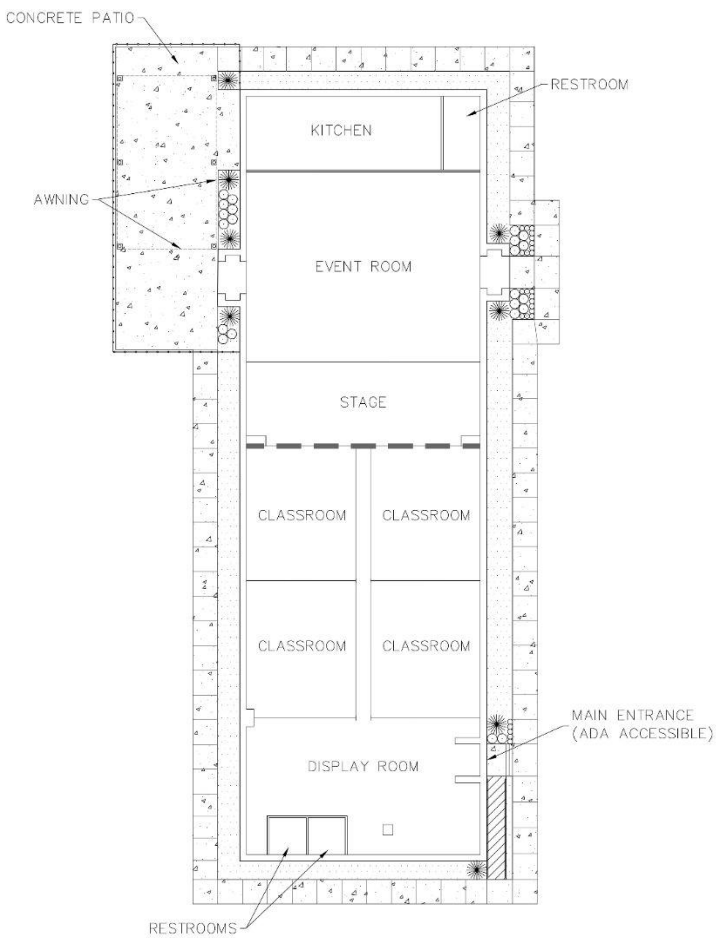


Figure 15 - Plan view of selected approach.

7.1 Structure

The original concrete foundation and stone walls remain standing on the site. In order to keep the original stone walls as the exterior of the building, the team decided to place steel structures within the frame of the north and south section of the building. The steel structures will be connected to the stone walls to prevent any shifting or movement of the walls. The frame of the steel structure was designed to be slightly taller than the original walls to ensure that the pitch of the roof would extend over the stone walls. The columns of the steel structure will be encased to prevent injury to visitors, but the rest of the steel structure will remain visible in the final design. The purpose of this is to make the stone walls visible on the interior of the building and to preserve the character of the original building. Two separate structures have been designed to meet the requirements of the existing structure. The span lengths between columns in both steel structures were heavily influenced by the original layout of the building due to the location of existing stone columns, existing window placement, etc. All steel structures have been designed in accordance with the AISC Steel Manual to ensure the integrity of the design.

7.2 ADA Accessibility

The original building was built prior to current ADA requirements, and therefore most of the existing building entrances were not ADA accessible. Each entrance was redesigned to meet ADA specifications and requirements to ensure that all persons have access to the building (see Figure 16). The final design of the building required to include proper bathroom facilities with ADA accessibility. Two restrooms in the south section have been designed in their original location, and an additional restroom has been included in the north section next to the kitchen as there is no ADA accessibility between the north and south sections of the building. With these three restrooms, there is access for all persons to utilize the facilities. Each restroom interior also meets ADA requirements to ensure ease of use (see Figure 17).

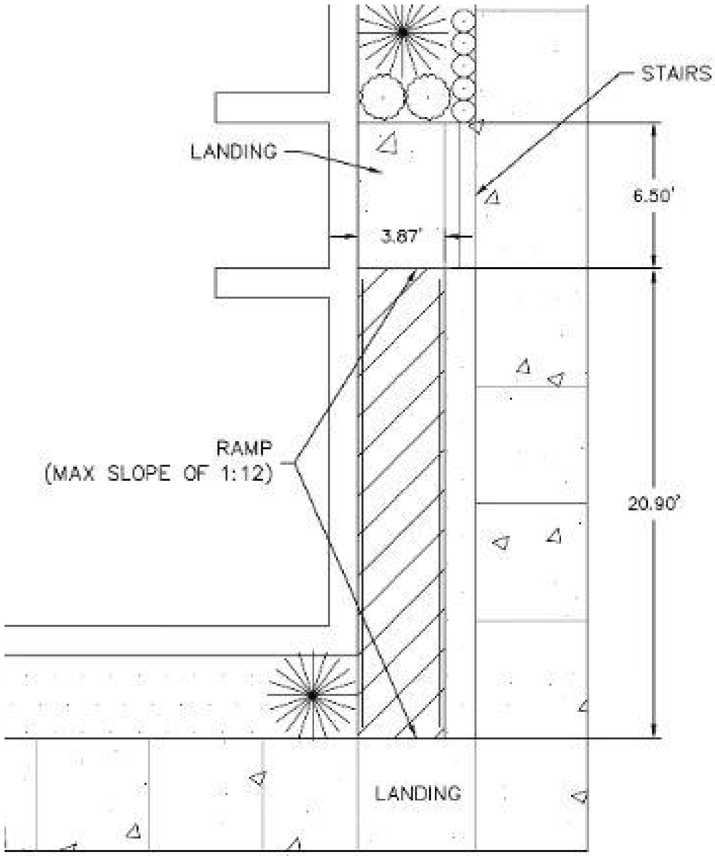


Figure 16 - Proposed ADA accessible entrance with ramp and guardrails.

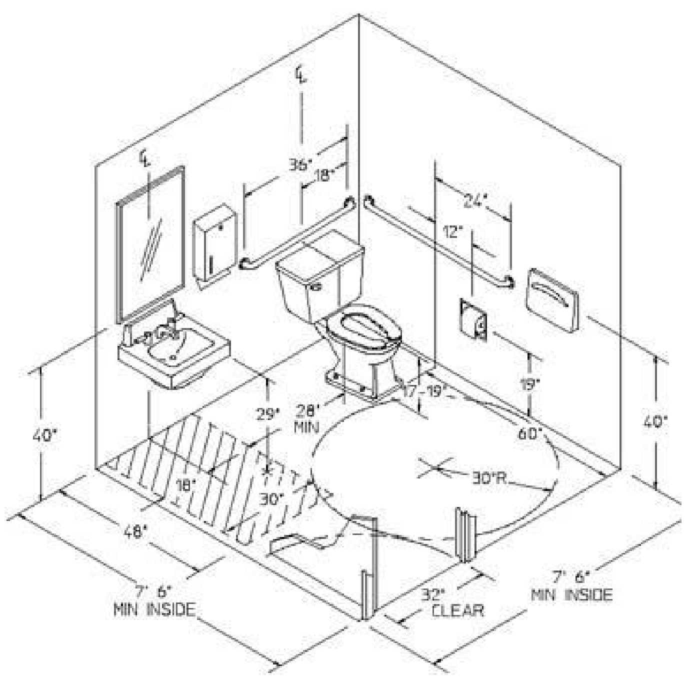


Figure 17 - Model of an ADA accessible restroom.
(Source: https://www.partitionsandstalls.com/ada-bathroom-layout.html)

7.3 Outdoor Awning and Patio

During a site visit, Carter G. Woodson alumni expressed their desire for the restored building to include an outdoor space connected to the north section of the building. To accommodate this, a concrete patio was designed on the west side of the north section (see Figure 18). An awning was designed to provide cover for the concrete patio. The awning was designed using the same steel as the interior steel frames to maintain consistency throughout the project. The client also expressed the desire to have the west kitchen window open up to the patio to allow for food to be passed directly from the kitchen to the patio. In order to include this in the final design, the existing stone surrounding the window must be cut out to make the window an accessible height. The concrete patio also extends underneath the west kitchen window to accommodate the foot traffic by this window. A chain link fence has been designed to enclose the entire patio.

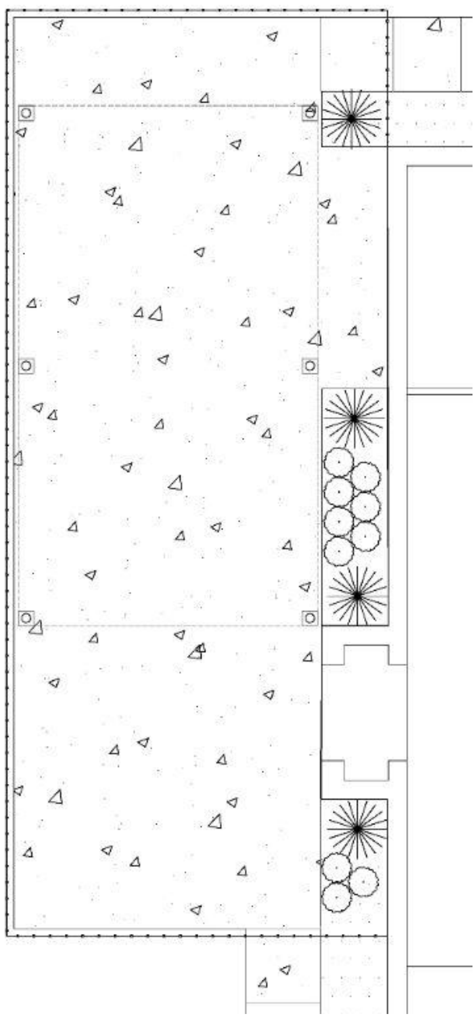


Figure 18 - Plan view of proposed concrete patio.

8. Summary of Engineering Design and Analysis

8.1 Structural

8.1.1 Design of Steel Structures

Both interior steel structures were designed within the standards of the AISC Steel Manual. The interior structures will be made with A992 Grade 50 steel. The layout of the structures were limited by the existing stone walls. All of the columns were placed to strategically avoid obstructing the building's entrances and windows. The loading on the structure was determined using the ASCE 7-22 standards and the LRFD load combinations. Once the total load of 60.89 lb/ft^2 was determined, the load was applied to the respective tributary areas of each structure. See Figure 19 below for the proposed layout of the northern steel structure.

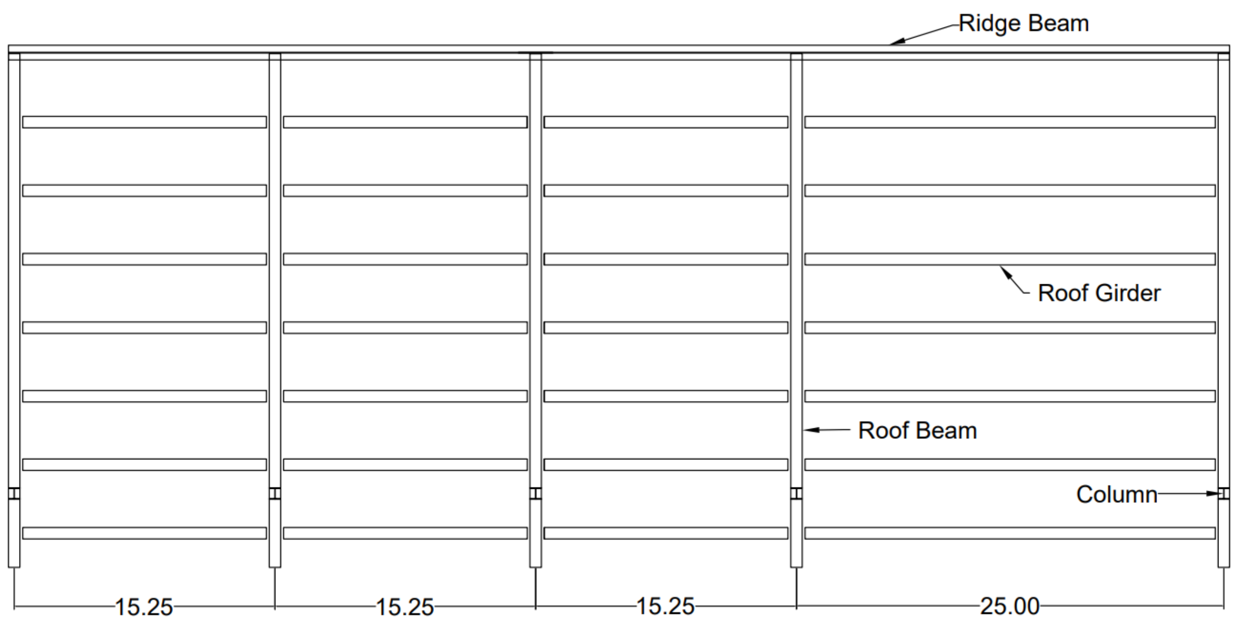


Figure 19 - Plan view of the north section roof layout.

The northern section has a tributary area of two feet on either side of the girder beams that extends the length of the beam. The irregular spacing of the roof beams due to constraints from the existing building required all the girders to be designed conservatively with the longest bay length of 25 feet. This allowed for all beams and columns connected to the girders to be identical to make for ease of construction while ensuring the safety of the design. After finding the proper distributed load on the beams, the structural analysis and design software SkyCiv was used to aid in the design process of the beams and columns in the north section. Figures 20-23 below show the structural analyses of the beams and columns modeled in the software.

Reactions

Support at	X	Y	Mx
0	0 kip	3.4881 kip	0 kip-ft
25	0 kip	3.4881 kip	0 kip-ft

Force Extremes

Result	Max	Min
Bending Moment	21.8007 kip-ft	0 kip-ft
Shear	3.4881 kip	-3.4881 kip
Displacement	0 in	-0.6659 in

Figure 20 - SkyCiv analysis of roof girder.

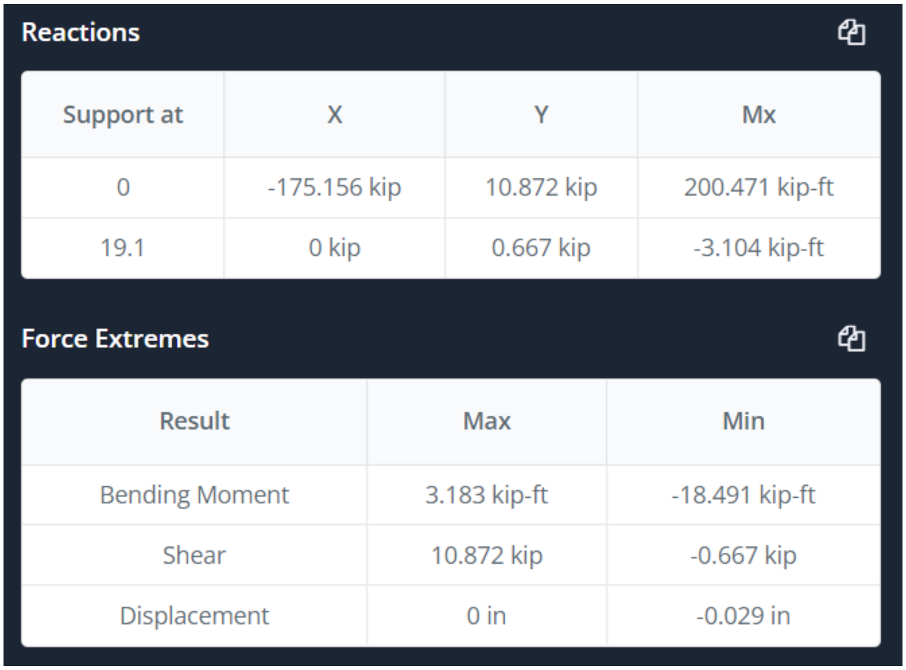
Reactions

Support at	X	Y	Mx
0	0 kip	19.644 kip	97.882 kip-ft
26.015	0 kip	30.182 kip	-87.215 kip-ft

Force Extremes

Result	Max	Min
Bending Moment	51.495 kip-ft	-101.319 kip-ft
Shear	19.644 kip	-23.078 kip
Displacement	0 in	-0.508 in

Figure 21 - SkyCiv analysis of roof beam.



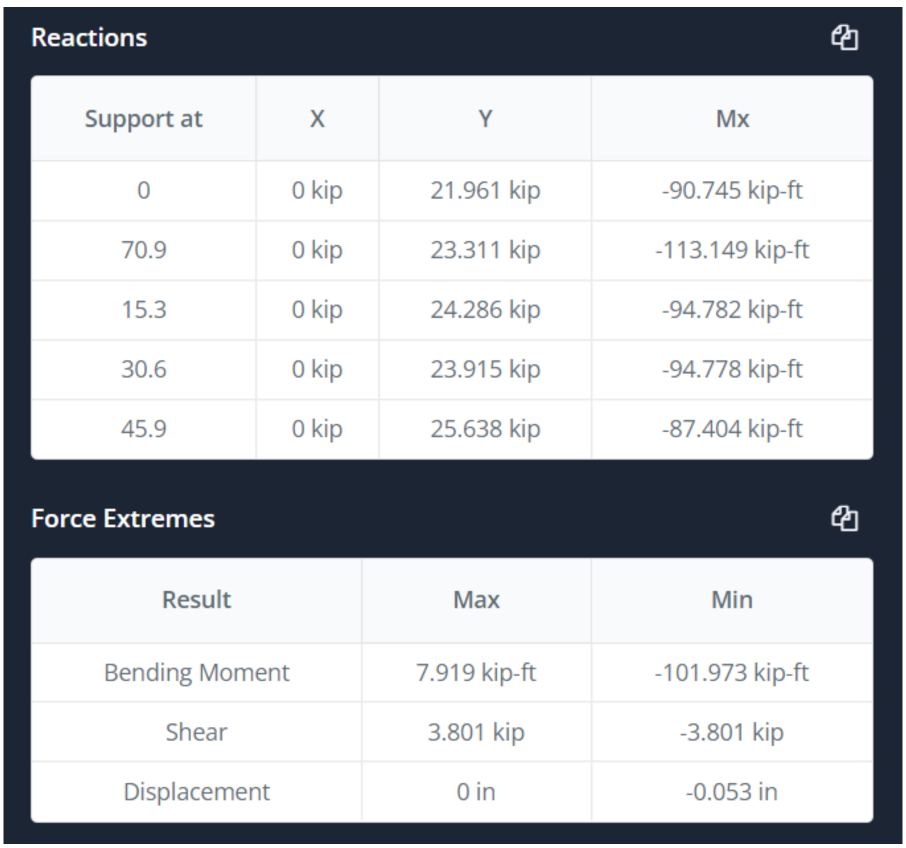
Reactions

Support at	X	Y	Mx
0	-175.156 kip	10.872 kip	200.471 kip-ft
19.1	0 kip	0.667 kip	-3.104 kip-ft

Force Extremes

Result	Max	Min
Bending Moment	3.183 kip-ft	-18.491 kip-ft
Shear	10.872 kip	-0.667 kip
Displacement	0 in	-0.029 in

Figure 22 - SkyCiv analysis of column.



Reactions

Support at	X	Y	Mx
0	0 kip	21.961 kip	-90.745 kip-ft
70.9	0 kip	23.311 kip	-113.149 kip-ft
15.3	0 kip	24.286 kip	-94.782 kip-ft
30.6	0 kip	23.915 kip	-94.778 kip-ft
45.9	0 kip	25.638 kip	-87.404 kip-ft

Force Extremes

Result	Max	Min
Bending Moment	7.919 kip-ft	-101.973 kip-ft
Shear	3.801 kip	-3.801 kip
Displacement	0 in	-0.053 in

Figure 23 - SkyCiv analysis of ridge beam.

