White Oak Conservation Restoration

Mahaska County, Iowa

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Section I. Executive Summary

White Oak Conservation Center is a focal point for Mahaska County, Iowa that currently provides scenic trail routes and access to a 20-acre lake, amongst other amenities for residents and visitors to enjoy. To keep this area as a destination spot, our student engineering group has proposed rehabilitation services in various sectors for the betterment of the site. Our student group has expertise and applicable experience in different areas of civil engineering design, such as structural, transportation, land development and water resource design. This experience aided the team's ability to provide quality and innovative designs for site improvements. One of the major amenities of White Oak Conservation Center is the 20-acre lake which includes boat ramp access and is used for recreation and fishing. Along with the lake, the site has mowed walking paths with multiple pedestrian bridges across small streams, as well as pavilions and playgrounds that can be accessed via gravel roads. The conservation area is a great attraction to citizens of Mahaska County as well as visitors who enjoy the many activities and scenery this site has to offer.

The scope of our restoration project can be broken down by design element. The design elements our team worked on were as follows: an evaluation of the lake, new pavilions, new pedestrian bridges, enhanced roadways, new parking lots, an upgraded boat ramp, rehabilitated walking trails, and a new playground area. An overview of the design process and end products for each amenity on site can be found below.

1. Lake Evaluation

   The White Oak Conservation Center is home to a 20-acre lake that is currently having problems with water quality. It is surrounded by four satellite ponds and a south subbasin. We created a HEC-HMS model to evaluate the inflows for the main lake. From this we found that all the satellite ponds flow into the lake. Our key finding was that greater than 90% of inflow into the main lake comes from the south subbasin. This indicates that most of the sediment inflow for the main lake is coming from the south. Because of this we are recommending hydraulic dredging to solve the sedimentation issue and to improve water quality. Sediment removal is recommended in the south end of the lake and in the deepest portion of the lake, near the upper northwest of the lake. Proper and routine maintenance is also recommended for the south subbasin sediment collection structures and the satellite ponds to maintain effectiveness and prevent a future dredging need.

2. Pavilions

   Designs to replace the existing steel pavilions were completed to improve aesthetics and cohesion with the natural aspects of the site. The pavilions were designed to be made of Douglas Fir wood, with a metal roof decking. The roofs were designed to be double gable and valley, bearing on sheathing. The framing members supporting this roof are purlins spaced two feet on center, which are connected to girders. The girders and architectural trusses that are exposed by the gable roof design bear on columns which are supported by knee braces. This knee brace connection is essential for minimizing the effect of wind forces. Both pavilions will utilize the existing slab that vary in size, with the northern Cardinal pavilion on a 45'x25' slab and the southern

Robin pavilion on a 25'x'25' slab. A cost estimate was performed for both the Cardinal and Robin pavilions. Calculations were performed using ASCE load combinations and NDS design specifications.

3. Pedestrian Bridges

Three of the most used and structurally flawed bridges on the site were redesigned to allow for safe travel for pedestrians and service vehicles. The boat ramp bridge, southeast bridge, and primary inlet bridge were designed by expanding the bridge deck of a U.S. Forest Service typical pedestrian bridge to 8'-9", meeting ADA requirements as well as allowing for easy access and multi-direction travel. Supporting the Douglas Fir wood frame of the bridge are concrete abutments designed using a 10,000-pound design vehicle loading. While varying in length and construction constraints, each bridge was designed with the goal of creating an aesthetically pleasing, reliable, and efficient trail network around the site.

4. Roadways

The existing roadways on site were deteriorating due to a long service life and a lack of maintenance. The roadways needed to be redesigned to improve function and safety. The design of the roadways began with our team surveying the existing roadways centerlines and edges of pavement. Then, the software Pave Xpress was used to determine how thick the different proposed pavement options would need to be to provide adequate support for the roadways. Once these thicknesses were known, cross-sections for the three roadway surfacing alternatives were created (HMA, PCC, Gravel). These cross-sections were then applied to both horizontal and vertical alignments to create a 3D roadway corridor. All grades along the roadways were designed to meet the AASHTO Rural Local Road Standards. Following the completion of the roadway design, a cost estimate was performed for the three surfacing options.

5. Parking Lots

The existing parking areas on site had no distinct parking stalls and were deteriorating due to a long service life and a lack of maintenance. They needed to be redesigned to improve function and safety. The design of the parking lots began with our team surveying the existing parking areas edges of pavement. After surveying the site, the number of parking spots needed was estimated to be twenty-two with three of them being ADA accessible for vans. Once the number of parking spots was determined, two separate layouts were created, one matching the existing geometry of the site, and one proposing new geometry for the parking lot areas. These different layouts were sent to the client who made the choice to move forward with the new geometry option. Using the new geometry, the parking lots were graded to SUDAS Section 8 Standards. Following the completion of the parking lot design, a cost estimate was performed for the three surfacing options (HMA, PCC, Gravel).

6. Boat Ramp

The design of the boat ramp aimed to enhance functionality, safety, and accessibility by addressing accumulated sediment concerns. By evaluating equipment options to clear the accumulated gravel, we optimized watercraft launching and retrieval processes in compliance with U.S. Army Corps of Engineers standards. Utilizing a telescopic front loader, we can efficiently remove sediment and mitigate pavement damage risks. Accessibility improvements, including trailer parking integration, ensure access for most users. The enhanced boat ramp offers an improved experience for recreational boaters while preserving its existing location and structure.

7. Trails

    The evaluation of the existing trail system reveals areas in need of improvement, targeting safety concerns and drainage issues. Specific spots with poor drainage or steep slopes are identified as requiring immediate attention to ensure user safety. In response, ABA compliant alternatives are proposed for heavily trafficked regions to alleviate strain and prevent erosion. Furthermore, alternatives for the proposed trail extension were developed, aiming to enhance the overall trail system and provide additional recreational opportunities while maintaining environmental sustainability, durability, and a natural aesthetic.

8. Playgrounds

    Due to the deterioration of the playground near the southern Robin pavilion, the client expressed interest in replacement items that would add a variety of activities for all ages on the site that differ from the newly implemented playground near the Cardinal pavilion. The recommended design items to adhere to the client's interests are prefabricated outdoor games of concrete cornhole and concrete ladder toss from Doty & Sons Concrete Products. A cost estimate was performed for the implementation of these new activities as well as demolition of the current playground.

The total estimated construction cost for the entire project amounts to $1.17 million. The total cost has been delineated across three distinct phasing plans, each with certain tasks. Project funding will primarily be sourced from the client; however, financial support can be pursued through grants provided by the Iowa Department of Natural Resources. These grants include, but are not limited to, the Resource Enhancement and Protection (REAP) program, the Wildlife Habitat Stamp Fund program, and the Water Recreation Access Cost-Share program. Further information about phasing, each task's cost, and funding opportunities is detailed in section VII.

In conclusion, the design recommendations for each sector of the restoration were based on efficiency, cost, design creativity, and practicality. A comprehensive drawing set was created for all plans for the scope of the project. Further explanations and design details are outlined in the body of this report. Calculations and subsequent technical information can be found in the appendices at the end of the report.

Section II. Organization Qualifications and Experience

1. Name of Organization

   We are a senior design group of eight students enrolled at the University of Iowa in the course Project Design & Management (CEE:4850).

2. Organization Location and Contact Information

   Our team contacted the client bi-weekly via email to update them and highlight any completed work. Our teams project manager, Cody Hall, was accessible via email at cody-hall@uiowa.edu.

3. Organization and Design Team Description

   Our team was composed of several engineering students who were proficient in many different areas of civil engineering design. Because of this, we offered engineering services in many different disciplines. These included transportation design, structural design, foundation design, land development, water resource design, and hydraulic modeling. With our organization offering services in these fields, we felt qualified to perform the scope of work requested by the client at the Mahaska County Conservation Board. Our team consisted of Maya Johnson (Pavilion Design and Playground Design), Evan Felts (Parking Lot and Roadway Design), Noah Lyon (Hydraulic Modeling and Lake Evaluation), Justin Japlon (Boat Ramp Design), Cody Hall (PM, Bridge Design), Cory Siegel (Parking Lot and Roadway Design), Aden Gomez (Hydraulic Modeling and Lake Evaluation), and Beau Benzing (Recreational Trail Design).

Section III. Design Services

Project Scope

The primary objective of this design project was to enhance the Mahaska County Conservation area to make it an attractive public destination, provide effective and economical solutions for the Mahaska County Conservation Board, and offer alternative plans of action for their consideration.

Design Tasks

1. Lake Evaluation
   a. Obtained a comprehensive depth survey of the entire lake and pond system.
   b. Developed a model illustrating water flow dynamics throughout the system.
   c. Determined the volume of sediment to be removed and identify specific locations.
   d. Proposed means and methods for sediment removal.
   e. Identified suitable locations for depositing or utilizing removed sediment.
2. Pavilions
   a. Designed new pavilions at the north and south ends, considering materials and aesthetics.

   b. Computed load calculations using ASCE 7-22 and NDS specifications.
   c. Designed pavilions such that existing foundations are sufficient for new loading.
   d. Created a cost estimate for each new pavilion and demolition of existing pavilions.
3. Pedestrian Bridges
   a. Designed new pedestrian bridges to replace three existing bridges on the southeast side of the site.
   b. Adapted U.S. Forestry Service typical pedestrian bridge designs to meet project goals and intended bridge usage.
   c. Computed load calculations for the bridges using a H5 design vehicle (10,000 lbs).
   d. Designed bridge abutments to support loading and withstand water and earth pressures.
   e. Created a cost estimate for each bridge and demolition of existing bridges.
4. Roadways
   a. Obtained survey points of the existing road centerlines and edges of pavement.
   b. Determined pavement and base thicknesses using Pave Xpress for different material alternatives.
   c. Created 3 separate roadway cross-sections (HMA, PCC, Gravel).
   d. Built 3D corridors based on the existing road centerline for each alternative roadway cross-section while adhering to AASHTO Rural Local Roads Guidelines in I.M. 3.210.
   e. Created a cost estimate for each roadway alternative.
5. Parking Lot
   a. Obtained survey points for the existing parking lot edge of pavements.
   b. Estimated the number of parking spots needed on site based on our client's insight.
   c. Proposed two alternative layouts for our client, one matching the existing circular geometry of the parking lots, one changing the geometry to be rectangular.
   d. Moved forward with the new geometry layout and followed SUDAS Section 8 Standards for all design criteria.
   e. Created a cost estimate for the proposed parking lots for three surfacing options (HMA, PCC, Gravel).
6. Boat Ramp
   a. Modeled the existing boat ramp with the adjacent parking lot.
   b. Resolved issues related to gravel accumulation at the top and bottom of the ramp.
   c. Addressed the challenge of organized trailer parking.
7. Trails
   a. Evaluate the existing trail system, identifying areas in need of improvement.

    b. Assess trail drainage to address safety concerns in specific spots with poor drainage or steep slopes, develop complying alternatives to high use regions.
    c. Evaluate and develop alternatives for the proposed trail extension.
8. Playgrounds
    a. Developed alternative amenities, outdoor games, to replace existing playground on south end.

Work Plan

See Appendix A.

<u>Section IV.</u> Constraints, Challenges, and Impacts

1. Constraints

The project's parameters were defined by various restrictions and limitations, encompassing a diverse array of factors. These constraints can be broadly categorized into overarching constraints, as detailed below, and specific constraints tailored to each design element, outlined subsequently.

General: Many of the constraints encountered on this site were due to the client not wanting to relocate many of the existing design elements. This was to minimize cost as well as to keep the layout comparable to what was previously on site. Additional constraints included ensuring construction access to locations where work was to be performed, and the funding available to our client for this project.

Pavilions: When redesigning the existing pavilions, our team was constrained by the location and the dimensions of the existing slabs. The client wanted to reuse the slabs for the new pavilions to save on pavement costs.

Bridges: The locations of the bridges on this site needed to be accessible for the construction crews and their equipment. Due to the dense terrain, this was a major restriction.

Roadways and Parking Lots: When looking at the current roadways and parking lots, our team was constrained to match the existing locations of these roads and parking lots.

Boat Ramp: A major constraint when modeling the boat ramp was keeping the ramp in its existing location. Furthermore, there was not much room to design pull-through trailer parking in the adjacent parking lot.

Trails: The large trail system that runs around the project site was constrained by the existing locations as well as the durability of the existing material that was used to previously construct these trails.

2. Challenges

Numerous observations made by our team during both the initial and subsequent visits to the site highlighted potential challenges. These hurdles emerged notably during the project's preliminary design phase. Each design element faced its own set of challenges, alongside overarching design issues affecting all design elements.

General: All design elements experienced the challenge of hindered access for pedestrians during the construction phase of the project.

Lake Evaluation: The main lake at the center of the site lacked reliable data on sediment type. The satellite ponds which flow directly into the main lake lacked reliable elevation data. This led to multiple assumptions being made by our team about the storage-elevation relationships of these ponds and assumptions on how much sediment needed to be removed from the main lake.

Pavilions: When designing the pavilions, a major challenge came from the reuse of the concrete slab. Our team had to ensure that the loads experienced by the new pavilion would not exceed that of the existing pavilion. If these loads were greater than the existing loads, we would not have been able to use the existing slab as it would not provide adequate support for our structure.

Bridges: The replacement of the bridges on site posed several challenges. The first being the lack of geotechnical data our team had for the project location. This made it difficult to determine the true soil properties which led to a lot of assumptions used during the abutment design. The next challenge our team faced with the bridge design was the fact that two of the existing bridges had very low decks, leading to more difficulty with abutment design.

Roadways: Existing roadway slopes were unreasonably high in specific areas due to the long service life and minimal maintenance done on the roadways. This posed challenges with ensuring the roadways were graded to AASHTO and SUDAS typical roadway standards. An additional issue with the roadway design was ensuring that the daylighting area did not overlap any existing amenities or trees.

Parking Lots: The existing parking lots were designed to be near amenities such as the pavilions, restrooms, and the boat launch while lacking specified parking stalls. The main challenge our team encountered with the design of the new parking lots was maintaining the ease of access to these existing amenities while changing the geometric layout of the parking lots and defining clear parking stalls for vehicles.

Boat Ramp: Managing accumulated gravel at the boat ramp posed a significant challenge, requiring careful removal to avoid structural damage. Proactive strategies were needed to prevent future buildup and ensure compliance with regulations.

Trails: The terrain that the trails lie on is very steep at certain locations. This led to many challenges of meeting slope and run distance standards while maintaining the existing geometry of the trails. This terrain also made it difficult to ensure proper slopes on the portion of the trail our client requested be extended. Additionally, determining specific areas to resurface, what materials to resurface these areas with, and how runoff would affect these materials was particularly tricky.

Playgrounds: A major challenge we ran into when redesigning the playground areas was determining the location for the new amenities so that it could accommodate outdoor games such as cornhole and ladder toss. This was particularly difficult due to the locations of the existing amenities on site.

Phasing: A challenge experienced by our team while creating a phasing plan for the project was determining which items were deemed as high priority. Our team needed to know this to provide a proper recommendation to the client on what order the project should be completed in to ensure efficient and logical results.

3. Societal Impact within the Community and/or State of Iowa

Enhanced community quality of life and resilience due to improved community gathering spaces and outdoor recreational opportunities. This benefits the community's health and wellbeing by offering exciting opportunities for exercise and a location to hold events, promoting community involvement.

Increased wildlife diversity, habitat, and abundance due to water quality improvements for the lake and satellite ponds. In turn increasing potential for nature-based tourism and wildlife sports.

Section V. Alternative Solutions That Were Considered

To determine the most efficient designs for the restoration of White Oak Conservation Center, several techniques were used to analyze each section of the project. To produce the best possible product for our client, the following alternative solutions were evaluated and presented to the client throughout the duration of the project.

1. Lake Evaluation

During the lake evaluation the use of the RUSLE equation was considered in determining the amount of sediment gain by the lake on a typical annual basis. Due to insufficient data for the surrounding area, the complexity of the site, and being able to achieve the requested scope without this process; an alternative was used. This alternative accounted for the fish habitat and boating usefulness. There were also alternatives for type of dredging, each having their pros and cons. This decision will be left for the client to decide based on their storage capability, funding, and equipment availability.

2. Pavilions

For the pavilions, a variety of designs, sizes, and materials were considered. The location of the pavilions was predetermined based on the existing concrete slabs that are recommended to be reused, so ultimately it was determined for the new designs to match the sizes of the current pavilions to maximize the use of the slab rather than implementing a unique design. Multiple roof types were considered as well, keeping in mind cohesion with the aesthetics of the area and resilient material types. Ultimately, the hip and valley roof design are the recommended choice for durability concerns as well as allowing for exposed trusses as an interesting architectural feature.

3. Pedestrian Bridges

With the natural wooded area aesthetic in mind, each of the U.S. Forest Service typical bridge designs was considered. The bridges vary in size, materials, load bearing structure, and more. Each of the designs provided by the U.S. Forest Service are unique and work well in certain parts of the country. When selecting which would fit best on the site, many variables were considered. Of the variables considered, the intended use of the bridge, the construction limitations, the access to materials, and the overall cost weighed most heavily in the decision. Ultimately, due to adaptability of the design, the Sawn Timber Stringer Trail Bridge was selected.

4. Roadways

During our initial site visit, our client expressed an interest in replacing the existing gravel roads with paved roads while citing a lack of funding as a major constraint. Our team designed roadways to match the existing road geometry, while creating three separate surfacing alternatives. The first alternative was a 4” HMA surface, the second alternative was a 6” PCC surface, and the third alternative was a 1” Gravel resurfacing of the existing roads to correct any rutting or settling that occurred. Gravel resurfacing of the roadways was the cheapest alternative; however, it did not provide any improvements to the issue of large quantities of sediment being washed down to the base of the boat ramp. Paving the road surface with HMA or PCC would help resolve this sediment issue, but it came with a much higher material cost. Ultimately it is the client’s choice on what to pursue for construction, but our team recommends paving the roads using HMA surfacing. This is a cheaper alternative than using PCC surfacing and addresses the issue of sedimentation. Proper maintenance on the roadways is needed to maintain the appropriate function of the roadways.

5. Parking Lots

Our client expressed interest in improving the existing parking lot areas around the project site. These areas lacked specific parking stall locations. Our team designed two different alternatives for the geometry of the parking lots. The first alternative was meant to match the existing shape of the parking lots on site while adding in well-defined parking stalls. The second alternative expanded the parking lot areas and changed the shape of these areas while implementing well-defined parking stalls and sidewalks. Both alternatives were sent to our client, and he ultimately chose to move forward with the new geometry alternative. This alternative allocated more spots on site and included one-way access into 2 of the parking lots. The alternative for the new geometry of the parking lot was designed with three separate surfacing alternatives. These alternatives ultimately used the same pavement thicknesses as the roadways to account for the ease of construction. These alternatives include a 4” HMA surface, a 6” PCC surface, and a 1” Gravel resurfacing to level out any rutting or settlement that had occurred. Gravel surfacing would be the cheapest option but would not address the sedimentation issue occurring at the base of the boat ramp. Paving the lots using HMA or PCC would resolve the sedimentation issue but would come at a higher cost. Ultimately it is the client’s choice on what surfacing to use for construction, but our team recommends paving the parking lots using HMA surfacing and paving the sidewalks using PCC surfacing. This alternative is cheaper than using PCC surfacing and will help counter the sedimentation issue occurring at the south lot near the boat ramp. This will also ensure the ease of construction as tying in HMA parking lots into HMA roads will be much easier than using a different surfacing material. Proper maintenance of the parking lots is needed to maintain the appropriate function of the parking lots.

6. Boat Ramp

During our assessment of suitable equipment for the boat ramp maintenance, we explored various options. Initially, we examined the feasibility of using a skid steer. However, it became evident that a skid steer lacks the capability to reach into the water adequately, thus limiting its ability to remove all accumulated sediment in the transition area (base) of the boat ramp. Subsequently, we considered employing an excavator. However, the substantial weight of an excavator posed concerns regarding potential damage to the boat ramp pavement merely from its traversal. Moreover, there existed a risk of unintentional damage to the submerged portion of the boat ramp if the excavator operator was unaware of the slab's location beneath the sediment. Consequently, our recommendation is to utilize a telescopic front loader, or a telehandler with a bucket, for sediment removal. This equipment offers the necessary reach to effectively clean out sediment while minimizing the risk of pavement damage to the boat ramp.

7. Trails

When evaluating the trails, we considered drainage ways, elevations, the Architectural Barries Act (ABA) standards for hiking trails, United States Forest Service Recommendations, all while attempting to maintain a natural feel with the selected materials. It was important to us to maintain the current trail alignments but do our best to improve and redesign specific segments, which made way for our most viable alternative of mixed surface materials (wood chips, gravel, and mowed grass) that meets code in high use areas.

8. Playgrounds

To better engage the community with new amenities at White Oak, alternatives to a traditional playground were considered for the area near the southern pavilion. Rather than updating the existing playground to be of similar standards to the new playground by the northern pavilion, various outdoor games were considered instead. Using the Doty & Sons Concrete Games Products, it is recommended to implement a cornhole and ladder toss amenities as a playground replacement due to their popularity and ease of use for the community. Using the Doty & Sons Concrete Games Products, it is recommended to implement cornhole and ladder toss amenities as a playground replacement due to their popularity and ease of use for the community.

Section VI. Final Design Details

1. Lake Evaluation

Design Storm Evaluation: A HEC-HMS model was created incorporating the following data. The design storm evaluated was a 100-year, 24-hour storm. We utilized the SCS Type 2 precipitation model using NOAA Atlas 14 for depth evaluation, assuming uniform precipitation for all subbasins. Subbasin areas, CN values, and lag time were determined using the NRCS water lag method and data from StreamStats. The elevation-area relationship for the main lake was established using topographic survey data. While the satellite ponds elevation-area was calculated using a percent reduction method. Outlet elevations and slopes for discharge pipes were determined using GPS and changes in elevation. Emergency spillways were assumed to be a broad crested weir with a coefficient of 3. More assumptions and details can be found in Appendix B.

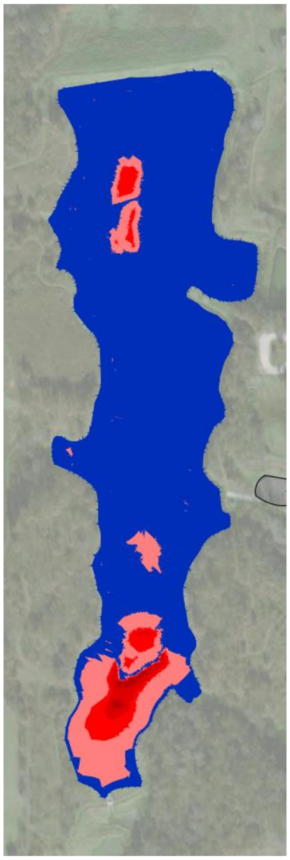


Figure 1: Lake Dredging Plan

Sediment Removal Plan: At a water surface elevation of 738 feet the lake has 810 square feet of area exceeding a 16' depth. To increase this area, 300 cubic yards of sediment needs to be removed from the current location. This is recommended to increase fish wintering ability. Additional dredging of 1,722 cubic yards is recommended in the south end of the lake. The majority of lake inflow occurs from the south subbasin, causing the accumulation of sediment there. Its removal would improve boating usefulness and increase summer habitat for wildlife. Therefore, 2,022 cubic yards of sediment must be removed. See Figure 1 for the graphical representation of sediment removal. In summary, increasing the area of 16' depth at the current deepest location would improve fish habitat in the winter season. Improving recreational activities such as fishing by deepening the south end, which is currently shallow and unusable for boating. Regular monitoring and assessment of the lake's condition post-sediment removal will be essential to ensure the desired outcomes are achieved.

2. Pavilions

Loading: Loads on the pavilion were calculated using ASCE 7-22. The risk category was determined to be category I based on its current and anticipated usage. The roof dead load was calculated based on a corrugated metal roof deck and 24/0 OSB sheathing. The snow load was determined based on the location in Cedar, Iowa. Wind loads were determined by analyzing the lateral force resisting system that are the moment frames created by use of knee braces between posts and trusses. These loads were used in ASD load combinations to determine the critical case for both lateral and vertical loading, keeping uplift in mind as an open-air structure.

Roof Framing: The roof framing consists of joists, runners, and trusses to support the double hip and valley roof design. All joists are to be 4x6 Douglas Fir, and girders are designed as 4x10 of the same material. Douglas Fir was chosen for its availability in the area, as well as its water resistance that will be important for an outdoor pavilion. Knee braces will be used as the lateral force resisting system, creating a moment frame between posts and trusses.

Columns: The columns will be 6x6 Douglas Fir timbers that will carry the loads from the joists and trusses down to the existing concrete slab. Since the posts will have to be retrofit to the existing slab and foundation, RPBZ Retrofit Connections from Simpson Wood Connectors will be used to limit uplift of the structure.

3. Pedestrian Bridges

Bridge Cross-Section: Each of the three bridges was adapted from the U.S. Forest Service Plans for a Sawn Timber Stringer Trail Bridge. The sawn timber stringers of the bridge are made of Douglas Fir wood and were selected due to the availability of the material and the ease of construction. Using the typical plan set as well as the Forest Service's *Sustainable Trail Bridge Design* manual, the bridge cross-section was adapted to meet the client's desires. The bridge deck was expanded to 8'-9" to meet ADA requirements and provide plenty of space for the client to drive their mowers and side-by-side's over.

Loading: Before designing the abutments, the bridge's loading was evaluated. First, the dead load from the bridge components was calculated and is shown in Reference 1 of Appendix D. After calculating the dead load, the live and snow load values were found using the AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges and the International Building Code. Both the live and snow loads are shown in Reference 2 of Appendix D. Once the dead, live, and snow loads were found, they were then combined according to the AASHTO LRFD Bridge Design Specifications. Once combined, the load was applied to each bridge in Autodesk Robot to determine the reaction forces, shear forces, and moment of the structure. The final loading on the bridge was an H5 design vehicle. In accordance with AASHTO specifications, the 10,000-pound design vehicle was applied to each bridge and the internal forces were analyzed.

Abutments: The abutments were designed to meet six major criteria: overturning moment, bearing capacity, stem flexure capacity, shear capacity, heel flexure capacity, and toe flexure capacity. To begin the analysis, the soil type from the site was estimated using the USDA Web Soil Survey. After determining the type of soil on the site, the soil's properties were estimated. Using the determined loading, as well as the properties of the soil, water, and concrete, the size of the abutments and their steel reinforcement were designed. The calculations for all the abutments on the site can be found in References 8 – 17 in Appendix D.

4. Roadways

Roadway Classification and Design Criteria: Our roadway design adheres strictly to the AASHTO Guidelines for Rural Local Roads in I.M. 3.210. Considering the specific parameters outlined, such as a design volume (ADT) under 400 vehicles with agricultural access, and the rolling terrain characteristics, thorough consideration was given to various design elements. The chosen design speed of 20 mph, stopping sight distance of 95 ft, and minimum K values for crest/sag vertical curves (5 ft) ensured optimal safety and functionality. Horizontal curve radii were set at a minimum of 75 ft to accommodate safe vehicle maneuvering. The traveled way was finalized at 20 ft, featuring 10 ft lane widths and 2 ft shoulders, with slopes adhering to a 3:1 ratio for horizontal to vertical elements. Additionally, the design considered minimum and maximum grade requirements of 0.5% and 12% with a typical grade of 2% to ensure smooth vehicle movement and drainage efficiency. Measures were also taken to address typical design considerations such as sight distances, cross-sectional elements, roadside clearances, and potential environmental impacts, ensuring compliance with local regulations and standards. For specific values, please refer to Appendix E.

Pavement Design: Pavement design calculations were conducted accurately based off the utilization of both SUDAS Pavement Thickness 5F-1 and Pave Xpress software, integrating detailed inputs specific to the three different pavement types the team moved forward with. For the HMA option, the recommended HMA surface course over a gravel base was derived from various inputs, including design life, reliability, traffic loadings, and material properties such as ESAL, CBR, and resilient modulus. Similarly, for the PCC alternative, the PCC surface course over the same gravel base was determined, factoring in parameters such as modulus of rupture, modulus of elasticity, and joint spacing. The Gravel alternative was designed with a gravel surface course over an existing gravel subbase, considering material type, thickness, and drainage coefficient for optimal performance. Detailed analysis was conducted to ensure pavement thicknesses were adequate to withstand anticipated traffic loads while considering local soil conditions, climate, and expected service life. For detailed calculations and inputs, please refer to Appendix E.

Final Corridor Design: In line with the established design parameters, final corridor designs were developed for each alternative to meet the project's requirements effectively. The HMA alternative features lanes with a total width of 20 feet, containing an HMA surface course supported by a gravel base. 2-foot shoulders were placed on both sides of our one-way roadway to provide additional safety margins. The PCC alternative mirrors this design, with PCC surface course lanes and gravel shoulders, ensuring robustness and durability. Meanwhile, the Gravel alternative maintains lanes with a gravel surface course on an existing gravel subbase, with shoulders on each side. Detailed cross-sectional drawings were developed for each alternative, illustrating layer thicknesses, material specifications, and drainage features. Each design iteration has been thoroughly crafted to optimize durability, safety, and cost-effectiveness, ensuring long-term performance and reliability on the rural local roads for this site. Typical design considerations such as shoulder widths, pavement markings, and signage were also integrated into the final designs to ensure compliance with standard road design practices and user expectations. For specific dimensions and details, please refer to Appendix E.