8.1.2 Connection Specifications for North Section:

Connections for the north section were designed following the guidelines in the textbook *Unified Design of Steel Structures, 3rd Ed.* by Louis F. Geschwindner, Judy Lui, and Charles J. Carter. These guidelines follow the standards laid out in the AISC Steel Manual. Since the North section was designed conservatively to serve the loading at the largest bay, all steel members are identical at each type of connection. Therefore, only one specification is provided for each connection type. See Figures 24-27 below to see the modeled connections specifications for the north section.

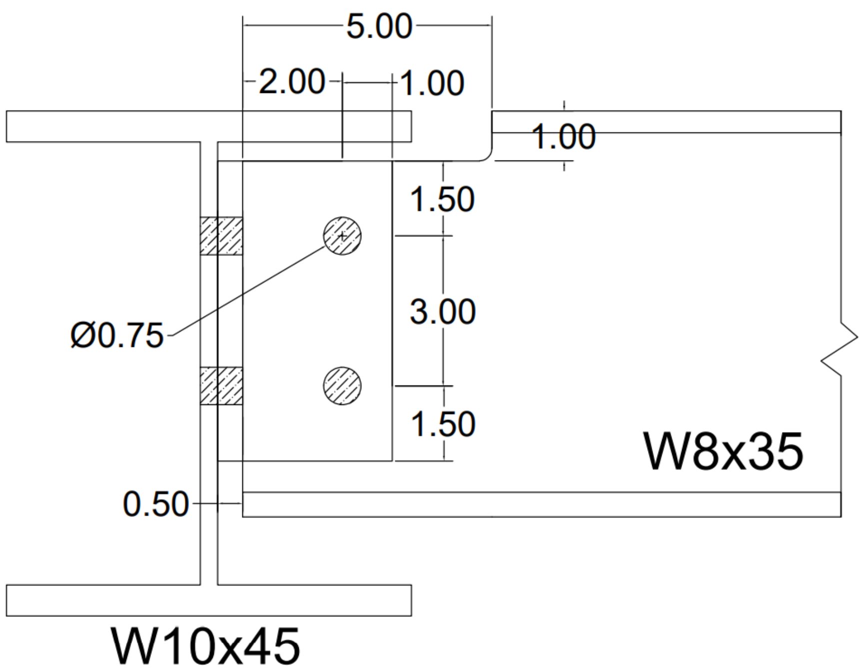


Figure 24 - Simple connection of roof girder to roof beams in the north section.

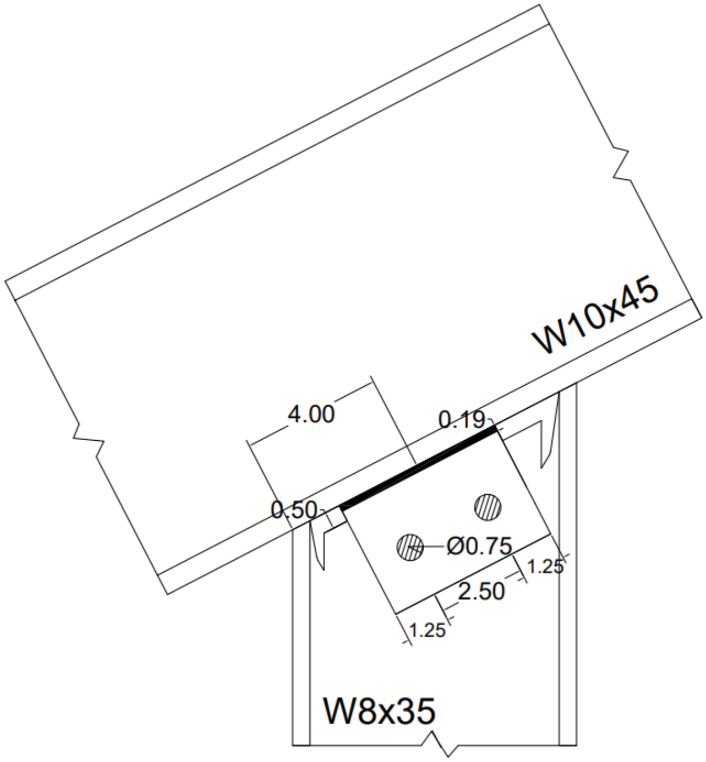


Figure 25 - Moment connection of roof beam to column in the north section.

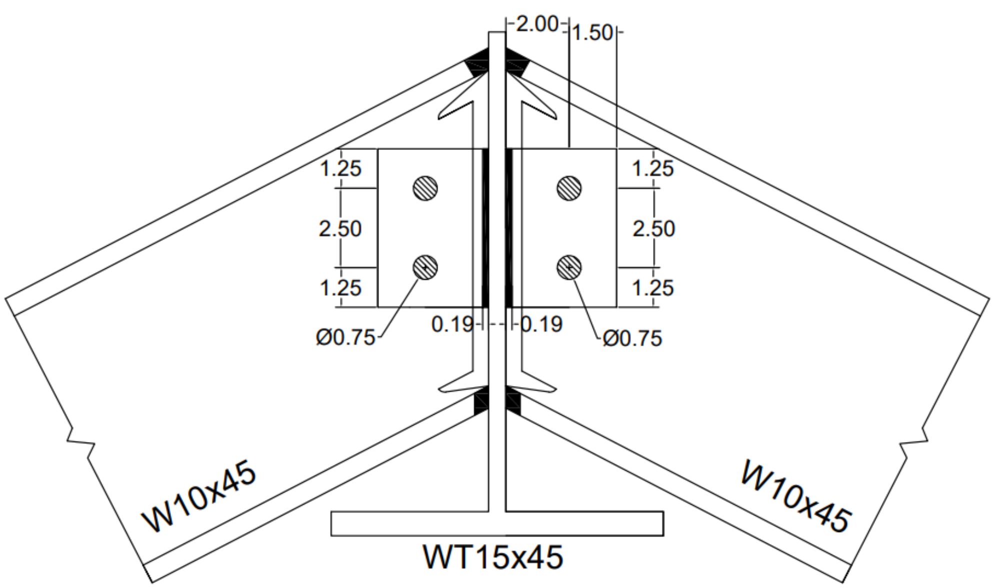


Figure 26 - Moment connection of roof beam to ridge beam in the north section.

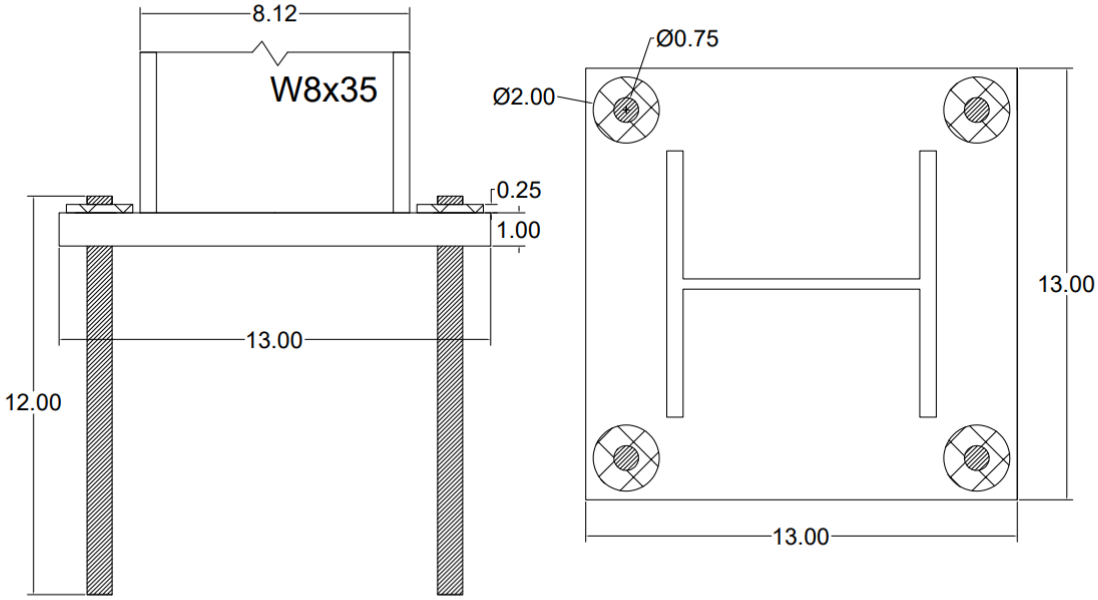


Figure 27 - Moment connection of column base plate in the north section.

8.1.3 Design of South Structure:
The south structure was designed in two separate pieces, a smaller section that is only a gable roof and a larger section that is a cross-gable roof. The smaller section was designed first because most of the sections would be able to be transferred over to the larger section. The steel type decided upon was A992 Grade 50 steel to match the northern structure and the wind load of 60.89 lb/ft² applied to the structure as well. Columns were placed to limit window blockage and to accommodate for the cross-gable roof. See Figure 28 for the proposed layout of the steel structure.

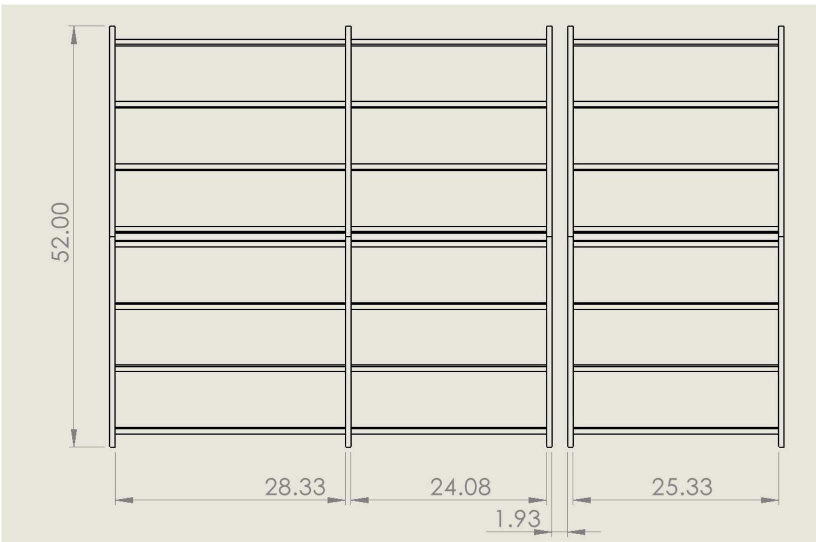


Figure 28 - South structure layout.

The beams have a tributary area of 8 feet by 25 feet and connect into the girders using a simple connection. Most of the beams and connections were identical because there was no variation in the loading. Many beams being identical allowed for most girders and columns to be identical as well. Hand calculations were done to find all of the sections as well as connections. The cross-gable was unable to be done because the design would hinder interior space and block one of the entrances. Figure 29 shows what would be the steel placement for a cross-gable section

Figure 29 - Cross-gable steel placement.

8.1.4 Connection Specifications for South Section:

Connections for the South section were designed following the guidelines in the textbook *Unified Design of Steel Structures, 3rd Ed.* by Louis F. Geschwindner, Judy Lui, and Charles J. Carter. These guidelines follow the standards laid out in the AISC Steel Manual. The column base connection was done using the AISC Base Plate and Anchor Rod Design Steel Guide 2nd Ed. The girder to girder connection is designed using the AISC Extended End-Plate Moment Connections Seismic and Wind Applications 2nd Ed. See Figures 30-33 to see the modeled connections specifications.

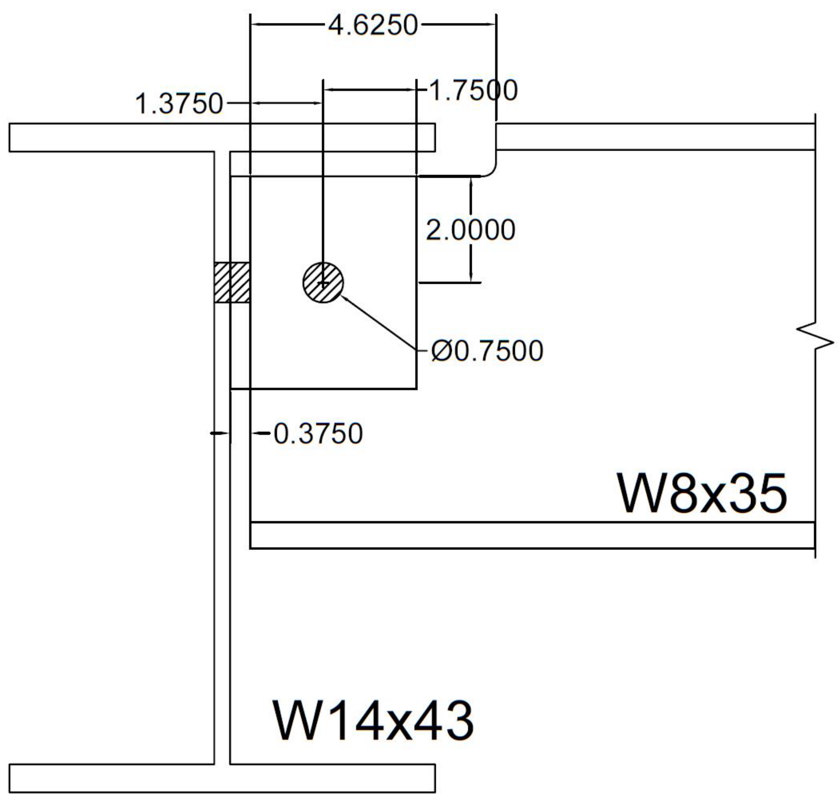


Figure 30 - Simple connection between beam and girder.

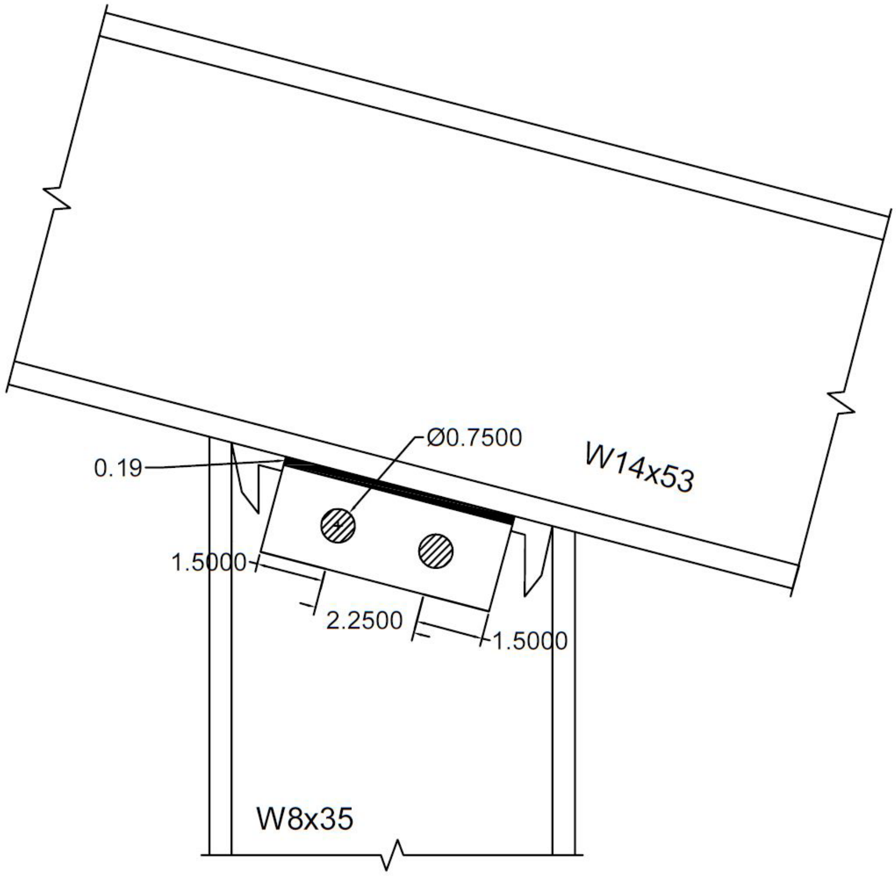


Figure 31 - Moment connection between girder and column

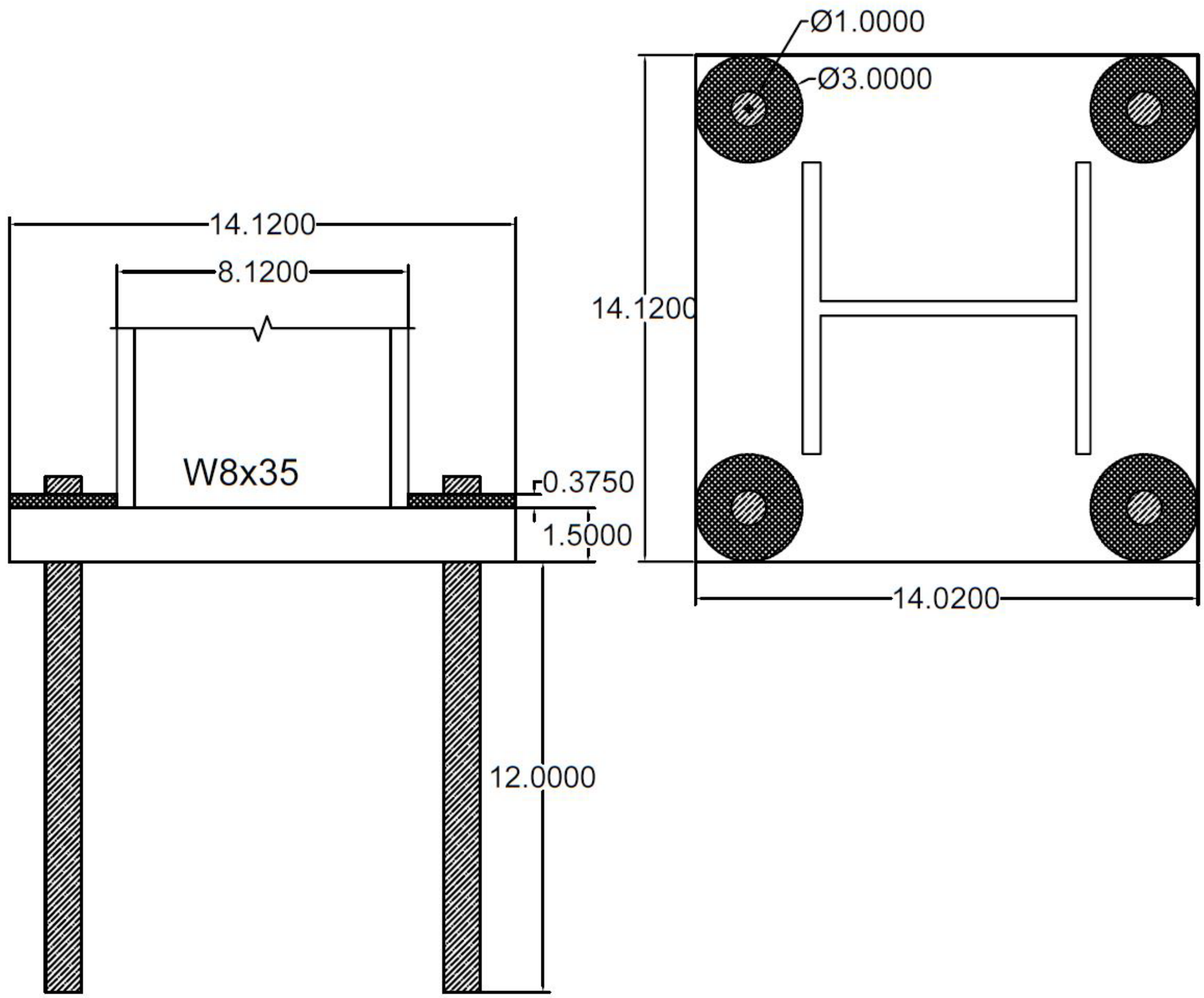
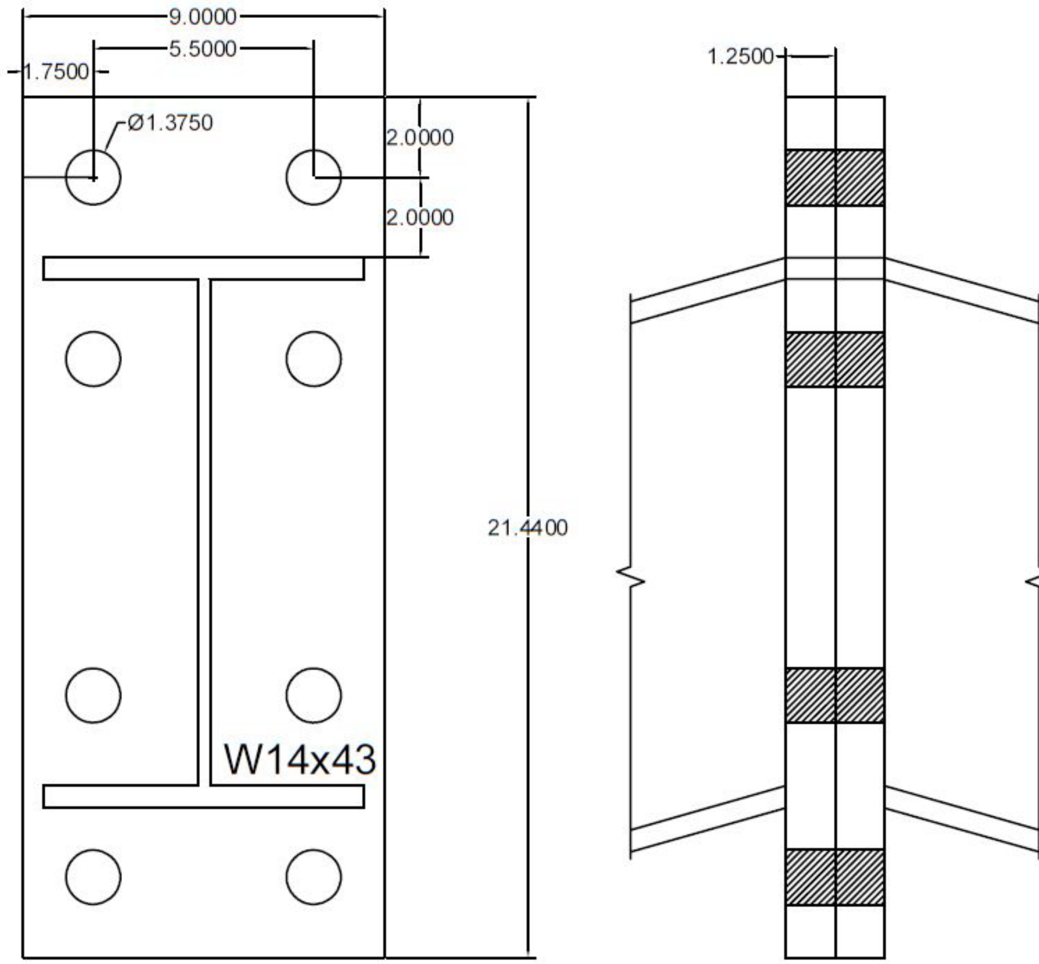


Figure 32 - Connection between column and ground



Hi

Figure 33 - Extended end plate connection between girders

8.1.5 Connection Specifications for Awning:

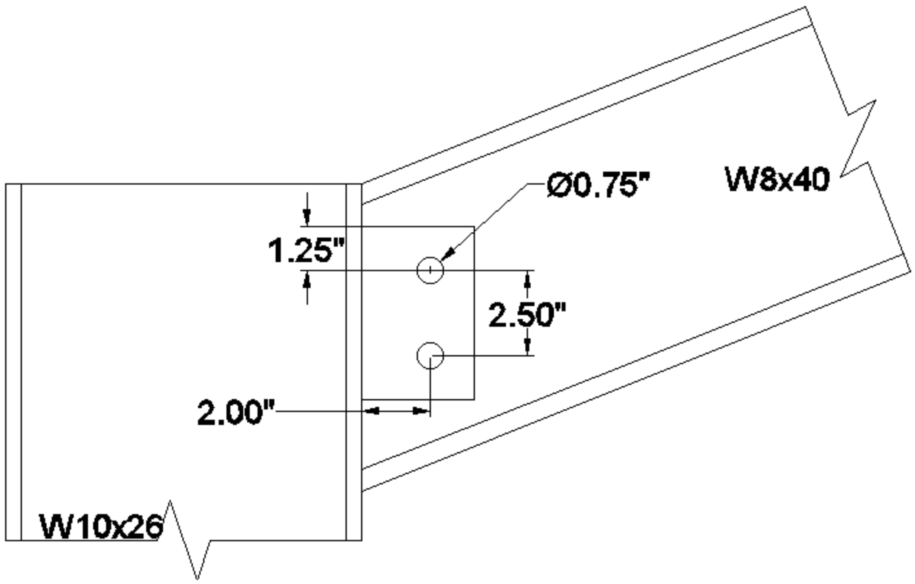


Figure 34 - Moment connection of beam to column.

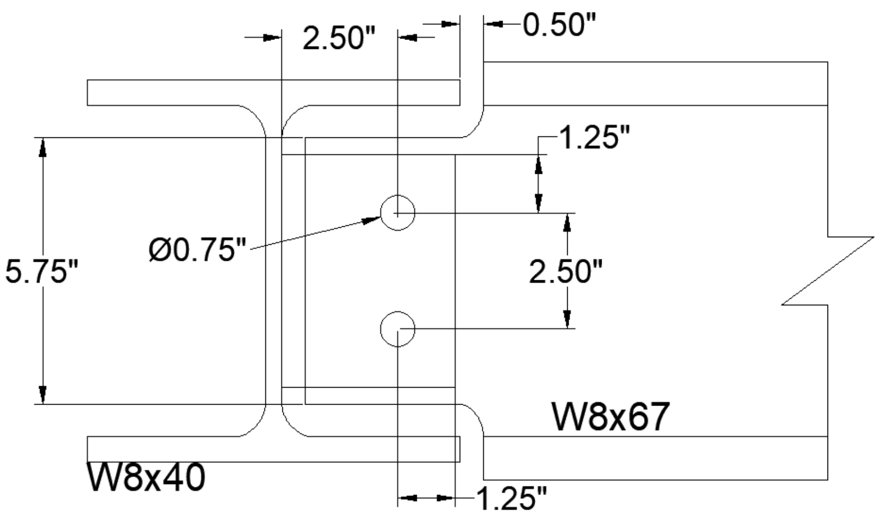


Figure 35 - Simple connection of girder to beam.

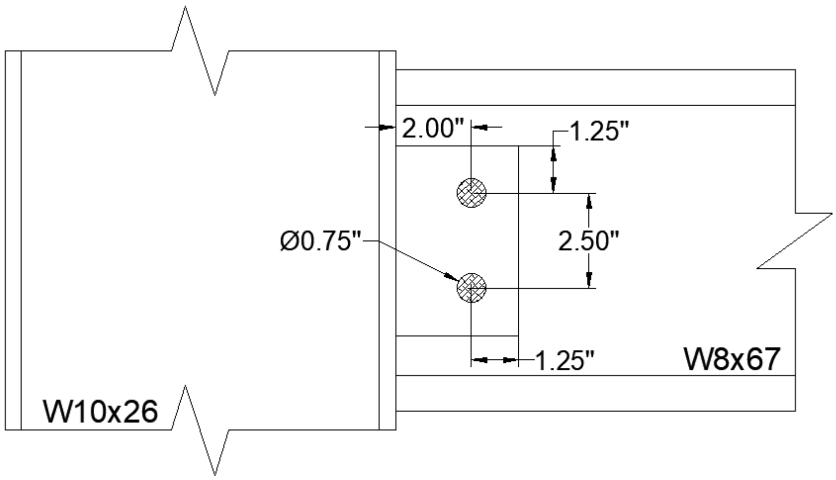


Figure 36 - Moment connection of girder to column.

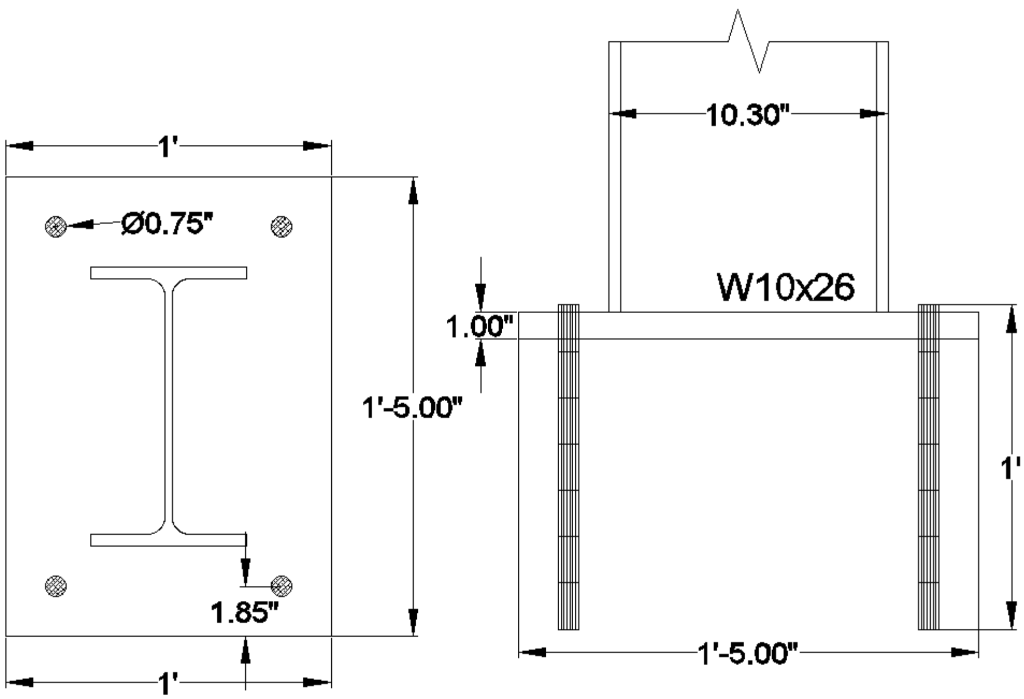


Figure 37 - Moment connection of column base plate.

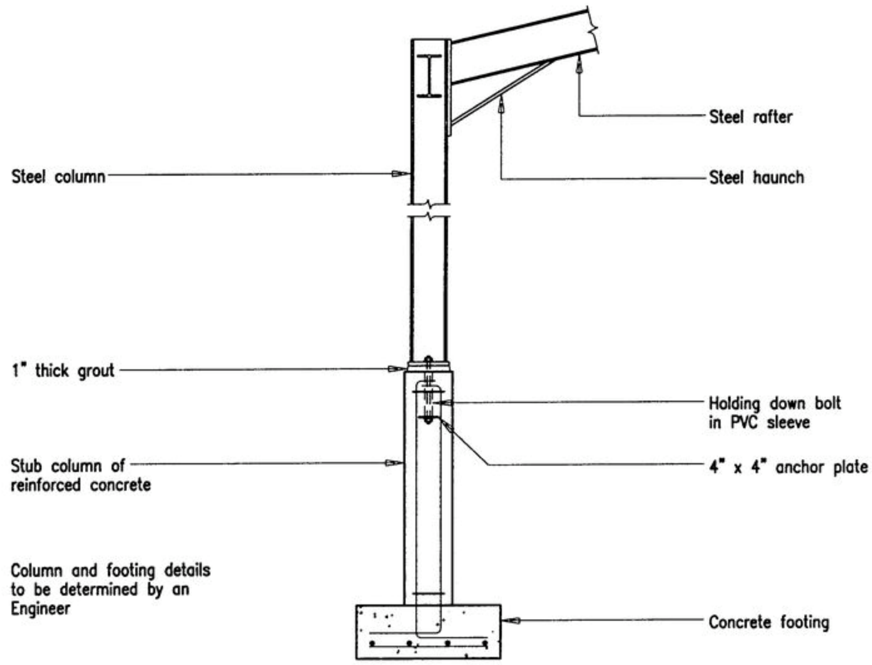


Figure 38 - Steel column connection for awning into concrete patio.
(Source: https://www.oas.org/cdmp/document/codedraw/sectiond.htm)

8.1.6 Steel Quantities

The steel beam quantities and lengths required to construct the infrastructure of the building and the awning are detailed below in Table 2. The table divides quantities into three categories: the north section, south section, and the awning. The section type(s), quantities, and lengths are provided for each type of member per category.

Steel Beams			
Member	Section	Quantity	Length
North Section			
Girders	W8x35	42	14.25'
	W8x35	14	24'
Roof Beams	W10x45	10	29.5'
Ridge Beam	WT15x45	1	70.9'
Columns	W8x35	10	19.1'
South Section			
Beams	W8x35	20	25'
	W8x35	3	8'
	W8x35	2	24'
	W8x35	2	15.75'
	W8x35	1	13.5'
	W8x35	1	17'
Girders	W14x43	10	27'
	W14x35	2	12'
Columns	W8x35	9	16'
	HSS	1	16’
Awning			
Girders	W8x67	10	17.5'
Beams	W8x40	3	20'
Columns	W10x26	3	16'
Columns	W10x26	3	8’

Table 2 - Steel beam quantities and lengths.

8.1.7 Interior Design

The interior design of the building includes walls, flooring, and a stage. The detailed design of the internal components was limited due to time constraints, but the construction will follow typical standards and specifications. Figure 39 shows the typical anatomy of an interior wall. The walls will have studs spaced 16 inches on center and will frame out all windows and doorways as specified below where it is necessary. The floor and stage will be constructed according to Figure 40. The joists will be spaced 16 inches on center and the headers will be spaced at five feet on center. The stage will be built at a higher elevation with four foot taller footings.

The building interior will include the construction of walls, flooring, and a stage. All of these will combine to create the layout (as shown in Figure 15 in Section 7). Due to limited time, the completed design of all interior elements were not able to be designed in detail, but this report has included typical specifications of each element.

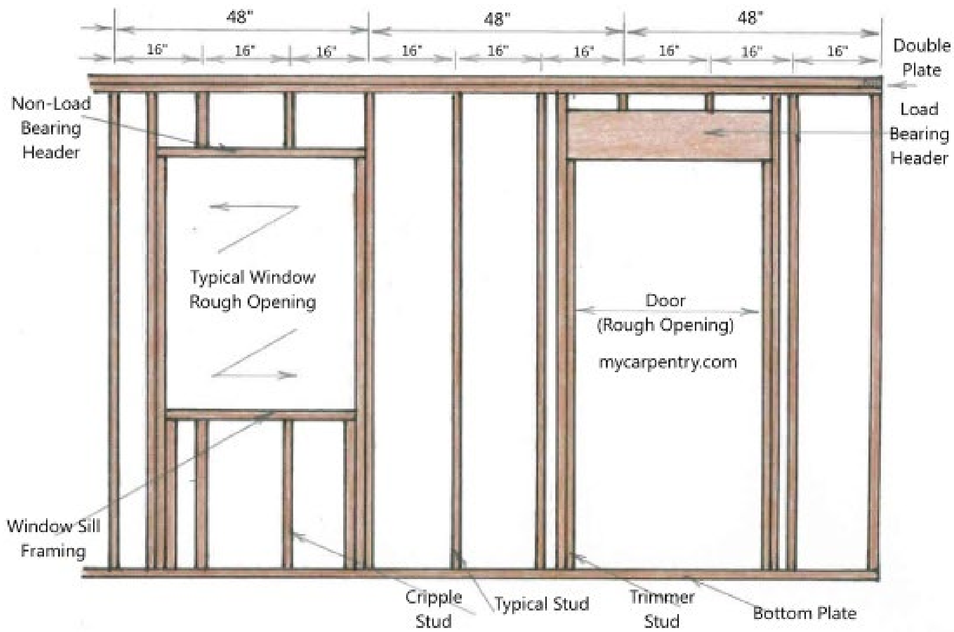


Figure 39 - Anatomy of an interior wall.
(Source: https://www.mycarpentry.com/framing-a-wall.html)

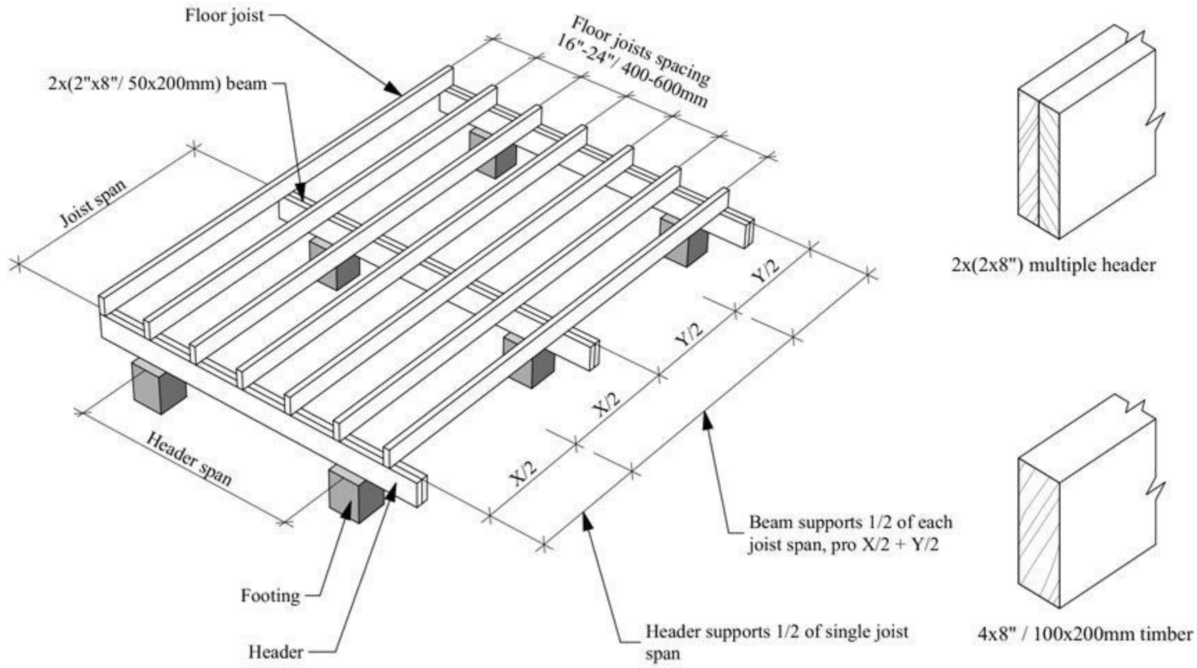


Figure 40 - A typical floor joist layout.
(Source: https://www.pinuphouses.com/floor-joist-spacing/)

8.2 Geotechnical / Foundation

The existing foundation on the project site will remain. The foundation has been inspected by an engineer within the past few years who found no issues. The only new construction will consist of concrete sidewalk around the building and an outdoor concrete patio. Construction will occur on the following two types of soil: taloka silt loam and okay loam. Both soils are generally conducive of a sturdy geotechnical foundation for construction of concrete structures. There is no area of concern in regard to the geotechnical aspects of the design.