5. Parking Lots

Minimum Design Requirements: Our parking lot design carefully adheres to typical minimum design standards sourced from industry guidelines from the Statewide Urban Design and Specifications (SUDAS), particularly Section 12A-2 E and Section 8B-1 A, C, D, E, and F. These standards encompass critical parameters such as surface firmness, slopes, widths, passing spaces, and ADA parking stall criteria. For instance, all entrances and exits are a minimum width of 24-feet while sidewalk widths meet the encouraged 5-foot minimum, as recommended by SUDAS. Detailed specifications and calculations are outlined in Appendix F, referencing specific SUDAS sections and tables consulted during design.

Parking Lot Grading and Drainage: Grading and drainage considerations are methodically based on a comprehensive analysis of industry standards, local regulations, and site-specific factors, specifically guided by the Statewide Urban Design and Specifications (SUDAS), Section 8B-1 E. Our approach ensures effective drainage and accessibility, with a typical slope of 1.5% facilitating proper drainage and adherence to ADA slope requirements. Minimum slopes of 0.5% are used where flat areas are prevalent, supplemented by additional drainage measures such as the discouragement of any slopes exceeding 5%. Detailed documentation with references to SUDAS sections consulted and calculations performed is provided in Appendix F for verification.

Pavement Type and Thickness: Pavement selection and thickness was tied into the same process that was used in the design of the roadways. Specifically reviewing design standards and materials specifications stated in the Statewide Urban Design and Specifications (SUDAS), specifically Section 8B-1 F. Factors like design life, traffic loads, and pavement thickness requirements are carefully considered to ensure durability. Utilizing detailed tables provided by SUDAS and the Pave Xpress software, pavement thickness is determined to accommodate both cars and trucks, with references cited in Appendix E for transparency.

Required Parking Stalls: Determining the necessary parking spots and stalls, including those compliant with ADA regulations from the guidelines of the Statewide Urban Design and Specifications (SUDAS), Section 8C-1, to establish precise criteria for dimensions and accessibility requirements. Calculations for stall projection and aisle width are strictly integrated into our design, ensuring full compliance with accessibility standards. Transparent documentation, along with explicit references to specific SUDAS sections consulted, is provided in Appendix F for thorough verification.

6. Boat Ramp

Methods: The accumulated sediment at the base and top of the boat ramp needs removal to prevent further buildup. Our recommendation is to employ a telescopic front loader, or a telehandler with a bucket, for sediment removal at the bottom of the boat ramp. This equipment provides the necessary reach to efficiently clean out sediment while minimizing the risk of pavement damage. At the top of the boat ramp, both the telehandler and the skid steer can be utilized to relocate the aggregate obstructing the drainage path. Rainwater flowing down the uphill road and parking lot is currently directed onto the boat ramp due to the obstruction caused by the aggregate pile. If the pile is removed and the parking lot is regraded slightly, water and sediment will be diverted away from the entrance to the boat ramp. Following heavy rainfall events, there is a possibility of aggregate accumulation obstructing the drainage path again. In such instances, manual intervention with a shovel or the use of a skid steer to relocate the aggregate is recommended.

Slope: The transition area of the boat ramp, where the ramp meets the water level, was not steep enough according to the U.S. Army Corps of Engineers (USACE). By removing the accumulated gravel in the transition area, the slope will be greater to facilitate smoother transitions for watercraft entering and exiting the water. These Grade adjustments are implemented to improve maneuverability and reduce wear on trailers.

Accessibility: Accessibility improvements are being made to ensure the existing boat ramp provides access for a diverse range of users. Trailer parking will be implemented in the adjacent parking lot for those who launch their boats.

7. Trails

Profiles: Updated alignments for the waterfront and main trails were made following ABA 1017.7.1. Due to this area being of high use with access to the boat ramp, docks, pavilions, and fishing outcroppings, we maintained all slopes below 8.33% with breaks between inclines. All trails are designed at 8’ to satisfy ABA 1017.4 passing spaces requirements. The proposed extension was not designed to be surfaced or meet ABA standards as the existing alignment would require a copious amount of fill. We recommend denoting this extension as a fire-break due to not accommodating vehicles, but also have offered an alternative utilizing switchbacks for a grade less prone to erosion and accessible by mowers and smaller equipment in Appendix H.

Materials: Due to a durability concern as well as a goal of maintaining a natural feel, it is recommended to use a few different surfacing alternatives. In the ABA compliant trail redesigns, 4” thick grade A crushed stone is proposed to provide suitable strength, durability, and drainage. In other lower use, high flow areas, wood chip surfacing was considered to improve strength and surface quality, but after undergoing a cost analysis was not found to be a suitable alternative. In the other low use areas with relatively good drainage and minimal erosion, maintaining the current grass pathways is a suitable course of action. Alignments and profiles for each existing trail are included in the CAD files to offer many analysis options when determining budget and intent for each area of the system. The recommended combination of surfaces for each trail are included in the trail detail sheet under Appendix H.

Drainage: The profile was designed with drainage in mind, being graded at 2% down from the center sloping in both directions to prevent puddling in accordance with the USFS *Basic Trail Design Manual* for unpaved trails. Grate drainage systems with connecting pipes under trail surface are recommended in high flow areas to improve drainage and mitigate erosion.

8. Playgrounds

Replacement Items: To diversify the site, it is recommended that new amenities near the southern pavilion be added in place of a playground redesign, while the newly updated playground near the northern pavilion will remain. The new amenities include various permanent outdoor games, prefabricated by Doty & Sons Concrete Products, Outdoor Concrete Games. The recommended games are concrete cornhole/bag toss (#BYOB5533 in Playgrounds, Reference 1) and ladder toss (#LT4232 in Playgrounds, Reference 2) because they are easy to use for a range of ages and abilities and require minimal additional equipment.

Section VII. Engineer's Cost Estimate

1. Cost Estimates

The total estimated cost for the project is $1.17 million. A 20% contingency fee was applied to the total project cost to account for any uncertainty within the design and construction phases. In the following pages, you will find a breakdown for each part the project.

Table 1: Total Cost Estimate

Project:	**White Oaks Conservation Area Restoration**		
Project Element	**Sheet Set**	**Cost**	
Lake Dredging	A	$	43,300.00
Robins Pavilion	B	$	25,000.00
Cardinal Pavilion	B	$	32,950.00
Boat Ramp Bridge	C	$	42,200.00
Southeast Bridge	C	$	38,300.00
Primary Inlet Bridge	C	$	30,800.00
Roadways	D	$	393,980.00
Parking Lots	E	$	326,839.00
Boat Ramp	F	$	900.00
Trail Resurfacing	G	$	34,170.00
Playgrounds	H	$	4,950.00
Contingency Fee			
Contingency (20%)		$	195,000.00
Total Cost			
Total Project Cost		$	1,168,389.00

Table 2: Dredging Cost Estimate

Project:	**Lake Dredging**						
Item	**Item Description**	**Item Code**	**Unit**	**Unit Cost***	**Quantity**	**Total Cost**	**Rounded Cost**
Option 1							
Hydraulic Dredging	CUTTER SUCTION DREDGING, PUMPED 1000' TO SHORE DUMP	352413.13.1000	CY	$ 21.42	2022	$ 43,304.16	$ 43,300.00
Option 2							
Mechanical Dredging	CLAMSHELL MECHANICAL DREDGING, BARGE MOUNTED EXCAVATION	352423.13.0500	CY	$ 29.78	2022	$ 60,215.19	$ 60,000.00
Total Cost							
Option 1 Hydraulic (Recommended)						$ 43,304.16	**$ 43,300.00**

Gordian; 32nd Edition (2018), Heavy Construction Costs with RSMeans Data; *Unit cost have been adjusted for inflation.

Table 3: Robin Pavilion Cost Estimate

Project:	Robin Pavilion						
Item	**Item Description**	**Source**	**Unit**	**Unit Cost**	**Quantity**	**Total Cost**	**Rounded Cost**
Materials							
Purlins	4 X 6 X 14' #2 S4S DOUGLAS FIR TIMBER	Menards	EA	$ 57.84	40	$ 2,313.60	$ 2,300.00
Girders	4 X 10 X 8' #2 S4S DOUGLAS FIR TIMBER	Menards	EA	$ 155.74	8	$ 1,245.92	$ 1,250.00
Columns	6 X 6 X 12' #2 S4S DOUGLAS FIR TIMBER	Menards	EA	$ 98.51	4	$ 394.04	$ 400.00
Roof Decking	36" X 14' PRO-RIB EMERALD GREEN STEEL PANEL	Menards	EA	$ 44.30	20	$ 886.00	$ 900.00
Column Connection	RPBZ ZMAX GALVANIZED RETROFIT POST BASE	Home Depot	EA	$ 7.02	16	$ 112.32	$ 100.00
Knee Brace Connection	KBS17 KNEE-BRACE STABILIZER	Home Depot	EA	$ 6.75	16	$ 108.00	$ 100.00
Gable Plate	OUTDOOR ACCENTS AVANT COLLECTION APGP ZMAX BLACK POWDER-COATED GABLE PLATE	Home Depot	EA	$ 80.72	4	$ 322.88	$ 325.00
Truss Connection	OUTDOOR ACCENTS AVANT COLLECTION APGP ZMAX BLACK POWDER-COATED T STRAP	Home Depot	EA	$ 19.98	12	$ 239.76	$ 250.00
Joist Connection	HU GALVANIZED JOIST HANGER	Home Depot	EA	$ 19.98	100	$ 1,998.00	$ 2,000.00
Nails	8D BRIGHT STEEL SMOOTH SHANK COMMON NAILS	Home Depot	10 LBS	$ 34.98	1	$ 34.98	$ 35.00
Screws	STRONG-DRIVE SDS HEAVY DUTY CONNECTORS	Home Depot	25 PK	$ 12.65	1	$ 12.65	$ 15.00
Construction							
Labor		ZipRecruiter	SF	$ 10.00	625	$ 6,250.00	$ 6,250.00
Metal Roof Installation		ZipRecruiter	SF	$ 5.00	715	$ 3,575.00	$ 3,575.00
Demolition of Existing Pavilions		ProMatcher	EA	$ 7,500.00	1	$ 7,500.00	$ 7,500.00
Total Cost							
Robin Pavilions						$ 24,993.15	**$ 25,000.00**

Table 4: Cardinal Pavilion Cost Estimate

Project:	Cardinal Pavilion						
Item	**Item Description**	**Source**	**Unit**	**Unit Cost**	**Quantity**	**Total Cost**	**Rounded Cost**
Materials							
Purlins	4 X 6 X 14' #2 S4S DOUGLAS FIR TIMBER	Menards	EA	$ 57.84	55	$ 3,181.20	$ 3,200.00
Girders	4 X 10 X 8' #2 S4S DOUGLAS FIR TIMBER	Menards	EA	$ 155.74	8	$ 1,245.92	$ 1,250.00
Columns	6 X 6 X 12' #2 S4S DOUGLAS FIR TIMBER	Menards	EA	$ 98.51	4	$ 394.04	$ 400.00
Roof Decking	36" X 14' PRO-RIB EMERALD GREEN STEEL PANEL	Menards	EA	$ 44.30	40	$ 1,772.00	$ 1,775.00
Column Connection	RPBZ ZMAX GALVANIZED RETROFIT POST BASE	Home Depot	EA	$ 7.02	16	$ 112.32	$ 100.00
Knee Brace Connection	KBS17 KNEE-BRACE STABILIZER	Home Depot	EA	$ 6.75	16	$ 108.00	$ 100.00
Gable Plate	OUTDOOR ACCENTS AVANT COLLECTION APGP ZMAX BLACK POWDER-COATED GABLE PLATE	Home Depot	EA	$ 80.72	4	$ 322.88	$ 325.00
Truss Connection	OUTDOOR ACCENTS AVANT COLLECTION APGP ZMAX BLACK POWDER-COATED T STRAP	Home Depot	EA	$ 19.98	12	$ 239.76	$ 250.00
Joist Connection	HU GALVANIZED JOIST HANGER	Home Depot	EA	$ 19.98	100	$ 1,998.00	$ 2,000.00
Nails	8D BRIGHT STEEL SMOOTH SHANK COMMON NAILS	Home Depot	10 LBS	$ 34.98	1	$ 34.98	$ 35.00
Screws	STRONG-DRIVE SDS HEAVY DUTY CONNECTORS	Home Depot	25 PK	$ 12.65	1	$ 12.65	$ 15.00
Construction							
Labor		ZipRecruiter	SF	$ 10.00	1125	$ 11,250.00	$ 11,250.00
Metal Roof Installation		ZipRecruiter	SF	$ 5.00	950	$ 4,750.00	$ 4,750.00
Demolition of Existing Pavilions		ProMatcher	EA	$ 7,500.00	1	$ 7,500.00	$ 7,500.00
Total Cost							
Cardinal Pavilions						$ 32,921.75	**$ 32,950.00**

Table 5: Boat Ramp Bridge Cost Estimate

Project:	**Boat Ramp Bridge**						
Item	**Item Description**	**Source**	**Unit**	**Unit Cost**	**Quantity**	**Total Cost**	**Rounded Cost**
Materials							
Abutments	BULK CONCRETE W/ REINFORCING	ConcreteNetwork.com	CY	$ 175.00	9	$ 1,575.00	$ 1,600.00
Sills for Girders	12" X 12" DOUGLAS FIR	Lumber Store by Carlwood	LF	$ 43.00	30	$ 1,290.00	$ 1,300.00
Girders	6" X 18" DOUGLAS FIR	Twin Creeks	LF	$ 55.00	240	$ 13,200.00	$ 13,200.00
Deck Planks	3" X 12" DOUGLAS FIR	Close Lumber	LF	$ 9.00	480	$ 4,320.00	$ 4,400.00
Running Planks	2" X 12" DOUGLAS FIR	Menards	LF	$ 3.00	480	$ 1,440.00	$ 1,500.00
Blocking	4" X 6" S4S DOUGLAS FIR	Menards	LF	$ 3.50	25	$ 87.50	$ 100.00
Handrail Posts	4" X 6" S4S DOUGLAS FIR	Menards	LF	$ 3.50	105	$ 367.50	$ 400.00
Railing	2" X 6" S4S DOUGLAS FIR	Menards	LF	$ 1.50	288	$ 432.00	$ 500.00
Rail Cap	2" X 8" S4S DOUGLAS FIR	Menards	LF	$ 1.75	96	$ 168.00	$ 200.00
Backing Planks	3" X 6" DOUGLAS FIR	Ashby Lumber	LF	$ 4.50	20	$ 90.00	$ 100.00
Connections	NAILS & BOLTS	Menards	LS	$ 300.00	1	$ 300.00	$ 300.00
Construction							
Demolition		ProMatcher	LS	$ 2,500.00	1	$ 2,500.00	$ 2,500.00
Earthwork			LS	$ 5,000.00	1	$ 5,000.00	$ 5,000.00
Labor		ZipRecruiter	HR	$ 23.00	480	$ 11,040.00	$ 11,100.00
Total Cost							
Boat Ramp Bridge						$ 41,810.00	**$ 42,200.00**

Table 6: Southeast Bridge Cost Estimate

Project:	**Southeast Bridge**						
Item	**Item Description**	**Source**	**Unit**	**Unit Cost**	**Quantity**	**Total Cost**	**Rounded Cost**
Materials							
Abutments	BULK CONCRETE W/ REINFORCING	ConcreteNetwork.com	CY	$ 175.00	6	$ 1,050.00	$ 1,100.00
Sills for Girders	12" X 12" DOUGLAS FIR	Lumber Store by Carlwood	LF	$ 43.00	30	$ 1,290.00	$ 1,300.00
Girders	6" X 18" DOUGLAS FIR	Twin Creeks	LF	$ 55.00	200	$ 11,000.00	$ 11,000.00
Deck Planks	3" X 12" DOUGLAS FIR	Close Lumber	LF	$ 9.00	400	$ 3,600.00	$ 3,600.00
Running Planks	2" X 12" DOUGLAS FIR	Menards	LF	$ 3.00	400	$ 1,200.00	$ 1,200.00
Blocking	4" X 6" S4S DOUGLAS FIR	Menards	LF	$ 3.50	25	$ 87.50	$ 100.00
Handrail Posts	4" X 6" S4S DOUGLAS FIR	Menards	LF	$ 3.50	105	$ 367.50	$ 400.00
Railing	2" X 6" S4S DOUGLAS FIR	Menards	LF	$ 1.50	240	$ 360.00	$ 400.00
Rail Cap	2" X 8" S4S DOUGLAS FIR	Menards	LF	$ 1.75	80	$ 140.00	$ 200.00
Backing Planks	3" X 6" DOUGLAS FIR	Ashby Lumber	LF	$ 4.50	20	$ 90.00	$ 100.00
Connections	NAILS & BOLTS	Menards	LS	$ 300.00	1	$ 300.00	$ 300.00
Construction							
Demolition		ProMatcher	LS	$ 2,500.00	1	$ 2,500.00	$ 2,500.00
Earthwork			LS	$ 5,000.00	1	$ 5,000.00	$ 5,000.00
Labor		ZipRecruiter	HR	$ 23.00	480	$ 11,040.00	$ 11,100.00
Total Cost							
Southeast Bridge						$ 38,025.00	**$ 38,300.00**

Table 7: Primary Inlet Bridge Cost Estimate

Project:	**Primary Inlet Bridge**						
Item	**Item Description**	**Source**	**Unit**	**Unit Cost**	**Quantity**	**Total Cost**	**Rounded Cost**
Materials							
Abutments	BULK CONCRETE W/ REINFORCING	ConcreteNetwork.com	CY	$ 175.00	4	$ 700.00	$ 700.00
Sills for Girders	12" X 12" DOUGLAS FIR	Lumber Store by Carlwood	LF	$ 43.00	20	$ 860.00	$ 900.00
Girders	6" X 18" DOUGLAS FIR	Twin Creeks	LF	$ 55.00	135	$ 7,425.00	$ 7,500.00
Deck Planks	3" X 12" DOUGLAS FIR	Close Lumber	LF	$ 9.00	270	$ 2,430.00	$ 2,500.00
Running Planks	2" X 12" DOUGLAS FIR	Menards	LF	$ 3.00	270	$ 810.00	$ 900.00
Blocking	4" X 6" S4S DOUGLAS FIR	Menards	LF	$ 3.50	25	$ 87.50	$ 100.00
Handrail Posts	4" X 6" S4S DOUGLAS FIR	Menards	LF	$ 3.50	85	$ 297.50	$ 300.00
Railing	2" X 6" S4S DOUGLAS FIR	Menards	LF	$ 1.50	162	$ 243.00	$ 300.00
Rail Cap	2" X 8" S4S DOUGLAS FIR	Menards	LF	$ 1.75	54	$ 94.50	$ 100.00
Backing Planks	3" X 6" DOUGLAS FIR	Ashby Lumber	LF	$ 4.50	20	$ 90.00	$ 100.00
Connections	NAILS & BOLTS	Menards	LS	$ 300.00	1	$ 300.00	$ 300.00
Construction							
Demolition		ProMatcher	LS	$ 1,000.00	1	$ 1,000.00	$ 1,000.00
Earthwork			LS	$ 5,000.00	1	$ 5,000.00	$ 5,000.00
Labor		ZipRecruiter	HR	$ 23.00	480	$ 11,040.00	$ 11,100.00
Total Cost							
Inlet Bridge						$ 30,377.50	**$ 30,800.00**

Table 8: Roadway Cost Estimate (HMA)

Project:	**Roadways**						
Item	**Item Description**	**Item Code**	**Unit**	**Unit Cost**	**Quantity**	**Total Cost**	**Rounded Cost**
Excavation - Class 10							
Cut/Fill	EXCAVATION, CLASS 10, ROADWAY AND BORROW	2102-2710070	CY	$ 4.36	1278.75	$ 5,575.35	$ 5,575.00
Backfill Placing	GRANULAR BACKFILL	2402-0425031	TON	$ 35.17	1516.28	$ 53,327.55	$ 53,500.00
1" Subbase Compaction	COMPACTION WITH MOISTURE AND DENSITY CONTROL	2107-0875000	CY	$ 1.28	245.68	$ 314.48	$ 315.00
6.5" Granular Base	GRANULAR SURFACING ON ROAD, CLASS A CRUSHED STONE	2312-8260051	TON	$ 39.91	1889.72	$ 75,418.84	$ 75,500.00
Pavement							
4" HMA Pavement	HOT MIX ASPHALT MIXTURE, COMMERICAL MIX (INCLUDES ASPHALT BINDER), AS PER PLAN	2303-0000100	TON	$ 144.71	1658.37	$239,982.72	$ 240,000.00
10.5" Gravel Shoulders	GRANULAR SHOULDERS, TYPE A	2121-7425010	TON	$ 26.55	610.53	$ 16,209.46	$ 16,200.00
4" Top Soil	TOPSOIL, STRIP, SALVAGE AND SPREAD	2105-8425015	CY	$ 6.17	185.62	$ 1,145.30	$ 1,150.00
Hydraulic Seeding	HYDRAULIC SEEDING	2601-2636070	ACRE	$ 4,199.05	0.35	$ 1,449.38	$ 1,450.00
Pavement Markings	PAINTED PAVEMENT MARKING, WATERBORNE OR SOLVENT-BASED	2527-9263109	STA	$ 21.94	13.31	$ 291.98	$ 290.00
Total Cost							
HMA Roadways						$393,715.05	**$ 393,980.00**

Source: Bid Tabs

Table 9: Parking Lot Cost Estimate (HMA)

Project:	Parking Lots						
Item	**Item Description**	**Item Code**	**Unit**	**Unit Cost**	**Quantity**	**Total Cost**	**Rounded Cost**
General							
Tree Relocation	TREE, TRANSPLANTING	2610-0000150	EACH	$ 2,500.00	1.00	$ 2,500.00	$ 2,500.00
Excavation - Class 10							
Cut/Fill	EXCAVATION, CLASS 10, ROADWAY AND BORROW	2102-2710070	CY	$ 4.36	1914.62	$ 8,347.75	$ 8,300.00
Backfill Placing	GRANULAR BACKFILL	2402-0425031	TON	$ 35.17	2718.76	$ 95,618.88	$ 95,500.00
1" Subbase Compaction	COMPACTION WITH MOISTURE AND DENSITY CONTROL	2107-0875000	CY	$ 1.28	135.63	$ 173.60	$ 175.00
6.5" Granular Base	GRANULAR SURFACING ON ROAD, CLASS A CRUSHED STONE	2312-8260051	TON	$ 39.91	1251.85	$ 49,961.28	$ 50,000.00
Pavement							
4" HMA Pavement	HOT MIX ASPHALT MIXTURE, COMMERICAL MIX (INCLUDES ASPHALT BINDER), AS PER PLAN	2303-0000100	TON	$ 144.71	1065.95	$ 154,253.35	$ 154,500.00
4" Top Soil	TOPSOIL, STRIP, SALVAGE AND SPREAD	2105-8425015	CY	$ 6.17	95.66	$ 590.20	$ 590.00
Hydraulic Seeding	HYDRAULIC SEEDING	2601-2636070	ACRE	$ 4,199.05	0.18	$ 746.90	$ 745.00
Pavement Markings	PAINTED PAVEMENT MARKING, WATERBORNE OR SOLVENT-BASED	2527-9263109	STA	$ 21.94	3.60	$ 78.98	$ 79.00
ADA SYMBOLS	PAINTED SYMBOLS AND LEGENDS, WATERBORNE OR SOLVENT-BASED	2527-9263137	EACH	$ 116.10	3.00	$ 348.30	$ 350.00
Sidewalk							
6" PCC Sidewalk	SIDEWALK, P.C. CONCRETE, 6 IN.	2511-7526006	SY	$ 97.41	145.07	$ 14,131.25	$ 14,100.00
Total Cost							
HMA Parking Lot						$324,250.51	**$ 326,839.00**

Source: Bid Tabs

Table 10: Boat Ramp Cost Estimate

Project:	Boat Ramp						
Construction	**Item Description**	**Source**	**Unit**	**Unit Cost**	**Quantity**	**Total Cost**	**Rounded Cost**
Operator Labor		ZipRecruiter	HR	$ 30.00	8	$ 240.00	$ 250.00
Telehandler Rental		BigRentz	LS	$ 650.00	1	$ 650.00	$ 650.00
Total Cost							
Boat Ramp						$ 890.00	**$ 900.00**

Table 11: Trail Resurfacing Cost Estimate (Gravel)

Project:	**Trail Redesign and Surfacing**						
Item	**Item Description**	**Item Code**	**Unit**	**Unit Cost**	**Quantity**	**Total Cost**	**Rounded Cost**
Excavation - Class 10							
Cut/Fill	EXCAVATION, CLASS 10, ROADWAY AND BORROW	2102-2710070	CY	$ 4.36	1690.04	$ 7,368.57	$ 7,370.00
ABA Surfacing							
4" Granular Base	GRANULAR SURFACING ON ROAD, CLASS A CRUSHED STONE	2312-8260050	CY	$ 39.91	375.94	$ 15,003.77	$ 15,000.00
Compaction of Base	COMPACTION WITH MOISTURE AND DENSITY CONTROL	2107-0875000	CY	$ 1.28	375.94	$ 481.20	$ 500.00
Secondary Surfacing							
4" Granular Base	GRANULAR SURFACING ON ROAD, CLASS A CRUSHED STONE	2312-8260050	CY	$ 39.91	273.86	$ 10,929.75	$ 10,950.00
Compaction of Base	COMPACTION WITH MOISTURE AND DENSITY CONTROL	B07Y342P2R	CY	$ 1.28	273.86	$ 350.54	$ 350.00
Total Cost							
Grade A Crushed Stone Surfacing/ Trail Redesign						$ 34,133.83	**$ 34,170.00**

Table 12: Playground Cost Estimate

Project:	**Playgrounds**						
Item	**Item Description**	**Source**	**Unit**	**Unit Cost**	**Quantity**	**Total Cost**	**Rounded Cost**
Materials							
Cornhole	CONCRETE CORNHOLE/BAG TOSS #BYOB5531	Doty & Sons Concrete	EA	$ 1,391.00	1	$ 1,391.00	$ 1,500.00
Ladder Toss	CONCRETE LADDER TOSS #LT4232	Doty & Sons Concrete	EA	$ 1,146.00	1	$ 1,146.00	$ 1,200.00
Construction							
Demolition of Existing		ProMatcher	EA	$ 2,000.00	1	$ 2,000.00	$ 2,000.00
Labor		ZipRecruiter	SF	$ 10.00	25	$ 250.00	$ 250.00
Total Cost							
Playgrounds						$ 4,787.00	**$ 4,950.00**

2. Funding

The project's funding will primarily come from the client, supplemented by potential support from governmental programs through the Iowa Department of Natural Resources. Among these, the Iowa Resource Enhancement and Protection (REAP) program stands out as a promising avenue for financial assistance. This program offers funding opportunities tailored to projects aimed at conserving and enhancing Iowa's natural and cultural resources, aligning closely with our project objectives.

Additionally, we intend to explore other funding sources such as the Iowa Wildlife Habitat Stamp Fund program and the Water Recreation Access Cost-Share program. These programs provide further opportunities for financial support, allowing us to address more of the expensive and critical aspects of our project. Notably, each program underscores a distinct focus on wildlife habitat or water recreation. This is something to keep in mind when answering the questions on the applications for these grants.

By aligning our project vision with the goals of these funding programs, we aim to secure the necessary resources to bring the White Oak Conservation Center to fruition. Through partnership and dedication, we are determined to realize our vision for environmental conservation and enhancement.

3. Phasing Plan

Given the extensive scope of design elements requiring refurbishing, the substantial total cost associated with their construction, and the constraints of our client's annual budget, our team has devised a strategic phasing plan to ensure the successful completion of the project.

Our devised phasing plan strategically divides the project into three distinct phases, each tailored to ensure cost-effectiveness and feasibility within a single construction year. Phase one addresses the site's utmost priorities, tackling critical elements with immediate significance. Phase two encompasses necessary components deemed important but of lesser urgency compared to those in phase one. Finally, phase three encompasses lower-priority items, allowing for their adoption by the client as needed or as funding becomes available.

The design elements our team deemed necessary to include in phase one are the completion of the lake dredging, the reconstruction of both pavilions on site, the reconstruction of the boat ramp bridge, and the reconstruction of the southeastern bridge. The two recommended bridges are necessary to include in phase one due to their structural flaws. Reconstructing the bridges early in the project will improve site access for maintenance crews by increasing the overall connectivity of the site. The lake dredging was assigned to phase one since sedimentation has been occurring for quite some time at this site, and it is necessary for the health of the lake that this be completed early on. The pavilions were selected to be in phase one with the intention of increasing visitation to the site early in the project. The total estimated construction cost for phase one is $218,100.00.