The outdoor concrete patio was designed to comfortably fit approximately 50-75 guests. The final dimensions for the concrete patio are 61.35' by 20.55' with a small additional section near the northeast corner to connect to the kitchen window. The small section is 16.30' by 4.45'. The total square footage for the patio is about 1,333 square feet. The concrete will be reinforced with wire meshing mid-depth to hold the weight of the steel awning and the foot traffic. The reinforcement was particularly necessary due to the design decision to not provide any joints on the patio. The awning weighs approximately 15,434 pounds. The awning is 20' x 35' (700 square feet) making the total load per square foot for the concrete patio to hold is about 22 pounds per square foot. The number was rounded up to 40 pounds per square foot to account for any dead and live loads that the patio will hold. To account for 40 pounds of load per square foot, the patio was designed with a concrete thickness of five inches. The chosen type of concrete was Portland cement concrete at a strength of 3,500 psi with a layer of gravel fill to be installed underneath the layer of concrete. See Figure 41 for details on the concrete patio installation. The chain link fence that encloses the patio has a circular concrete footing that will be poured approximately two feet below the existing surface with a radius of 4”. See Figure 42 for details on the chain link fence installation.

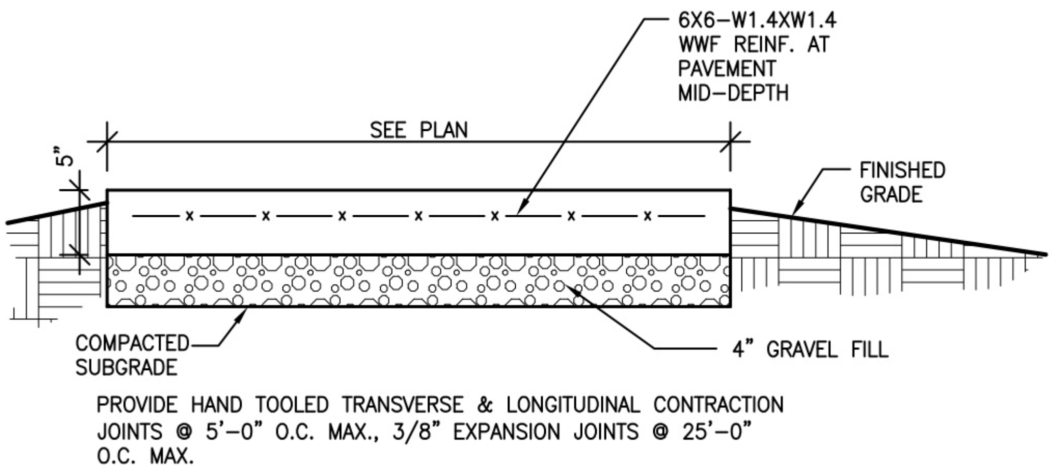


Figure 41 - Concrete pavement detail for outdoor patio and sidewalks.
(Source: https://www.architekwiki.com/details/sidewalk-concrete-walkway-paving)

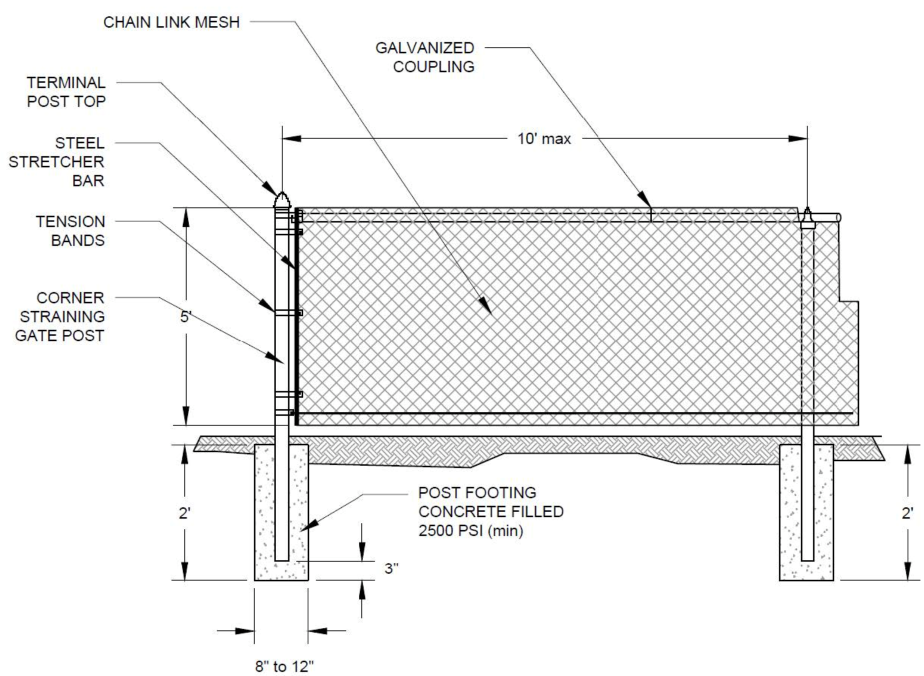


Figure 42 - Footing detail for chain link fence.
(Source: https://www.northmiamifl.gov/799/Chain-Link)

8.3 Environmental Effects

Although the project site is small and contains a pre-built foundation, it is essential to conduct an environmental analysis to ensure the design of additional concrete structures will not impact any endangered species in the surrounding area. To do so, a report was created with Information for Planning and Consultation (IPaC) to get an overview of the endangered species in the area and to see if any of the endangered species have a critical habitat within the project site boundaries.

From the report, it appears there is a presence of the following endangered or threatened species:

- Gray bats
- Northern long-eared bats
- Piping plover
- Red knot
- American burying beetle
- Monarch butterfly

However, there is only a critical habitat for the piping plover and a proposed critical habitat for the red knot. Further action needs to be taken to receive permission to construct within the project limits. See full IPaC report in Appendix 13.2 for further details.

8.4 Construction and Constructability

The primary goal of this project is to analyze the feasibility of rehabilitating the existing structure. The majority of the steel structures are to be placed in the interior of the existing stone walls. The original design plan was to place columns six inches away from the walls to allow for installation of threaded rods to hold the columns. However, the major hindrance in this design is the existing concrete foundation that makes proper installation of threaded rods more difficult. This is an issue that needs further consideration. This design provides sufficient spacing between the column-girder connection and the wall to allow for ease of construction. Additionally, the awning was designed separate from the main steel frame to eliminate the need for extra steel or labor costs.

9. Sustainability Analysis

The sustainability for this project was analyzed by following the *Envision: Sustainable Infrastructure Framework Guidance Manual* created by Institute for Sustainable Infrastructure. The manual is divided into five categories, Quality of Life, Leadership, Resource Allocation, Natural World, and Climate and Resilience. Each of the five categories is divided into several specific subcategories. The number of subcategories that are addressed on a project are relative to the rate of sustainability on a project. The Carter G. Woodson Rehabilitation Project addresses the five subcategories below.

9.1 Quality of life


QUALITY OF LIFE: WELLBEING

QL1.1 Improve Community Quality of Life

26 POINTS

INTENT

Improve the net quality of life of all communities affected by the project and mitigate negative impacts to communities.

METRIC

Measures taken to assess community needs and improve quality of life while minimizing negative impacts.

Envision: Sustainable Infrastructure Framework Guidance Manual version 3

QL1.1 Improve Community Quality of life: The town of Tullahassee is in need of a community building. Through the rehabilitation efforts laid out in this report they are given a building that addresses their needs and wants. Through many meetings with town officials and community alumni, the designed building provides the town with a replica of their original gathering place that offers space for community events, access to a kitchen, and classroom spaces to use for any type of activity that would benefit the town.


QUALITY OF LIFE: WELLBEING

QL1.6 Minimize Construction Impacts

8 POINTS

INTENT

Minimize or eliminate the temporary inconveniences associated with construction.

METRIC

Extent of issues addressed through construction management plans.

Envision: Sustainable Infrastructure Framework Guidance Manual version 3

QL1.6 Minimize Construction Impacts: The rehabilitation efforts taken throughout the project have allowed the minimization of construction impacts. By using the existing foundation and stone walls there is no demolition that will take place. Eliminating the demolition of the building would cut down on the construction timeline and the job would be completed quicker. The building is also one of two buildings on the plot of land. This would minimize the need to alter the communities daily activities and provide lots of open land to place construction equipment.

9.2 Resource Allocation

RESOURCE ALLOCATION: MATERIALS

RA1.2 Use Recycled Materials

16 POINTS

INTENT
Reduce the use of virgin natural resources and avoid sending useful materials to landfills by specifying reused materials, including structures, and material with recycled content.

METRIC
Percentage of project materials that are reused or recycled. Plants, soil, rock, and water are not included in this credit.

Envision: Sustainable Infrastructure Framework Guidance Manual version 3

RA1.2 Use Recycled Materials: One of the main goals of this rehabilitation project was to use the existing structure as the base for the structural design. By utilizing the existing structure, the quantity of required project materials was reduced significantly. This not only cuts costs, but also allows the school to maintain its original exterior appearance. The community of Tullahassee hoped to maintain the original outward appearance of the building, and the team was able to accommodate this request.

RESOURCE ALLOCATION: ENERGY

RA2.2 Reduce Construction Energy Consumption

12 POINTS

INTENT
Conserve resources and reduce greenhouse gases and air pollutant emissions by reducing energy consumption during construction.

METRIC
The number of strategies implemented on the project during construction that reduce energy consumption and emissions.

Envision: Sustainable Infrastructure Framework Guidance Manual version 3

RA 2.2 Reduce Construction Energy Consumption: Due to the utilization of the existing structure, the construction of this project avoids the need for demolition. Avoiding demolition removed the need for demolition equipment, the clean up of demolished materials, and reduced the amount of new materials to be delivered to the site. This reduces the number of trucks, truck trips, and amount of equipment needed on the project site, and in turn reduces the overall fuel consumption required throughout the duration of the project.

9.3 Leadership

LEADERSHIP: COLLABORATION

LD1.3 Provide for Stakeholder Involvement

18 POINTS	**INTENT** Early and sustained stakeholder engagement and involvement in project decision making.	**METRIC** Establishment of sound and meaningful programs for stakeholder identification, early and sustained engagement, and involvement in project decision making.

Envision: Sustainable Infrastructure Framework Guidance Manual version 3

LD1.3 Provide for Stakeholder Involvement: The team initiated weekly meetings with the client to keep all parties informed. The client's input at these meetings was directly integrated into the design of the building. The team planned a site visit in early February to meet with the client in-person. This helped establish a relationship early on in the design of the project. The final design aligns with the clients needs and wants for the project.

10. Risk and Uncertainty Considerations

Throughout the design process, the team ran into several obstacles that limited accuracy and contributed to risk and uncertainty in the final design. The biggest obstacle was the design of the original cross-gable roof located at the southeast entrance. The steel sections required to make the cross-gable would interfere with the entrance and a large section of open area in the southeastern section of the building. The required steel columns would also make it impossible for the southeast entrance to be ADA compliant. As this is the primary entrance to the building, it was necessary to keep this entrance accessible. One alternative solution that was considered was to install a wooden cross-gable section onto the main interior steel frames through a plywood base. This would allow the roof to maintain its original look and function. However, it was unknown if the plywood base would be able to handle the added stresses of the wind load from the cross-gable section and, additionally, it was difficult to determine which connections would make this design possible. Therefore, instead of a steel or wooden cross-gable, a steel gable section was designed to allow for a functional roof with a gutter along the base to prevent molding between the roof and existing stone walls.

Some of the other areas of concern included the stability of the existing stone walls, material conditions of the existing stone walls, and the existing grading/elevations. The existing stone walls were not designed to handle earthquake excitations. Structures built during the 1940s often were not designed to withstand earthquakes. Many buildings structurally similar to the school have been destroyed due to earthquake activity. However, the risk of structural failure is dramatically reduced as the roof is designed to be connected to the interior steel frames. This removal of the lump mass will decrease displacements of the wall during earthquakes and/or heavy wind activity.

The condition of the stone walls was also unknown. This was due to many factors: potential damage from the 2012 fire, years of being left to the elements, and multiple sections of stone wall that were cut out after the initial construction for bay doors. There are also multiple sections of the existing wall with no sandstone pieces attached. This was likely due to a poor or incomplete job of pouring concrete during the original construction of the building. The existence of air pockets within the walls also created a concern for potential plant growth that could cause further damage to the structure. The wall sections adjacent to the bay doors would have to be completely reconstructed and rebuilt to match the pre-existing walls, which would prove to be a difficult and expensive undertaking.

The final area of concern was a lack of survey information about the existing site, including the site elevations, contours, and existing utility structures within the project area. Without a proper survey, the site could not be properly graded. Although the project site appeared mostly flat upon inspection, a survey would be necessary to verify. The design of the new building was done under the assumption that the building or surrounding area do not exceed typical, acceptable slopes. In the case that the existing structure was found to be exceeding typical slopes, the design would have to be adjusted to account for this to avoid any potential damage to the existing or new structure. The exact site contours should also be verified to ensure the drainage run-off direction is the same as expected from existing contours found on Google Maps.

11. Project Cost Estimate

The cost estimate for this project was determined using RSMeans. See Figure 43 below for an itemized list of the materials needed to construct the project. The estimate has subsections designated by the different design components of the project. By dividing the cost estimate into the different components, the town of Tullahassee will be able to easily remove the optional components once there is a finalized budget. This also gives the town a clear idea of the cost-heavy components of the project. The primary and required components of the project include the north and south interior steel structures, the concrete sidewalks, and the roof. These sections are essential to maintain the overall stability of the building. The optional components include the concrete patio, landscaping, and the awning. The cost of the interior could be lowered as well if more open space is desired. The unit cost of each item includes both the material cost and construction cost. The quantities of all items were estimated using the known dimensions of the design elements. The quantities were also rounded up to err on the side of caution. The subtotal of the project is a sum of each subsection. A 40% contingency was then added to the subtotal. This was determined by adding a standard 20% contingency with an additional 20% to account for the risks and uncertainties in the cross-gable section of the roof and the condition of the existing stone walls as mentioned in Section 10. After adding the contingency, the project total was estimated to be $731,000.

Engineer's Cost Estimate
100% Final Plans - Undergraduate Engineers & Preservation

Item	Unit	Quantity	Unit Cost	Total Cost
North Section				
W8x35 A992	LF	1125.5	$79.00	$88,915
W10x45 A992	LF	295	$80.00	$23,600
WT15x45 A992	LF	71	$100.00	$7,100
2" - 3/4" A325-N Bolts/Washers	EA	264	$7.00	$1,848
1' - 3/4" Anchor Rod/Washer	EA	40	$18.00	$720
1"x13"x13" Base Plate	SF	12	$70.00	$840
3/8" Connection Plate	SF	3	$32.00	$96
3.5x3.5x.5 Angle	EA	224	$17.00	$3,808
10' - 1"x8"	EA	120	$30.00	$3,600
			Total =	**$130,527**
South Section				
W8x35 A992	LF	559	$79.00	$44,161
W8x58 A992	LF	230	$115.00	$26,450
W14x43 A992	LF	270	$88.00	$23,760
5.25"x2"x3/8" Plate	SF	1.5	$30.00	$45
1.5"x14"x14" Base Plate	EA	10	$275.00	$2,750
1.25"x9"x21.5" End Plates	EA	10	$210.00	$2,100
3.5"x3.5"x3/8" A36 Angles	EA	96	$7.00	$672
3/4"x3" A325 Bolts/Washers	EA	148	$10.00	$1,480
1 3/8" A325 Bolts	EA	80	$3.00	$240
1.5"x3/8" Washers	EA	80	$5.00	$400
1 1/8"x3/8" Washers	EA	40	$2.00	$80
18"x1" A36 Anchor Rods	EA	40	$20.00	$800
8' - 1"x8"	LF	60	$15.00	$900
			Total =	**$103,838**
Awning				
W10x26 A992	LF	72	$59.00	$4,248
W8x40 A992	LF	60	$80.00	$4,800
W8x67 A992	LF	175	$132.00	$23,100
3/4"x2"Bolts / washers	EA	52	$6.00	$312
1/2"x2.5"x3/8" Plate	SF	14	$26.00	$364
1"x12"x17" Base Plate	EA	6	$68.00	$408
1"x12" Anchor Rods/ Washers	EA	24	$16.00	$384
3.5"x3.5"x0.5"Angle	EA	40	$17.00	$680
			Total =	**$34,296**

Table 3 - Detailed cost estimate.

Roof				
Plywood	SF	10650	$5.00	$53,250
Shingles	SF	10650	$6.00	$63,900
			Total =	**$117,150**
Concrete Patio				
5" Concrete Pavement	SF	1350	$10.00	$13,500
Wire Mesh Reinforcement	SF	1350	$1.00	$1,350
Chain Link Fence	LF	125	$20.00	$2,500
			Total =	**$17,350**
Concrete Sidewalk				
5" Concrete Pavement	SY	2000	$10.00	$20,000
			Total =	**$20,000**
Interior				
Light Framing	BF	1464	$2.00	$2,928
5/8" Drywall	SF	5600	$8.00	$44,800
Headers	LF	1385	$2.00	$2,770
4x8 Joists	LF	5428	$2.00	$10,856
Subflooring	SF	5930	$2.00	$11,860
Flooring	SF	5930	$4.00	$23,720
			Total =	**$96,934**
Landscape				
Deciduous Shrubs	EA	47	$30.00	$1,410
Evergreen Shrubs	EA	8	$30.00	$240
			Total =	**$1,650**

Project Subtotal =	**$521,745**
Contingency (40%) =	$208,698
Project Total =	**$731,000**

Table 4 - Continued detailed cost estimate.

12. Project Summary and Conclusions

In summary, Undergraduate Engineering & Preservation was successful in creating a rehabilitation design for the Carter G. Woodson school in Tullahassee, Oklahoma. These designs included multiple steel structures to support a roof and an awning, the layout of the interior, ADA-compliant entrances and restrooms, and a concrete patio with sidewalks surrounding the building. Rehabilitation of the structure will allow Tullahasse to have a new space to hold events for the community. The patio, the large gathering room, the stage, and the classrooms all have many potential uses for the community, and the display room in the south of the structure will give space to display the history of the school.

Although the proposed design will not be constructed immediately due to other projects already in progress, the additional time allows for some concerns about the existing structure and cross-gable roof to be inspected and handled appropriately. Not all design concerns were able to be met with the current design or cost estimate as they are out of the scope of the project. These include the stage, electrical system, plumbing, and HVAC system. The design and cost of these elements will be left for a professional to assess.

13. Appendices

13.1 - References

- **Wagoner County Building Permit Requirements**
  - https://www.ok.gov/wagonercounty/documents/Building%20Permit%20Requirements.pdf
- **Oklahoma Historic Preservation Standards and Guidelines**
  - https://omes.ok.gov/sites/g/files/gmc316/f/HPStandardsGuidelines.pdf
- **American Society of Civil Engineers 7-16 (ASCE)**
  - https://ascelibrary.org/doi/book/10.1061/9780784414248
- **American Society of Civil Engineers 7-22 (ASCE)**
  - https://sp360.asce.org/PersonifyEbusiness/Merchandise/Product-Details/productId/276865145?_ga=2.105837515.955683875.1649796822-623055044.1646761971
- **International Building Code (IBC)**
  - https://www.ok.gov/oubcc/documents/2021%2009%2014%20IBC%202018%20Permanent%20Rule.pdf
- **International Existing Building Code (IEBC)**
  - https://www.ok.gov/oubcc/documents/2021%2009%2014%20IEBC%202018%20Permanent%20Rule.pdf
- **International Fire Code (IFC)**
  - https://www.ok.gov/oubcc/documents/2021%2009%2014%20Permanent%20Rule%20IFC%202018.pdf
- **Americans with Disabilities Act (ADA)**
  - https://www.ok.gov/odc/documents/SmallTownADA.pd
- **Abandoned Oklahoma**
  - https://abandonedok.com/carter-woodson-school/
- **Institute for Sustainable Infrastructure, Envision Manual version 3**
  - https://canvas.okstate.edu/courses/120856/files/13269517/download?download_frd=1
- **Roof Online**
  - https://roofonline.com/weights-measures/weight-of-roofing-materials/
- **Unified Design of Steel Structures, 3rd Ed.**
- **American Institute of Steel Construction Base Plate and Anchor Design, 2nd Ed.**
  - https://www.construccionenacero.com/sites/construccionenacero.com/files/u11/ci33_321_aisc_design_guide_1_-_column_base_plates_-_2nd_edition.pdf
- **American Institute of Steel Construction Steel Construction Manual, 15th Ed.**
- **American Institute of Steel Construction Extended End Plate Moment Connections Seismic and Wind Applications, 2nd Ed.**
  - http://www.abarsazeha.com/images/ScinteficResources/DesignGuide/DG04.pdf

13.2 - Site Investigation Information

13.2.1 Information for Planning and Consultation Report

IPaC **U.S. Fish & Wildlife Service**

IPaC resource list

This report is an automatically generated list of species and other resources such as critical habitat (collectively referred to as *trust resources*) under the U.S. Fish and Wildlife Service's (USFWS) jurisdiction that are known or expected to be on or near the project area referenced below. The list may also include trust resources that occur outside of the project area, but that could potentially be directly or indirectly affected by activities in the project area. However, determining the likelihood and extent of effects a project may have on trust resources typically requires gathering additional site-specific (e.g., vegetation/species surveys) and project-specific (e.g., magnitude and timing of proposed activities) information.

Below is a summary of the project information you provided and contact information for the USFWS office(s) with jurisdiction in the defined project area. Please read the introduction to each section that follows (Endangered Species, Migratory Birds, USFWS Facilities, and NWI Wetlands) for additional information applicable to the trust resources addressed in that section.

Location

Wagoner County, Oklahoma

Local office

Oklahoma Ecological Services Field Office

📞 (918) 581-7458
📠 (918) 581-7467

9014 East 21st Street
Tulsa, OK 74129-1428

http://www.fws.gov/southwest/es/Oklahoma/

Endangered species

This resource list is for informational purposes only and does not constitute an analysis of project level impacts.

The primary information used to generate this list is the known or expected range of each species. Additional areas of influence (AOI) for species are also considered. An AOI includes areas outside of the species range if the species could be indirectly affected by activities in that area (e.g., placing a dam upstream of a fish population even if that fish does not occur at the dam site, may indirectly impact the species by reducing or eliminating water flow downstream). Because species can move, and site conditions can change, the species on this list are not guaranteed to be found on or near the project area. To fully determine any potential effects to species, additional site-specific and project-specific information is often required.

Section 7 of the Endangered Species Act **requires** Federal agencies to "request of the Secretary information whether any species which is listed or proposed to be listed may be present in the area of such proposed action" for any project that is conducted, permitted, funded, or licensed by any Federal agency. A letter from the local office and a species list which fulfills this requirement can **only** be obtained by requesting an official species list from either the Regulatory Review section in IPaC (see directions below) or from the local field office directly.

For project evaluations that require USFWS concurrence/review, please return to the IPaC website and request an official species list by doing the following:

1. Draw the project location and click CONTINUE.
2. Click DEFINE PROJECT.
3. Log in (if directed to do so).
4. Provide a name and description for your project.
5. Click REQUEST SPECIES LIST.

Listed species[1] and their critical habitats are managed by the Ecological Services Program of the U.S. Fish and Wildlife Service (USFWS) and the fisheries division of the National Oceanic and Atmospheric Administration (NOAA Fisheries[2]).

Species and critical habitats under the sole responsibility of NOAA Fisheries are **not** shown on this list. Please contact NOAA Fisheries for species under their jurisdiction.

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1. Species listed under the Endangered Species Act are threatened or endangered; IPaC also shows species that are candidates, or proposed, for listing. See the listing status page for more information. IPaC only shows species that are regulated by USFWS (see FAQ).
2. NOAA Fisheries, also known as the National Marine Fisheries Service (NMFS), is an office of the National Oceanic and Atmospheric Administration within the Department of Commerce.

The following species are potentially affected by activities in this location:

Mammals

NAME	STATUS
Gray Bat Myotis grisescens Wherever found No critical habitat has been designated for this species. https://ecos.fws.gov/ecp/species/6329	Endangered
Northern Long-eared Bat Myotis septentrionalis Wherever found No critical habitat has been designated for this species. https://ecos.fws.gov/ecp/species/9045	Threatened

Birds

NAME	STATUS
Piping Plover Charadrius melodus There is **final** critical habitat for this species. The location of the critical habitat is not available. https://ecos.fws.gov/ecp/species/6039	Threatened
Red Knot Calidris canutus rufa Wherever found There is **proposed** critical habitat for this species. The location of the critical habitat is not available. https://ecos.fws.gov/ecp/species/1864	Threatened

Insects

NAME	STATUS
American Burying Beetle Nicrophorus americanus No critical habitat has been designated for this species. https://ecos.fws.gov/ecp/species/66	Threatened
Monarch Butterfly Danaus plexippus Wherever found No critical habitat has been designated for this species. https://ecos.fws.gov/ecp/species/9743	Candidate

Critical habitats

Potential effects to critical habitat(s) in this location must be analyzed along with the endangered species themselves.

THERE ARE NO CRITICAL HABITATS AT THIS LOCATION.

Migratory birds

Certain birds are protected under the Migratory Bird Treaty Act[1] and the Bald and Golden Eagle Protection Act[2].

Any person or organization who plans or conducts activities that may result in impacts to migratory birds, eagles, and their habitats should follow appropriate regulations and consider implementing appropriate conservation measures, as described below.

1. The Migratory Birds Treaty Act of 1918.
2. The Bald and Golden Eagle Protection Act of 1940.

Additional information can be found using the following links:

- Birds of Conservation Concern http://www.fws.gov/birds/management/managed-species/birds-of-conservation-concern.php
- Measures for avoiding and minimizing impacts to birds http://www.fws.gov/birds/management/project-assessment-tools-and-guidance/conservation-measures.php
- Nationwide conservation measures for birds http://www.fws.gov/migratorybirds/pdf/management/nationwidestandardconservationmeasures.pdf

THERE ARE NO MIGRATORY BIRDS OF CONSERVATION CONCERN EXPECTED TO OCCUR AT THIS LOCATION.

Tell me more about conservation measures I can implement to avoid or minimize impacts to migratory birds.

Nationwide Conservation Measures describes measures that can help avoid and minimize impacts to all birds at any location year round. Implementation of these measures is particularly important when birds are most likely to occur in the project area. When birds may be breeding in the area, identifying the locations of any active nests and avoiding their destruction is a very helpful impact minimization measure. To see when birds are most likely to occur and be breeding in your project area, view the Probability of Presence Summary. Additional measures or permits may be advisable depending on the type of activity you are conducting and the type of infrastructure or bird species present on your project site.

What does IPaC use to generate the migratory birds potentially occurring in my specified location?

The Migratory Bird Resource List is comprised of USFWS Birds of Conservation Concern (BCC) and other species that may warrant special attention in your project location.

The migratory bird list generated for your project is derived from data provided by the Avian Knowledge Network (AKN). The AKN data is based on a growing collection of survey, banding, and citizen science datasets and is queried and filtered to return a list of those birds reported as occurring in the 10km grid cell(s) which your project intersects, and that have been identified as warranting special attention because they are a BCC species in that area, an eagle (Eagle Act requirements may apply), or a species that has a particular vulnerability to offshore activities or development.

Again, the Migratory Bird Resource list includes only a subset of birds that may occur in your project area. It is not representative of all birds that may occur in your project area. To get a list of all birds potentially present in your project area, please visit the AKN Phenology Tool.

What does IPaC use to generate the probability of presence graphs for the migratory birds potentially occurring in my specified location?

The probability of presence graphs associated with your migratory bird list are based on data provided by the Avian Knowledge Network (AKN). This data is derived from a growing collection of survey, banding, and citizen science datasets .

Probability of presence data is continuously being updated as new and better information becomes available. To learn more about how the probability of presence graphs are produced and how to interpret them, go the Probability of Presence Summary and then click on the "Tell me about these graphs" link.

How do I know if a bird is breeding, wintering, migrating or present year-round in my project area?

To see what part of a particular bird's range your project area falls within (i.e. breeding, wintering, migrating or year-round), you may refer to the following resources: The Cornell Lab of Ornithology All About Birds Bird Guide, or (if you are unsuccessful in locating the bird of interest there), the Cornell Lab of Ornithology Neotropical Birds guide. If a bird on your migratory bird species list has a breeding season associated with it, if that bird does occur in your project area, there may be nests present at some point within the timeframe specified. If "Breeds elsewhere" is indicated, then the bird likely does not breed in your project area.

What are the levels of concern for migratory birds?

Migratory birds delivered through IPaC fall into the following distinct categories of concern:

1. "BCC Rangewide" birds are Birds of Conservation Concern (BCC) that are of concern throughout their range anywhere within the USA (including Hawaii, the Pacific Islands, Puerto Rico, and the Virgin Islands);
2. "BCC - BCR" birds are BCCs that are of concern only in particular Bird Conservation Regions (BCRs) in the continental USA; and
3. "Non-BCC - Vulnerable" birds are not BCC species in your project area, but appear on your list either because of the Eagle Act requirements (for eagles) or (for non-eagles) potential susceptibilities in offshore areas from certain types of development or activities (e.g. offshore energy development or longline fishing).

Although it is important to try to avoid and minimize impacts to all birds, efforts should be made, in particular, to avoid and minimize impacts to the birds on this list, especially eagles and BCC species of rangewide concern. For more information on conservation measures you can implement to help avoid and minimize migratory bird impacts and requirements for eagles, please see the FAQs for these topics.

Details about birds that are potentially affected by offshore projects

For additional details about the relative occurrence and abundance of both individual bird species and groups of bird species within your project area off the Atlantic Coast, please visit the Northeast Ocean Data Portal. The Portal also offers data and information about other taxa besides birds that may be helpful to you in your project review. Alternately, you may download the bird model results files underlying the portal maps through the NOAA NCCOS Integrative Statistical Modeling and Predictive Mapping of Marine Bird Distributions and Abundance on the Atlantic Outer Continental Shelf project webpage.

Bird tracking data can also provide additional details about occurrence and habitat use throughout the year, including migration. Models relying on survey data may not include this information. For additional information on marine bird tracking data, see the Diving Bird Study and the nanotag studies or contact Caleb Spiegel or Pam Loring.

What if I have eagles on my list?

If your project has the potential to disturb or kill eagles, you may need to obtain a permit to avoid violating the Eagle Act should such impacts occur.

Proper Interpretation and Use of Your Migratory Bird Report

The migratory bird list generated is not a list of all birds in your project area, only a subset of birds of priority concern. To learn more about how your list is generated, and see options for identifying what other birds may be in your project area, please see the FAQ "What does IPaC use to generate the migratory birds potentially occurring in my specified location". Please be aware this report provides the "probability of presence" of birds within the 10 km grid cell(s) that overlap your project; not your exact project footprint. On the graphs provided, please also look carefully at the survey effort (indicated by the black vertical bar) and for the existence of the "no data" indicator (a red horizontal bar). A high survey effort is the key component. If the survey effort is high, then the probability of presence score can be viewed as more dependable. In contrast, a low survey effort bar or no data bar means a lack of data and, therefore, a lack of certainty about presence of the species. This list is not perfect; it is simply a starting point for identifying what birds of concern have the potential to be in your project area, when they might be there, and if they might be breeding (which means nests might be present). The list helps you know what to look for to confirm presence, and helps guide you in knowing when to implement conservation measures to avoid or minimize potential impacts from your project activities, should presence be confirmed. To learn more about conservation measures, visit the FAQ "Tell me about conservation measures I can implement to avoid or minimize impacts to migratory birds" at the bottom of your migratory bird trust resources page.

Facilities

National Wildlife Refuge lands

Any activity proposed on lands managed by the National Wildlife Refuge system must undergo a 'Compatibility Determination' conducted by the Refuge. Please contact the individual Refuges to discuss any questions or concerns.

THERE ARE NO REFUGE LANDS AT THIS LOCATION.

Fish hatcheries

THERE ARE NO FISH HATCHERIES AT THIS LOCATION.

Wetlands in the National Wetlands Inventory

Impacts to NWI wetlands and other aquatic habitats may be subject to regulation under Section 404 of the Clean Water Act, or other State/Federal statutes.

For more information please contact the Regulatory Program of the local U.S. Army Corps of Engineers District.

WETLAND INFORMATION IS NOT AVAILABLE AT THIS TIME

This can happen when the National Wetlands Inventory (NWI) map service is unavailable, or for very large projects that intersect many wetland areas. Try again, or visit the NWI map to view wetlands at this location.

Data limitations

The Service's objective of mapping wetlands and deepwater habitats is to produce reconnaissance level information on the location, type and size of these resources. The maps are prepared from the analysis of high altitude imagery. Wetlands are identified based on vegetation, visible hydrology and geography. A margin of error is inherent in the use of imagery; thus, detailed on-the-ground inspection of any particular site may result in revision of the wetland boundaries or classification established through image analysis.

The accuracy of image interpretation depends on the quality of the imagery, the experience of the image analysts, the amount and quality of the collateral data and the amount of ground truth verification work conducted. Metadata should be consulted to determine the date of the source imagery used and any mapping problems.

Wetlands or other mapped features may have changed since the date of the imagery or field work. There may be occasional differences in polygon boundaries or classifications between the information depicted on the map and the actual conditions on site.

Data exclusions

Certain wetland habitats are excluded from the National mapping program because of the limitations of aerial imagery as the primary data source used to detect wetlands. These habitats include seagrasses or submerged aquatic vegetation that are found in the intertidal and subtidal zones of estuaries and nearshore coastal waters. Some deepwater reef communities (coral or tuberficid worm reefs) have also been excluded from the inventory. These habitats, because of their depth, go undetected by aerial imagery.

Data precautions

Federal, state, and local regulatory agencies with jurisdiction over wetlands may define and describe wetlands in a different manner than that used in this inventory. There is no attempt, in either the design or products of this inventory, to define the limits of proprietary jurisdiction of any Federal, state, or local government or to establish the geographical scope of the regulatory programs of government agencies. Persons intending to engage in activities involving modifications within or adjacent to wetland areas should seek the advice of appropriate federal, state, or local agencies concerning specified agency regulatory programs and proprietary jurisdictions that may affect such activities.

13.3 Client Meetings

13.3.1 Client Meeting 1/28/22

This was the first meeting with the client, as well as with members of the history department. The main agenda was for introductions and to start getting an idea of what both groups will be doing.

13.3.2 Client Meeting 1/31/22

The purpose of this meeting was to form a basis for what the town was wanting from the projects and some of the history of the town. Discussion over the town having a bi-annual homecoming as well as family gatherings made the project be focused on providing for the community as a whole. The shell of the building is salvageable and is wanted to be kept as well as there not being any present documents over the building. An OU group was going to be in town for two days and the group was extended an invite. Grants and other forms of revenue weren't defined because of trying to finish audits.

13.3.3 Client Meeting 2/6/22

Only one member was able to go to Tullahassee and gather information from the town about what they would like to see from the building. Discussions with the mayor and others from the community gave a large amount of ideas for what can be done with the building and set a layout for what can be done. Major things that were asked were to have the building have a kitchen, a stage, classrooms, and an outside area for the community to use. An awning on the West side of the building was proposed and greatly liked as well as extending a window to allow for access to the kitchen from the outside. Viewing the building also showed that the structure was stable, but needed major repairs to the walls from the fire and where the fire department cut out bay doors.

13.3.4 Client Meeting 2/9/22

This meeting was to have the whole group see the structure and get measurements of the structure to be able to start designing the structure. Discussions over the ideas from 2/6/22 happened to ensure clarity for the whole group as well as the mayor.

13.3.5 Client Meeting 2/20/22

Discussion over the decision matrix and collecting some extra measurements.

13.3.6 Client Meeting 4/8/22

Discussion over the design up to that point and what was left of the semester. A brief discussion over a cost analysis happened and the mayor was looking forward to it.

13.4 Data and Analysis

13.4.1 Hand Calculations - Design of North Section and SkyCiv Results

Load Combinations:

Loading – $DL = 3.35\ lb/ft^2$
$L_R = 20\ lb/ft^2$
$S = 26.17$ (includes ice)
$W = 30\ lb/ft^2$

Load Combinations: 1. $1.4(3.35) = 4.69$

2. $1.2(3.35)+1.6(0)+.5(20) = 14.02$
$1.2(3.35)+1.6(0)+.5(26.17) = 17.11$

3. $1.2(3.35)+1.6(26.17)+.5(30) = \underline{60.89}$
4. $1.2(3.35)+(30)+.5(26.17) = 47.11$ (-12.90)

5. $1.2(3.35)+1.0(1)+.2(26.17) = 10.25$
6. $.9(3.35)+1(30) = 33.02$
$.9(3.35)+1(-30) = \underline{-26.99}$

Design of Girder Beams:

Tributary Area – $15.25' \times 4' = 61\ ft^2$

· Assume A992 steel $F_y = 50\ ksi$, $F_u = 65\ ksi$

Loading – $60.89\ lb/ft^2 \times 4' = .244\ k/ft$

$M_u = \frac{.244(15.25)^2}{8} = 7.09 = \cancel{\frac{24.31}{1.14}} = \cancel{21.32}\ k\cdot ft$ @ $L_b = 15.25'$

Table 3-10: Try $\underline{W8\times10}$ → $\phi M_n = 33.0\ k\cdot ft$

w/ self weight: $w = .254\ k/ft$, $M_u = 7.38\ k\cdot ft$ good!

Tributary Area – $25' \times 4' = 100\ ft^2$

Loading – $60.89 \times 4' = .244\ k/ft$

$M_u = \frac{.244(25)^2}{8} = 19.06$ $\cancel{= 93.75\ k/ft}$ @ $L_b = 25\ ft$

Table 3-10: Try $\underline{W8\times35}$ → $\phi M_n = 87\ k\cdot ft$

w/ self weight: $P_u = .279\ k/ft$, $M_u = 21.80\ k\cdot ft$ good!

$\underline{W8\times10}$: Tension – $P_n = 50(5.26) = 263\ k$ (✓)
Shear – $V_n = .6(50)(8.14\times.23) = 40.2 > 1.937$ (✓)

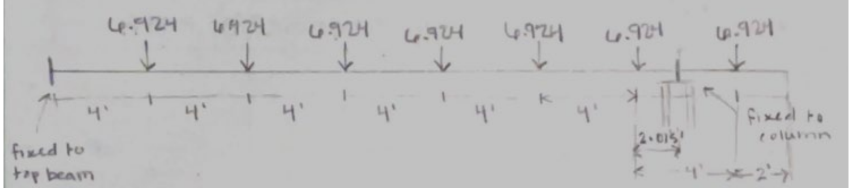
$\underline{W8\times35}$: Tension – $P_n = 50(10.3) = 515\ k$ (✓)
Shear – $V_n = .6(50)(8.12\times.31) = 75.52 > 3.462$ (✓)

– point loads on beam = $\underline{6.924\ k}$

Design of Roof Beams:

Roof Beams: 30' long

* worst case you have $\frac{1}{2}$ of a 25' girder & ½ of a 15.25' girder resting on one point. ⇒ 40.42 k @ each point



- Solved using SkiCiv Software

$M_{max} = 50.242$ kip·ft , $V_{max} = 19.055$ kip

W10×45 → Try this

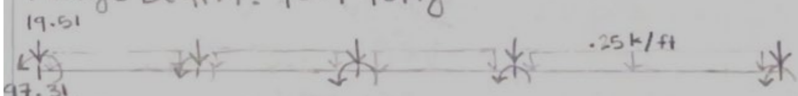
w/ SW included : $M_{max} = 51.4952$ k·ft , $V_{max} = 19.64$ kip

ϕM_n = good!