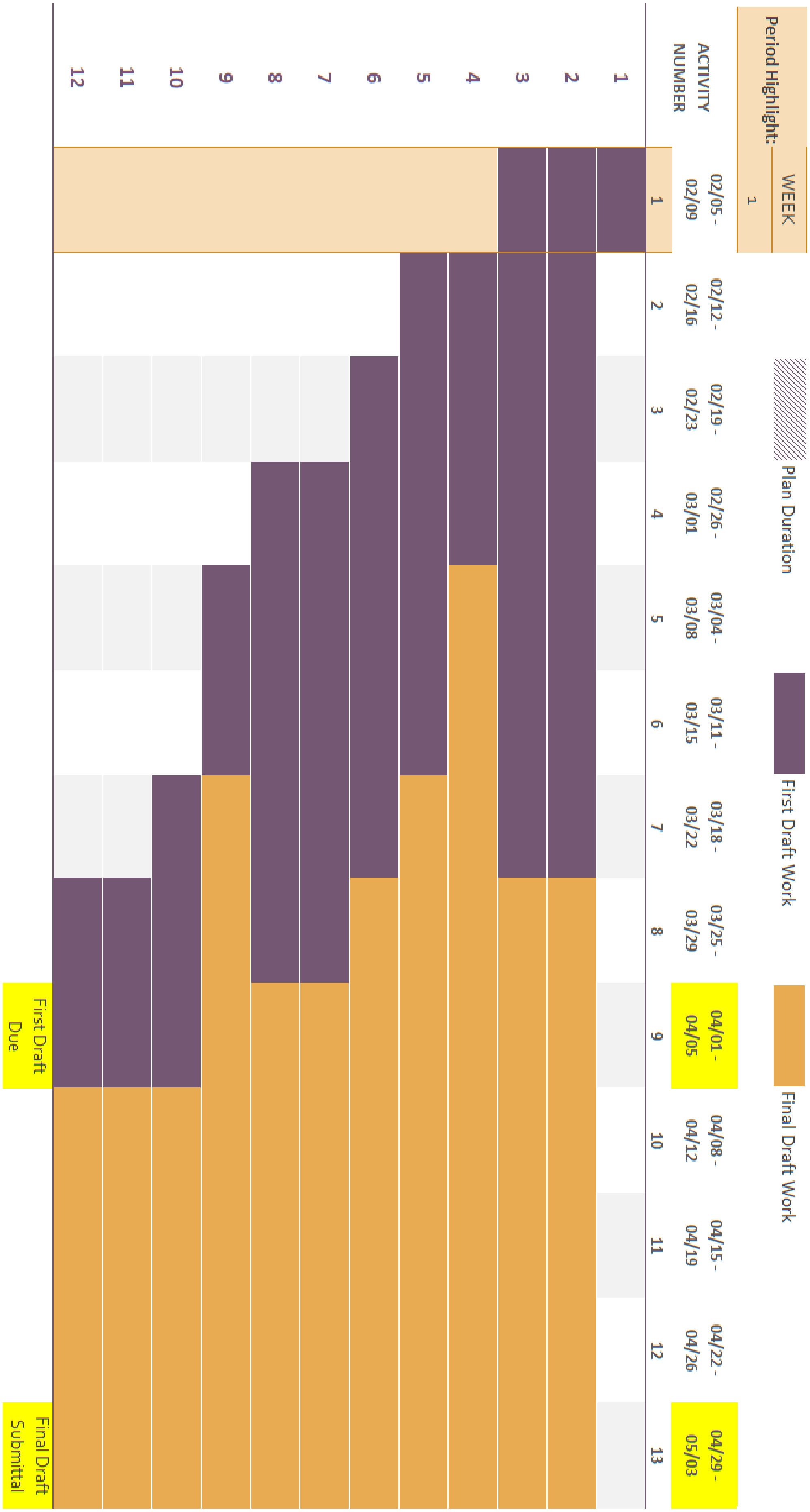
The elements we are recommending in phase two are the boat ramp clean up, the paving of the southern roadway and parking lot, and the rehabilitation of the main high use trail system. Our team is emphasizing the rehabilitation of the main trial system first to ensure that the trails become stabilized. Our team recommends the boat ramp clean up following the construction of the bridges and the rehabilitation of the main trail system. By waiting until this phase to clean up the boat ramp, our goal is to ensure any sediment added to the foot of boat ramp from machinery during phase 1 is also cleaned up.

Following the boat ramp clean up, the adjacent parking lot and roadway should be regraded and paved. Paving in this phase is strategically scheduled so that no heavy equipment needed for pavilion, trail, or bridge reconstruction would need to drive on the new pavement and cause unnecessary wear and tear. Once these areas are paved, the main source for the boat ramps sedimentation will be stabilized. The total estimated construction cost for phase two is $316,483.00.

Lastly, phase three tasks include the paving of the remaining roadways and parking lots, reconstruction of the primary inlet bridge, rehabilitating the secondary use trail system, and construction of the new playground areas. Our team identified these tasks as lower priority since they are not in failing condition or contributing to sedimentation issues with the main lake. The rest of the roadways and parking lots are not a source of sediment in the lake, and with the high cost to regrade and pave these areas, it is not a high priority task. The primary inlet bridge on the southern portion of the site is in adequate condition and a reconstruction is only necessary to increase safety and improve access for maintenance crews. The secondary trail system is in adequate condition and is not as heavily used as the main system, making it a less pressing task. The playground construction is the lowest priority item due to its low cost and ease of construction. The total estimated construction cost for phase three is $633,483.00.

Appendix A: Work Plan

ACTIVITY	TASK LEAD	FIRST DRAFT START	FIRST DRAFT DURATION	FINAL DRAFT START	FINAL DRAFT DURATION	ACTIVITY NUMBER
1. Data Collection	Cody Hall	1	1	1	1	**1**
2. Lake Evaluation	Noah Lyon / Aden Gomez	1	7	1	13	**2**
3. Pavilion Design	Maya Johnson	1	7	1	13	**3**
4. Roadway Design	Cory Siegel / Evan Felts	2	3	2	13	**4**
5. Parking Lot Design	Cory Siegel / Evan Felts	2	5	2	13	**5**
6. Pedestrian Bridge Design	Cody Hall	3	5	3	13	**6**
7. Trail System Analysis	Beau Benzing	4	5	4	13	**7**
8. Boat Ramp Redesign	Justin Japlon	4	5	4	13	**8**
9. Playground Design	Maya Johnson	5	2	5	13	**9**
10. Project Report	Cody Hall	7	3	7	13	**10**
11. Project Presentation	Cody Hall	8	2	8	13	**11**
12. Project Poster	Cody Hall	8	2	8	13	**12**


Period Highlight:
WEEK
1
Plan Duration
First Draft Work
Final Draft Work
ACTIVITY NUMBER
1
2
3
4
5
6
7
8
9
10
11
12
02/05 - 02/09
02/12 - 02/16
02/19 - 02/23
02/26 - 03/01
03/04 - 03/08
03/11 - 03/15
03/18 - 03/22
03/25 - 03/29
04/01 - 04/05
04/08 - 04/12
04/15 - 04/19
04/22 - 04/26
04/29 - 05/03
1
2
3
4
5
6
7
8
9
10
11
12
13
First Draft Due
Final Draft Submittal

Appendix B: Lake Evaluation

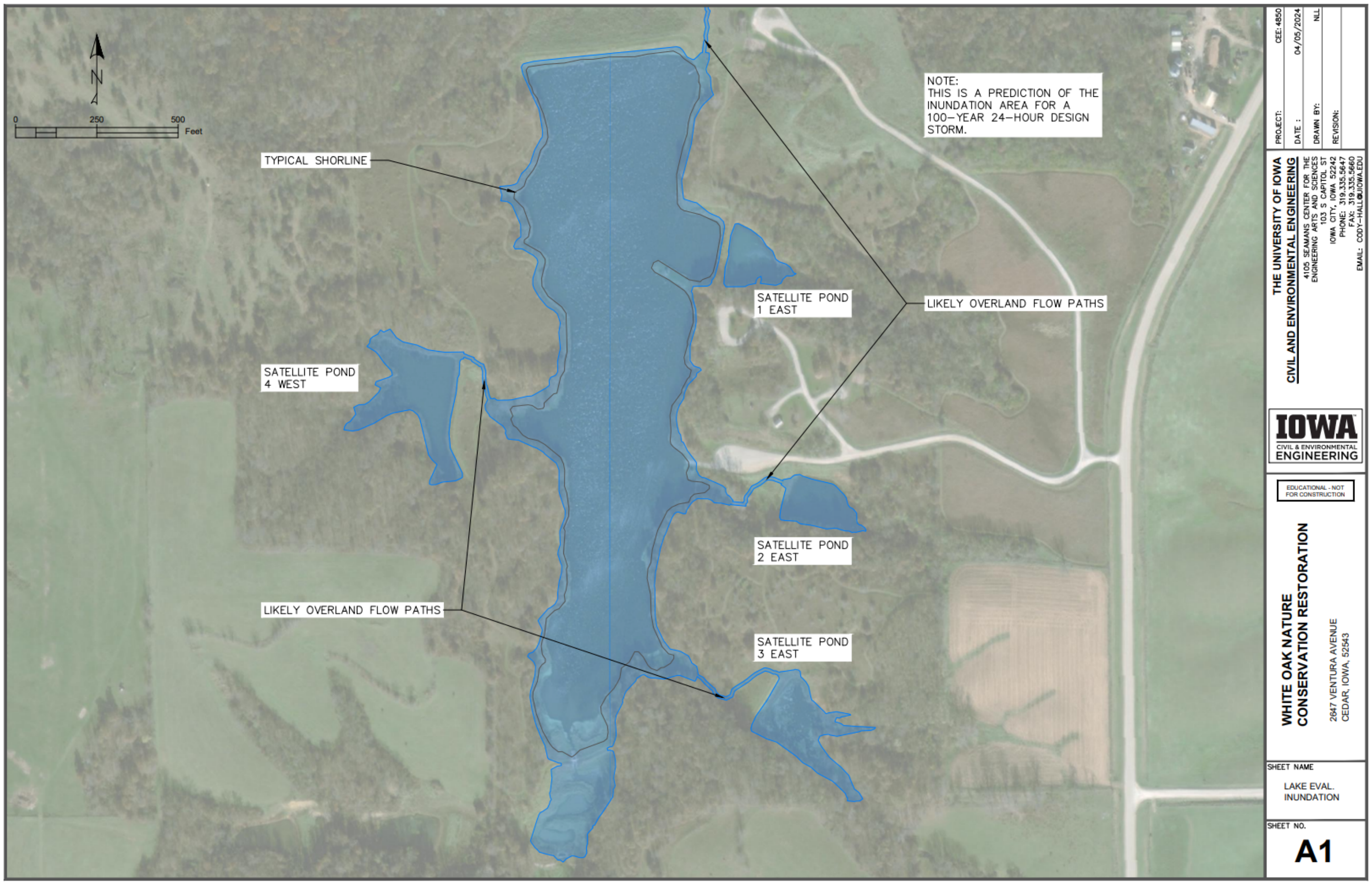


Exhibit 1: Lake inundation map due to a 100-Year 24-Hour design storm.

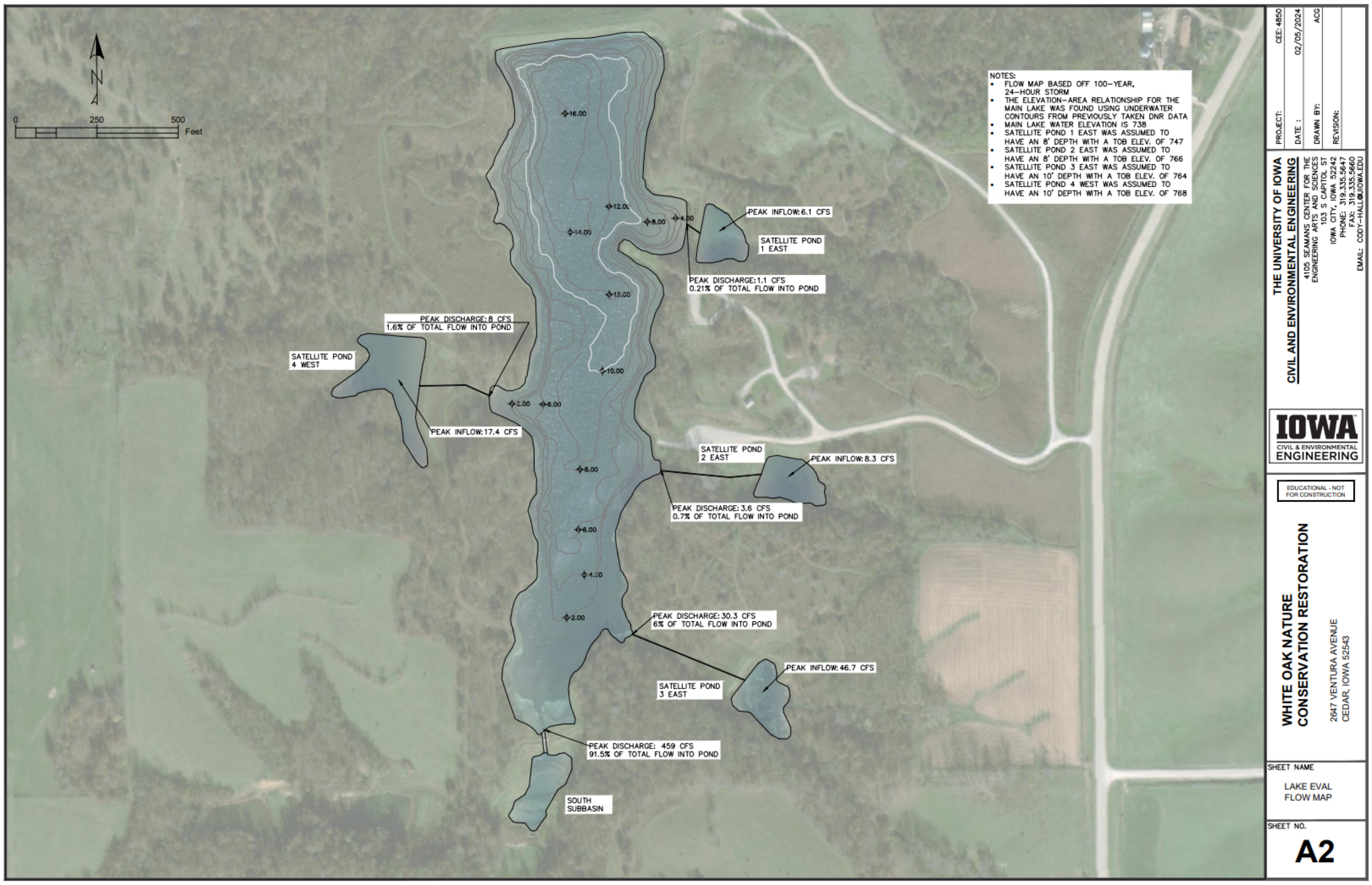


Exhibit 2: Flow map describing how the satellite ponds interact with the main lake.

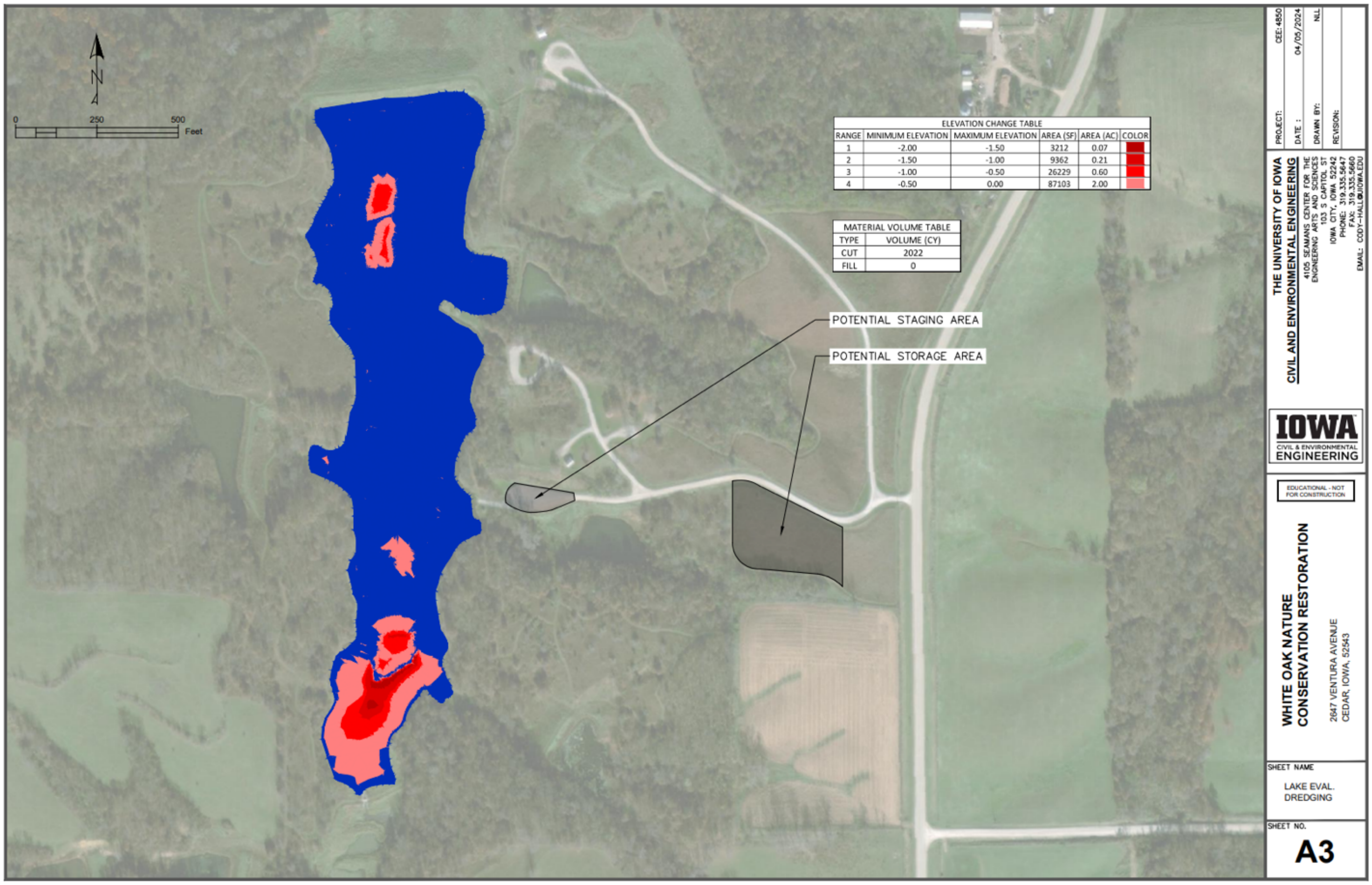


Exhibit 3: Recommended areas for dredging in the main lake.

Design Storm Assumptions

- The design storm evaluated was a 100-year, 24-hour storm.
- The SCS Type 2 precipitation model using NOAA Atlas 14 for depth was used for evaluation.
- Precipitation is assumed uniform for all subbasins.

Subbasins Assumptions

- Subbasin areas were determined using Streamstats.
- The loss method used was SCS Curve Number, CN calculated using data from Streamstats.
- The transform method used was SCS Unit Hydrograph, lag time calculated using data from Streamstats.

Satellite Ponds Assumptions

- The satellite pond area-elevation was found by using percent reduction equations using the embankment data from the topographic survey.

- Satellite Pond 1 East (NE satellite pond) was assumed to have an 8’ depth with a top of bank storage elevation of 747 feet.
- Satellite Pond 2 East (E satellite pond) was assumed to have an 8’ depth with a top of bank storage elevation of 766 feet.
- Satellite Pond 3 East (SE satellite pond) was assumed to have a 10’ depth with a top of bank storage elevation of 764 feet.
- Satellite Pond 1 West (W satellite pond) was assumed to have a 10’ depth with a top of bank storage elevation of 768 feet.
- Outlet elevations for satellite ponds were shot using GPS and recorded at the site visit on 2/14/2024

Main Lake

- The elevation-area relationship for the main lake was found using underwater contours from DNR data provided by Lewis Bruce. (See Figure 1)

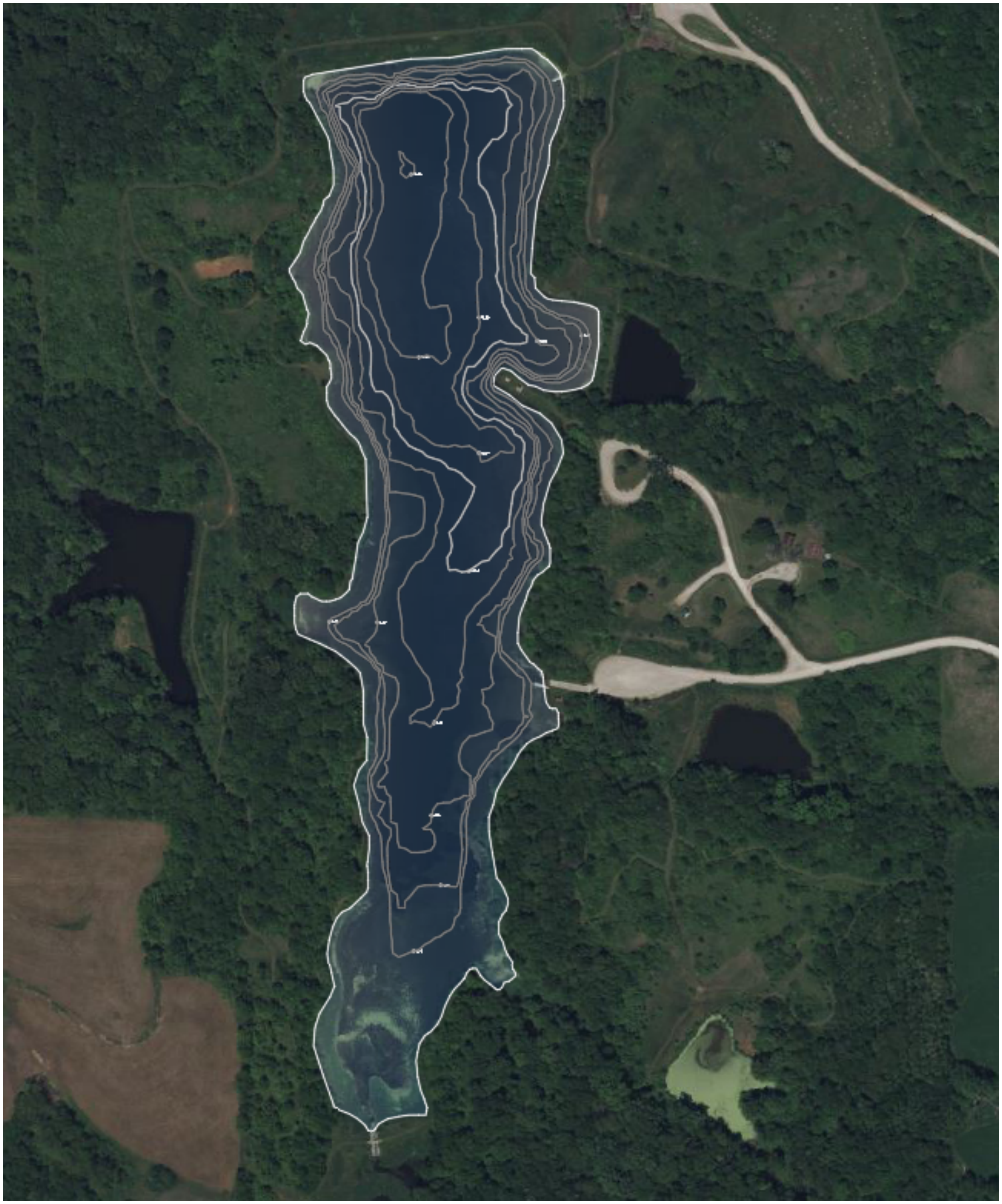


Figure 1: Main lake contour data from DNR provided survey.

Outlets and Spillways

- Outlet elevations for satellite ponds and main lake were shot with a GPS unit and recorded at the site visit on 2/14/2024.
- Manning's n values are all correlated with the pipe material documented on site, only one that was questionable was the north outlet pipe (this was assumed as ductile iron).
- Slopes for the discharge pipes were found using changes in elevation and length.
- Emergency spillways were assumed broad-crested with a coefficient of 3.
- Spillway elevations and lengths were determined using modified lidar based contour data.

HEC-HMS

- The following data was input into HEC-HMS (See Figure 2)
- The HEC-HMS model was created. (See Figure 3)

Project: White Oak Lake Model

Simulation Run: 100-Year

Simulation Start: 1 January 2024, 01:00

Simulation End: 5 January 2024, 23:00

HMS Version: 4.11

Executed: 26 March 2024, 15:20

Global Parameter Summary - Subbasin

	Area (MI2)
Element Name	**Area (MI2)**
East 1 Subbasin	0.01
East 2 Subbasin	0.01
East 3 Subbasin	0.05
South Subbasin	0.69
West Subbasin	0.02

	Downstream
Element Name	**Downstream**
East 1 Subbasin	Satellite Pond 1 East
East 2 Subbasin	Satellite Pond 2 East
East 3 Subbasin	Satellite Pond 3 East
South Subbasin	Main Lake
West Subbasin	Satellite Pond 4 West

	Loss Rate: Scs	
Element Name	**Percent Impervious Area**	**Curve Number**
East 1 Subbasin	0	74.4
East 2 Subbasin	0	64.6
East 3 Subbasin	0	74.5
South Subbasin	0	73.9
West Subbasin	0	66.7

	Transform: Scs	
Element Name	**Lag**	**Unitgraph Type**
East 1 Subbasin	4.26	Standard
East 2 Subbasin	7.77	Standard
East 3 Subbasin	11.69	Standard
South Subbasin	48.81	Standard
West Subbasin	9.46	Standard

Global Results Summary

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
East 1 Subbasin	0.01	6.1	01Jan2024, 13:00	4.22
East 2 Subbasin	0.01	8.34	01Jan2024, 13:00	3.18
East 3 Subbasin	0.05	46.68	01Jan2024, 13:00	4.23
South Subbasin	0.69	459.02	01Jan2024, 14:00	4.16
Satellite Pond 3 East	0.05	30.28	01Jan2024, 14:00	4.65
West Subbasin	0.02	17.43	01Jan2024, 13:00	3.4
Satellite Pond 4 West	0.02	7.96	01Jan2024, 15:00	3.41
Satellite Pond 2 East	0.01	3.62	01Jan2024, 15:00	3.25
Satellite Pond 1 East	0.01	1.11	01Jan2024, 16:00	4.2
Main Lake	0.78	41.49	01Jan2024, 24:00	3.8

Figure 2: HEC-HMS input and results summary.

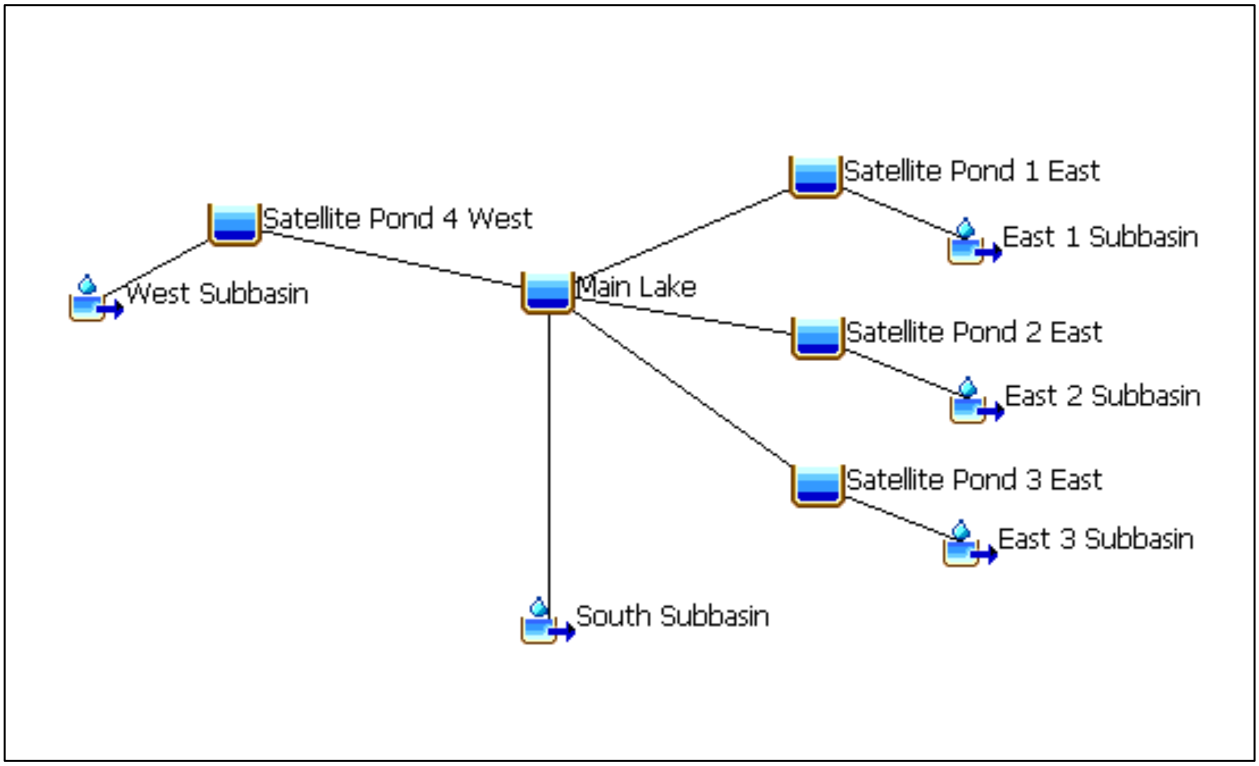


Figure 3: HEC-HMS model layout.

Sediment Removal Assumptions

- At a water surface elevation of 738, the lake has 810 square feet of area exceeding a 16' depth.
- The new 16' depth area is 3581sq. Ft, which equates to 300 cubic yards of sediment that would need to be removed.
- At the South end, expanding recreational boating activity available area will produce an increase in sediment removal by 1,722 cubic yards.
- Increased the area of the 16' depth at the current location to increase habitat life for fish over the winter.
- Increased depth at the south end of the pond to increase recreational activity such as fishing, the south end was also very shallow and unusable on a boat.