$\phi V_n = 75.5\text{ k} > 19.64\text{ k}$ ∴ good!

Design of Ridge Beam:

Ridge Beam: 70.9' long



- Using SkyCiv

$M_{max} = 6.51$ k·ft , $V_{max} = 3.125$ kip

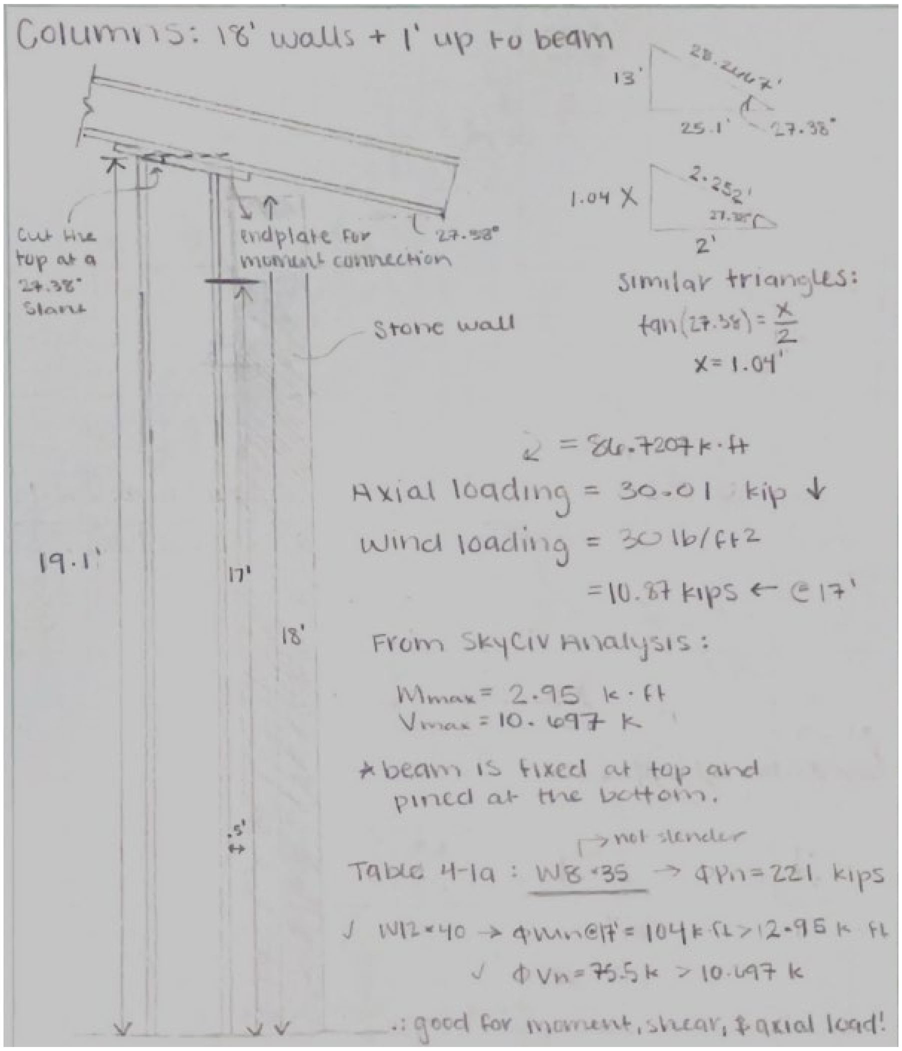
for WT's : Yielding – $M_n = (57.7)(50) \leq 1.6(32)(50) = 2560 = 213$ k·ft

WT13.5×42 LTB – $L_b = 25'$, $L_p = 7.57'$, $L_r = 52.97'$

$L_p < L_b < L_r$ – $M_n = 213 - (213 - 133.4)\left(\frac{25 - 7.59}{52.97 - 7.59}\right) = 182$ k·ft ✓

(slender) FLB – $M_n = \frac{.7(29000)(32)}{\left(\frac{10.5}{2 \times .76}\right)^2} = 1134.42$ k·ft ✓

Design of Columns:



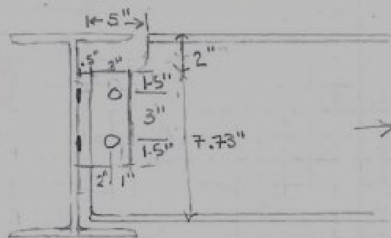
Design of beam to girder connection:

Roof Girder to beam: W8×35 to W10×45   10×39

\+ Simple connection

\+ Double-angle, bolted-bolted

R_u = 7 kips, all steel is A992 Gr. 50. Use L3½×3½×½ angles.
Flange coped 2". Use 3/4" A325-N bolts in standard holes.



→ initial layout

1. $\phi r_n = 17.9\text{ k}(2) = 35.8$ kips $N = \frac{7}{35.8} = .19$ → need less than 1 bolt - lets do 2!

2. W8×35 - $t_w = .31$

1st bolt

$l_c = 1.5 - \frac{1}{2}(3/4 + 1/16) = 1.09 < 1.5$ (2×3/4) → tearout controls

$R_n = 1.2 l_c t F_u = 1.2(1.09)(.31)(65) = (26.4\text{ kips}).75 = 19.8\text{ k}$

18.5 k < 35.8 k → nominal bolt strength controls over design shear strength

2nd bolt

$l_c = 3 - (3/4 + 1/16) = 2.1875 > 1.5$ (13/16) → bearing controls

$R_n = 2.4(3/4)(.31)(65) = (36.27\text{ k}).75 = 27.20 < 35.8\text{ k}$

$\phi R_n = 25.5 + 18.5 = 44\text{ k} > 7\text{ k}$ → can support load!

3. L3½×3½×½

1st bolt

$l_c = 1.5 - \frac{1}{2}(13/16) = 1.09 < 1.5$ → tearout

$R_n = 1.2(1.09)(.5)(65) = (42.51\text{ k}).75 = 31.88\text{ k} < 35.8\text{ k}$

2nd bolt

$L_c = 3 - (13/16) = 2.1875 > 1.5$ → bearing

$R_n = .75(2.4)(3/4)(.5)(65) = 43.875 > 35.8\text{ k}$

1st	2nd
35.8	35.8 k
18.5	25.5 k
31.88×2	43.875 k×2

$\phi R_n = 18.5 + 25.5 = 44\text{ k} > 7\text{ k}$ ✓

4. Min depth: $T = 5.75''(1/2) = 2.85''$ → depth is $6'' > 2.875''$ — good! ✓

5. Shear yield of beam

$\phi V_n = 75.7\text{ k} > 7\text{ kips}$ ✓

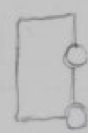
6. Check yield @ cope

$\phi V_n = \phi(.6)F_y h t_w = 1.0(.6)(50)(7.12)(.31) = 66.22\text{ k} > 7\text{ k}$ – good ✓

7. Check rupture @ cope

$\phi V_n = .75(.6)(65)(7.12 - 2(13/16))(.31) = 49.83\text{k} > 7\text{k}$ → good! ✓

8. Check beam web for block shear



$A_{nt} = (1.75 - \frac{1}{2}(\underbrace{3/4 + 1/16 + 1/16}_{.875}))(.31) = .41\text{ in}^2$

$A_{gv} = 6(.31) = 1.86\text{ in}^2$

$A_{nv} = (6 - 1.5(3/4 + 1/16 + 1/16))(.31) = 1.43\text{ in}^2$

$F_u A_{nt} = 65(.41) = 26.65\text{ k}$

$.6 F_y A_{gv} = .6(50)(1.86) = 55.8\text{ k}$

$.6 F_u A_{nv} = .6(65)(1.43) = 55.77\text{ k}$

$\phi R_n = .75(55.8 + 26.65) = 61.84\text{ k}$

$61.84\text{ k} > 7\text{ k}$ — good! ✓

9. Check coped beam flexural strength

$M_u = 7\text{k}(5.5) = 38.5\text{ k}\cdot\text{in}$

$\frac{c}{d} < 1 \rightarrow f = \frac{2(5)}{8.12} = 1.23$ $\qquad$ $\frac{c}{h_o} > 1 \rightarrow k = 2.2\left(\frac{7.63}{5}\right)^{1.65} = 4.42$

$k_1 = 1.23 \times 4.42 = 5.44 > 1.61$

$\lambda_p = .46\sqrt{\frac{5.44(29000)}{50}} = 25.84$ $\qquad$ $\lambda = 32.07$

$M_p = F_y Z = 50(34.7) = 1735$

$M_y = F_y S = 50(31.2) = 1560$

good! ✓

$M_n = 1735 - (1735 - 1560)\left(\frac{32.07}{25.84} - 1\right) = 1692.8\text{ k}\cdot\text{in} > 38.5\text{ k}\cdot\text{in}$

10. Check angles for rupture, yield, block shear

$A_{nv} = (6 - 2(.875)).5 = 2.125\text{ in}^2$ $\qquad$ $\phi V_n = 2(\phi .6 F_u A_{gv}) = 2(.75 \times .6 \times 65 \times 2.125)$

$= 124 > 7$ ✓

$A_{gv} = 6(.5) = 3\text{ in}^2$ $\qquad$ $\phi V_n = 2 \times .6 \times 50 \times 3 = 180 > 7$ good! ✓

$A_{nt} = (1 - \frac{1}{2}(.875)).5 = .28\text{ in}^2$ $\qquad$ $65 \times .28 = 18.2$

$.6(50)(2.25) = 67.5$

$.6(65)(1.59) = 62.01$

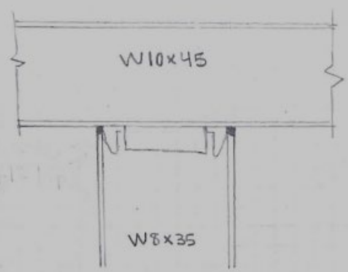
$A_{gv} = 4.5(.5) = 2.25\text{ in}^2$

$A_{nv} = (4.5 - 1.5(.875)).5 = 1.59\text{ in}^2$ $\qquad$ $\phi R_n = 2(.75(18.2 + 62.01))$

$= 120.32 > 7$ — good! ✓

Design of beam to column connection:

Roof beam to column: W10×39 to W8×35



10×39: $b_f = 8"$ W8×35: $b_f = 8"$

+ moment connection
+ $M_u = 202$ k·ft
  $V_u = 11.12$ k
+ use 3/4" A325-N bolts and E70 electrodes
+ all steel is A992 Gr. 50

Beam: W10×45
$d = 10.1$ in, $b_f = 8.02"$, $t_w = .35$ in, $t_f = .62$ in, $Z = 54.9\ in^3$

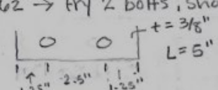
Column: W8×35
$d = 8.12$ in, $b_f = 8.02"$, $t_w = .31$ in, $t_f = .495$ in

1. Flexural strength of beam
$\phi M_n = \frac{.9(50)(54.9)}{12} = 206$ k·ft > 202 k·ft — good!

2. Flange to column weld:
CJP weld using E70 electrodes.

3. Web plate:
min bolts = $\frac{11.12}{17.9} = .62 \rightarrow$ try 2 bolts, should be conservative!

$T/2 = 3.75$ in $\rightarrow$



4. Bolt bearing strength:
1st/ $L_c = 1.25 - \frac{1}{2}(13/16) = .84 < 1.5 \rightarrow$ tearout
$\phi R_n = .75 \times 1.2 \times .84 \times .375 \times 65 = 18.43$ k
2nd/ $L_c = 2.5 - (13/16) = 1.6875 > 1.5 \rightarrow$ bearing
$\phi R_n = .75(2.4)(.75)(.375)(65) = 32.91$ k

design strength = 17.9 kips

$\phi R_n = 2(17.9) = 35.8$ k
35.8 k > 11.12 k
good! ✓

5. Shear yield of plate:
$A_{gv} = .375(5) = 1.875\ in^2$
$\phi V_n = 1.0(.6)(50)(1.875) = 56.25 > 11.12$ good! ✓

6. Shear rupture of plate:
$A_{nv} = (5 - 2(.875)).375 = 1.22\ in^2$
$\phi V_n = .75(.6)(50)(1.22) = 27.45 > 11.12$ good! ✓

7. Block shear of plate:
$A_{nt} = (1.25 - \frac{1}{2}(.875)).375 = .31\ in^2$
$A_{gv} = 3.75 \times .375 = 1.41\ in^2$
$A_{nv} = (3.75 - 1.5(.875)).375 = .92\ in^2$
$(.31)65 = 20.15$ k
$.6(50)(1.41) = 42.3$ k
$.6(65)(.92) = 35.9$ k

$\phi R_n = .75(35.9 + 20.15) = 42.1$ k
42.1 k > 11.12 k — good! ✓

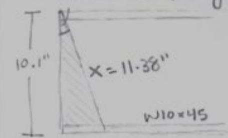
8. Plate to beam weld:
- Fillet welds on either side of the plate

$D = \frac{11.12}{(2 \times 1.392 \times 5)} = .80$ sixteenths of in.

use a 3/16" weld, the minimum for a 3/8" plate.

Design of beam to ridge beam:

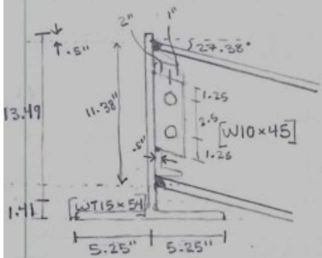
Roof Beam to Ridge Beam: W10×45 to WT15×45



$\frac{10.1}{\cos(27.38)} = 11.38"$

$\tan(62.62) = \frac{4.98}{X}$

$X = 2.58$



+ moment connection

+ $M_u = 113.15$ k·ft
  $V_u = 25.44$ k

+ use 3/4" A325-N bolts in std. holes and E70 electrodes

+ all steel in A992 Gr. 50

1. $\phi M_n = \frac{.9(50)(54.9)}{12} = 206$ k·ft > 113.15 k·ft

   $\phi M_n = \frac{.9(50)(57.7)}{12} = 216$ k·ft > 113.15 k·ft

2. Flange to column weld:
   CJP weld using E70 electrodes

3. bolts needed: $\frac{25.44}{17.9} = 1.43 = 2$ bolts!

4. Bolt bearing strength:
   - configuration is the same as roof beam to column.
   - $\phi R_n = 32.91$ k & 18.43 k > 17.9 k $\phi R_n = 2(17.9) = 35.8$ k > 25.44 k — good! ✓

5. Shear yield of plate:
   - plate is the same size & thickness as roof beam to column.
   - $\phi V_n = 56.25$ k > 25.44 k — good! ✓

6. Shear rupture of plate:
   - from previous connection $\phi R_n = 27.45$ k > 25.44 k — good! ✓

7. Block shear of plate:
   $\phi R_n = 42.1$ k > 25.44 k — good! ✓

8. Plate to beam weld:
   - Fillet welds on either side of the plate
     $D = \frac{25.44}{(2 \times 1.392 \times 5)} = 1.84$ sixteenths of an inch

   use a 3/16" weld, the minimum for a 3/8" plate.

Design of column base plate:

Moment Base Plate design: W8x35 column

1. Required Strength

$M_u = 3.1$ k·ft

$P_u = 175$ k

2. $N > d + 2(2") = 12.12" = 13"$

$B > b_f + 2(2") = 12.02" = 13"$ Try 13"×13" base plate

3. e & e_{crit}

$e = \frac{3.1}{175} = .02$ in $\quad f_{p(max)} = \phi_c(.85f'_c)\sqrt{\frac{A_2}{A_1}} = .65 \times .85 \times 4 \times 1 = 2.21$ ksi

$q_{max} = f_{p(max)} \times B = 2.21 \times 13 = 28.73$ k/in

$e_{crit} = \frac{13}{2} - \frac{175}{2(28.73)} = 3.45$ in

$e < e_{crit} \rightarrow$ can be designed as small moment

4. Bearing Length (Y).

$Y = N - 2e = 13 - 2(.02) = 12.96 \quad q = 13.5 < q_{max}$ — good! ✓

5. Plate thickness.

$m = \frac{N - .95d}{2} = \frac{13 - .95(8.12)}{2} = 2.64$ in

$f_p = \frac{175}{13 \times 12.96} = 1.04$ ksi $\quad Y > m$, so $t_{p(req)} = 1.5(2.64)\sqrt{\frac{1.04}{36}}$

$= .67$ in

Check using n:

$n = \frac{13 - .8(8.02)}{2} = 3.29$ in $\rightarrow t_{p(req)} = 1.5(3.29)\sqrt{\frac{1.04}{36}} = .84$ in

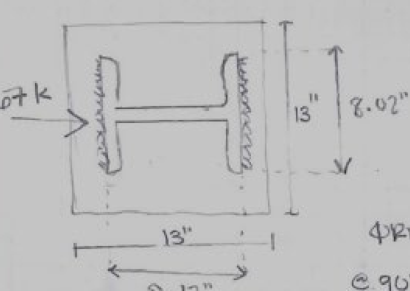
controls

use base plate 1"×13"×1'-1"

6. rod size

(4) 3/4" rods, ASTM F1554, Gr. 36 : length = 12"

7. Design welds connecting column to base plate:



+ Two transverse welds

+ Base plate 1" thick

· try 1/8" fillet weld

$F_{EXX} = E70$

$\phi R_n = \phi F_{nw} A_{we}$

@ 90° = $.6F_{EXX}(1.5) = .6(70)(1.5) = 63$

$A_{we} = .707(.125)(8.02 + 8.02) = 1.42$

$\phi R_n = .75(63)(1.42) = 67.10$ kips $> .67$ k

13.4.2 Hand Calculations - Design of South Section

Design of Beams:

Beam design for south section

lateral beams :

Load : 60.49 lb/ft^2

Distributed load : 487.12 lb/ft = 0.49 kip/ft

Max moment : $\frac{wl^2}{8} = \frac{(0.49 k/ft)(25 ft)^2}{8} = 38.3$ k·ft

$L_b = 25$ ft

spacing : 8 ft ∴ 4 beams per side

Max shear : $w\left(\frac{l}{2}\right) = 0.49 \text{ kip/ft}\left(\frac{25 ft}{2}\right) = 6.125$ kip

try W 8 X 35

meets moment requirement

check shear :

$A_w = T \cdot t_w = 5.75 in \cdot 0.31 in = 1.78 in^2$

$C_{v1} = 1.0$

$V_n = 0.6 F_y A_w C_{v1} = 0.6 \cdot 50 k/in^2 \cdot 1.78 in^2 \cdot 1 = 53.5 k > 6.125 kip$

serviceability check :

$l/240 = 25 ft/240 = 0.1 ft = 1.2 in$

$$\Delta_{max} = \frac{5wl^4}{384EI} = \frac{5 \cdot 0.49 kip/ft \cdot \left(\frac{1 ft}{12 in}\right)\left(25 ft \cdot \frac{12 in}{1 ft}\right)^4}{384 (29000 ksi)(127 in^4)} = 1.17 in$$

$1.2 in > 1.17 in$

∴ it works

Design of 30 ft section beams:

Beam design 30ft section

Load: 0.49k/ft

Max moment: $\frac{wl^2}{8} = \frac{(0.49k/ft)(30ft)^2}{8} = 55.1k\cdot ft$

$L_b = 30ft$

max shear $= w\left(\frac{l}{2}\right) = 0.49kip/ft\left(\frac{30ft}{2}\right) = 7.35kip$

check W6x35

meets moment requirement

check shear

$A_w = 1.78in^2$

$V_n = 0.6F_y \cdot A_w \cdot C_v = 0.6 \cdot 50ksi \cdot 1.78in^2 \cdot 1 = 53.5k > 7.35kip$

serviceability

$l/240 = 30ft/240 = 0.125ft = 1.5in$

$\Delta_{max} = \frac{5wl^4}{384EI} = \frac{5 \cdot 0.49k/ft \cdot \left(\frac{1ft}{12in}\right)\left(30ft \cdot \frac{12in}{1ft}\right)^4}{384(29000ksi)(127.2in^4)} = 2.42in$

$2.42in > 1.5in$ ∴ need new section

try W8x58

$\Delta_{max} = \frac{5wL^4}{384EI} = \frac{5 \cdot 0.49k/ft \cdot \frac{1ft}{12in}\left(30ft \cdot \frac{12in}{1ft}\right)^4}{384(29000ksi)(228in^4)} = 1.35in$

$1.35in < 1.5$ ∴ good

Design of Girder:

Girder design

Length: 27 ft

unsupported length: 24.52 ft

Point Loads: two 6.56 kip loads in the middle, two 4.24 kip at the top, two 5,77 kip loads at the connection to the column

Max shear:

$$V_1 = \frac{Pb^2}{L^3}(3a+b) = \frac{8.48\,kip \cdot (24)^2}{(24.52\,ft)^3}(3 \cdot 0.52\,ft + 24) + \frac{13.12\,kip \cdot (16\,ft)^2}{(24.52\,ft)^3}(3 \cdot 8.52 + 16)$$

$$- \frac{13.12\,kip \cdot (8\,ft)^2}{(24.52\,ft)^3}(3 \cdot 16.52 + 8)$$

$$= 14.64\,kip \leftarrow \text{max shear}$$

$$V_2 = \frac{Pa^2}{L^3}(3b+a) = \frac{13.12\,kip \cdot (16.52\,ft)^2}{(24.52\,ft)^3}(3 \cdot 8\,ft + 16.52\,ft) - \frac{13.12\,kip \cdot (8.52\,ft)^2}{(24.52\,ft)^3}(3 \cdot 16 + 8.52)$$

$$- \frac{8.48\,kip \cdot (0.52\,ft)^2}{(24.52\,ft)^3}(3 \cdot 24 + 0.52)$$

$$= 6.18\,kip$$

Max moment: $M_{max} = 75.18\,kip \cdot ft$

$C_b = 1$

try 14x43

meets moment requirement

$A_w = T \cdot t_w = 10\,7/8\,in \cdot 5/16\,in = 3.4\,in^2$

$V_n = 0.6 \cdot f_y \cdot A_w \cdot C_v = 0.6 \cdot 50\,ksi \cdot 3.4\,in^2 \cdot 1 = 102\,kip > 14.64\,kip$

check serviceability

$l/240 = 24.52\,ft/240 = 0.1\,ft = 1.23\,in$

$\Delta_{max} = 0.15\,in$

$0.15\,in < 1.23$ ∴ it is good

Design of Column:

Column Design for South Section

Length: 16 ft

Axial Load: 20.82 kip + 0.7 kip = 21.527 kip

windloading: 30 lb/ft² $\cdot$ cos(15.7) = 28.9 kip

$$M_{max} = \frac{PL}{8} = \frac{28.9\,kip \cdot 16\,ft}{8} = 57.8\,k \cdot ft$$

$$V_{max} = \frac{P}{2} = \frac{28.9\,kip}{2} = 14.45\,kip$$

try W8x35

Axial: 241 kip > 21.82 kip

$\phi M_n = 72.2\ k \cdot ft > 57.8\ k \cdot ft$

$\phi V_n = 75.5\,k > 28.9\,kip$

Design of Girder to Column Connection:

Girder to column connection

Design Webplate:

using 3/4" A325-N bolts, $\phi R_n = 17.9$ kips

$$n = \frac{28.9 \text{ kip}}{17.9} = 1.6 \therefore 2 \text{ bolts are needed}$$

length of plate = 1.5in + 1.5in + 3 · 3/4in = 5.25in

width of plate = 2in

thickness = 3/8in

Bolt bearing strength:

$l_c = l_{e} - \frac{1}{2} d_h = 1.5in - \frac{1}{2}(3/4 + 1/8\ in) = 1.5in - 7/16in = 17/16in < 2(3/4) = 1.5in$

$\phi R_n = 1.2 l_c t F_u = 1.2(17/16in)(3/8in)(58\ ksi) = 27.7$ kip

for other bolt

$l_c = s - d_h = (3 \cdot 3/4in) - 7/8in = 11/8in < 2(3/4) = 1.5in$

$\phi R_n = 1.2 l_c t F_u = 1.2(11/8in)(3/8in)(58ksi) = 35.9$ kips

$\phi R_{n\,total} = 27.7$ kip + 35.9 kip = 63.6 kip > 28.9 kip

Check plate for shear yield:

$A_{gv} = t \cdot L = (3/8in)(5.25in) = 1.97in^2$

$\phi V_n = \phi\, 0.6 F_y A_{gv} = 1.0 \cdot 0.6 \cdot 36 \cdot 1.97 = 42.6$ kip > 28.9 kip

Check plate for shear rupture:

$A_{nv} = (L - n(d_h + 1/8))t = (5.25in - 2(3/4 + 1/8in))\ 3/8in = 1.31\ in^2$

$\phi V_n = \phi\, 0.6 F_u A_{nv} = 0.75(0.6(58)(1.31\ in^2)) = 34.3$ kip > 28.9 kip

Check block shear:

$A_{nt} = (l_{ev} - \frac{1}{2}(d_h + 1/8))t_w = (1in - \frac{1}{2}(7/8in))\ 3/8in = 0.21in^2$

$A_{gv} = L t_w = 3.75in \cdot 3/4in = 1.41in^2$

$A_{nv} = (L - (n - 0.5)(d_h + 1/16))t_w = (3.75in - 1.5(7/8in))(3/8in) = 0.91in^2$

Girder to column connection

Determine tension rupture

$F_u A_{nt} = 58(0.21 in^2) = 12.18 kip$

Check shear yield versus shear rupture

$0.6 F_y A_{gv} = 0.6(36)(1.41 in^2) = 30.5 kip$

$0.6 F_u A_{nv} = 0.6(58)(0.91) = 31.7 kip$

$\Phi R_n = 0.75(30.5 kip + 12.18 kip) = 32 kip > 28.9 kip$

Flange to girder weld:

use CJP welds with E70

Plate to Girder weld

using a fillet weld on each side of the plate

$D = \frac{28.9}{(2(1.392)(5.25))} = 1.98$ sixteenths of an in.

$\therefore$ use a 3/16 in weld

Design of Beam to Girder Connection:

Beam to Girder connection

Coped beam with double angle

Number of bolts required:

Using 3/4" A325-N Bolts

$\phi R_n = 17.9$ kip

due to double shear: $n_s \phi R_n = 2(17.9\text{ kip}) = 35.8$ kip

number of bolts required: $N = \frac{R_u}{n_s(\phi R_n)} = \frac{7.35\text{ kip}}{35.8} = 0.21$ ∴ 1 bolt

Bolt strength at holes in Beam Web:

W8X35, $t_w = 0.31$ in, 1 in cope, 3 1/2 X 3 1/2 X 3/8 A36 Angle that's 4" Long

$L_c = L_{ev} - \frac{1}{2} d_h = 2\text{ in} - 0.5(3/4\text{ in} + 1/8\text{ in}) = 1.56\text{ in} > 2(3/4") = 1.5\text{ in}$

$\phi R_n = 0.75 \cdot 2.4\, d\, t\, F_u = 0.75 \cdot 2.4(3/4\text{ in})(0.31\text{ in})(65\text{ ksi}) = 27.2$ kip

$\phi R_n = 27.2\text{ kip} > 6.125\text{ kip}$

Bolt Strength at holes in Angle Leg:

$L_c = L_{ev} - \frac{1}{2} d_h = 2\text{ in} - \frac{1}{2}(\frac{3}{4}\text{ in} + 1/8\text{ in}) = 1.56\text{ in} > 2(3/4") = 1.5\text{ in}$

$\phi R_n = 0.75 \cdot 2.4\, d\, t\, F_u = 0.75 \cdot 2.4(3/4\text{ in})(3/8\text{ in})(58\text{ ksi}) = 29.3$ kip

29.3 kip > 17.9 kip ∴ bolt shear will control and meets requirements

Bolt strength at supporting member

Since the beam portion of the connection is controlled by bolt shear, the same will be for the girder and it will meet requirements

Check shear yield of the beam at the cope:

$\phi V_n = \phi\, 0.6\, F_y\, h\, t_w = 1 \cdot 0.6 \cdot 50 \cdot (5\,3/4 - 1\text{ in}) \cdot 0.31\text{ in} = 44\text{ kip} > 6.125\text{ kip}$

Check shear rupture of the beam at the cope

$\phi V_n = \phi\, 0.6\, F_u A_e = 0.75 \cdot 0.6 \cdot 65\text{ ksi} \cdot (5\,3/4\text{ in} - 1\text{ in} - 7/8\text{ in}) \cdot 0.31\text{ in} = 35\text{ kip} > 6.125\text{ kip}$

Check beam for block shear:

$A_{nt} = (L_{eh} - \frac{1}{2}(d_h + 1/8))t_w = (1.75\text{ in} - 0.5(3/4\text{ in} + 1/8) \cdot 0.31\text{ in} = 0.41\text{ in}^2$

$A_{gv} = L t_w = 4\text{ in}(0.31\text{ in}) = 1.24\text{ in}^2$

beam to girder connection

$A_{nv} = (L-(n-0.5)(d_h+1/8))t_w = (4in - 0.5(3/4in + 1/8)) \cdot 0.31in = 1.1 in^2$

$F_u A_{nt} = 65(0.41in^2) = 26.7kip$

$0.6F_y A_{gv} = 0.6(1.24in^2)(50) = 37.2kip$

$0.6 F_u A_{nv} = 0.6(65)(1.1) = 42.9kip$

$\phi R_n = 0.75(26.7kip + 42.9kip) = 52.2kip > 6.125kip$

check flexural strength of the coped beam:

$M_u = R_u e = 6.125kip \cdot 5in = 30.625 kip\cdot in$

$\lambda = h_c/t_w = 4.75in/0.31in = 15.3$

$\lambda_p = 0.46\sqrt{\frac{k_1 E}{F_y}} = 0.46\sqrt{\frac{4.72 \cdot 29000}{50}} = 24.1 > 15.3 \therefore M_n = M_p$

$\phi M_n = 0.9 \cdot F_y \cdot Z_{net} = 0.9 \cdot 50 \cdot 16.1in^3 = 726.3 kip\cdot in > 30.625kip\cdot in$

$Z_{net} = 16.1in^3$

Angles:

check for shear rupture:

$A_{nv} = (l - n(d_h + 1/8))t_a = (4in - 1(3/4in + 1/8in))3/8in = 1.17in^2$

$\phi V_n = 2(\phi 0.6 F_u A_{nv}) = 2(0.75(0.6)(58)(1.17)) = 61kip > 6.125kip$

check for shear yield:

$A_{gv} = l \cdot t_a = 4in \cdot 3/8in = 1.5in^2$

$\phi V_n = 2(\phi 0.6 F_y A_{gv}) = 2(1 \cdot 0.6 \cdot 36 \cdot 1.5) = 64.8 > 6.125kip$

check for block shear:

$A_{nt} = (l_{eh} - \frac{1}{2}(d_n + 1/8))t_w = (1.75in - 0.5(3/4in + 1/8))(3/8in) = 0.41in^2$

$A_{gv} = Lt_w = 1.24in^2$

$A_{nv} = (L-(n-0.5)(d_n+1/8))t_w = (4-0.5(3/4in+1/8)) \cdot 0.31 = 1.1in^2$

$F_u A_{nt} = 58(0.41in^2) = 23.78kip$

$0.6F_y A_{gv} = 0.6(1.24in^2)(36) = 26.8kip$

$0.6 F_u A_{nv} = 0.6(58)(1.1in^2) = 38.3kip$

$\phi R_n = 2(0.75(26.8 + 23.78)) = 75.9kip > 6.125kip$

Design of Column Base Connection:

Column to Ground

Axial load: 12.83 kip

Moment: 57.8k·ft

trial baseplate size:

$N > d + (2)(3.0\,in) = 8.12in + 2\cdot 3.0in = 14.12in$

$B > b_f + (2)(3.0\,in) = 8.02in + 2\cdot 3.0in = 14.02in$

Determine e and e_{crit}

$$e = \frac{M_u}{P_u} = \frac{57.8k\cdot ft}{12.83k} = 4.52ft \cdot 12in/ft = 54.2in$$

$$f_{p(max)} = \phi_c (0.85 f'_c)\sqrt{\frac{A_2}{A_1}}$$

$$= 0.65(0.85)(4)(1)$$

$$= 2.2ksi$$

$$q_{max} = f_{pmax} \cdot B$$

$$= 2.2\,ksi \cdot 14.02in = 30.84\,kip/in$$

$$e_{crit} = \frac{N}{2} - \frac{P_u}{2q_{max}} = \frac{14.12in}{2} - \frac{12.83kip}{2\cdot 30.84kip/in} = 6.85in$$

$e > e_{crit}$

$$f = \frac{N}{2} - 1.5 = \frac{14.12in}{2} - 1.5in = 5.56in$$

$$\left(f + \frac{N}{2}\right)^2 = \left(5.56in + \frac{14.12in}{2}\right)^2 = 159.3in^2$$

$$\frac{2P_u(e+f)}{q_{max}} = \frac{2(12.83kip)(54.2in + 5.56in)}{30.84kip/in} = 49.7in^2 < 159.3in^2$$

bearing Length → $Y = \left(f + \frac{N}{2}\right) \pm \sqrt{\left(f+\frac{N}{2}\right)^2 - \frac{2P_u(e+f)}{q_{max}}} = 12.62in \pm \sqrt{159.3in^2 - 49.7in^2} = 2.62in$

bolt pretension → $T_u = qY - P_u = (30.84)(2.62in) - (12.83) = 68kip$

plate thickness:

$$m = \frac{N - 0.95d}{2} = \frac{14.12in - 0.95(8.12)}{2} = 3.2in$$

$$Y \le m \therefore t_{p(req)} = 2.11\sqrt{\frac{f_p Y(m - Y/2)}{F_y}}$$

Column to Ground

$$t_{p\,(req)} = 2.11\sqrt{\frac{2.2\,ksi \cdot 2.62\,in\left(3.2\,in - \frac{2.62}{2}\right)}{36}} = 1.16\,in$$

at tension interface:

$$X = \frac{N}{2} - \frac{d}{2} - 1.5 = \frac{14.12}{2} - \frac{8.12}{2} - 1.5 = 1.5\,in$$

$$t_{p\,(req)} = 2.11\sqrt{\frac{T_u X}{B F_y}} = 2.11\sqrt{\frac{68\,kip \cdot 1.5\,in}{14.02\,in \cdot 36\,ksi}} = 0.95\,in$$

check the value of n

$$n = \frac{B - 0.8 b_f}{2} = \frac{14.02\,in - 0.8 \cdot 8.02\,in}{2} = 3.8\,in$$

$$t_{p\,req} = 2.11\sqrt{\frac{f_{py}(n - Y/2)}{F_y}} = 2.11\sqrt{\frac{2.2\,ksi \cdot 2.62\,in\left(3.8\,in - \frac{2.62}{2}\right)}{36}} = 1.33\,in$$

the 1.33 in controls, use 1.5 in-plate

use rod size 1" of grade 36 with a hole diameter of 1 7/8 in, washer sizes of 3 in and 3/8 in thick.

try an embedment of 12 in

$$\phi N_{cbg} = \phi \Psi_3\, 16\sqrt{f'_c}\, h_{ef}^{5/3}\left(\frac{A_N}{A_{No}}\right) = 0.7 \cdot 1.25 \cdot 16\sqrt{4000}\,(12)^{5/3}\left(\frac{84\,in^2}{432}\right)$$

$$= 111.4\,kip\ \checkmark$$

Girder to Girder Connection:

Girder to Girder connection

Extended End-Plate moment connection

$M_{pe} = 1.1 R_y F_y Z_x = 1.1 \cdot 1.1 \cdot 50 \cdot 69.6 = 4210.6 k \cdot in$

$L_p = \min \begin{cases} d/2 = 13.7/2 = 6.85 in & \leftarrow \text{use this one} \\ 3b_f = 3(8 in) = 24 in \end{cases}$

$M_{uc} = M_{pe} + V_u L_p = 4210.8 kip \cdot in + 12.27 kip \cdot 6.85 in = 4294.8 kip \cdot in$

trial values:

$b_p = b_f + 1 = 8 in + 1 = 9 in$

$g = 5\tfrac{1}{2} in$

$p_{fi} = 2 in$
$p_{fo} = 2 in$
$d_e = 1\tfrac{5}{8} in$

$F_{yp} = 50 ksi$
$F_{up} = 65 ksi$

$F_t = 90 ksi$ (A-325 bolts)

$h_o = 13.7 + 2 - \frac{0.53}{2} = 15.44 in$

$h_1 = 13.7 - 0.53 - 2 - \frac{0.53}{2} = 10.91 in$

Required bolt diameter:

$d_{b\,req} = \sqrt{\frac{2 M_{uc}}{\pi \phi F_t (h_o + h_1)}} = \sqrt{\frac{2(4294.8 kip \cdot in)}{\pi (0.75)(90)(15.44 + 10.91 in)}} = 1.24 in$ try $1\tfrac{3}{8} in$

bolt tensile strength:

$P_t = F_t A_b = 90 ksi \cdot 1.48 in^2 = 133.6 kip$

$M_{np} = 2P_t (h_o + h_1) = 2(133.6 kip)(15.44 in + 10.91 in) = 7032.7 kip \cdot in$

$\phi M_{np} = 0.75 \cdot 7032.7 kip \cdot in = 5274.5 > 4294.8 kip \cdot in$

Determine End plate thicknesses:

$s = \frac{1}{2}\sqrt{b_p g} = \frac{1}{2}\sqrt{9 in (5.5)} = 3.52 in > 2 in$

$Y_p = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{fi}} + \frac{1}{s}\right) + h_o\left(\frac{1}{p_{fo}}\right) - \frac{1}{2}\right] + \frac{2}{g}\left[h_1(p_{fi} + s)\right]$

$= \frac{9}{2}\left[15.44 in\left(\frac{1}{2 in} + \frac{1}{3.52 in}\right) + 10.91 in\left(\frac{1}{2 in}\right) - \frac{1}{2}\right] + \frac{2}{5.5}\left[15.44 in(2 in + 3.52 in)\right]$

Girder to Girder

$= 107.8 in$

$t_{preq} = \sqrt{\frac{1.11\,\phi M_{np}}{\phi_b F_{yp} Y_p}} = \sqrt{\frac{1.1(5274.0)}{0.9(50)(107.8)}} = 1.09 in$ ∴ use $1\frac{1}{4}$in (A572 Gr. 50 steel)

Check shear yielding of extended portion of endplate:

$\phi R_n = 0.9(0.6 F_{yp}) b_p t_p = 0.9(0.6)(50)(9)(1.25) = 304$ kips

Calculate factored beam flange force:

$F_{fu} = \frac{M_{uc}}{(d_b - t_{fb})} = \frac{4294.8 \text{ kip·in}}{13.7 - 0.53} = 326$ kips

$\frac{F_{fu}}{2} = 163$ kips $\leq \phi R_n$ ∴ 304 kips is ok

Check shear rupture:

$A_n = [b_p - 2(d_b + 3/16)]t_p = [9.0 - 2(1\frac{3}{8} + 3/16)]\,1.25 = 7.34 in^2$

$\phi R_n = 0.75(0.6 F_{uf}) A_n = 0.75 \cdot 0.6 \cdot 65 \cdot 7.34 in^2 = 214.7$ kip

163 kip $\leq \phi R_n$ ∴ 214.7 kip, ok

Check compression bolts shear rupture strength:

$V_u = 12.27$ kip $\leq \phi R_n = \phi n_b F_v A_b = 0.75 \cdot 4 \cdot 54 \cdot 1.48 in^2 = 239.8$ ksp

$V_u \leq \phi R_n$ ∴ 239.8 kips ok

Check compression bolts bearing/tearout:

Endplate:

Bearing strength $= 2.4 d_b t_p F_u = 2.4 \cdot 1.375 \cdot 1.25 \cdot 65 = 268.1$ kip

Tearout outer bolts:

$L_c = (2.0 + 0.53 + 2.0) - (1.375 + 1/16) = 3.1$ in

$R_n = 1.2 L_c t_p F_u = 1.2 \cdot 3.1 \cdot 1.25 \cdot 65 = 302.3$ kip > 268.1 kip

$\phi R_n = 4 \cdot 0.75 \cdot 268.1$ kip $= 804.3$ kip $> V_u$

Design welds:

Beam flanges to end plate:

CJP welds with 5/16 backing fillets that are back gouged and AWS TC-U4b-GF

Girder to Girder

for the inside of the flange

Beam web to End-plate weld:

$$D = \frac{0.6\ F_{yb}\ t_{wb}}{2(1.392)} = \frac{0.6 \cdot 50 \cdot 0.305}{2 \cdot 1.392} = 3.29$$

use 3/8 in fillet weld

$$\frac{d_b}{2} - t_{fb} = 6.32$$

$$D = \frac{12.27}{2(1.392)(6.32)} = 0.687$$

use 3/8 in fillet weld

13.4.3 Hand Calculations - Design of Awning

Design of Column and Beam:

Beam design awning

Load 60.89 lb/ft^2

Spacing 17.5 ft

$60.89 \times 17.5 = 1065.5$ lb/ft

$w = 1.065$ k/ft

$\frac{wL^2}{8} = \frac{1.065 \times 20^2}{8} = 53.25$ k.ft

$V = \frac{wL}{2} = \frac{1.065 \times 20}{2} = 10.65$ kip

$F_y = 50$

$A_{req} = \frac{1.065 \times 12}{0.9F_y} = 0.284$ in^2

$L/240 = \frac{20}{240} = 0.083 = 1$ in

Try W8 × 40

$A_w = T \cdot t_w = 5.75 \times 0.36 = 2.07$ in^2

$V_n = 0.6 F_y A_w C = 0.6 \times 50 \times 1.44 \times 1 = 62.1 > 10.65$ kip ok

$\Delta_{max} = \frac{5wL^4}{384EI} = \frac{5 \times 1.065 \times (20 \times 12)^4 \times \frac{1}{12}}{384 \times 29000 \times 146} = 0.90$ in < 1 in ok

column design

wind load $30 \times \cos 21.8 = 27.8$ kips

$M = \frac{PL}{8} = \frac{27.8 \times 16}{8} = 55.6$ k ft

$V = \frac{P}{2} = 13.9$ kip

W 10 × 26

$\phi M_n = 66.4 > 55.6$ k.ft

$\phi V_n = 80.3 > 13.9$ kips

Design of Girder:

girder design

$(60.89 \times 5) \times 17.5 = 5327.875$ lb/ft $= 5.32$ k/ft

point load

$V = \frac{Pb}{L} = \frac{5.32 \times 20}{20} + \frac{5.32 \times 15}{20} - \frac{5.32 \times 10}{20} = 6.65$ kip

$m_{max} = M_{max} = 33.3$ kip ft

$M_{A\,1/4} = 20$ kipft

$M_{B\,1/2} = 33.3$ kipft

$M_{C\,3/4} = 24$ kipft

$C_b = \frac{12.5 M_{ax}}{2.5 M_{max} + 3M_A + 4M_B + 3M_C} = 1.19$

Try W8 x67

$A_w = T \times t_w = 5.75 \times 0.57 = 3.2$

$V_n = 0.6 \times 50 \times 3.2 \times 1 = 96 > 6.65$ ok

$\frac{L}{240} = 1.7$ in

$D_{max} = 1.3 < 1.7$ ok

Design of Connection Beam to Column:

Design a direct-welded to column moment connection

The beam W8X40

$d = 8.25 in$ $t_w = 0.360 in$ $Z = 39.8 in^3$

$b_f = 8.07 in$ $t_f = 0.560 in$

The column W10X26

$d = 10.3 in$ $t_w = 0.260 in$

$d_f = 5.77 in$ $t_f = 0.440 in$

$\phi M_n = \phi M_p = \phi F_y Z = \frac{0.9 \times 50 \times 39.8}{12} = 149.25 > 108.8$ ft-kips

$\phi r_n = 17.9$ kips 3/4 in A325 N bolt

$n = \frac{24.55}{17.9} = 1.37$

try two-bolt connection with spacing 2.5 in and end distances 1.25 in $L = 5$ in which is greater than $T/2 = 2.8$ in

try $t = 3/8$

bolt bearing strength

$L_c = L_{ev} - \frac{1}{2} d_h = 1.25 - \frac{1}{2}(3/4 + 1/16) = 0.84 < 1.25$

$R_n = 1.2 L_c t F_u = 1.2 \times 0.84 \times 0.375 \times 58 = 22.18$ kips

$\phi R_n = 0.75 \times 22.18$ kip $= 16.6$ kip

$l_c = s - d_h = 2.5 - (3/4 + 1/16) = 1.68 > 1.25$

$R_n = 2.4 d t F_u = 2.4(3/4)(0.375)(58) = 39.2$ kips

$\phi R_n = 29.4$ kips

strength two bolt connection

$\phi R_n = 2(17.9) = 35.8 > 24.55$ kips

check plate for shear yield

$A_{gv} = tL = 0.375(5) = 1.875 in^2$

$\phi V_n = \phi\, 0.6 F_y A_{gv} = 1 \times 0.6 \times 36 \times 1.875 = 40.5 > 24.55$ kips check the plant for shear rupture

$A_{nv} = (L - n(d_n + 1/16))t = (5 - 2(3/4 + 1/8))(0.375) = 1.2 \text{ in}^2$

$\phi V_n = \phi\, 0.6 F_u A_{nv} = 0.75 \times 0.6 \times 58 \times 1.2 = 31.32 > 24.55$ kips

Check. shear block

$A_{nt} = 1.25 - 1/2(3/4 + 1/8)(0.375) = 0.305 \text{ in}^2$

$A_{gv} = Lt_w$
$= 3.75 \times 0.375 = 1.4 \text{ in}^2$

$A_{nv} = (3.75 - 2.5(3/4 + 1/8))(.375) = 0.585 \text{ in}^2$

$F_u A_{nt} = 58 \times 0.305 = 17.7$ kips

$0.6 F_y A_{gv} = 0.6(36)(1.4) = 30.24$ kips
$0.6 F_u A_{nv} = 0.6(58)(0.585) = 20.358$ kips
$\phi R_n = 0.75(30.24 + 20.358) = 37.9 > 24.55$ kips

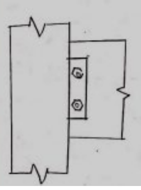
plate to column weld

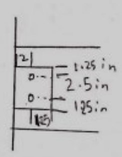
$$D = \frac{24.55}{(2(1.392)(5))} = 1.76$$

use 3/16 in weld, minimum weld for 3/8 in plate

Final design

two 3/4 in A325-N bolts in 3/8 x 2.5-1/2 x 1'-0 plate.





Design of Connection Girder to Column:

Design a direct-welded girder to column connection

girder W8X67

$d = 9$ in $t_w = 570$ $Z = 70.1\ in^3$

$d_f = 8.28$ in $t_f = 0.935$

Column W10X26

$d = 10.3$ in $t_w = 0.260$ in

$d_f = 5.77$ in $t_f = 0.440$ in

$\phi M_n = \phi M_p = \phi F_y Z = \dfrac{0.9 \times 50 \times 70.1}{12} = 262.875 > 89$ ft-kips

$\phi r_n = 17.9$ kips 3/4 in A325N bolt

$n = \dfrac{20.55}{17.9} = 1.148$ try two bolt connection with spacing 2.5 in and end distances 1.25 in which is grater than $T/2 = 2.8$ in

$L = 5$ try plate $t = 3/8$

$L_c = l_{ev} - 1/2\, d_h = 1.25 - 1/2(3/4 + 1/16)\ 0.84 < 1.25$

$R_n = 1.2 L_c t F_u = 1.2(0.84)(0.375)(58) = 39.2$ kips

$\phi R_n = 29.4$ kips

$L_c = S - d_n = 2.5 - (3/4 + 1/16) = 1.68 > 1.25$

$R_n = 2.4\, d t F_u = 2.4(3/4)(0.375)(58) = 39.2$ kips $\phi R_n = 29.4$ kips

For two bolt connection

$\phi R_n = 2 \times 17.9 = 35.8 > 20.55$ kips

Check the plate for shear yield

$A_{gv} = tL = 0.375 \times 5 = 1.875\ in^2$

$\phi V_n = 0.75 \times 0.6 \times 58 \times 1.875 = 40.5 > 20.55$ kips

check plate for shear rupture.

$A_{nv} = (L - n(d_h + 1/16)t = 5 - 2(3/4 + 1/8))(0.375) = 1.2$ in

$\phi V_n = \phi 0.6 F_u A_{nv} = 0.75 \times 0.6 \times 58 \times 1.2 = 31.32 > 20.55$ kips

Check block Shear

$A_{nt} = (L_{eh} - 1/2\,(d_h + 1/16))\,t_w$

$= 1.25 - 1/2\,(3/4 + 1/8) \times 0.375 = 0.305\ in^2$.

$A_{gv} = L t_w$

$= 3.75 \times .375 = 1.4\ in^2$

$A_{nv} = (L - (n - 0.5)(d_h + 1/16))\,t_w$

$= 3.75 - 3(3/4 + 1/8) \times 0.375 = 0.42\ in^2$

$F_u A_{nt} = 58 \times 0.305 = 17.7$ kips

$0.6\,F_u A_{nv} = 0.6 \times 58 \times 0.42 = 14.616$ kips

$0.6\,F_y A_{gv} = 0.6 \times 36 \times 1.4 = 30.24$ kips

$U_{bs} = 1.0$

$\phi R_n = 0.75(14.616 + 30.24) = 44.856 > 20.55$

plate to column weld

1.392 kips per 1/16 in of weld per in of length

$D = \dfrac{20.55}{(2(1.392)(5))} = 1.476$

use 3/16 in weld, minimum weld for the 3/8 in plate

Final design

two 3/4 in A325-N bolt in 3/8 x 2.5 -1/2 x 1'-0 plate

Design of Connection Girder to Beam:

Design shear tab connection girder to beam

W8X40 beam A36

W8X67 girder A992

two 3/4 A325N bolts

bolt shear strenght

$\phi r_n = 17.9$ kip/bolt

$e = \frac{a}{2} = \frac{2.5}{2} = 1.25$

$L_c = l_{ev} - \frac{1}{2} d_h = 1.25 - \frac{1}{2}(3/4 + 1/16) = 0.84 < 1.25$

$R_n = 1.2 L_c t_p F_u = 1.2(0.84)(0.250)(58) = 14.616$ kips

$\phi R_n = 10.9$ kips

$L_c = 2.5 - h = 2.5 - (3/4 + 1/16) = 1.68 > 1.25$ in

$R_n = 2.4\, d\, t_p F_u = 2.4 \times 3/4 \times 0.250 \times 58 = 26.1$ kips

$\phi R_n = 14.6$ kips

bolt bearing and tearout strength

A992 thickness is 0.360

$R_n = 2.4\, d\, t_w F_u = 2.4 \times 3/4 \times 0.360 \times 65 = 42.12$ kips

$\phi R_n = 31.59$ kips control

block shear strength

$A_{nt} = (L_{eh} - \frac{1}{2}(d_n + 1/16)) t_w$

$= (1.25 - \frac{1}{2}(3/4 + 1/8))\, 0.250 = 0.203\ in^2$

$A_{gv} = L t_w$

$= 3.75 \times 0.250 = 0.9375\ in^2$

$A_{nv} = (3.75 - 3(3/4 + 1/8))(.250) = 0.28\ in^2$

$F_u A_{nt} = 58(0.203) = 11.77$ kips

$0.6 F_y A_{gv} = 0.6 \times 36 \times 0.9375 = 20.25$

$0.6 F_u A_{nv} = 0.6 \times 58 \times 0.28 = 9.744$

$\phi R_n = 0.75(20.25 + 1.0(11.77)) = 24$ kips

Connection Strength based on the bolts in holes

The bottom bolt

$\phi R_n = 14.3$ kips

The othr bolt

$\phi R_n = 17.9$ kips

$\phi R_n = 14.3 + 17.9 = 32.2$ kips

design shear yield strength

$A_{gv} = L t_p = 5 \times 0.250 = 1.25\ in^2$

$\phi V_n = \phi .6 F_y A_{gv} = 1 \times 0.6 \times 36 \times 1.25 = 27$ kips

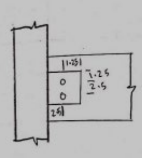
design shear rupture

$A_{nv} = (L - n(d_n + 1/16)) t_p = (5 - 2(3/4 + 1/8))(0.250) = 0.8125\ in^2$

$\phi V_n = \phi .6 F_u A_{nv} = 0.75(0.6(58)(0.8125)) = 21.2$ kips

$t_{min} = \frac{3.09 D}{F_u} = \frac{3.09 \times 2.5}{65} = 0.12$ in

$\phi R_n = 24$ kips



Design of Column Base Plate Connection:

Column W10x26

$d = 10.3\,in$

$b_f = 5.77\,in$

$f'_c = 4\,ksi$

$N > d + 2 \times 3\,in = 16.3$

$B > d_f + 2 \times 3\,in = 11.77$

$N = 17$

$B = 12$

$e = \frac{M_u}{P_u} = 4\,in$

$f_p = \phi_c (0.85 f'_c) \sqrt{\frac{A_2}{A_1}} = 0.65 \times 0.85 \times 4 \times 1$

$= 2.21\,ksi$

$q = f_p \times B$

$= 2.21 \times 12 = 26.52\,kip/in$

$e_{crit} = \frac{N}{2} - \frac{P_u}{2q}$

$= 1/2 \times [17 - 13.9/26.52]$

$= 8.23\,in$

$e < e_{crit}$

Determine length Y

$Y = N - 2e = 17 - 2 \times 4 = 9\,in$

$q = \frac{P_u}{Y} = 1.54\,kips\,in < 26.52$ ok

$m = \frac{N - 0.95d}{2} = \frac{17 - 0.95 \times 10.3}{2} = 3.67\,in$

$f_p = \frac{P_u}{BY} = 0.068\,ksi$

$t_p = 1.5m \sqrt{\frac{f_p}{F_y}}$

$= 1.5 \times 3.67 \sqrt{\frac{0.068}{36}}$

$\simeq 1$

Check thickness

$n = \frac{B - 0.8b_f}{2} = \frac{12 - 0.8 \times 5.77}{2} = 3.69\,in$

$t_{p_{req}} = 1.5 \times 3.64 \sqrt{\frac{0.068}{36}}$

$\simeq 1\,in$

use a base plate 1 1/2" x 17 x 1'-7"

four 3/4 in diameter rods ASTM F1554 Grade 56 rod length

13.5 - Team Management Plan

Team Name - Undergraduate Engineering and Preservation

Team Members

Frances Boyd is a senior civil engineering student at Oklahoma State University. After graduation, she will begin working remotely for Olsson, Inc. She has five years of work experience from two previous internships - one with Olsson, Inc as a general civil intern and one with R.L. Shears Company as a landscape architect technician. Both internships have contributed to proficiency in AutoCAD Civil 3D, grading design, and other general land development design.

Gracie Fink is a senior at Oklahoma State University. Her plans after graduation include starting her career working for POWER Engineers in their Fort Worth office. She completed two internships with POWER Engineers where she helped design multiple overhead transmission lines. She is proficient in PLSCadd and has a lot of experience working in a team setting.

Justin Hoppe is a senior in civil engineering at Oklahoma State University graduating in May of 2022, and will be working with Tanner Consulting in Tulsa. He has previous experience as an undergraduate researcher working with steel connections, which has given more insight into structural design. On top of structural design, he has other interests in geotechnics. Some of the skills he has are being concise while thinking and not stopping until the task is done.

Ali Almutairi is a senior at Oklahoma State University in civil engineering and graduating in May of 2022. He will be working for Kuwait Ministry of Electricity and Water. His previous experience as an undergraduate student include steel design, structural analysis and concrete.

Leadership Plan

- **Manager** is the person that makes sure everyone is staying on task and will be the primary mediator of any issues the group may face. The team member filling this role will be Gracie.
- **Coordinator** is the person who will be in direct contact with our clients and school contacts. The team member filling this role will be Ali
- **Editor** will be in charge of making sure all of our submissions are cohesive and have no grammatical errors. They will make sure papers and slideshows are presented as one document. The team member filling this role will be Frances.
- **Secretary** will be in charge of taking notes during meetings and class periods. They will be in charge of collecting all the ideas we come up with and keeping them organized. The team member filling this role will be Justin.

Communication Plan

Our communication plan is based on ensuring everyone will know what is happening by using One Note to keep track of our schedule and what tasks need to be done. On top of the One Note, every Monday we will have a group discussion about what each member has accomplished over the weekend and what we still need to work on throughout the week. We plan on being in person in class and can meet in zoom in case we need to do extra work as a team or if someone becomes sick. In the case of someone becoming sick, they would need to alert the team so we can meet over zoom at the beginning of class and participate during class time.

Meeting Schedule

We have identified that the group can meet outside of class on Tuesday nights at 7:00PM over zoom. This will be a time to discuss any issues/concerns we may have but we plan to address the bigger issues during class time when we can meet in person.

Preliminary Team Goals -

- The first goal for the team is to communicate with all members of the team to complete the project. We have arranged to utilize all meeting times to ensure a proactive approach when counseling our client, and plan to communicate continuously.
- The second goal for the team is to create an inclusive environment for teamwork and teambuilding.
- The third goal for the team is to apply all the knowledge that we have learned from our education at Oklahoma State University as a civil engineering student.
- The fourth goal for the team is to improve our presentation skills by practicing before the presentation day to make sure all team members feel confident.
- The fifth goal for the team was to design proposals that would encompass the needs of our client when evaluating several alternatives before the final design.

Tasks and Milestone Plan

Tasks will be laid out in the OneNote and everyone can see the progress of all tasks. We will evaluate tasks assigned at the beginning of every week and make adjustments as needed. Every Monday we will provide an update to the team of where we are on our tasks. If a problem of missing deadlines is reoccurring, we will have a group meeting to discuss these issues face to face. We will be understanding of life getting in the way of deadlines but each member will need to come to the group in advance with missed deadlines and a plan of how they will accomplish their task.

Team Vision

As a group we will aim to keep open and continuous lines of communication with our campus partners in an effort to preserve the historical importance of the site. As a team we will be open to all ideas and support one another in achieving deadlines throughout the project.