Unit Pricing and Inflation Factors

- Unit pricing source: Gordian; 32nd Edition, Heavy Construction Costs with RSMeans data
- Inflation Factor Source: https://edzarenski.com/category/inflation-indexing/

Appendix C: Pavilions

Reference 1: Prefabricated Bench Pavilion Specifications

Sentinel Mountain Shelter Model 98-93

Specifications

Description: Table Shelter
Size: 10' x 10'
Roof Pitch: 3/12

Roof Style: Gable
Options Shown: Hi Rib Steel Roof, 4' square table with 4 bench seats (no back) using 2" x 4" recycled plastic planks

Features

- Clear spans
- 6x6 by 3/16 steel posts
- 4' square table, 2" x 4" recycled plastic planks
- 4 bench seats (3 optional), no back, 2" x 4" recycled plastic planks
- Polyester powder coated
- Wind load: 90 mph class C
- Snow Load: 30 lbs
- Custom designs available
- USA Made

Options

- Hot dipped galvanized
- Stain or clear sealer for wood members
- Increased wind & snow load available
- Variety of roof pitches available
- Wood post: square
- Steel post: square
- Roof options: hi rib, standing seam, cedar, asphalt, tile, etc.
- Gutters & downspouts
- Reader boards, tables, benches and bike racks
- Chemical resistant Natur-Kote primer for harsh environments

Natural Structures
PO Box 270, Baker City, OR 97814 (541) 523-0224 (800) 252-8475 www.naturalstructures.com - info@naturalstructures.com

July 2021

Reference 2: Pavilion Loading

White Oak Conservation Area - Pavilion Loading

Assumptions:

Risk Category = I

Site Soil Class = Default

Exposure category = Partially exposed

Surface Roughness = C

Using ASCE 7 Hazard Tool:

Elevation: $z_e := 776\ ft$

Wind speed: $V := 103\ mph$

Live Load: $w_L := 20\ psf$
Ordinary, flat, pitched
and curved roof

Mahaska County: $p_g := 33\ psf$
ASCE Hazard Tool
Risk Category 1

Exposure factor: $C_e := 1.0$
Partially exposed
Surface roughness C
ASCE 7-22 Table 3.7-1

Thermal factor: $C_t := 1.2$
Unheated structure
ASCE 7-22 Table 7.3-2

Slope roof factor: $C_s := 1.0$
Ct = 1.2
4:12 pitch
ASCE 7-22 Figure 7.4-1c

Span length: $L_1 := 25\ ft$ $L_2 := 45\ ft$

Flat roof snow load: $p_f := 0.7 \cdot C_e \cdot C_t \cdot p_g = 27.72\ psf$

Balanced Snow Load: $p_S := C_s \cdot p_f = 27.72\ psf$

Unbalanced Snow Load:

$$\gamma := 0.13 \cdot \left(\frac{p_g}{psf}\right) + 14 = 18.29$$

$$l_{u1} := \sqrt{\left(\frac{L_1}{6 \cdot ft}\right)^2 + \left(\frac{L_1}{2 \cdot ft}\right)^2} = 13.176 \qquad l_{u2} := \sqrt{\left(\frac{L_2}{6 \cdot ft}\right)^2 + \left(\frac{L_2}{2 \cdot ft}\right)^2} = 23.717$$

$$h_{D1} := 0.43 \cdot \sqrt[3]{l_{u1}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} = 2.601 \qquad h_{D2} := 0.43 \cdot \sqrt[3]{l_{u2}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} = 3.164$$

$$p_{US1} := \frac{h_{D1} \cdot \gamma}{\sqrt{12}} \cdot psf = 13.732\ psf \qquad p_{US2} := \frac{h_{D2} \cdot \gamma}{\sqrt{12}} \cdot psf = 16.704\ psf$$

Wind loading: Open building with pitched free roof: Figure 27.3-5

Wind directionality factor: $K_d := 0.85$

Topographic factor: $K_{zt} := 1.0$

Ground elevation factor: $K_e := e^{-0.0000362 \cdot \frac{z_e}{ft}} = 0.972$

Gust-effect factor: $G := 0.85$ Rigid building

Internal pressure coefficient: $GC_{pi} := 0$ Open building

Velocity pressure coefficient: $h_{eave} := 9\ ft$ $h_{peak} := 13\ ft + 2\ in$

$$h := \frac{h_{peak} + h_{eave}}{2} = 11.083\ ft$$

$$\alpha := 9.8$$

$$K_h := 2.41 \cdot \left(\frac{15}{\frac{h}{ft}}\right)^{\frac{2}{\alpha}} = 2.564$$

Velocity pressure: $q_h := 0.00256 \cdot K_h \cdot K_{zt} \cdot K_e \cdot \left(\frac{V}{mph}\right)^2 = 67.694$

Angle: $\theta := \text{atan}\left(\frac{4}{12}\right) = 18.435\ °$

Dimensions:

Robin: $L_{Robin} := 25\ ft$ $\quad \frac{h}{L_{Robin}} = 0.443$

Cardinal: $L_{CardinalTrans} := 45\ ft$ $\quad \frac{h}{L_{CardinalTrans}} = 0.246$

$L_{CardinalLong} := 25\ ft$ $\quad \frac{h}{L_{CardinalLong}} = 0.443$

Net Pressure Coefficients:

Clear Wind Flow:

Case A: $C_{NWAClear} := 1.1$

$$C_{NLAClear} := -0.4 + (\theta - 15\ °) \cdot \left(\frac{0.1 + 0.4}{22.5\ ° - 15\ °}\right) = -0.171$$

Case B:

$$C_{NWBClear} := 0.1 + (\theta - 15\ °) \cdot \left(\frac{-0.1 - 0.1}{22.5\ ° - 15\ °}\right) = 0.008$$

$$C_{NLBClear} := -1.1 + (\theta - 15\ °) \cdot \left(\frac{-0.8 + 1.1}{22.5\ ° - 15\ °}\right) = -0.963$$

Obstructed Wind Flow:

Case A: $C_{NWAObs} := -1.2$

$$C_{NLAObs} := -1 + (\theta - 15\ °) \cdot \left(\frac{-1.2 - 1}{22.5\ ° - 15\ °}\right) = -2.008$$

Case B:

$$C_{NWBObs} := -0.6 + (\theta - 15\ °) \cdot \left(\frac{-0.8 + 0.6}{22.5\ ° - 15\ °}\right) = -0.692$$

$$C_{NLBObs} := -1.6 + (\theta - 15\ °) \cdot \left(\frac{-1.7 + 1.6}{22.5\ ° - 15\ °}\right) = -1.646$$

Design wind pressures:

$p_{WAClear} := q_h \cdot K_d \cdot G \cdot C_{NWAObs} \cdot psf = -58.69\ psf$

$p_{LAClear} := q_h \cdot K_d \cdot G \cdot C_{NLAObs} \cdot psf = -98.19\ psf$

$p_{WBClear} := q_h \cdot K_d \cdot G \cdot C_{NWBClear} \cdot psf = 0.41\ psf$

$p_{LBClear} := q_h \cdot K_d \cdot G \cdot C_{NLBClear} \cdot psf = -47.08\ psf$

$p_{WAObs} := q_h \cdot K_d \cdot G \cdot C_{NWAObs} \cdot psf = -58.69\ psf$

$p_{LAObs} := q_h \cdot K_d \cdot G \cdot C_{NLAObs} \cdot psf = -98.19\ psf$

$p_{WBObs} := q_h \cdot K_d \cdot G \cdot C_{NWBObs} \cdot psf = -33.83\ psf$

$p_{LBObs} := q_h \cdot K_d \cdot G \cdot C_{NLBObs} \cdot psf = -80.49\ psf$

Vertical loads:

$w_{wwy} := p_{WAClear} \cdot \sin(\theta) = -18.56\ psf$

$w_{wly} := p_{LAClear} \cdot \sin(\theta) = -31.05\ psf$

Horizontal loads:

$w_{wwx} := p_{WAClear} \cdot \cos(\theta) = -55.679\ psf$

$w_{wlx} := p_{LAClear} \cdot \cos(\theta) = -93.151\ psf$

Steel roof: $w_{dsteel} := 3\ psf$

Plywood sheathing: $w_{dOSB} := 0.4\ psf$ Per 1/8" thickness

Plywood waterproofing: $w_{dwp} := 3\ psf$ Per 1" thickness

Total dead load: $w_D := w_{dsteel} + (3 \cdot w_{dOSB}) + w_{dwp} = 7.2\ psf$

Factored vertical load: $w_u := \max(w_D + w_L, w_D + p_S, w_D + 0.6 \cdot w_{wly}, w_D + 0.75 \cdot w_L + 0.75 \cdot p_S + 0.75 \cdot (0.6 \cdot w_{wly})) = 34.92\ psf$

Factored horizontal load: $w_{wind} := 0.6 \cdot 93.4\ psf = 56.04\ psf$

Reference 3: Design of Sheathing

Design of Roof Sheathing

Unblocked OSB. 1 span

1. Bending controls

$l_1 := 24\ \mathbf{in}$

$F_b S \quad \geq \quad \dfrac{w_u \cdot l_1^{\,2}}{10} = 167.616\ \dfrac{\mathbf{lbf \cdot in}}{\mathbf{ft}}$

2. Span rating = 24/0

$F_b S := 300\ \dfrac{\mathbf{lbf \cdot in}}{\mathbf{ft}}$

$EI := 60000\ \dfrac{\mathbf{lbf \cdot in^2}}{\mathbf{ft}}$

$F_s := 130\ \dfrac{\mathbf{lbf}}{\mathbf{ft}}$

3. Adjustment factors

$C_D := 1.15 \qquad C_t := 1.0 \qquad C_M := 1.0 \qquad C_S := 1.0$

4. Adjusted design values

$F_b S' := F_b S \cdot C_D \cdot C_t \cdot C_M \cdot C_S = 345\ \dfrac{\mathbf{lbf \cdot in}}{\mathbf{ft}}$

$EI' := EI \cdot C_t \cdot C_M \cdot C_S = 60000\ \dfrac{\mathbf{lbf \cdot in^2}}{\mathbf{ft}}$

$F'_s := F_s \cdot C_t \cdot C_M \cdot C_S = 130\ \dfrac{\mathbf{lbf}}{\mathbf{ft}}$

5. Allowable uniform loads

$w_b := \dfrac{10 \cdot F_b S'}{l_1^{\,2}} = 71.875\ \mathbf{psf}$

$w_s := \dfrac{20 \cdot F'_s}{12 \cdot \left(l_1 - 3.5\ \mathbf{in}\right)} = 126.829\ \mathbf{psf}$

$\Delta_{LL} := \dfrac{l_1}{240} = 0.1\ \mathbf{in} \qquad w_{DLL} := \dfrac{1743 \cdot EI' \cdot \Delta_{LL}}{12 \cdot l_1^{\,4}} = 31.521\ \mathbf{psf}$

$\Delta_{tot} := \dfrac{l_1}{180} = 0.133\ \mathbf{in} \qquad w_{DTL} := \dfrac{1743 \cdot EI' \cdot \Delta_{tot}}{12 \cdot l_1^{\,4}} = 42.028\ \mathbf{psf}$

6. Design check

Bending:	$w_u = 34.92\ psf$	$\le$	$w_b = 71.875\ psf$	OK
Shear:	$w_u = 34.92\ psf$	$\le$	$w_s = 126.829\ psf$	OK
Short term deflection:	$p_S = 27.72\ psf$	$\le$	$w_{DLL} = 31.521\ psf$	OK
Long term deflection:	$p_S = 27.72\ psf$	$\le$	$w_{DTL} = 42.028\ psf$	OK

Select 24/0 OSB Sheathing

7. Thickness - Panel Design Specification Table 11

$$t := \frac{3}{8}\ in$$

8. Panel edge support - Manual for Engineered Wood Construction Table M9.4-1

Table M9.4-1 Panel Edge Support[2]

Sheathing Span Rating	Maximum Recommended Span (in.) With Edge Support	Maximum Recommended Span (in.) Without Edge Support
24/0	24	19.2[1]
24/16	24	24
32/16	32	28
40/20	40	32
48/24	48	36

1. 20 in. for 3/8 and 7/16 performance category panels, 24 in. for 15/32 and 1/2 performance category panels.
2. Additional edge support is recommended when panel widths are less than 24 inches. Edge support requirements should be obtained from the manufacturer.

With edge support: 24 in

Without edge support: 20 in

Edge support required -> use tongue and groove

Table M9.4-2 Minimum Nailing for Wood Structural Panel Applications

Application	Recommended Nail Size & Type	Nail Spacing (in.) Panel Edges	Nail Spacing (in.) Intermediate Supports
Single Floor–Glue-nailed installation[5]	**Ring- or screw-shank**		
16, 20, 24 oc, 3/4 performance category or less	6d[2]	6	12
24 oc, 7/8 or 1 performance category	8d[2]	6	12
32, 48 oc, (32-in. span (c-c) application)	8d[2]	6	12
48 oc, (48-in. span (c-c) application)	8d[2]	6	6
Single Floor–Nailed-only installation	**Ring- or screw-shank**		
16, 20, 24 oc, 3/4 performance category or less	6d	6	12
24 oc, 7/8 or 1 performance category	8d	6	12
32, 48 oc, (32-in. span application)	8d	6	12
48 oc, (48-in. span application)	8d[2]	6	6
Sheathing–Subflooring[1]	**Common smooth, ring- or screw-shank**		
7/16 to 1/2 thick performance category	6d	6	12
7/8 performance category or less	8d	6	12
Thicker panels	10d	6	6
Sheathing–Wall sheathing	**Common smooth, ring- or screw-shank or galvanized box[3]**		
7/16 performance category or less	6d	6	12
Over 7/16 performance category	8d	6	12
Sheathing–Roof sheathing	**Common smooth, ring- or screw-shank[3]**		
5/16 to 1 performance category	8d	6	12[4]
Thicker panels	8d ring- or screw-shank or 10d common smooth	6	12[4]

1. 8d common nails may be substituted if ring- or screw-shank nails are not available.
2. 10d ring-shank, screw-shank, or common nails may be substituted if supports are dry in accordance with NDS.
3. Other code approved fasteners may be used.
4. For spans 48 in. or greater, space nails 6 in. at all supports.
5. Use only adhesives conforming to ASTM D3498.

Nail size = 8d

Nail spacing:

6 in for panel edges

12 in for intermediate supports

24/0 OSB sheathing, 0.375 in thick

Reference 4: Design of Purlins

Design of Purlins

Factored load: $q_u := 35\ psf$

Tributary width: $w_T := 2\ ft$ $\quad w_u := q_u \cdot w_T = 70\ plf$

Max. moment and shear: $M := \dfrac{w_u \cdot (12.5\ ft)^2}{8} = 1.367\ kip \cdot ft$

$V := \dfrac{w_u \cdot 12.5\ ft}{2} = 0.438\ kip$ $\quad R := V$

4x6 Douglas Fir-Larch No.2: $L := 12.5\ ft$

Bending: $F_b := 900\ psi$

Shear: $F_v := 180\ psi$

Young's Modulus: $E := 1600\ ksi$ $\quad E_{min} := 580\ ksi$

Section properties: $b := 3.5\ in$ $\quad d := 5.5\ in$

$A := b \cdot d = 19.25\ in^2$

$I := \dfrac{1}{12} \cdot b \cdot d^3 = 48.526\ in^4$

$S := \dfrac{I}{\frac{d}{2}} = 17.646\ in^3$

Adjustment factors:

$C_D := 1.15$ $\quad C_i := 1.0$ $\quad C_T := 1.0$

$C_{Mb} := 0.85$ $\quad C_{MV} := 0.97$ $\quad C_F := 1.3$

$C_r := 1.15$ $\quad C_t := 1.0$ $\quad C_{fu} := 1.05$

Beam stability factor, CL: $l_u := 12.5\ ft$ $\quad \frac{l_u}{d} = 27.273 \quad > 7$

$l_e := 1.63 \cdot l_u + d = 20.833\ ft$

$E'_{min} := E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T = 580\ ksi$

$R_B := \sqrt{\frac{l_e \cdot d}{b^2}} = 10.595 \qquad F_{bE} := \frac{1.2 \cdot E'_{min}}{R_B^{\ 2}} = 6200.7\ psi$

$F_{bstar} := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i \cdot C_r = 1547\ psi$

$$C_L := \frac{1 + \left(\frac{F_{bE}}{F_{bstar}}\right)}{1.9} - \sqrt{\left(\frac{1 + \left(\frac{F_{bE}}{F_{bstar}}\right)}{1.9}\right)^2 - \frac{\frac{F_{bE}}{F_{bstar}}}{0.95}} = 0.984$$

Working stresses: $f_b := \frac{M}{S} = 929.752\ psi$

$f_v := \frac{3\ V}{2 \cdot b \cdot d} = 34.091\ psi$

Adjusted design values:

$F'_b := F_b \cdot C_D \cdot C_{Mb} \cdot C_t \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 1358.875\ psi$

$F'_v := F_v \cdot C_D \cdot C_{MV} \cdot C_t \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 310.143\ psi$

Check design:

Positive bending:

$F'_b = 1358.875\ psi \quad > \quad f_b = 929.752\ psi$ OK

$OCR := \frac{f_b}{F'_b} = 0.684$

Shear:

$F'_v = 310.143\ psi \quad > \quad f_v = 34.091\ psi$ OK

$OCR := \frac{f_v}{F'_v} = 0.11$

Deflection:

$$I := \frac{1}{12} \cdot b \cdot d^3 = 48.526 \; in^4$$

$$w_{LT} := \left(w_D + \frac{p_S}{2} \right) \cdot w_T = 0.042 \; \frac{kip}{ft} \qquad w_{ST} := \left(\frac{p_S}{2} \right) \cdot w_T = 0.028 \; \frac{kip}{ft}$$

$$\delta_{LT} := \frac{5 \cdot w_{LT} \cdot (12.5 \; ft)^4}{384 \cdot E \cdot I} = 0.298 \; in \qquad \delta_{ST} := \frac{5 \cdot w_{ST} \cdot (12.5 \; ft)^4}{384 \cdot E \cdot I} = 0.196 \; in$$

$$\delta_{tot} := (1.5 \cdot \delta_{ST}) + \delta_{LT} = 0.592 \; in$$

$\delta_{ST} = 0.196 \; in$ < $\Delta_{ST} := \frac{L}{360} = 0.417 \; in$ OK

$\delta_{tot} = 0.592 \; in$ < $\Delta_{tot} := \frac{L}{240} = 0.625 \; in$ OK

Purlins are 4x6 Douglas Fir-Larch No.2

*Designed for worst case purlin, use the same size for each

Reference 5: Design of Girders

Design of Girders

Length: $L := 18\ ft$

Factored load: $w_u := \frac{(8 \cdot R)}{L} = 0.194\ \frac{kip}{ft}$

Max. moment and shear: $M := \frac{w_u \cdot (L)^2}{8} = 7.875\ kip \cdot ft$

$V := \frac{w_u \cdot L}{2} = 1.75\ kip$

4x10 Douglas Fir-Larch No.2:

Bending: $F_b := 1500\ psi$

Shear: $F_v := 175\ psi$

Section properties: $b := 3.5\ in$ $d := 9.25\ in$

$A := b \cdot d = 32.375\ in^2$

$I := \frac{1}{12} \cdot b \cdot d^3 = 230.84\ in^4$

$S := \frac{I}{\frac{d}{2}} = 49.911\ in^3$

Adjustment factors: $C_D := 1.15$ $C_i := 1.0$

$C_{Mb} := 0.85$ $C_{MV} := 0.97$ $C_F := 1.1$

$C_r := 1.15$ $C_t := 1.0$ $C_{fu} := 1.05$

Beam stability factor, CL:

$l_u := 22.5\ ft$ $\qquad \dfrac{l_u}{d} = 29.189 \quad > 7$

$l_e := 1.63 \cdot l_u + d = 37.446\ ft$

$E'_{min} := E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T = 580\ ksi$

$R_B := \sqrt{\dfrac{l_e \cdot d}{b^2}} = 18.42 \qquad F_{bE} := \dfrac{1.2 \cdot E'_{min}}{{R_B}^2} = 2051.3\ psi$

$F_{bstar} := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i \cdot C_r = 2182\ psi$

$$C_L := \frac{1+\left(\dfrac{F_{bE}}{F_{bstar}}\right)}{1.9} - \sqrt{\left(\frac{1+\left(\dfrac{F_{bE}}{F_{bstar}}\right)}{1.9}\right)^2 - \frac{\dfrac{F_{bE}}{F_{bstar}}}{0.95}} = 0.791$$

Working stresses:

$f_b := \dfrac{M}{S} = 1893.353\ psi$

$f_v := \dfrac{3\ V}{2 \cdot b \cdot d} = 81.081\ psi$

Adjusted design values:

$F'_b := F_b \cdot C_D \cdot C_{Mb} \cdot C_t \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 1539.895\ psi$

$F'_v := F_v \cdot C_D \cdot C_{MV} \cdot C_t \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 205.017\ psi$

Check design:

Positive bending:

$F'_b = 1539.895\ psi \quad > \quad f_b = 1893.353\ psi$ OK

$OCR := \dfrac{f_b}{F'_b} = 1.23$

Shear:

$F'_v = 205.017\ psi \quad > \quad f_v = 81.081\ psi$ OK

$OCR := \dfrac{f_v}{F'_v} = 0.395$

Deflection:

$$I := \frac{1}{12} \cdot b \cdot d^3 = 230.84\ in^4$$

$$w_{LT} := \left(w_D + \frac{p_S}{2}\right) \cdot w_T = 0.042\ \frac{kip}{ft} \qquad w_{ST} := \left(\frac{p_S}{2}\right) \cdot w_T = 0.028\ \frac{kip}{ft}$$

$$\delta_{LT} := \frac{5 \cdot w_{LT} \cdot L^4}{384 \cdot E \cdot I} = 0.269\ in \qquad \delta_{ST} := \frac{5 \cdot w_{ST} \cdot L^4}{384 \cdot E \cdot I} = 0.177\ in$$

$$\delta_{tot} := (1.5 \cdot \delta_{ST}) + \delta_{LT} = 0.535\ in$$

$\delta_{ST} = 0.177\ in$ < $\Delta_{ST} := \frac{L}{360} = 0.6\ in$ OK

$\delta_{tot} = 0.535\ in$ < $\Delta_{tot} := \frac{L}{240} = 0.9\ in$ OK

Girders are 4x10 Douglas Fir-Larch No.2

*Designed for worst case girder, use the same size for each

Reference 6: Design of Columns, Trusses, and Knee-Braces

Design of Columns

Unbraced length: $K_e = 0.972$ $\quad l_e := 9\ ft$

Loading on columns: $P := V = 1.75\ kip$ $\quad w := 0.6 \cdot w_{wind} \cdot l_e = 0.303\ klf$

Design values: $F_c := 1150\ psi$ $\quad F_b := 1500\ psi$

$E := 1600\ ksi$ $\quad E_{min} := 580\ ksi$

Section size: $b := 5.5\ in$ $\quad d := 5.5\ in$

$A := b \cdot d = 30.25\ in^2$ $\quad S := \dfrac{b \cdot d^2}{6} = 27.729\ in^3$

Working stresses: $f_c := \dfrac{P}{A} = 57.851\ psi$

$M := \dfrac{w \cdot l_e^{\ 2}}{8} = 3.064\ kip \cdot ft$ $\quad f_b := \dfrac{M}{S} = 1325.963\ psi$

Adjustment factors: $C_D := 1.0$ $\quad C_M := 1.0$ $\quad C_t := 1.0$ $\quad C_F := 1.0$

$C_i := 1.0$ $\quad C_T := 1.0$ $\quad c := 0.8$ $\quad C_L := 1.0$

Column stability factor: $F_{cstar} := F_c \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i$

$E'_{min} := E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T$

$$F_{cE} := \frac{0.822 \cdot E'_{min}}{\left(\dfrac{l_e}{d}\right)^2} = \left(1.236 \cdot 10^3\right)\ psi$$

$$C_P := \frac{1 + \left(\dfrac{F_{cE}}{F_{cstar}}\right)}{2 \cdot c} - \sqrt{\left(\frac{\left(1 + \dfrac{F_{cE}}{F_{cstar}}\right)}{2 \cdot c}\right)^2 - \frac{\dfrac{F_{cE}}{F_{cstar}}}{c}} = 0.715$$

Adjusted design value: $F'_c := F_{cstar} \cdot C_P = 822.751\ psi$

$F'_b := F_b \cdot C_D \cdot C_{Mb} \cdot C_t \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 1539.563\ psi$

Amplification factor: $\beta_1 := \dfrac{1}{1 - \dfrac{f_c}{F_{cE}}} = 1.049$

Check design:

Compression stress:	$f_c = 57.9\ psi$	<	$F'_c = 822.8\ psi$	OK
Max. slenderness ratio:	$\lambda := \dfrac{l_e}{d} = 19.636$	<	50	OK
Bearing parallel to grain (crushing):	$f_{cnet} := f_c = 57.9\ psi$	<	$F_{cstar} = 1150\ psi$	OK
Bending:	$f_b = 1325.963\ psi$	<	$F'_b = 1540\ psi$	OK

Demand capacity ratios: $DCR_c := \dfrac{f_c}{F'_c} = 0.07$ $\qquad DCR_b := \dfrac{f_b}{F'_b} = 0.861$

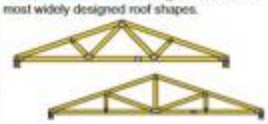
Columns are 6x6 Douglas Fir-Larch visually graded timbers.

Design of Trusses

Alpine truss designs are engineered to meet specific span, configuration and load conditions. The shapes and spans shown here represent only a fraction of the millions of designs produced by Alpine engineers.

Total load(PSF) / Duration factor, Live load(PSF), Roof type	55/1.15, 40 snow shingle; 55/1.15, 30 snow tile			47/1.15, 30 snow shingle			40/1.15, 20 snow shingle			40/1.25, 20 ** shingle, **construction or rain, not snow load		
Top Chord	2x4	2x6	2x6	2x4	2x6	2x6	2x4	2x6	2x6	2x4	2x6	2x6
Bottom Chord	2x4	2x4	2x6	2x4	2x4	2x6	2x4	2x4	2x6	2x4	2x4	2x6
Pitch	Spans in feet to out of bearing											
2/12	24	24	33	27	27	37	31	31	43	33	33	46
2.5/12	29	29	39	33	33	45	37	38	52	39	40	55
3/12	34	34	46	37	39	53	40	44	60	43	46	64
3.5/12	39	39	53	41	44	61	44	50	65	47	52	70
4/12	41	43	59	43	49	64	46	56	69	49	57	74
5/12	44	52	67*	46	58	69*	49	66	74*	53	66	80*
6/12	46	60*	69*	47	67*	71*	51	74*	76*	55	74*	82*
7/12	47	67*	70*	48*	72*	72*	52*	77*	77*	56*	80*	83*

Common – Truss configurations for the most widely designed roof shapes.



2x4 is suitable for chords and webs, recommend to use 4x10 (same as girders) for top/bottom chords and web members for consistency.

Design of Knee-Braces

Used to create moment frame connection between columns and trusses, use 4x6 for consistency with purlin size.

Reference 7: Design of Connections and Weight Check

Design of Connections

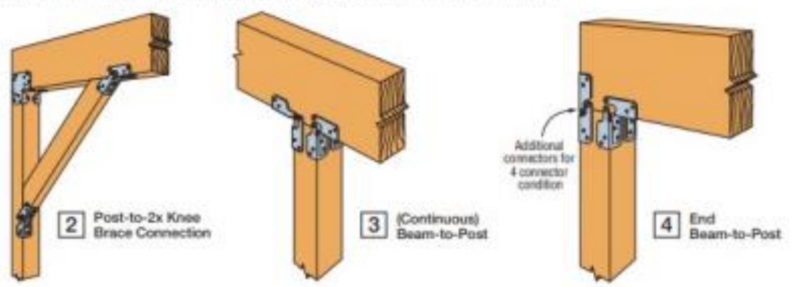
Knee-brace to column and beam connection:

Braces are 45 degrees

KBS17 knee-brace stabilizer

Model No.	Dimensions (in.)		Type of Connection	Connectors per Joint	Fasteners Each Connector	Direction of Load	Allowable Loads (160)				Code Ref.
							In-Service Moisture Content				
							≤ 19%		> 19%		
	W	L					DF/SP	SPF/HF	DF/SP	SPF/HF	
KBS1Z	1½	3	1	2	(12) 0.131 x 2½	F_1 – Brace angle = 45°	1,175	1,010	1,055	860	IBC, FL, LA
						F_1 – Brace angle = 30° or 60°	835	720	835	720	
			2	1	(12) 0.131 x 1½	F_1 – Brace angle = 45°	630	540	470	385	
						F_1 – Brace angle = 30° or 60°	510	440	395	330	
			3	4	(12) 0.131 x 2½	Uplift	1,160	1,000	1,160	1,000	
						Lateral	1,725	1,480	1,725	1,480	
			4	2	(12) 0.131 x 2½	Uplift	540	465	540	465	
						Lateral	485	420	430	370	
				4		Uplift	900	775	900	775	
						Lateral	1,270	1,095	1,270	1,095	

1. Allowable loads have been increased for wind or earthquake loading with no further increase allowed. Reduce where other loads govern.
2. For braces installed at intermediate angles, allowable loads may be interpolated between loads listed for brace angle = 45° and those listed for brace angle = 30° or 60°.
3. **Fasteners:** Nail dimensions in the table are listed diameter by length. See pp. 21–22 for fastener information.



End beam to post:

Retro-fitting posts to concrete foundation

RPBZ Connector-Only Values

Model No.	Part Qty.	Post Size	Fasteners: Base Connection[4,5] Type	Fasteners: Base Connection[4,5] Qty.	Fasteners: Post Type	Fasteners: Post Qty.	Allowable Connector Loads (DF/SP) Uplift (160)	Allowable Connector Loads (DF/SP) F_2 (160)	Allowable Connector Loads (DF/SP) F_3 (160)	Code Ref.
	Connection To Concrete									
RPBZ	1	4x, 6x	⅝" Anchor bolt or ¼" Titen® 2 screw	2 anchors or 4 screws	¼" x 1½" SDS	4	1,500	860	485	IBC, FL
RPBZ	2	4x, 6x	⅝" Anchor bolt or ¼" Titen® 2 screw	4 anchors or 8 screws	¼" x 1½" SDS	8	2,235	1,115	1,115	IBC, FL
	Connection To Wood Framing									
RPBZ	1	4x, 6x	¼" x 3" SDS	4	¼" x 1½" SDS	4	1,335	860	485	IBC, FL
RPBZ	2	4x, 6x	¼" x 3" SDS	8	¼" x 1½" SDS	8	2,235	1,115	1,115	IBC, FL
RPBZ	1	4x, 6x	¼" x 1½" SDS	4	¼" x 1½" SDS	4	845	860	485	IBC, FL
RPBZ	2	4x, 6x	¼" x 1½" SDS	8	¼" x 1½" SDS	8	1,825	1,115	1,115	IBC, FL

RPBZ Anchorage-to-Concrete   1/4" x 1 1/2" SDS Heavy-Duty Connector screws

RPBZ Anchorage-to-Concrete Values

Model No.	Part Qty.	Post Size	Fasteners: Base Connection Type	Fasteners: Base Connection Qty.	Allowable Anchorage Loads: Uplift Uncracked	Allowable Anchorage Loads: Uplift Cracked	Allowable Anchorage Loads: F_2	Allowable Anchorage Loads: F_3
	Corner – Post Flush to Edge							
RPBZ	1	4x, 6x	¼" x 1¾" Titen 2 screw	4	750	—	820	820
RPBZ	1	4x, 6x	⅝"-diameter anchor	2	1,520	1,065	510	510
	Away From Edge							
RPBZ	1	4x, 6x	¼" x 1¾" Titen 2 screw	4	850	—	935	935
RPBZ	1	4x, 6x	⅝"-diameter anchor	2	2,190	1,565	1,265	1,265
RPBZ	2	4x, 6x	¼" x 1¾" Titen 2 screw	8	1,500	—	1,645	1,645
RPBZ	2	4x, 6x	⅝"-diameter anchor	4	3,635	2,595	1,730	1,730

1. Allowable load for design shall not exceed minimum of Connector Only Value and Anchorage to Concrete Value.
2. Allowable connector loads are based on DF/SP lumber. For SPF/HF, multiply table loads by 0.86.
3. Double 2x4s may be used in lieu of 4x4 post.
4. For installation on 6x or larger members, if four RPBZ post bases are used, allowable loads may be taken to be 1.5 x the tabulated two-part value.
5. For installations into concrete, the minimum compressive strength is f'_c = 2,500 psi. Designer is responsible for concrete member uplift design.
6. Away-From-Edge loads require face of wood post to be a minimum of 2½" away from near edge of concrete on all four sides of the post.
7. Allowable anchorage to concrete uplift and shear loads for the ⅝" diameter anchors are calculated per ACI 318-14. Shear loads assume cracked concrete while uplift loads consider both cracked and uncracked concrete values, and all are qualified for Wind and Seismic Design Categories A&B.
8. Embedment depth for these post-install anchors must be a minimum of 2¾" and are for use with SET-3G® or AT-XP® structural anchoring adhesives or Titen HD® screw anchors.
9. Allowable uplift and shear loads for the Titen® 2 masonry screws do not carry a particular "cracked" or "uncracked" designation.
10. Titen®2 masonry screws and non-stainless-steel Titen HD® screw anchors should be used only in interior-dry and non-corrosive environments.
11. Threads on Strong-Drive® SDS Heavy-Duty Connector screws installed into wood framing must be fully engaged into a structural wood member.

Allowable anchorage loads:

$$P_{all} := 3635\ \mathbf{psi} \cdot 1.5 = 785.16\ \mathbf{ksf}$$

Modified for 6x or larger members, columns are 6x6.

$$F_2 := 1730\ \mathbf{psi} \cdot 1.5 = 373.68\ \mathbf{ksf}$$

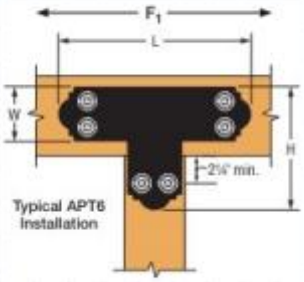
$$F_3 := 1730\ \mathbf{psi} \cdot 1.5 = 373.68\ \mathbf{ksf}$$

Truss chord-to-web:

Beam-to-Column Ties and Flat Straps

Model No.	Ga.	Dimensions (in.) W	L	H	Fastener Qty Column	Beam	DF/SP Allowable Loads Uplift/Tension (160)	F_1 (160)	Code Ref.
APL4	12	3	8¼	8¼	2	4	1,155	670	IBC, FL
APL6	12	5	11¼	11¼	4	6	1,905	1,340	
APT4	12	3	13½	8¼	2	4	1,330	1,015	
APT6	12	5	17½	11¼	4	8	2,130	1,425	
APST4	12	3	11¼	—	—	4	1,505	—	—
APST6	12	5	11¼	—	—	4	1,505	—	

1. Allowable loads have been increased for wind or earthquake loading with no further increase allowed. Reduce where other loads govern.
2. Connector table loads and fastener quantities are listed for two parts. For single part installations, use half the listed values.
3. All fasteners are Outdoor Accents® SDWS22312DBB structural wood screws inserted through an STN22 washer.
4. Fasteners sold separately.



Typical APT6 Installation

Truss gable plate:

Gable Plates

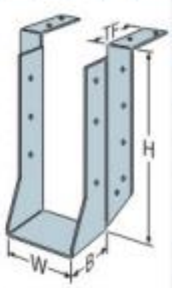
Model No.	Ga.	Roof Pitch	Angle (deg.)	Dimensions (in.) W	H	L	Fastener Qty. Beam	Center Column	Angled Struts	DF/SP Allowable Loads Uplift (160)	Code Ref.
APGP612	12	6:12	27°	5	20½	36	16	8	16	3,925	—
APGP812		8:12	34°								
APGP1212		12:12	45°								

1. Allowable loads have been increased for wind or earthquake loading with no further increase allowed. Reduce where other loads govern.
2. Connector table loads and fastener quantities are listed for two parts. For single part installations, use half the listed values.
3. Uplift loads apply to the connection between the center vertical post and the beam.
4. All fasteners are a Simpson Strong-Tie® SDWS22312DBB inserted through an STN22 washer. Quantities listed are for two parts.
5. Fasteners sold separately.

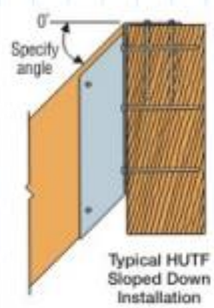
APGP1212 Shown

Joist-to-girder:

Joist or Purlin Size	Model No.	Ga.	Dimensions W	Dimensions H	Dimensions B	Dimensions TF	Fasteners (in.) Header	Fasteners (in.) Joist	DF/SP Allowable Loads Uplift (160)	DF/SP Allowable Loads Floor (100)	DF/SP Allowable Loads Snow (115)	DF/SP Allowable Loads Roof (125)	Installed Cost Index (ICI)	Code Ref.
Sawn Lumber Sizes														
DBL 2x16	WP216-2	12	3⅛	15	2½	2 9/16	(2) 0.148 x 3	(2) 0.148 x 3	—	3,300	3,300	3,300	Lowest	IBC, FL, LA
	HU216-2TF	12	3⅛	15	2½	2½	(20) 0.162 x 3½	(8) 0.148 x 3	1,400	4,050	4,050	4,050	34%	
TPL 2x16	HU216-3TF	12	4 11/16	15	2½	2½	(20) 0.162 x 3½	(8) 0.162 x 3½	1,640	4,050	4,050	4,050	Lowest	
3x4	HU34TF	12	2 9/16	3 3/16	2½	2½	(8) 0.162 x 3½	(2) 0.148 x 1½	370	2,050	2,050	2,050	•	
3x6	HU36TF	12	2 9/16	5⅜	2½	2½	(10) 0.162 x 3½	(4) 0.148 x 1½	705	2,785	2,785	2,785	•	
3x8	HU38TF	12	2 9/16	7⅛	2½	2½	(12) 0.162 x 3½	(4) 0.148 x 1½	640	3,265	3,265	3,265	•	
3x10	HU310TF	12	2 9/16	9⅛	2½	2½	(14) 0.162 x 3½	(6) 0.148 x 1½	1,220	3,945	3,945	3,945	•	
3x12	WP312	12	2 9/16	11	2½	2 9/16	(2) 0.148 x 3	(2) 0.148 x 1½	—	3,300	3,300	3,300	•	
	HU312TF	12	2 9/16	11	2½	2½	(16) 0.162 x 3½	(6) 0.148 x 1½	1,140	4,590	4,590	4,590	•	
3x14	WP314	12	2 9/16	13	2½	2 9/16	(2) 0.148 x 3	(2) 0.148 x 1½	—	3,300	3,300	3,300	•	
	HU314TF	12	2 9/16	13	2½	2½	(18) 0.162 x 3½	(8) 0.148 x 1½	1,065	4,030	4,030	4,030	•	
3x16	WP316	12	2 9/16	15	2½	2 9/16	(2) 0.148 x 3	(2) 0.148 x 1½	—	3,300	3,300	3,300	•	
	HU316TF	12	2 9/16	15	2½	2½	(20) 0.162 x 3½	(8) 0.148 x 1½	1,125	4,050	4,050	4,050	•	
4x3	HU43TF	12	3 9/16	3	2½	2½	(8) 0.162 x 3½	(2) 0.148 x 3	330	2,600	2,600	2,600	•	—
4x4	HU44TF	12	3 9/16	3 3/16	2½	2½	(8) 0.162 x 3½	(2) 0.148 x 3	370	2,050	2,050	2,050	Lowest	
4x6	HU46TF	12	3 9/16	5⅜	2½	2½	(10) 0.162 x 3½	(4) 0.148 x 3	815	2,785	2,785	2,785	28%	
	WP46	12	3 9/16	5⅜	2½	2 9/16	(2) 0.148 x 3	(2) 0.148 x 1½	—	3,300	3,300	3,300	•	



HU410TF



Weight Check

Unit weight of Douglas Fir-Larch No. 2: $\gamma_{wood} := 31 \ \frac{lbf}{ft^2}$

Estimated unit weight of steel for existing pavilions (based on W12x50): $\gamma_{steel} := 50 \ \frac{lbf}{ft^2}$

Smaller unit weight for wood for same size pavilion will yield a smaller weight for the replacement pavilions, existing concrete slabs will be sufficient with retrofitting of columns.

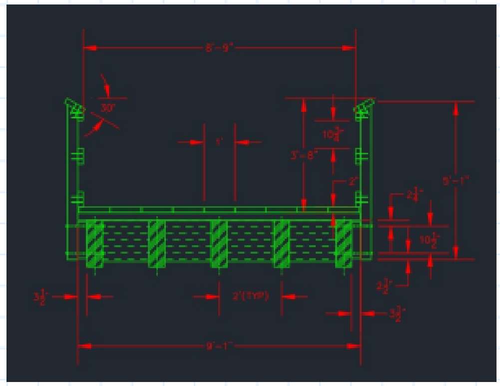
Appendix D: Pedestrian Bridges

Reference 1: Dead Load Calculations

Dead Load Calculations

Date: 03/04/2024

Bridge Cross-Section:



Bridge Components:

Continuous Components:

- Girders (6" x 18" Douglas Fir beams)
- Deck (3" x 12" x 109" horizontal Douglas Fir planks)
- Running Planks (2" x 12" Douglas Fir planks)
- Rails (2" x 6" S4S Rail)
- Rail Cap (2" x 8" S4S Rail Cap)

Spaced Components:

- Posts (4" x 6" x 5'-1" Post Fabricated)
- Rail Attachment Blocking (4" x 6" x 1'-2" S4S Blocking)

Calculations: (per girder) $N_{girder} := 5$

--> For continuous components: $w_{cont} = \gamma \cdot A \cdot \left(\frac{N_x}{N_{girder}} \right)$

--> For spaced components: $w_{spaced} = \frac{P}{s} \cdot \left(\frac{N_x}{N_{girder}} \right)$

Material		Unit Weight (kcf)
Aluminum Alloys		0.175
Bituminous Wearing Surfaces		0.140
Cast Iron		0.450
Cinder Filling		0.060
Compacted Sand, Silt, or Clay		0.120
Concrete	Lightweight	0.110 to 0.135
	Normal Weight with $f'_c \leq 5.0$ ksi	0.145
	Normal Weight with $5.0 < f'_c \leq 15.0$ ksi	$0.140 + 0.001 f'_c$
Loose Sand, Silt, or Gravel		0.100
Soft Clay		0.100
Rolled Gravel, Macadam, or Ballast		0.140
Steel		0.490
Stone Masonry		0.170
Wood	Hard	0.060
	Soft	0.050
Water	Fresh	0.0624
	Salt	0.0640
Item		Weight per Unit Length (klf)
Transit Rails, Ties, and Fastening per Track		0.200

Table 3.5.1-1: Unit Weights

Dead Load Calculations

Date: 03/04/2024

- Deck (3" x 12" x 109" horizontal Douglas Fir planks)

$A_{deck} := 3\ \mathbf{in} \cdot 109\ \mathbf{in} = 2.271\ \mathbf{ft}^2$ $\quad N_{deck} := 1$ $\quad \gamma_{softwood} := 50\ \frac{\mathbf{lb}}{\mathbf{ft}^3}$

$w_{deck} := \gamma_{softwood} \cdot A_{deck} \cdot \left(\frac{N_{deck}}{N_{girder}}\right) = 22.708\ \frac{\mathbf{lb}}{\mathbf{ft}}$ $\quad$ (Table 3.5.1-1)

- Girders (6" x 18" Douglas Fir beams)

$A_{girder} := 6\ \mathbf{in} \cdot 18\ \mathbf{in} = 0.75\ \mathbf{ft}^2$ $\quad N_{girder} = 5$ $\quad \gamma_{softwood} = 50\ \frac{\mathbf{lb}}{\mathbf{ft}^3}$

$w_{girder} := \gamma_{softwood} \cdot A_{girder} \cdot \left(\frac{N_{girder}}{N_{girder}}\right) = 37.5\ \frac{\mathbf{lb}}{\mathbf{ft}}$

*TABLE-1: SOLID SAWN STRINGER SIZE REQUIREMENTS - LRFD

**STRINGER SPAN (FEET)	TIMBER SPECIES - DOUGLAS FIR - LARCH GRADE - NO.1				
	DESIGN LOADING IN POUNDS PER SQUARE FOOT				
	PEDESTRIAN LIVE LOAD		GROUND SNOW LOAD		
	***65	90	120	150	200
●10	3" X 8"	3" X 10"	3" X 12"	4" X 10"	4" X 12"
●15	4" X 10"	4" X 12"	4" X 14"	4" X 16"	6" X 12"
●20	4" X 14"	6" X 12"	6" X 12"	6" X 14"	6" X 16"
▲25	6" X 14"	6" X 14"	6" X 16"	6" X 18"	6" X 20"
▲30	6" X 16"	6" X 18"	6" X 20"	6" X 20"	8" X 20"

- Running Planks (2" x 12" Douglas Fir planks)

$A_{planks} := 2\ \mathbf{in} \cdot 12\ \mathbf{in} = 0.167\ \mathbf{ft}^2$ $\quad N_{planks} := 9$ $\quad \gamma_{softwood} = 50\ \frac{\mathbf{lb}}{\mathbf{ft}^3}$

$w_{planks} := \gamma_{softwood} \cdot A_{planks} \cdot \left(\frac{N_{planks}}{N_{girder}}\right) = 15\ \frac{\mathbf{lb}}{\mathbf{ft}}$

- Rails (2" x 6" S4S Rail)

$A_{rail} := 2\ \mathbf{in} \cdot 6\ \mathbf{in} = 0.083\ \mathbf{ft}^2$ $\quad N_{rail} := 6$ $\quad \gamma_{hardwood} := 60\ \frac{\mathbf{lb}}{\mathbf{ft}^3}$

$w_{rail} := \gamma_{hardwood} \cdot A_{rail} \cdot \left(\frac{N_{rail}}{N_{girder}}\right) = 6\ \frac{\mathbf{lb}}{\mathbf{ft}}$ $\quad$ (Table 3.5.1-1)

- Rail Cap (2" x 8" S4S Rail Cap)

$A_{railcap} := 2\ \mathbf{in} \cdot 8\ \mathbf{in} = 0.111\ \mathbf{ft}^2$ $\quad N_{railcap} := 2$ $\quad \gamma_{hardwood} = 60\ \frac{\mathbf{lb}}{\mathbf{ft}^3}$

$w_{railcap} := \gamma_{hardwood} \cdot A_{railcap} \cdot \left(\frac{N_{railcap}}{N_{girder}}\right) = 2.667\ \frac{\mathbf{lb}}{\mathbf{ft}}$

Dead Load Calculations

Date: 03/04/2024

- Posts (4" x 6" x 5'-1" Post Fabricated)

$V_{post} := 4\ in \cdot 6\ in \cdot 61\ in = 0.847\ ft^3$ $\quad N_{post} := 2$ $\quad s_{post} := 5\ ft$ $\quad \gamma_{hardwood} = 60\ \frac{lb}{ft^3}$

$P_{post} := V_{post} \cdot \gamma_{hardwood} = 50.833\ lb$ $\quad w_{post} := \frac{P_{post}}{s_{post}} \cdot \left(\frac{N_{post}}{N_{girder}}\right) = 4.067\ \frac{lb}{ft}$

- Rail Attachment Blocking (4" x 6" x 1'-2" S4S Blocking)

$V_{blocking} := 4\ in \cdot 6\ in \cdot 14\ in = 0.194\ ft^3$ $\quad N_{blocking} := 2$ $\quad \gamma_{hardwood} = 60\ \frac{lb}{ft^3}$

$P_{blocking} := V_{blocking} \cdot \gamma_{hardwood} = 11.667\ lb$ $\quad s_{blocking} := 5\ ft$

$w_{blocking} := \frac{P_{blocking}}{s_{blocking}} \cdot \left(\frac{N_{blocking}}{N_{girder}}\right) = 0.933\ \frac{lb}{ft}$

- **Total Dead Load**

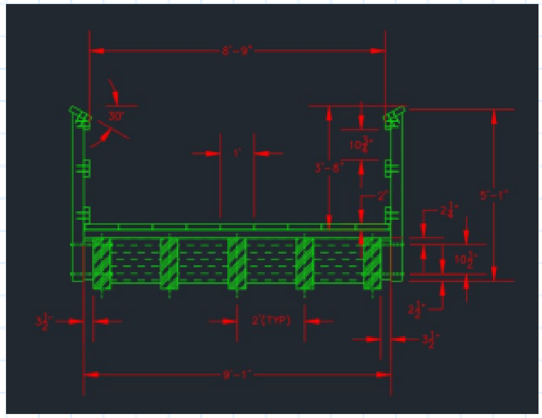
$$w_{DC} := w_{girder} + w_{deck} + w_{planks} + w_{rail} + w_{railcap} + w_{post} + w_{blocking} = 88.875\ \frac{lb}{ft}$$

Reference 2: Live and Snow Load Calculations

Live and Snow Load Calculations

Date: 03/08/2024

Bridge Cross-Section:



Bridge Components:

Width: $width := 9.083\ ft$

Live Load Calculations: (per girder) $N_{girder} := 5$

- Design Live Load: $LL := 90\ \frac{lb}{ft^2}$ <-- from Section 3.1 of *AASHTO LRFD Guide Specifications for the Design of Pedestrain Bridges*

$w_{LL} := LL \cdot width = 817.47\ \frac{lb}{ft}$

Snow Load Calculations: (per girder) $N_{girder} = 5$

- Design Snow Load: $S := 20\ \frac{lb}{ft^2}$ $w_S := S \cdot width = 181.66\ \frac{lb}{ft}$

FIGURE 1608.2 GROUND SNOW LOADS, p_g, FOR THE UNITED STATES (psf)

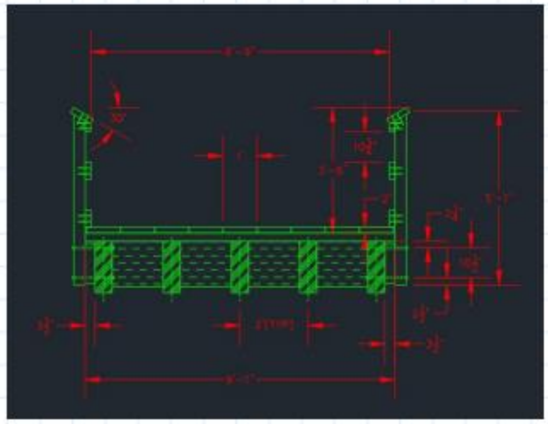
--> Figure 1608.2 is from Chapter 16 of the IBC

Reference 3: Load Combinations

Load Combinations

Date: 03/08/2024

Bridge Cross-Section:



Bridge Components:

Width: $width := 9.083\ ft$

Load Combinations: (per girder) $N_{girder} := 5$

--> Table is from the U.S. Forest Service Sustainable Trail Bridge Design Manual and is based on the AASHTO LRFD Bridge Design Specifications

Table 10—Load combinations and load factors the Forest Service uses to design trail bridge components.

	Description	Load combination and load factors*
	Live loads	1.25 × D + 1.75 × L
	Snow load only	1.25 × D + 1.75 × S
Max Loading Scenario ->	Snow trail users (case 1)	1.25 × D + 1.75 × L + 0.50 × S
	Snow trail users (case 2)	1.25 × D + 1.00 × L + 1.75 × S
	Snow groomer	1.25 × D + 1.75 × L + 1.75 × S
	Covered bridge (case 3)	1.25 × D + 1.75 × L + 0.50 × S
	Covered bridge (case 4)	1.25 × D + 1.00 × L + 1.75 × S

* Abbreviations: D = Dead loads L = Live loads S = Snow loads

$w_{DC} := 88.875\ \frac{lb}{ft}$ $w_{LL} := 817.47\ \frac{lb}{ft}$ $w_S := 181.66\ \frac{lb}{ft}$

$$w_{combined1} := (1.25 \cdot w_{DC}) + (1.75 \cdot w_{LL}) + (0.50 \cdot w_S) = 1632.496\ \frac{lb}{ft}$$

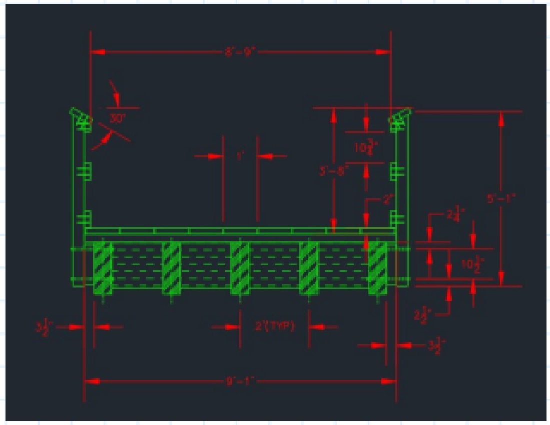
$$w_{combined2} := (1.25 \cdot w_{DC}) + (1.00 \cdot w_{LL}) + (1.75 \cdot w_S) = 1246.469\ \frac{lb}{ft}$$

Reference 4: Vehicle Load Calculations

Vehicle Load Calculations

Date: 03/08/2024

Bridge Cross-Section:



Bridge Components:

Width: $width := 9.083\ ft$

Live Load Calculations: (per girder) $N_{girder} := 5$

- Per AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges:

- Clear Deck Width between 7 and 10 feet

Design Vehicle = H5

Table 3.2-1—Design Vehicle

Clear Deck Width	Design Vehicle
7 to 10 ft	H5
Over 10 ft	H10

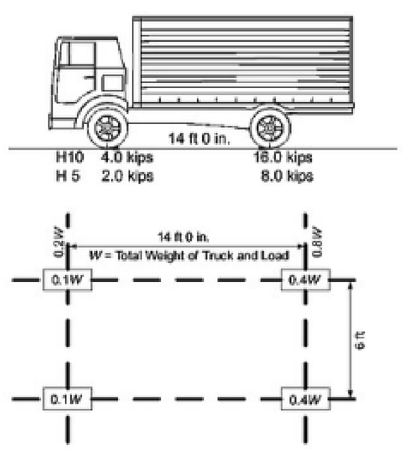


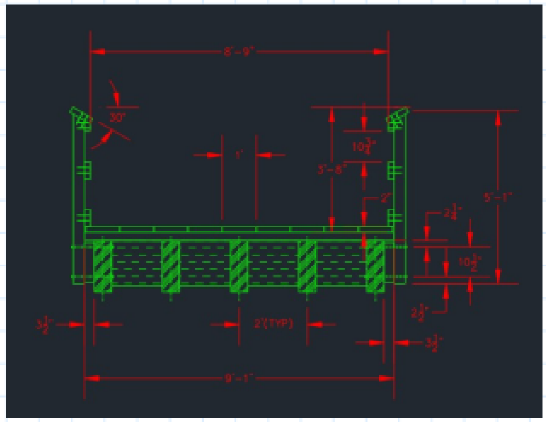
Figure 3.2-1—Maintenance Vehicle Configurations

Reference 5: Bridge 1 Forces

Bridge 1 Forces

Date: 03/29/2024

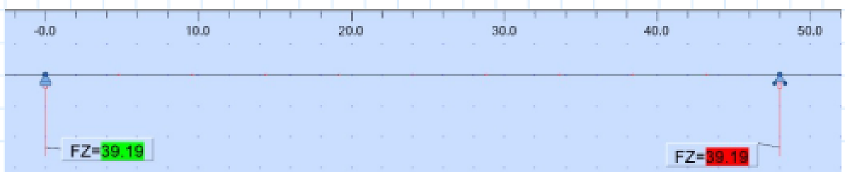
Bridge Cross-Section:



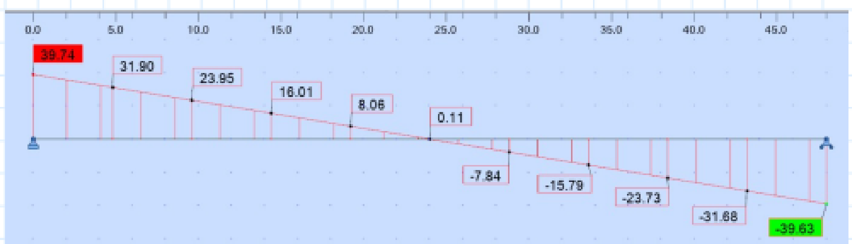
Combined Load Force Diagrams: $w_{combined} = (1.25 \cdot w_{DC}) + (1.75 \cdot w_{LL}) + (0.50 \cdot w_S)$

--> Without Midspan Pier:

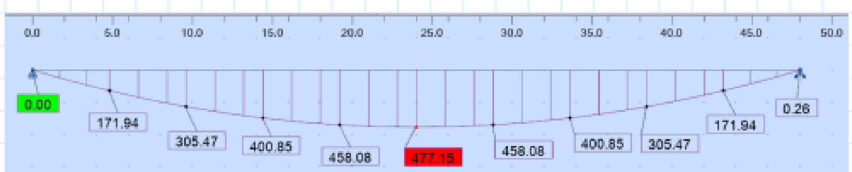
Reaction Forces



Shear Forces



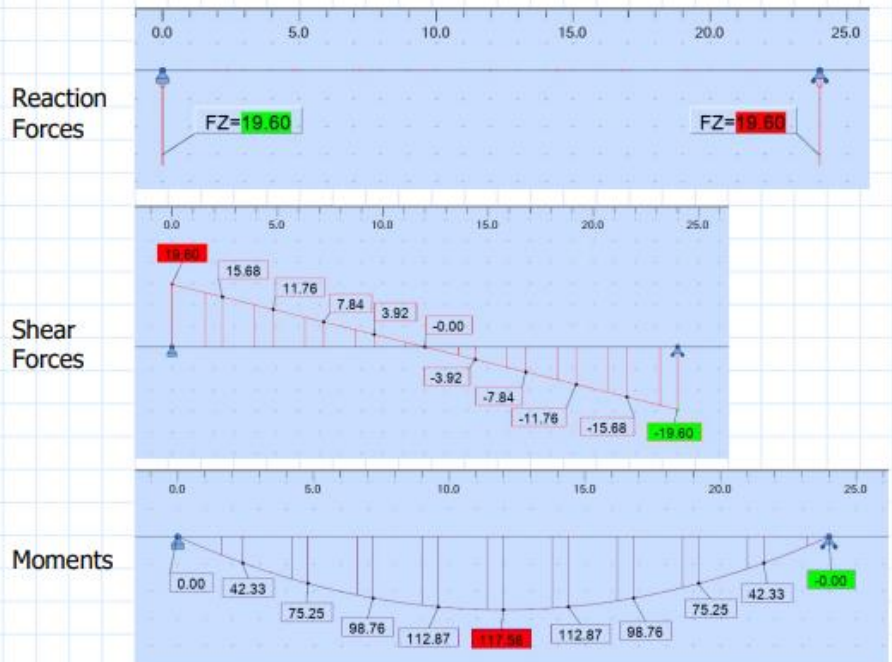
Moments



Bridge 1 Forces

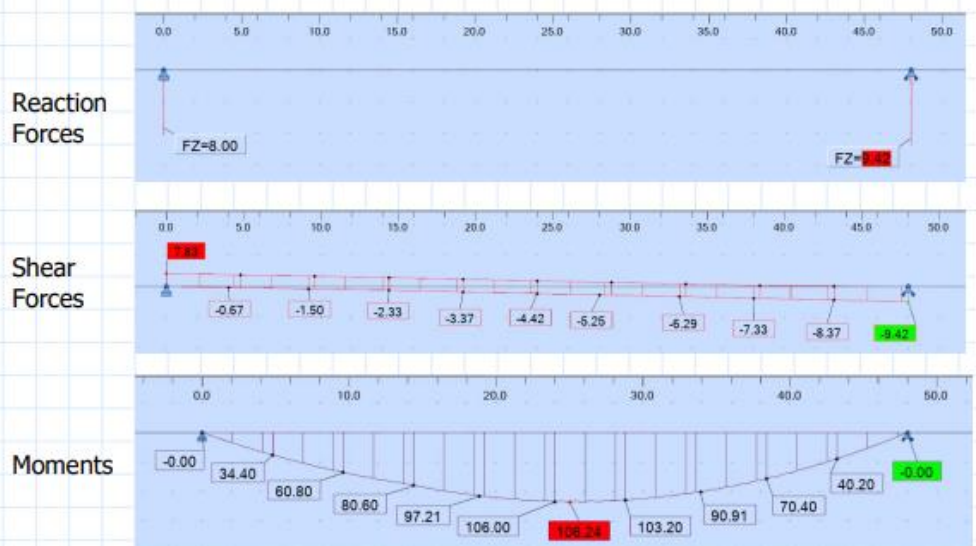
Date: 03/29/2024

--> With Midspan Pier: 2 spans, each with Length = 24 ft



H5 Truck Loading Force Diagrams:

--> Without Midspan Pier:

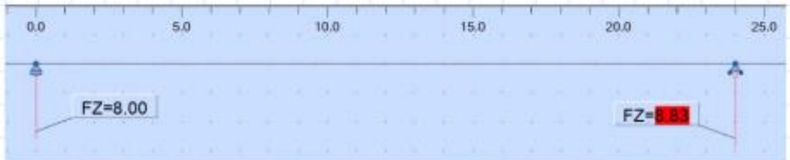


Bridge 1 Forces

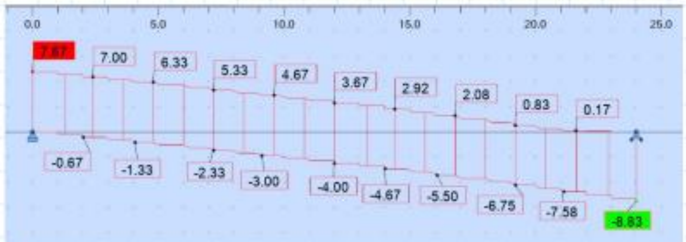
Date: 03/29/2024

--> With Midspan Pier: 2 spans, each with Length = 24 ft

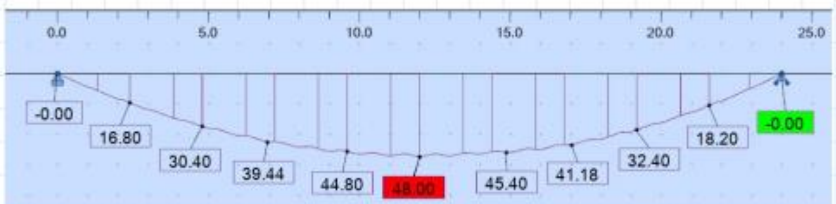
Reaction Forces



Shear Forces



Moments

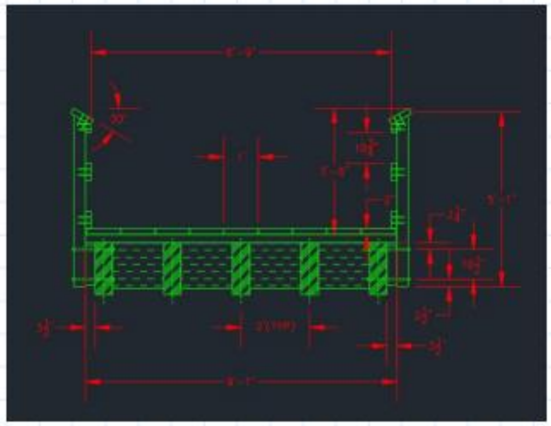


Reference 6: Bridge 2 Forces

Bridge 2 Forces

Date: 03/29/2024

Bridge Cross-Section:

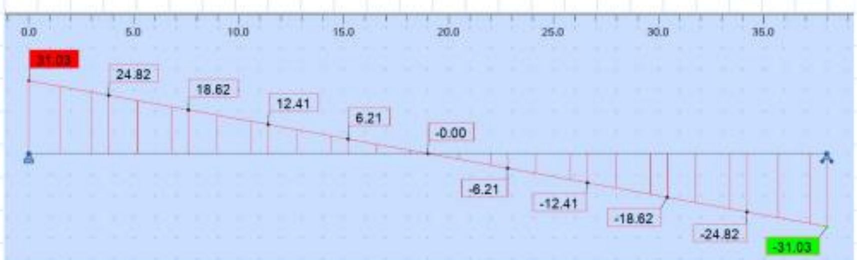


Combined Load Force Diagrams: $w_{combined} = (1.25 \cdot w_{DC}) + (1.75 \cdot w_{LL}) + (0.50 \cdot w_S)$

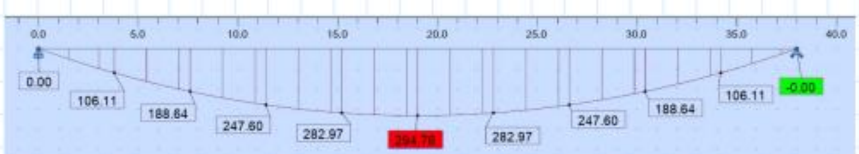
--> Without Midspan Pier:

Reaction Forces

Shear Forces



Moments



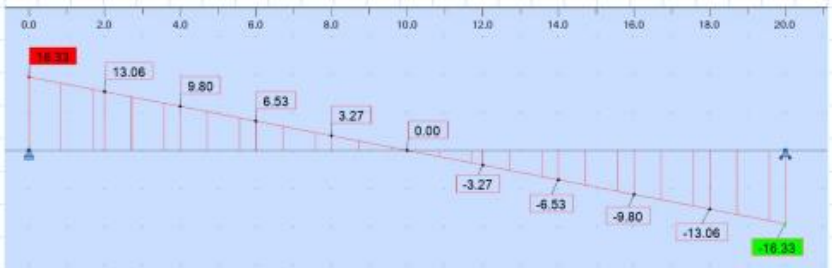
Bridge 2 Forces

Date: 03/29/2024

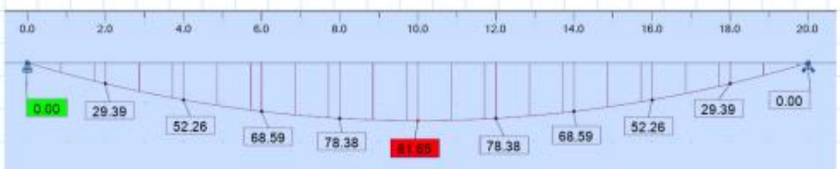
--> With Midspan Pier: 2 spans, each with Length = 20 ft

Reaction Forces

Shear Forces



Moments

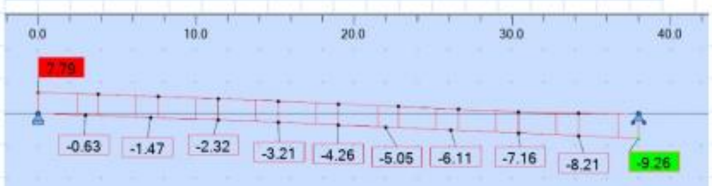


H5 Truck Loading Force Diagrams:

--> Without Midspan Pier:

Reaction Forces

Shear Forces



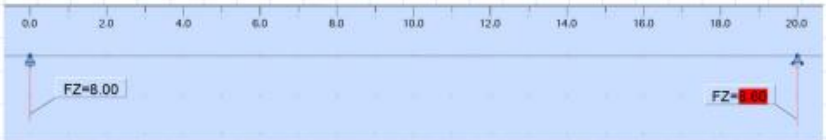
Moments

Bridge 2 Forces

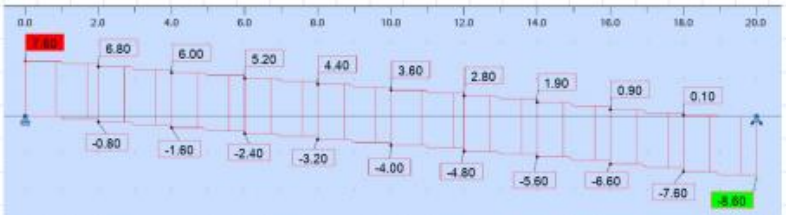
Date: 03/29/2024

--> With Midspan Pier: 2 spans, each at Length = 20 ft

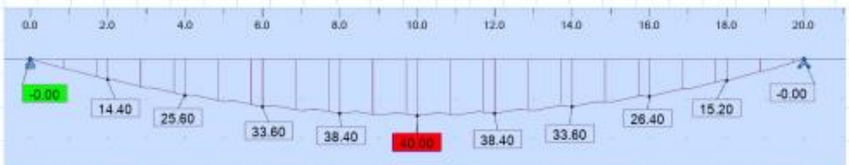
Reaction Forces



Shear Forces



Moments

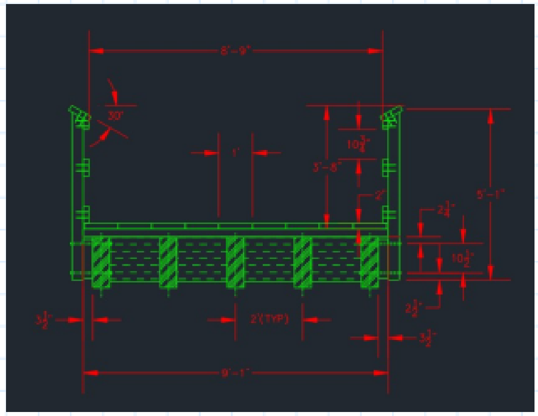


Reference 7: Bridge 3 Forces

Bridge 3 Forces

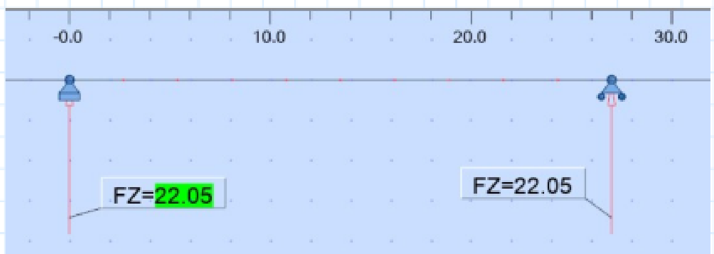
Date: 03/29/2024

Bridge Cross-Section:

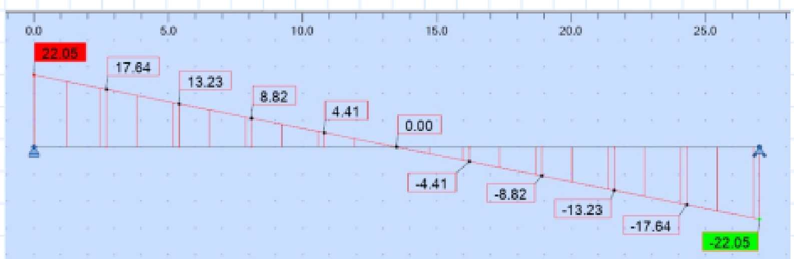


Combined Load Force Diagrams: $w_{combined} = (1.25 \cdot w_{DC}) + (1.75 \cdot w_{LL}) + (0.50 \cdot w_S)$

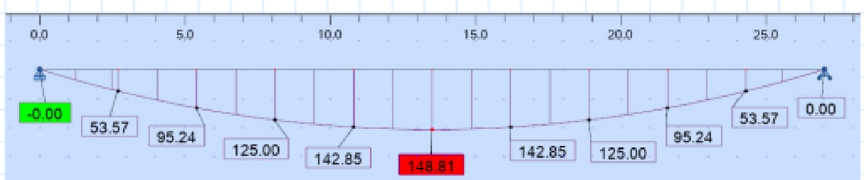
Reaction Forces



Shear Forces



Moments

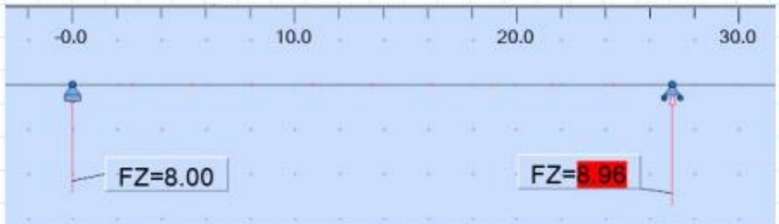


Bridge 3 Forces

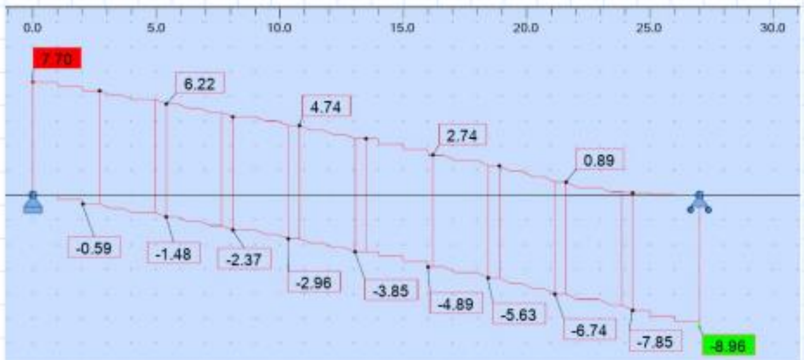
Date: 03/29/2024

H5 Truck Loading Force Diagrams:

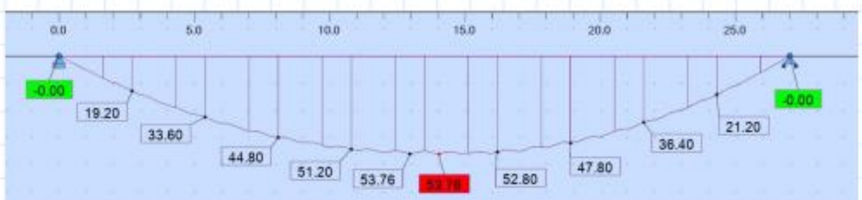
Reaction Forces



Shear Forces



Moments



Reference 8: Bridge 1 Abutment Calculations with Water

Retaining Wall Design Based on ACI 318-02

Bridge Surface =	743.00	ft		
Top of abutment =	740.08	ft		
Water surface elevation =	738.00	ft		
Under bridge ground elevation =	739.50	ft		

INPUT DATA & DESIGN SUMMARY

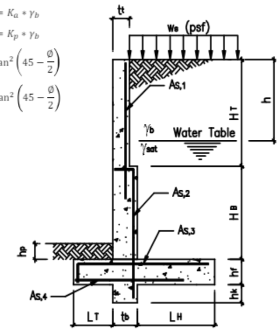
CONCRETE STRENGTH	f_c'	=	3	ksi
REBAR YIELD STRESS	f_y	=	40	ksi
LATER SOIL PRESSURE	$P_a=k_a \gamma_b$	=	58.83	pcf (equivalent fluid pressure)
ACTIVE EARTH PRESSURE COEFF.	K_a	=	0.490	
PASSIVE EARTH PRESSURE COEFF.	K_p	=	2.040	
FRICTION ANGLE	ϕ	=	20	deg
BACKFILL SPECIFIC WEIGHT	γ_b	=	120	pcf
SATURATED SPECIFIC WEIGHT	γ_{sat}	=	133.4	pcf
WATER SPECIFIC WEIGHT	γ_w	=	62.4	pcf
CONCRETE UNIT WEIGHT	γ_c	=	145	pcf
WATER TABLE	h	=	2.08	ft
PASSIVE PRESSURE	P_p	=	244.75	pcf
SURCHARGE WEIGHT	w_s	=	240	psf
FRICTION COEFFICIENT	μ	=	0.3	
ALLOW SOIL PRESSURE	Q_a	=	1.5	ksf — IBC Table 1806.2
THICKNESS OF TOP STEM	t_t	=	12	in
THICKNESS OF KEY & STEM	t_b	=	12	in
TOE WIDTH	L_T	=	1	ft
HEEL WIDTH	L_H	=	2	ft
HEIGHT OF TOP STEM	H_T	=	1.5	ft
HEIGHT OF BOT. STEM	H_B	=	1.5	ft
FOOTING THICKNESS	h_f	=	12	in
KEY DEPTH	h_k	=	0	in
SOIL OVER TOE	h_p	=	29.00	in
	y	=	3	ft
	L	=	4.00	ft
	B	=	10.00	ft

$P_a = K_a * \gamma_b$

$P_p = K_p * \gamma_b$

$K_a = \tan^2\left(45 - \frac{\emptyset}{2}\right)$

$K_p = \tan^2\left(45 - \frac{\emptyset}{2}\right)$



TOP STEM REINF. ($A_{s,1}$)	#	6	@	16	in o.c., (Caution > 18"o.c. max. ACI 14.3.5)	A_{bar} = 0.44	in^2
$A_{s,1}$ LOCATION (0=at soil face, 1=at middle)		0	at soil face				
BOT. STEM REINF. ($A_{s,2}$)	#	6	@	16	in o.c.	A_{bar} = 0.44	in^2
$A_{s,2}$ LOCATION (0=at soil face, 1=at middle)		0	at soil face				
TOP REINF.OF FOOTING ($A_{s,3}$)	#	5	@	16	in o.c., (Caution > 18"o.c. max. ACI 7.12.2.2)	A_{bar} = 0.31	in^2
BOT. REINF.OF FOOTING ($A_{s,4}$)	#	5	@	16	in	A_{bar} = 0.31	in^2

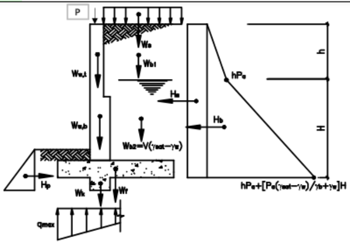
ANALYSIS

SERVICE LOADS

$H_b = 0.5 P_a h^2 + h P_a H + 0.5 [P_a (\gamma_{sat} - \gamma_w) / \gamma_b + \gamma_w] H^2$ = 0.40 kips

Where h = 2.0833 ft , H = 1.9167 ft

$H_s = w_s P_a (H_T + H_B + h_f) / \gamma_b$	=	0.47	kips
$H_p = 0.5 P_p (h_p + h_f + h_k)^2$	=	1.43	kips
$W_s = w_s (L_H + t_b - t_t)$	=	0.48	kips
$W_b = W_{b1} + W_{b2}$	=	0.77	kips
Where W_{b1} = 0.50 kips , W_{b2}	=	0.27	kips
$W_f = h_f (L_H + t_b + L_T) \gamma_c$	=	0.58	kips
$W_k = h_k\ t_b\ \gamma_c$	=	0.00	kips
$W_{w,t} = t_t\ H_T\ \gamma_c$	=	0.22	kips
$W_{w,b} = t_b\ H_B\ \gamma_c$	=	0.22	kips
$P = ((R_{... } + R_{...}) * N_{...}) / B$	=	14.22	kips



FACTORED LOADS

$\gamma H_b = 1.6 H_b$	=	0.65	kips
$\gamma H_s = 1.6 H_s$	=	0.75	kips
$\gamma W_s = 1.6 W_s$	=	0.77	kips
$\gamma W_b = 1.2 W_b$	=	0.93	kips
$\gamma W_f = 1.2 W_f$	=	0.70	kips
$\gamma W_k = 1.2 W_k$	=	0.00	kips
$\gamma W_{w,t} = 1.2 W_{w,t}$	=	0.26	kips
$\gamma W_{w,b} = 1.2 W_{w,b}$	=	0.26	kips
$\gamma P = 1.75 P$	=	24.88	kips

OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_b	0.40	0.65	1.42	0.57	0.92
H_s	0.47	0.75	2.00	0.94	1.51
Σ	0.88	1.40		**1.51**	2.42
	(kip)	(kip)	(ft)	(kip-ft)	(kip-ft)

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.48	0.77	3.00	1.44	2.30
W_b	0.77	0.93	3.00	2.32	2.78
W_f	0.58	0.70	2.00	1.16	1.39
W_k	0.00	0.00	1.50	0.00	0.00
$W_{w,t}$	0.22	0.26	1.50	0.33	0.39
$W_{w,b}$	0.22	0.26	1.50	0.33	0.39
P	14.22	24.88	1.50	21.32	37.31
Σ	16.48	27.79		**26.89**	44.57
	(kip)	(kip)	(ft)	(kip-ft)	(kip-ft)

OVERTURNING FACTOR OF SAFETY

$$SF = \frac{\Sigma Wx}{\Sigma Hy} = 17.76 > 1.5$$

PASS

CHECK SOIL BEARING CAPACITY (ACI 318-02 SEC.15.2.2)

$$L = L_T + t_b + L_H = 4.00 \text{ ft}$$

$$e = \frac{L}{2} - \frac{\Sigma Wx - \Sigma Hy}{\Sigma W} = 0.46 \text{ ft}$$

$$q_{MAX} = \begin{cases} \frac{\Sigma W\left(1+\frac{6e}{L}\right)}{BL}, & for \quad e \le \frac{L}{6} \\ \frac{\Sigma W}{3B(0.5L-e)}, & for \quad e > \frac{L}{6} \end{cases} = 0.70 \text{ ksf} < Q_a = 1.5 \text{ ksf}$$

PASS

CHECK FLEXURE CAPACITY. $A_{s,1}$ & $A_{s,2}$. FOR STEM (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

h = 2.0833 ft , H' = 0.9167 ft

$A = w_s P_a / \gamma_b$ = 118 plf

$B = h P_a$ = 123 plf

$C = [P_a (\gamma_{sat} - \gamma_w) / \gamma_b + \gamma_w] H'$ = 21 plf

At base of top stem

M_u = 0.60 ft-kips

V_u = 0.79 kips

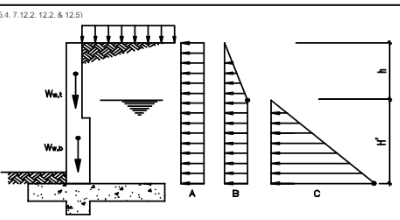
P_u = 25.40 kips

At base of bottom stem

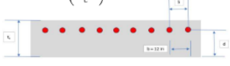
M_u = 0.60 ft-kips

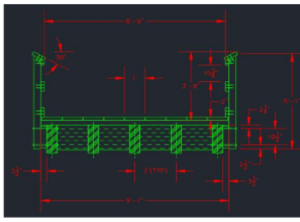
V_u = 0.79 kips

P_u = 25.40 kips



			At top stem		At base of bottom stem			
$\phi M_n = \phi\left[A_S f_y\left(d - \frac{A_S f_y - P_u}{1.7 b f_c'}\right)\right]$		=	13.37 PASS	ft-kips ,	13.37 PASS	ft-kips	>	M_u
$a = \frac{A_s f_y}{0.85 * b * f_c'}$, $c = \frac{a}{\beta_1}$ where	a	=	0.58	in ,	0.58	in		
	c	=	0.68	in ,	0.68	in		
	d	=	10.00	in ,	10.00	in		
$\varepsilon_s = \left(\frac{d-c}{c}\right) * 0.003$	ε_s	=	0.041 YIELDED		0.041 YIELDED			
	b	=	12	in ,	12	in		
	ϕ	=	0.9	,	0.9			
	A_s	=	0.44	in^2 ,	0.44	in^2		
	ρ	=	0.003		0.003			
	β_1	=	0.85		0.85			
$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f_c'}{f_y}\frac{87}{87+f_y}\right)$		=	0.028 PASS		0.028 PASS		>	ρ
$\rho_{MIN} = 0.0018\frac{t}{d}$		=	0.002 PASS		0.002 PASS		<	ρ





**client may need to add soil to match bridge surface to trail

Bridge Length = 48 ft

2 span bridge

Act. Earth Press. Coeff:	K_a =	$\tan^2(45-(\phi/2))$ =	K_a :	
Active Earth Pressure:	P_a =	$K_a * \gamma_{soil}$ =	<1	K_a = D/B
Pass. Earth Press. Coeff:	K_p =	$\tan^2(45+(\phi/2))$ =	>1	$K_a = \tan^{-1}(D/B)$
Passive Earth Pressure:	P_p =	$K_p * \gamma_{soil}$ =		

TABLE 1806.2 PRESUMPTIVE LOAD-BEARING VALUES

CLASS OF MATERIALS	VERTICAL FOUNDATION PRESSURE (psf)	LATERAL BEARING PRESSURE (psf/ft below natural grade)	LATERAL SLIDING RESISTANCE	
			Coefficient of friction[a]	Cohesion (psf)[b]
1. Crystalline bedrock	12,000	1,200	0.70	—
2. Sedimentary and foliated rock	4,000	400	0.35	—
3. Sandy gravel and gravel (GW and GP)	3,000	200	0.35	—
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC)	2,000	150	0.25	—
5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)	1,500	100	—	130

For SI: 1 pound per square foot = 0.0479kPa, 1 pound per square foot per foot = 0.157 kPa/m.
a. Coefficient to be multiplied by the dead load.
b. Cohesion value to be multiplied by the contact area, as limited by Section 1806.3.2.

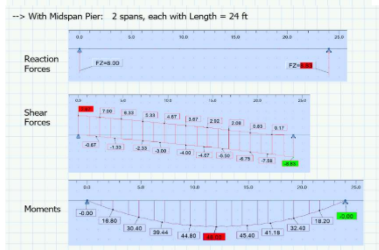
Table 5-17. Typical compacted densities and optimum moisture contents for USCS soil types (after Carter and Bentley, 1991).

Soil Description	USCS Class	Compacted Dry Unit Weight (lb/ft3)	Compacted Dry Unit Weight (kN/m3)	Optimum Moisture Content (%)
Gravel/sand mixtures:				
well-graded, clean	GW	125-134	19.6-21.1	8-11
poorly-graded, clean	GP	115-125	18.1-19.6	11-14
well-graded, small silt content	GM	119-134	18.6-21.1	8-12
well-graded, small clay content	GC	115-125	18.1-19.6	9-14
Sands and sandy soils:				
well-graded, clean	SW	109-131	17.2-20.6	9-16
poorly-graded, small silt content	SP	94-119	15.7-18.6	12-21
well-graded, small silt content	SM	109-125	17.2-19.6	11-16
well-graded, small clay content	SC	106-125	16.7-19.6	11-19
Fined-grained soils of low plasticity:				
silts	ML	94-119	14.7-18.6	12-24
clays	CL	94-119	14.7-18.6	12-24
organic silts	OL	81-100	12.7-15.7	21-33
Fine-grained soils of high plasticity:				
silts	MH	69-94	10.8-14.7	24-40
clays	CH	81-105	12.7-18.6	19-36
organic clays	OH	66-100	10.3-15.7	21-45

1. Overturning Analysis

Combined Loading:

H5 Truck Loading:



1. Overturning Analysis

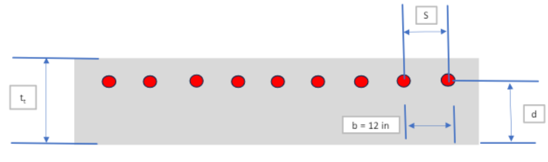
2. Bearing Capacity Analysis

2. Bearing Capacity Analysis

3. Flexure Capacity Analysis

$$M_u = (A*H)(0.5*H) + (B*H')(0.5*H') + (0.5*B*h)\left(H' + \frac{1}{3}h\right) + (0.5*C*H')\left(\frac{1}{3}H'\right)$$

$$V_u = (A*H) + (B*H') + (0.5*B*h) + (0.5*C*H')$$



$\beta_1 = 0.85$	for $0 < f_c' \le 4000$ psi
$\beta_1 = 0.85 - 0.05(f_c' - 4000)/1000$	for 4000 psi $< f_c' \le 8000$ psi
$\beta_1 = 0.65$	for $f_c' > 8000$ psi

3. Flexure Capacity Analysis

4. Shear Capacity Analysis

CHECK SHEAR CAPACITY FOR STEM (ACI 318-02 SEC.15.5.2, 11.1.3.1, & 11.3)

$$V_{allowable} = 2\phi b d \sqrt{f'_c}$$

	At top stem		At base of bottom stem			
=	9.86	kips ,	9.86	kips	>	V_u
	PASS		PASS			

where ϕ = 0.75 (ACI 318-02, Section 9.3.2.3) ϕ = 0.75

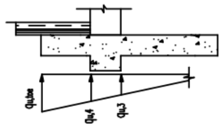
4. Shear Capacity Analysis

5. Heel Flexure Capacity Analysis

CHECK HEEL FLEXURE CAPACITY $A_{s,3}$ FOR FOOTING (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f'_c}{f_y}\frac{87}{87+f_y}\right) = 0.0278 \qquad \rho_{MIN} = \frac{0.0018}{2}\frac{h_f}{d} = 0.0054$$

$$M_{u,3} = \begin{cases} \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{(q_{u,3}+2q_{u,heel})bL_H^2}{6}, & for \quad e_u \le \frac{L}{6} \\ \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{q_{u,3}bS^2}{6}, & for \quad e_u > \frac{L}{6} \end{cases} = 0.91 \text{ ft-kips}$$



$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\frac{M_{u,3}}{0.383bd^2 f'_c}}\right)}{f_y} = 0.0067$$

PASS PASS

where							
d	=	2.00	in	$q_{u,toe}$	=	0.89	ksf
b	=	12.00	in	$q_{u,heel}$	=	0.50	ksf
e_u	=	0.46	ft	$q_{u,3}$	=	0.69	ksf
S	=	16.00	in	ϕ	=	0.9	

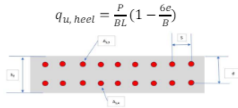
$$q_{u,toe} = \frac{P}{BL}\left(1+\frac{6e}{B}\right)$$

$$q_{u,heel} = \frac{P}{BL}\left(1-\frac{6e}{B}\right)$$

$$\phi M_n = \phi\left[A_S f_y\left(d - \frac{A_S f_y - P_u}{1.7bf'_c}\right)\right] = 0.91 \text{ ft-kips}$$

$(A_{s,3})_{required}$ = 0.13 in^2/ft < $A_{s,3}$ = 0.31 in^2/ft

PASS



5. Heel Flexure Capacity Analysis

6. Toe Flexure Capacity Analysis

CHECK TOE FLEXURE CAPACITY $A_{s,4}$ FOR FOOTING (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f'_c}{f_y}\frac{87}{87+f_y}\right) = 0.028 \qquad \rho_{MIN} = MIN\left(\frac{4}{3}\rho, \ \frac{0.0018}{2}\frac{h_f}{d}\right) = 0.0001$$

$$M_{u,4} = \frac{(q_{u,4}+2q_{u,toe})bL_T^2}{6} - \frac{L_T^2}{2L}\gamma w_f = 0.34 \text{ ft-kips}$$

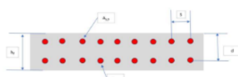
where			
d	=	10.00	in
$q_{u,4}$	=	0.79	ksf
b	=	12.00	in
A_s	=	0.31	in^2
ϕ	=	0.9	
ρ	=	0.0022	
		PASS	PASS

$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\frac{M_{u,4}}{0.383bd^2 f'_c}}\right)}{f_y} = 0.0001$$

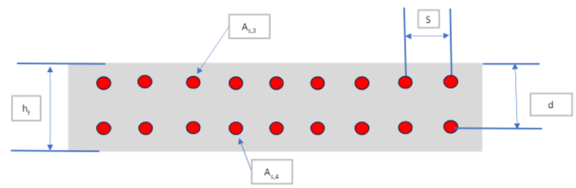
$$\phi M_n = \phi\left[A_S f_y\left(d - \frac{A_S f_y - P_u}{1.7bf'_c}\right)\right] = 0.62 \text{ ft-kips}$$

$(A_{s,4})_{required}$ = 0.02 in^2/ft < $A_{s,4}$ = 0.31 in^2/ft

PASS



toe - tension bottom
heel - tension top



6. Toe Flexure Capacity Analysis

7. Key Capacity Analysis

CHECK KEY CAPACITY FOR FOOTING

$1.5 (H_b + H_s)$ = 1.31 kips < $H_p + \mu \Sigma W$ = 6.37 kips

PASS

Techincal References:

1. Alan Williams: "Structuiral Engineering Reference Manual", Professional Publications, Inc, 2001.
2. Alan Williams: "Structuiral Engineering License Review Problems and Solutions", Oxford University Press, 2003.

Reference 9: Bridge 1 Abutment Calculations without Water

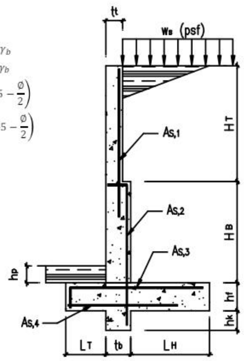
Retaining Wall Design Based on ACI 318-02

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=		3	ksi					
REBAR YIELD STRESS	f_y	=		40	ksi					
LATER SOIL PRESSURE	P_a	=		58.83	pcf (equivalent fluid pressure)					
PASSIVE PRESSURE	P_p	=		244.75	pcf					
ACTIVE EARTH PRESSURE COEFF.	K_a	=		0.490						
PASSIVE EARTH PRESSURE COEFF.	K_p	=		2.040						
FRICTION ANGLE	ϕ	=		20	deg					
SURCHARGE WEIGHT	w_s	=		240	psf					
FRICTION COEFFICIENT	μ	=		0.3						
ALLOW SOIL PRESSURE	Q_a	=		1.5	ksf	IBC Table 1806.2				
THICKNESS OF TOP STEM	t_t	=		12	in					
THICKNESS OF KEY & STEM	t_b	=		12	in					
TOE WIDTH	L_T	=		1	ft					
HEEL WIDTH	L_H	=		2	ft					
HEIGHT OF TOP STEM	H_T	=		1.5	ft					
HEIGHT OF BOT. STEM	H_B	=		1.5	ft					
FOOTING THICKNESS	h_f	=		12	in					
KEY DEPTH	h_k	=		0	in					
SOIL OVER TOE	h_p	=		29.00	in					
	y	=		3	ft					
	L	=		4.00	ft					
	B	=		10	ft					
UNIT WEIGHT OF SOIL	γ_b	=		120	pcf					
UNIT WEIGHT OF CONCRETE	γ_c	=		145	pcf					
TOP STEM REINF. ($A_{s,1}$)	1	#		6	@	16	in o.c., at middle	A_{bar} =	0.44	in^2
BOT. STEM REINF. ($A_{s,2}$)	2	#		6	@	16	in o.c., at each face	A_{bar} =	0.44	in^2
TOP REINF.OF FOOTING ($A_{s,3}$)		#		5	@	16	in	A_{bar} =	0.31	in^2
BOT. REINF.OF FOOTING ($A_{s,4}$)		#		5	@	16	in	A_{bar} =	0.31	in^2

$$P_a = K_a * \gamma_b \qquad P_p = K_p * \gamma_b$$

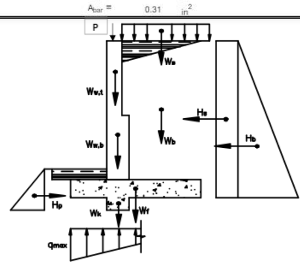
$$K_a = \tan^2\left(45 - \frac{\emptyset}{2}\right) \qquad K_p = \tan^2\left(45 - \frac{\emptyset}{2}\right)$$



ANALYSIS

SERVICE LOADS

$H_b = 0.5\, P_a (H_T + H_B + h_f)^2$	=	0.47	kips
$H_s = w_s P_a (H_T + H_B + h_f) / \gamma_b$	=	0.47	kips
$H_p = 0.5\, P_p (h_p + h_f + h_k)^2$	=	1.43	kips
$W_s = w_s (L_H + t_b - t_t)$	=	0.48	kips
$W_b = [H_T (L_H + t_b - t_t) + H_B L_H]\, \gamma_b$	=	0.72	kips
$W_f = h_f (L_H + t_b + L_T)\, \gamma_c$	=	0.58	kips
$W_k = h_k\, t_b\, \gamma_c$	=	0.00	kips
$W_{w,t} = t_t\, H_T\, \gamma_c$	=	0.22	kips
$W_{w,b} = t_b\, H_B\, \gamma_c$	=	0.22	kips
$P = R_{comb} + R_{H5}$	=	14.22	kips



FACTORED LOADS

$\gamma H_b = 1.6\, H_b$	=	0.75	kips
$\gamma H_s = 1.6\, H_s$	=	0.75	kips
$\gamma W_s = 1.6\, W_s$	=	0.77	kips
$\gamma W_b = 1.2\, W_b$	=	0.86	kips
$\gamma W_f = 1.2\, W_f$	=	0.70	kips
$\gamma W_k = 1.2\, W_k$	=	0.00	kips
$\gamma W_{w,t} = 1.2\, W_{w,t}$	=	0.26	kips
$\gamma W_{w,b} = 1.2\, W_{w,b}$	=	0.26	kips
$\gamma P = 1.75\, P$	=	24.88	kips

OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_b	0.47	0.75	1.33	0.63	1.00
H_s	0.47	0.75	2.00	0.94	1.51
Σ	0.94	1.51		**1.57**	2.51
	(kip)	(kip)	(ft)	(kip-ft)	(kip-ft)

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.48	0.77	3.00	1.44	2.30
W_b	0.72	0.86	3.00	2.16	2.59
W_f	0.58	0.70	2.00	1.16	1.39
W_k	0.00	0.00	1.50	0.00	0.00
$W_{w,t}$	0.22	0.26	1.50	0.33	0.39
$W_{w,b}$	0.22	0.26	1.50	0.33	0.39
P	14.22	24.88	1.50	21.32	37.31
Σ	16.43	27.73		**26.74**	44.39
	(kip)	(kip)	(ft)	(kip-ft)	(kip-ft)

OVERTURNING FACTOR OF SAFETY

$$SF = \frac{\Sigma Wx}{\Sigma Hy} = 17.04 > 1.5$$

PASS

CHECK SOIL BEARING CAPACITY (ACI 318-02 SEC.15.2.2)

$$L = L_T + t_b + L_H = 4.00 \text{ ft}$$

$$e = \frac{L}{2} - \frac{\Sigma Wx - \Sigma Hy}{\Sigma W} = 0.47 \text{ ft}$$

$$q_{MAX} = \begin{cases} \dfrac{\Sigma W\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{\Sigma W}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases} = 0.70 \text{ ksf} < Q_a = 1.5 \text{ ksf}$$

PASS

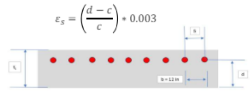
CHECK FLEXURE CAPACITY, $A_{s,1}$ & $A_{s,2}$, FOR STEM (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$M_u = \left(\frac{w_s P_a}{\gamma_b} * H\right)(0.5 * H) + (0.5 * Pa * H^2)\left(\frac{1}{3} * H\right) = 0.79 \text{ ft-kips}$$

P_u = 25.40 kips

$$\phi M_n = \phi\left[A_s f_y \left(d - \frac{A_s f_y - P_u}{1.7 b f_c'}\right)\right]$$

$$a = \frac{A_s f_y}{0.85 * b * f_c'} \qquad c = \frac{a}{\beta_1} \qquad \varepsilon_s = \left(\frac{d-c}{c}\right) * 0.003$$



where		At top stem		At base of bottom stem			
ϕM_n	=	13.37 PASS	ft-kips ,	13.37 PASS	ft-kips	>	M_u
a	=	0.58	in ,	0.58	in		
c	=	0.68	in ,	0.68	in		
d	=	10.00	in ,	10.00	in		
ε_s	=	0.041 YIELDED		0.041 YIELDED			
b	=	12	in ,	12	in		
ϕ	=	0.9	,	0.9			
A_s	=	0.44	in^2 ,	0.44	in^2		
ρ	=	0.003		0.003			
β_1	=	0.85		0.85			
$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f_c'}{f_y}\frac{87}{87+f_y}\right)$	=	0.028 PASS		0.028 PASS		>	ρ
$\rho_{MIN} = 0.0018\frac{t}{d}$	=	0.002 PASS		0.002 PASS		<	ρ

Act. Earth Press. Coeff:	K_a =	$\tan^2(45-(\phi/2))$ =
Active Earth Pressure:	P_a =	$K_a * \gamma_{soil}$ =
Pass. Earth Press. Coeff:	K_p =	$\tan^2(45+(\phi/2))$ =
Passive Earth Pressure:	P_p =	$K_p * \gamma_{soil}$ =

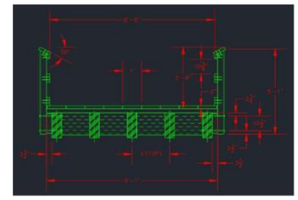


TABLE 1806.2 PRESUMPTIVE LOAD-BEARING VALUES

CLASS OF MATERIALS	VERTICAL FOUNDATION PRESSURE (psf)	LATERAL BEARING PRESSURE (psf/ft below natural grade)	LATERAL SLIDING RESISTANCE	
			Coefficient of friction[a]	Cohesion (psf)[b]
1. Crystalline bedrock	12,000	1,200	0.70	—
2. Sedimentary and foliated rock	4,000	400	0.35	—
3. Sandy gravel and gravel (GW and GP)	3,000	200	0.35	—
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC)	2,000	150	0.25	—
5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)	1,500	100	—	130

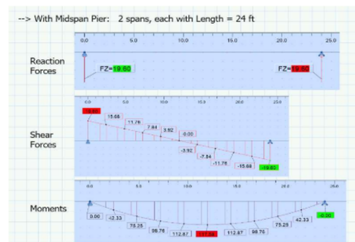
For SI: 1 pound per square foot = 0.0479kPa, 1 pound per square foot per foot = 0.157 kPa/m.
a. Coefficient to be multiplied by the dead load.
b. Cohesion value to be multiplied by the contact area, as limited by Section 1806.3.2.

Table 5-17. Typical compacted densities and optimum moisture contents for USCS soil types (after Carter and Bentley, 1991).

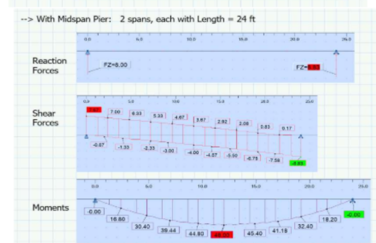
Soil Description	USCS Class	Compacted Dry Unit Weight (lb/ft3)	Compacted Dry Unit Weight (kN/m3)	Optimum Moisture Content (%)
Gravel/sand mixtures:				
well-graded, clean	GW	125-134	19.6-21.1	8-11
poorly-graded, clean	GP	115-125	18.1-19.6	11-14
well-graded, small silt content	GM	119-134	18.6-21.1	8-12
well-graded, small clay content	GC	115-125	18.1-19.6	9-14
Sands and sandy soils:				
well-graded, clean	SW	109-131	17.2-20.6	9-16
poorly-graded, small silt content	SP	94-119	15.7-18.6	12-21
well-graded, small silt content	SM	109-125	17.2-19.6	11-16
well-graded, small clay content	SC	106-125	16.7-19.6	11-19
Fined-grained soils of low plasticity:				
silts	ML	94-119	14.7-18.6	12-24
clays	CL	94-119	14.7-18.6	12-24
organic silts	OL	81-100	12.7-15.7	21-33
Fine-grained soils of high plasticity:				
silts	MH	69-94	10.8-14.7	24-40
clays	CH	81-100	12.7-18.6	19-36
organic clays	OH	66-100	10.3-15.7	21-45

1. Overturning Analysis

Combined Loading: --> With Midspan Pier: 2 spans, each with Length = 24 ft



H5 Truck Loading: --> With Midspan Pier: 2 spans, each with Length = 24 ft

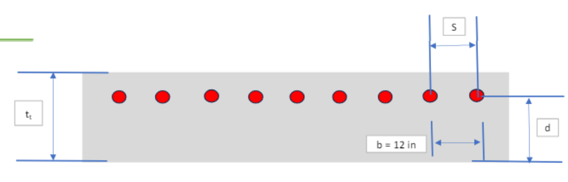


1. Overturning Analysis

2. Bearing Capacity Analysis

2. Bearing Capacity Analysis

3. Flexure Capacity Analysis



β_1 = 0.85		for $0 < f'_c \le 4000$ psi
β_1 = 0.85 − 0.05(f'_c − 4000)/1000		for 4000 psi < $f'_c \le 8000$ psi
β_1 = 0.65		for $f'_c > 8000$ psi

$$M_u = \gamma\left(\frac{P_a y^3}{6} + \frac{P_a y^2 w_s}{2\gamma_b}\right)$$

$$P_u = \gamma W_u$$

$$V = \gamma\left(\frac{P_a y^2}{2} + \frac{w_s P_a y}{\gamma_b}\right)$$

3. Flexure Capacity Analysis

4. Shear Capacity Analysis

CHECK SHEAR CAPACITY FOR STEM (ACI 318-02 SEC.15.5.2, 11.1.3.1, & 11.3)

		At top stem		At base of bottom stem			
$V_u = \left(\frac{w_s P_a}{\gamma_b} * H\right) + (0.5 * Pa * H^2)$	=	0.62	kips ,	0.62	kips		
$V_{allowable} = 2\phi b d \sqrt{f'_c}$	=	9.86 PASS	kips ,	9.86 PASS	kips	>	V

where ϕ = 0.75 (ACI 318-02, Section 9.3.2.3) ϕ = 0.75

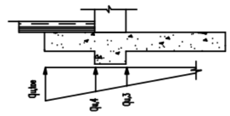
4. Shear Capacity Analysis

5. Heel Flexure Capacity Analysis

CHECK HEEL FLEXURE CAPACITY. $A_{s,3}$. FOR FOOTING (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f'_c}{f_y}\frac{87}{87+f_y}\right) = 0.0278 \qquad \rho_{MIN} = \frac{0.0018}{2}\frac{h_f}{d} = 0.0054$$

$$M_{u,3} = \begin{cases} \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{(q_{u,3} + 2q_{u,heel}) b L_H^2}{6}, & for \quad e_u \le \frac{L}{6} \\ \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{q_{u,3} b S^2}{6}, & for \quad e_u > \frac{L}{6} \end{cases} = 0.853 \text{ ft-kips}$$



$$\rho = \frac{0.85 f'_c\left(1 - \sqrt{1 - \frac{M_{u,3}}{0.383 b d^2 f'_c}}\right)}{f_y} = 0.0062$$

PASS PASS

where							
d_{heel}	=	2.00	in	$q_{u,toe}$	=	0.89	ksf
b	=	12.00	in	$q_{u,heel}$	=	0.50	ksf
e_u	=	0.47	ft	$q_{u,3}$	=	0.69	ksf
S	=	16.00	ft	ϕ	=	0.9	

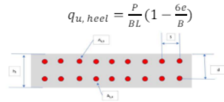
$$q_{u,toe} = \frac{P}{BL}\left(1 + \frac{6e}{B}\right)$$

$$q_{u,heel} = \frac{P}{BL}\left(1 - \frac{6e}{B}\right)$$

$$\phi M_n = \phi\left[A_S f_y\left(d - \frac{A_S f_y - P_u}{1.7 b f'_c}\right)\right] = 0.91 \text{ ft-kips}$$

$(A_{S,3})_{required}$ = 0.13 in^2/ft < $A_{S,3}$ = 0.31 in^2/ft

PASS



5. Heel Flexure Capacity Analysis

6. Toe Flexure Capacity Analysis

CHECK TOE FLEXURE CAPACITY. $A_{s,4}$. FOR FOOTING (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f'_c}{f_y}\frac{87}{87+f_y}\right) = 0.0278 \qquad \rho_{MIN} = MIN\left(\frac{4}{3}\rho, \ \frac{0.0018}{2}\frac{h_f}{d}\right) = 0.0001$$

$$M_{u,4} = \frac{(q_{u,4} + 2q_{u,toe}) b L_T^2}{6} - \frac{L_T^2}{2L}\gamma w_f = 0.34 \text{ ft-kips}$$

$$\rho = \frac{0.85 f'_c\left(1 - \sqrt{1 - \frac{M_{u,4}}{0.383 b d^2 f'_c}}\right)}{f_y} = 0.0001$$

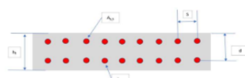
where			
d_{toe}	=	10.00	in
$q_{u,4}$	=	0.79	ksf
b	=	12.00	in
A_s	=	0.31	in^2
ϕ	=	0.9	
ρ	=	0.0022	

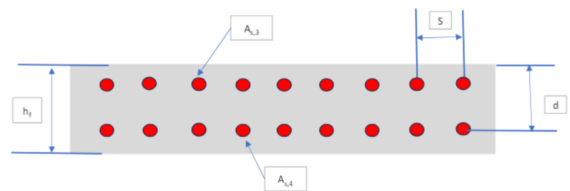
PASS PASS

$$\phi M_n = \phi\left[A_S f_y\left(d - \frac{A_S f_y - P_u}{1.7 b f'_c}\right)\right] = 0.62 \text{ ft-kips}$$

$(A_{S,4})_{required}$ = 0.02 in^2/ft < $A_{S,4}$ = 0.31 in^2/ft

PASS





6. Toe Flexure Capacity Analysis

7. Key Capacity Analysis

CHECK KEY CAPACITY FOR FOOTING

$1.5 (H_b + H_s)$ = 1.41 kips < $H_p + \mu \Sigma W$ = 6.36 kips

PASS

Techincal References:

1. Alan Williams: "Structuiral Engineering Reference Manual", Professional Publications, Inc, 2001.
2. Alan Williams: "Structuiral Engineering License Review Problems and Solutions", Oxford University Press, 2003.

Reference 10: Bridge 1 Pier Calculations with Water

Retaining Wall Design Based on ACI 318-02

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	3	ksi
REBAR YIELD STRESS	f_y	=	40	ksi
LATER SOIL PRESSURE	$P_a = K_a\gamma_b$	=	58.83	pcf (equivalent fluid pressure)
ACTIVE EARTH PRESSURE COEFF.	K_a	=	0.490	
PASSIVE EARTH PRESSURE COEFF.	K_p	=	2.040	
FRICTION ANGLE	ϕ	=	20	deg
BACKFILL SPECIFIC WEIGHT	γ_b	=	120	pcf
SATURATED SPECIFIC WEIGHT	γ_{sat}	=	0	pcf
WATER SPECIFIC WEIGHT	γ_w	=	62.4	pcf
CONCRETE UNIT WEIGHT	γ_c	=	145	pcf
WATER TABLE	n	=	2.08	ft
PASSIVE PRESSURE	P_p	=	244.75	pcf
SURCHARGE WEIGHT	w_s	=	240	psf
FRICTION COEFFICIENT	μ	=	0.3	
ALLOW SOIL PRESSURE	Q_a	=	1.5	ksf — IBC Table 1806.2
THICKNESS OF TOP STEM	t_t	=	12	in
THICKNESS OF KEY & STEM	t_b	=	12	in
TOE WIDTH	L_T	=	2	ft
HEEL WIDTH	L_H	=	4	ft
HEIGHT OF TOP STEM	H_T	=	1.5	ft
HEIGHT OF BOT. STEM	H_B	=	1.5	ft
FOOTING THICKNESS	h_f	=	12	in
KEY DEPTH	h_k	=	0	in
SOIL OVER TOE	h_p	=	29.00	in
	y	=	3	ft
	L	=	7.00	ft
	B	=	10.00	ft

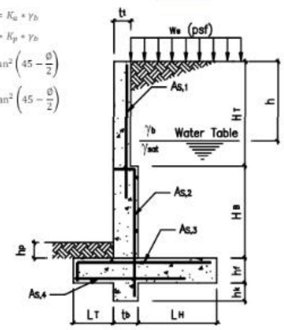
Bridge Surface =	743.00 ft
Top of abutment =	740.08 ft
Water surface elevation =	738.00 ft
Under bridge ground elevation =	739.50 ft

$$P_a = K_a * \gamma_b$$

$$P_p = K_p * \gamma_b$$

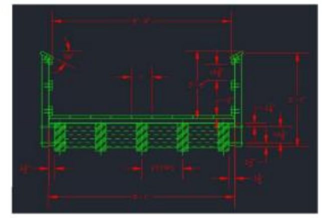
$$K_a = \tan^2\left(45 - \frac{\phi}{2}\right)$$

$$K_p = \tan^2\left(45 - \frac{\phi}{2}\right)$$



TOP STEM REINF. ($A_{s,1}$)	#	6	@	16	in o.c., (Caution > 18"o.c. max. ACI 14.3.5)	A_{bar} = 0.44 in²
$A_{s,1}$ LOCATION (0=at soil face, 1=at middle)		0	at soil face			
BOT. STEM REINF. ($A_{s,2}$)	#	6	@	16	in o.c.	A_{bar} = 0.44 in²
$A_{s,2}$ LOCATION (0=at soil face, 1=at middle)		0	at soil face			
TOP REINF OF FOOTING ($A_{s,3}$)	#	6	@	16	in o.c., (Caution > 18"o.c. max. ACI 7.12.2.2)	A_{bar} = 0.44 in²
BOT. REINF OF FOOTING ($A_{s,4}$)	#	6	@	16	in	A_{bar} = 0.44 in²

Pier @ 24 ft



Act. Earth Press. Coeff:	K_a =	$\tan^2(45-(\phi/2))$ =	K_o :	
Active Earth Pressure:	P_a =	$K_a * \gamma_{soil}$ =	<1	K_a = D/B
Pass. Earth Press. Coeff:	K_p =	$\tan^2(45+(\phi/2))$ =	>1	$K_a = \tan^{-1}(D/B)$
Passive Earth Pressure:	P_p =	$K_p * \gamma_{soil}$ =		

TABLE 1806.2 PRESUMPTIVE LOAD-BEARING VALUES

CLASS OF MATERIALS	VERTICAL FOUNDATION PRESSURE (psf)	LATERAL BEARING PRESSURE (psf/ft below natural grade)	LATERAL SLIDING RESISTANCE Coefficient of friction	Cohesion (psf)
1. Crystalline bedrock	12,000	1,200	0.70	—
2. Sedimentary and foliated rock	4,000	400	[illegible]	—
3. Sandy gravel and gravel (GW and GP)	3,000	200	[illegible]	—
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC)	2,000	150	0.25	—
5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)	1,500	100	—	130

For SI: [illegible]

Table 5.17. Typical compacted densities and optimum moisture contents for USCS soil types (after Carter and Bentley, 1991).

Soil Description	USCS Class	Compacted Dry Unit Weight (lb/ft³)	Compacted Dry Unit Weight (kN/m³)	Optimum Moisture Content (%)
Gravel/sand mixtures				
well-graded, clean	GW	[illegible]	[illegible]	[illegible]
poorly-graded, clean	GP	[illegible]	[illegible]	[illegible]
well-graded, small silt content	GM	[illegible]	[illegible]	[illegible]
well-graded, small clay content	GC	[illegible]	[illegible]	[illegible]
Sands and sandy soils				
well-graded, clean	SW	[illegible]	[illegible]	[illegible]
poorly-graded, small silt content	SP	[illegible]	[illegible]	[illegible]
well-graded, small silt content	SM	[illegible]	[illegible]	[illegible]
well-graded, small clay content	SC	[illegible]	[illegible]	[illegible]
Fined-grained soils of low plasticity				
silts	ML	[illegible]	[illegible]	[illegible]
clays	CL	[illegible]	[illegible]	[illegible]
organic silts	OL	[illegible]	[illegible]	[illegible]
Fine-grained soils of high plasticity				
silts	MH	[illegible]	[illegible]	[illegible]
clays	CH	[illegible]	[illegible]	[illegible]
organic clays	OH	[illegible]	[illegible]	[illegible]

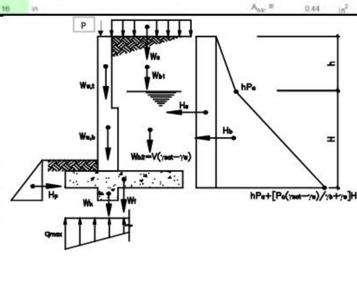
ANALYSIS

SERVICE LOADS

$H_b = 0.5 P_a h^2 + h P_a H + 0.5 [P_a (\gamma_{sat} - \gamma_w) / \gamma_b + \gamma_w] H^2$ = 0.33 kips

Where h = 2.0833 ft , H = 1.9167 ft

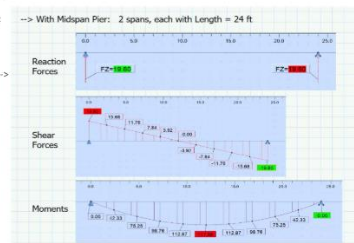
$H_s = w_s P_a (H_T + H_B + h_f) / \gamma_b$	=	0.47	kips
$H_p = 0.5 P_p (h_p + h_f + h_k)^2$	=	1.43	kips
$W_s = w_s (L_H + t_b - t_t)$	=	0.96	kips
$W_b = W_{b1} + W_{b2}$	=	0.52	kips
Where W_{b1} = 1.00 kips , W_{b2}	=	-0.48	kips
$W_f = h_f (L_H + t_b + L_T) \gamma_c$	=	1.02	kips
$W_k = h_k t_b \gamma_c$	=	0.00	kips
$W_{w,t} = t_t H_T \gamma_c$	=	0.22	kips
$W_{w,b} = t_b H_B \gamma_c$	=	0.22	kips
$P = ((R_{comb.} + R_{HS}) * N_{span}) / B$	=	28.43	kips



1. Overturning Analysis

Combined Loading: --> With Midspan Pier: 2 spans, each with Length = 24 ft

**Multiply by 2 -->



FACTORED LOADS

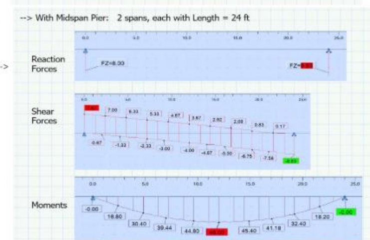
γH_b = 1.6 H_b	=	0.52	kips
γH_s = 1.6 H_s	=	0.75	kips
γW_s = 1.6 W_s	=	1.54	kips
γW_b = 1.2 W_b	=	0.63	kips
γW_f = 1.2 W_f	=	1.22	kips
γW_k = 1.2 W_k	=	0.00	kips
$\gamma W_{w,t}$ = 1.2 $W_{w,t}$	=	0.26	kips
$\gamma W_{w,b}$ = 1.2 $W_{w,b}$	=	0.26	kips
γP = 1.75 P	=	49.75	kips

OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_b	0.33	0.52	1.61	0.52	0.84
H_s	0.47	0.75	2.00	0.94	1.51
Σ	0.80	1.27		1.47	2.35
	(kip)	(kip)	(ft)	(kip-ft)	(kip-ft)

HS Truck Loading: --> With Midspan Pier: 2 spans, each with Length = 24 ft

**Multiply by 2 -->



RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.96	1.54	5.00	4.80	7.68
W_b	0.52	0.63	5.00	2.61	3.13
W_f	1.02	1.22	3.50	3.55	4.26
W_k	0.00	0.00	2.50	0.00	0.00
$W_{w,t}$	0.22	0.26	2.50	0.54	0.65
$W_{w,b}$	0.22	0.26	2.50	0.54	0.65
P	28.43	49.75	2.50	71.08	124.38
Σ	31.36	53.65		83.12	140.76
	(kip)	(kip)	(ft)	(kip-ft)	(kip-ft)

OVERTURNING FACTOR OF SAFETY

$$SF = \frac{\Sigma Wx}{\Sigma Hy} = 56.70 > 1.5$$

PASS

1. Overturning Analysis

CHECK SOIL BEARING CAPACITY (ACI 318-02 SEC.15.2.2)

$$L = L_T + t_b + L_H = 7.00 \text{ ft}$$

$$e = \frac{L}{2} - \frac{\Sigma Wx - \Sigma Hy}{\Sigma W} = 0.90 \text{ ft}$$

$$q_{MAX} = \begin{cases} \dfrac{\Sigma W\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{\Sigma W}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases} = 0.79 \text{ ksf} < Q_a = 1.5 \text{ ksf}$$

PASS

2. Bearing Capacity Analysis

2. Bearing Capacity Analysis

3. Flexure Capacity Analysis

CHECK FLEXURE CAPACITY, $A_{s,1}$ & $A_{s,2}$, FOR STEM (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

h = 2.0833 ft , H' = 0.9167 ft

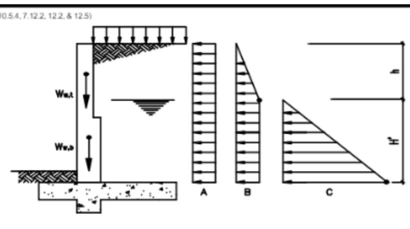
A = ws Pa / γb =	118	plf
B = h Pa =	123	plf
C = [Pa (γsat - γw) / γb + γw] H' =	-18	plf

At base of top stem

M_u =	0.58	ft-kips
V_u =	0.78	kips
P_u =	50.27	kips

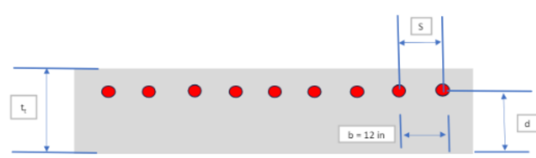
At base of bottom stem

M_u =	0.58	ft-kips
V_u =	0.78	kips
P_u =	50.27	kips



$$M_u = (A*H)(0.5*H) + (B*H')(0.5*H') + (0.5*B*h)\left(H' + \frac{1}{3}h\right) + (0.5*C*H')\left(\frac{1}{3}H'\right)$$

$$V_u = (A*H) + (B*H') + (0.5*B*h) + (0.5*C*H')$$

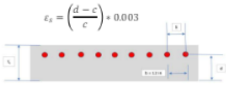


$$\phi M_n = \phi\left[A_S f_y\left(d - \frac{A_S f_y - P_u}{1.7 b f_c'}\right)\right]$$

		At top stem		At base of bottom stem			
ϕM_n	=	13.90	ft-kips ,	13.90	ft-kips	>	M_u
		PASS		PASS			

$$a = \frac{A_s f_y}{0.85 * b * f_c'} \qquad c = \frac{a}{\beta_1} \qquad \text{where}$$

$$\varepsilon_s = \left(\frac{d-c}{c}\right) * 0.003$$



		At top stem		At base of bottom stem	
a	=	0.58	in ,	0.58	in
c	=	0.68	in ,	0.68	in
d	=	10.00	in ,	10.00	in
ε_s	=	0.041		0.041	
		YIELDED		YIELDED	
b	=	12	in ,	12	in
φ	=	0.9	,	0.9	
A_s	=	0.44	in² ,	0.44	in²
ρ	=	0.003		0.003	
β_1	=	0.85		0.85	

$\beta_1 = 0.85$	for $0 < f'_c \le 4000$ psi
$\beta_1 = 0.85 - 0.05(f'_c - 4000)/1000$	for 4000 psi $< f'_c \le 8000$ psi
$\beta_1 = 0.65$	for $f'_c > 8000$ psi

$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f_c'}{f_y}\frac{87}{87+f_y}\right) = 0.028 \quad 0.028 > \rho$$

PASS PASS

$$\rho_{MIN} = 0.0018\frac{t}{d} = 0.002 \quad 0.002 < \rho$$

PASS PASS

3. Flexure Capacity Analysis

4. Shear Capacity Analysis

CHECK SHEAR CAPACITY FOR STEM (ACI 318-02 SEC.15.5.2, 11.1.3.1, & 11.3)

$$V_{allowable} = 2\phi bd\sqrt{f'_c}$$

	At top stem		At base of bottom stem			
=	9.86	kips ,	9.86	kips	>	V_u
	PASS		PASS			

where ϕ = 0.75 (ACI 318-02, Section 9.3.2.3) ϕ = 0.75

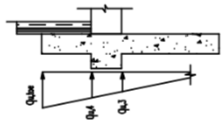
4. Shear Capacity Analysis

5. Heel Flexure Capacity Analysis

CHECK HEEL FLEXURE CAPACITY $A_{s,3}$ FOR FOOTING (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f'_c}{f_y}\frac{87}{87+f_y}\right) = 0.0278 \qquad \rho_{MIN} = \frac{0.0018}{2}\frac{h_f}{d} = 0.0036$$

$$M_{u,3} = \begin{cases} \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{(q_{u,3}+2q_{u,heel})bL_H^2}{6}, & for \quad e_u \le \frac{L}{6} \\ \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{q_{u,3}bS^2}{6}, & for \quad e_u > \frac{L}{6} \end{cases} = 1.63 \text{ ft-kips}$$



$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\frac{M_{u,3}}{0.383bd^2 f'_c}}\right)}{f_y} = 0.0052$$

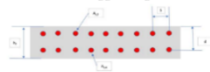
PASS PASS

where							
d	=	3.00	in	$q_{u,toe}$	=	1.18	ksf
b	=	12.00	in	$q_{u,heel}$	=	0.35	ksf
e_u	=	0.90	ft	$q_{u,3}$	=	0.83	ksf
S	=	16.00	in	ϕ	=	0.9	

$$q_{u,toe} = \frac{P}{BL}\left(1+\frac{6e}{B}\right)$$

$$q_{u,heel} = \frac{P}{BL}\left(1-\frac{6e}{B}\right)$$

$$\phi M_n = \phi\left[A_S f_y\left(d - \frac{A_S f_y - P_u}{1.7bf'_c}\right)\right] = 1.68 \text{ ft-kips}$$



$(A_{s,3})_{required}$ = 0.15 in²/ft < $A_{s,3}$ = 0.44 in²/ft

PASS

5. Heel Flexure Capacity Analysis

6. Toe Flexure Capacity Analysis

CHECK TOE FLEXURE CAPACITY $A_{s,4}$ FOR FOOTING (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

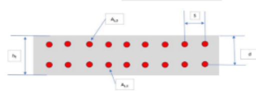
$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f'_c}{f_y}\frac{87}{87+f_y}\right) = 0.028 \qquad \rho_{MIN} = MIN\left(\frac{4}{3}\rho, \ \frac{0.0018}{2}\frac{h_f}{d}\right) = 0.0007$$

$$M_{u,4} = \frac{(q_{u,4}+2q_{u,toe})bL_T^2}{6} - \frac{L_T^2}{2L}\gamma w_f = 1.85 \text{ ft-kips}$$

where			
d	=	10.00	in
$q_{u,4}$	=	0.94	ksf
b	=	12.00	in
A_s	=	0.44	in²
ϕ	=	0.9	
ρ	=	0.0031	
		PASS	PASS

$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\frac{M_{u,4}}{0.383bd^2 f'_c}}\right)}{f_y} = 0.0005$$

$$\phi M_n = \phi\left[A_S f_y\left(d - \frac{A_S f_y - P_u}{1.7bf'_c}\right)\right] = 1.94 \text{ ft-kips}$$

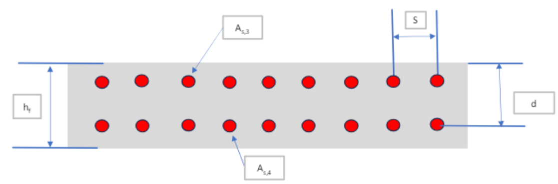


$(A_{s,4})_{required}$ = 0.06 in²/ft < $A_{s,4}$ = 0.44 in²/ft

PASS

toe - tension bottom

heel - tension top



6. Toe Flexure Capacity Analysis

7. Key Capacity Analysis

CHECK KEY CAPACITY FOR FOOTING

1.5 (H_b + H_s) = 1.19 kips < $H_p + \mu\Sigma W$ = 10.84 kips

PASS

Techincal References:

1. Alan Williams: "Structuiral Engineering Reference Manual", Professional Publications, Inc, 2001.
2. Alan Williams: "Structuiral Engineering License Review Problems and Solutions", Oxford University Press, 2003.

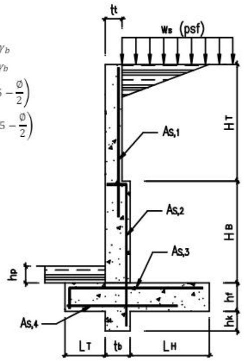
Reference 11: Bridge 1 Pier Calculations without Water

Retaining Wall Design Based on ACI 318-02

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	3	ksi						
REBAR YIELD STRESS	f_y	=	40	ksi						
LATER SOIL PRESSURE	P_a	=	58.83	pcf (equivalent fluid pressure)						
PASSIVE PRESSURE	P_p	=	244.75	pcf						
ACTIVE EARTH PRESSURE COEFF.	K_a	=	0.490							
PASSIVE EARTH PRESSURE COEFF.	K_p	=	2.040							
FRICTION ANGLE	ϕ	=	20	deg						
SURCHARGE WEIGHT	w_s	=	240	psf						
FRICTION COEFFICIENT	μ	=	0.3							
ALLOW SOIL PRESSURE	Q_a	=	1.5	ksf	IBC Table 1806.2					
THICKNESS OF TOP STEM	t_t	=	12	in						
THICKNESS OF KEY & STEM	t_b	=	12	in						
TOE WIDTH	L_T	=	2	ft						
HEEL WIDTH	L_H	=	4	ft						
HEIGHT OF TOP STEM	H_T	=	1.5	ft						
HEIGHT OF BOT. STEM	H_B	=	1.5	ft						
FOOTING THICKNESS	h_f	=	12	in						
KEY DEPTH	h_k	=	0	in						
SOIL OVER TOE	h_p	=	29.00	in						
	y	=	3	ft						
	L	=	7.00	ft						
	B	=	10	ft						
UNIT WEIGHT OF SOIL	γ_b	=	120	pcf						
UNIT WEIGHT OF CONCRETE	γ_c	=	145	pcf						
TOP STEM REINF. ($A_{s,1}$)	1	#	6	@	16	in o.c., at middle	A_{bar} =	0.44	in^2	
BOT. STEM REINF. ($A_{s,2}$)	2	#	6	@	16	in o.c., at each face	A_{bar} =	0.44	in^2	
TOP REINF.OF FOOTING ($A_{s,3}$)		#	6	@	16	in	A_{bar} =	0.44	in^2	
BOT. REINF.OF FOOTING ($A_{s,4}$)		#	6	@	16	in	A_{bar} =	0.44	in^2	

$$P_a = K_a * \gamma_b$$
$$P_p = K_p * \gamma_b$$
$$K_a = \tan^2\left(45 - \frac{\emptyset}{2}\right)$$
$$K_p = \tan^2\left(45 - \frac{\emptyset}{2}\right)$$



Act. Earth Press. Coeff: K_a = $\tan^2(45-(\phi/2))$ =

Active Earth Pressure: P_a = $K_a*\gamma_{soil}$ =

Pass. Earth Press. Coeff: K_p = $\tan^2(45+(\phi/2))$ =

Passive Earth Pressure: P_p = $K_p*\gamma_{soil}$ =

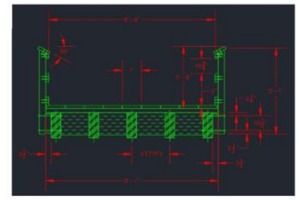
TABLE 1806.2 PRESUMPTIVE LOAD-BEARING VALUES

CLASS OF MATERIALS	VERTICAL FOUNDATION PRESSURE (psf)	LATERAL BEARING PRESSURE (psf/ft below natural grade)	LATERAL SLIDING RESISTANCE	
			Coefficient of friction[a]	Cohesion (psf)[b]
1. Crystalline bedrock	12,000	1,200	0.70	—
2. Sedimentary and foliated rock	4,000	400	0.35	—
3. Sandy gravel and gravel (GW and GP)	3,000	200	0.35	—
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC)	2,000	150	0.25	—
5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)	1,500	100	—	130

For SI: 1 pound per square foot = 0.0479kPa, 1 pound per square foot per foot = 0.157 kPa/m.
a. Coefficient to be multiplied by the dead load.
b. Cohesion value to be multiplied by the contact area, as limited by Section 1806.3.2.

Table 5.17. Typical compacted densities and optimum moisture contents for USCS soil types (after Carter and Bentley, 1991).

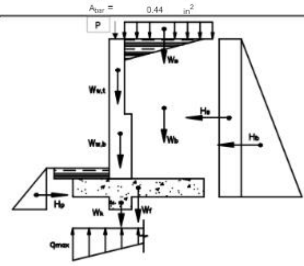
Soil Description	USCS Class	Compacted Dry Unit Weight		Optimum Moisture Content (%)
		(lb/ft3)	(kN/m3)	
Gravel/sand mixtures:				
well-graded, clean	GW	125-134	19.6-21.1	8-11
poorly-graded, clean	GP	115-125	18.1-19.6	11-14
well-graded, small silt content	GM	119-134	18.6-21.1	8-12
well-graded, small clay content	GC	115-125	18.1-19.6	9-14
Sands and sandy soils:				
well-graded, clean	SW	109-131	17.2-20.6	9-16
poorly-graded, small silt content	SP	94-119	15.7-18.6	12-21
well-graded, small silt content	SM	109-125	17.2-19.6	11-16
well-graded, small clay content	SC	106-125	16.7-19.6	11-19
Fined-grained soils of low plasticity:				
silts	ML	94-119	14.7-18.6	12-24
clays	CL	94-119	14.7-18.6	12-24
organic silts	OL	81-100	12.7-15.7	21-33
Fine-grained soils of high plasticity:				
silts	MH	69-94	10.8-14.7	24-40
clays	CH	81-106	12.7-16.6	19-36
organic clays	OH	66-100	10.3-15.7	21-45



ANALYSIS

SERVICE LOADS

$H_a = 0.5\,P_a\,(H_T + H_B + h_f)^2$	=	0.47	kips
$H_s = w_s\,P_a\,(H_T + H_B + h_f)\,/\,\gamma_b$	=	0.47	kips
$H_p = 0.5\,P_p\,(h_p + h_f + h_k)^2$	=	1.43	kips
$W_s = w_s\,(L_H + t_b - t_t)$	=	0.96	kips
$W_b = [H_T\,(L_H + t_b - t_t) + H_B\,L_H]\,\gamma_b$	=	1.44	kips
$W_f = h_f\,(L_H + t_b + L_T)\,\gamma_c$	=	1.02	kips
$W_k = h_k\,t_b\,\gamma_c$	=	0.00	kips
$W_{w,t} = t_t\,H_T\,\gamma_c$	=	0.22	kips
$W_{w,b} = t_b\,H_B\,\gamma_c$	=	0.22	kips
$P = R_{comb} + R_{H5}$	=	28.43	kips



FACTORED LOADS

$\gamma H_a = 1.6\,H_a$	=	0.75	kips
$\gamma H_s = 1.6\,H_s$	=	0.75	kips
$\gamma W_s = 1.6\,W_s$	=	1.54	kips
$\gamma W_b = 1.2\,W_b$	=	1.73	kips
$\gamma W_f = 1.2\,W_f$	=	1.22	kips
$\gamma W_k = 1.2\,W_k$	=	0.00	kips
$\gamma W_{w,t} = 1.2\,W_{w,t}$	=	0.26	kips
$\gamma W_{w,b} = 1.2\,W_{w,b}$	=	0.26	kips
$\gamma P = 1.75\,P$	=	49.75	kips

OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_a	0.47	0.75	1.33	0.63	1.00
H_s	0.47	0.75	2.00	0.94	1.51
Σ	0.94	1.51		**1.57**	2.51
	(kip)	(kip)	(ft)	(kip-ft)	(kip-ft)

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.96	1.54	5.00	4.80	7.68
W_b	1.44	1.73	5.00	7.20	8.64
W_f	1.02	1.22	3.50	3.55	4.26
W_k	0.00	0.00	2.50	0.00	0.00
$W_{w,t}$	0.22	0.26	2.50	0.54	0.65
$W_{w,b}$	0.22	0.26	2.50	0.54	0.65
P	28.43	49.75	2.50	71.08	124.38
Σ	32.28	54.76		**87.72**	146.27
	(kip)	(kip)	(ft)	(kip-ft)	(kip-ft)

OVERTURNING FACTOR OF SAFETY

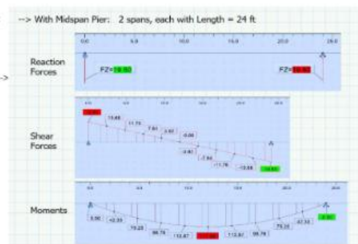
$$SF = \frac{\Sigma Wx}{\Sigma Hy} = 55.91 > 1.5$$

PASS

1. Overturning Analysis

Combined Loading: --> With Midspan Pier: 2 spans, each with Length = 24 ft

**Multiply by 2 -->



H5 Truck Loading: --> With Midspan Pier: 2 spans, each with Length = 24 ft

**Multiply by 2 -->

1. Overturning Analysis

CHECK SOIL BEARING CAPACITY (ACI 318-02 SEC.15.2.2)

$$L = L_T + t_b + L_H = 7.00 \text{ ft}$$

$$e = \frac{L}{2} - \frac{\Sigma Wx - \Sigma Hy}{\Sigma W} = 0.83 \text{ ft}$$

$$q_{MAX} = \begin{cases} \frac{\Sigma W\left(1+\frac{6e}{L}\right)}{BL}, & for \quad e \le \frac{L}{6} \\ \frac{\Sigma W}{3B(0.5L-e)}, & for \quad e > \frac{L}{6} \end{cases} = 0.79 \text{ ksf} < Q_a = 1.5 \text{ ksf}$$

PASS

2. Bearing Capacity Analysis

2. Bearing Capacity Analysis

CHECK FLEXURE CAPACITY, $A_{s,1}$ & $A_{s,2}$, FOR STEM (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

3. Flexure Capacity Analysis

$$M_u = \left(\frac{w_s P_a}{\gamma_b} * H\right)(0.5*H) + (0.5*Pa*H^2)\left(\frac{1}{3}*H\right) = 0.79 \text{ ft-kips}$$

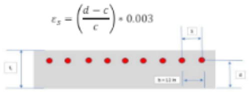
P_u = 50.27 kips

$$\phi M_n = \phi\left[A_s f_y\left(d - \frac{A_s f_y - P_u}{1.7 b f_c'}\right)\right]$$

		At top stem		At base of bottom stem			
ϕM_n	=	13.90	ft-kips ,	13.90	ft-kips	>	M_u
		PASS		PASS			

$$a = \frac{A_s f_y}{0.85 * b * f_c'} \qquad c = \frac{a}{\beta_1} \qquad \text{where}$$

$$\varepsilon_s = \left(\frac{d-c}{c}\right) * 0.003$$



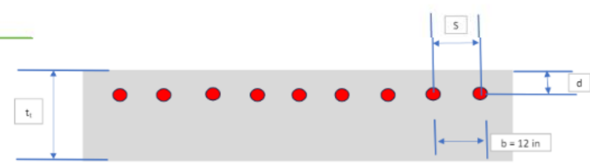
	=	At top stem		At base of bottom stem	
a	=	0.58	in ,	0.58	in
c	=	0.68	in ,	0.68	in
d	=	10.00	in ,	10.00	in
ε_s	=	0.041		0.041	
		YIELDED		YIELDED	
b	=	12	in ,	12	in
ϕ	=	0.9		0.9	
A_s	=	0.44	in^2 ,	0.44	in^2
ρ	=	0.003		0.003	
β_1	=	0.85		0.85	

$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f_c'}{f_y}\frac{87}{87+f_y}\right) = 0.028 \;\; 0.028 \; > \rho$$

PASS PASS

$$\rho_{MIN} = 0.0018\frac{t}{d} = 0.002 \;\; 0.002 \; < \rho$$

PASS PASS



$\beta_1 = 0.85$	for $0 < f'_c \le 4000$ psi
$\beta_1 = 0.85 - 0.05(f'_c - 4000)/1000$	for 4000 psi $< f'_c \le 8000$ psi
$\beta_1 = 0.65$	for $f'_c > 8000$ psi

$$M_u = \gamma\left(\frac{P_a y^3}{6} + \frac{P_a y^2 w_s}{2\gamma_b}\right)$$

$$P_u = \gamma W_w$$

$$V = \gamma\left(\frac{P_a y^2}{2} + \frac{w_s P_a y}{\gamma_b}\right)$$

3. Flexure Capacity Analysis

4. Shear Capacity Analysis

CHECK SHEAR CAPACITY FOR STEM (ACI 318-02 SEC.15.5.2, 11.1.3.1, & 11.3)

		At top stem		At base of bottom stem			
$V_u = \left(\frac{w_s P_a}{\gamma_b} * H\right) + (0.5 * Pa * H^2)$	=	0.62	kips ,	0.62	kips		
$V_{allowable} = 2\phi b d \sqrt{f'_c}$	=	9.86 PASS	kips ,	9.86 PASS	kips	>	V
where ϕ = 0.75 (ACI 318-02, Section 9.3.2.3)	ϕ =	0.75					

4. Shear Capacity Analysis

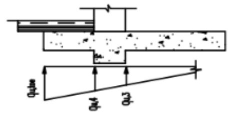
5. Heel Flexure Capacity Analysis

CHECK HEEL FLEXURE CAPACITY, $A_{s,3}$, FOR FOOTING (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f'_c}{f_y}\frac{87}{87+f_y}\right) = 0.0278$$

$$\rho_{MIN} = \frac{0.0018}{2}\frac{h_f}{d} = 0.0036$$

$$M_{u,3} = \begin{cases} \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{(q_{u,3} + 2q_{u,heel})bL_H^2}{6}, & for \quad e_u \le \frac{L}{6} \\ \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{q_{u,3} b S^2}{6}, & for \quad e_u > \frac{L}{6} \end{cases} = 3.594 \text{ ft-kips}$$



$$\rho = \frac{0.85 f'_c\left(1 - \sqrt{1 - \frac{M_{u,3}}{0.383 b d^2 f'_c}}\right)}{f_y} = 0.0123$$

PASS PASS

where							
d_{heel}	=	3.00	in	$q_{u,toe}$	=	1.17	ksf
b	=	12.00	in	$q_{u,heel}$	=	0.39	ksf
e_u	=	0.83	ft	$q_{u,3}$	=	0.84	ksf
S	=	16.00	ft	ϕ	=	0.9	

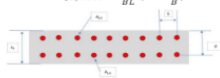
$$q_{u,toe} = \frac{P}{BL}\left(1 + \frac{6e}{B}\right)$$

$$q_{u,heel} = \frac{P}{BL}\left(1 - \frac{6e}{B}\right)$$

$$\phi M_n = \phi\left[A_S f_y\left(d - \frac{A_S f_y - P_u}{1.7 b f'_c}\right)\right] = 3.67 \text{ ft-kips}$$

$(A_{S,3})_{required}$ = 0.34 in^2 / ft < $A_{S,3}$ = 0.44 in^2 / ft

PASS



5. Heel Flexure Capacity Analysis

6. Toe Flexure Capacity Analysis

CHECK TOE FLEXURE CAPACITY, $A_{s,4}$, FOR FOOTING (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f'_c}{f_y}\frac{87}{87+f_y}\right) = 0.0278$$

$$\rho_{MIN} = MIN\left(\frac{4}{3}\rho, \ \frac{0.0018}{2}\frac{h_f}{d}\right) = 0.0007$$

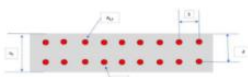
$$M_{u,4} = \frac{(q_{u,4} + 2q_{u,toe})bL_T^2}{6} - \frac{L_T^2}{2L}\gamma w_f = 1.85 \text{ ft-kips}$$

where			
d_{toe}	=	10.00	in
$q_{u,4}$	=	0.95	ksf
b	=	12.00	in
A_s	=	0.44	in^2
ϕ	=	0.9	
ρ	=	0.0031	

PASS PASS

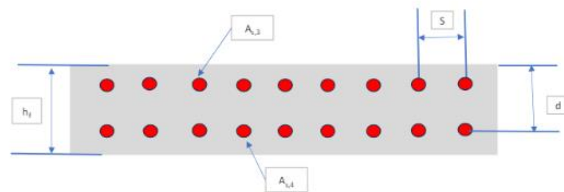
$$\rho = \frac{0.85 f'_c\left(1 - \sqrt{1 - \frac{M_{u,4}}{0.383 b d^2 f'_c}}\right)}{f_y} = 0.0005$$

$$\phi M_n = \phi\left[A_S f_y\left(d - \frac{A_S f_y - P_u}{1.7 b f'_c}\right)\right] = 1.94 \text{ ft-kips}$$



$(A_{S,4})_{required}$ = 0.06 in^2 / ft < $A_{S,4}$ = 0.44 in^2 / ft

PASS



6. Toe Flexure Capacity Analysis

7. Key Capacity Analysis

CHECK KEY CAPACITY FOR FOOTING

1.5 (H_b + H_s) = 1.41 kips < H_p + μ ΣW = 11.11 kips

PASS

Techincal References:

1. Alan Williams: "Structuiral Engineering Reference Manual", Professional Publications, Inc, 2001.
2. Alan Williams: "Structuiral Engineering License Review Problems and Solutions", Oxford University Press, 2003.

Reference 12: Bridge 2 Abutment Calculations with Water

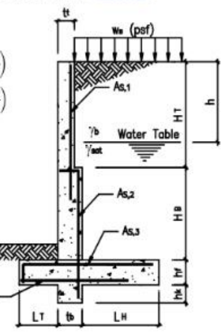
Retaining Wall Design Based on ACI 318-02

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	3	ksi
REBAR YIELD STRESS	f_y	=	40	ksi
LATER SOIL PRESSURE	$P_a = K_a \gamma_b$	=	58.83	pcf (equivalent fluid pressure)
ACTIVE EARTH PRESSURE COEFF.	K_a	=	0.490	
PASSIVE EARTH PRESSURE COEFF.	K_p	=	2.040	
FRICTION ANGLE	ϕ	=	20	deg
BACKFILL SPECIFIC WEIGHT	γ_b	=	120	pcf
SATURATED SPECIFIC WEIGHT	γ_{sat}	=	133.4	pcf
WATER SPECIFIC WEIGHT	γ_w	=	62.4	pcf
CONCRETE UNIT WEIGHT	γ_c	=	145	pcf
WATER TABLE	h	=	5.08	ft
PASSIVE PRESSURE	P_p	=	244.75	pcf
SURCHARGE WEIGHT	w_s	=	240	psf
FRICTION COEFFICIENT	μ	=	0.3	
ALLOW SOIL PRESSURE	Q_a	=	1.5	ksf (IBC Table 1806.2)
THICKNESS OF TOP STEM	t_t	=	12	in
THICKNESS OF KEY & STEM	t_b	=	12	in
TOE WIDTH	L_T	=	1	ft
HEEL WIDTH	L_H	=	2.5	ft
HEIGHT OF TOP STEM	H_T	=	1	ft
HEIGHT OF BOT. STEM	H_B	=	1	ft
FOOTING THICKNESS	h_f	=	12	in
KEY DEPTH	h_k	=	0	in
SOIL OVER TOE	h_p	=	23.00	in
	y	=	2	ft
	L	=	4.50	ft
	B	=	10.00	ft

Bridge Surface	=	746.00	ft
Top of abutment	=	743.08	ft
Water surface elevation	=	738.00	ft
Under bridge ground elevation	=	743.00	ft

$$P_a = K_a * \gamma_b \qquad P_p = K_p * \gamma_b$$
$$K_a = \tan^2\left(45 - \frac{\phi}{2}\right) \qquad K_p = \tan^2\left(45 + \frac{\phi}{2}\right)$$



TOP STEM REINF. ($A_{s,1}$)	#	6	@	16	in o.c., (Caution > 18" o.c. max, ACI 14.3.5)	A_{bar} =	0.44	in²
$A_{s,1}$ LOCATION (0=at soil face, 1=at middle)		0	at soil face					
BOT. STEM REINF. ($A_{s,2}$)	#	6	@	16	in o.c.	A_{bar} =	0.44	in²
$A_{s,2}$ LOCATION (0=at soil face, 1=at middle)		0	at soil face					
TOP REINF OF FOOTING ($A_{s,3}$)	#	6	@	16	in o.c., (Caution > 18" o.c. max, ACI 7.12.2.2)	A_{bar} =	0.44	in²
BOT. REINF. OF FOOTING ($A_{s,4}$)	#	6	@	16	in	A_{bar} =	0.44	in²

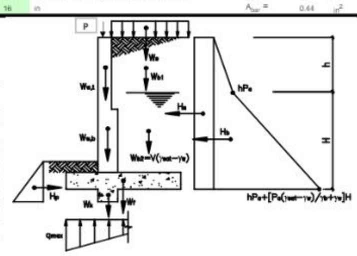
ANALYSIS

SERVICE LOADS

$H_b = 0.5\,P_a h^2 + h P_a H + 0.5\,[P_a(\gamma_{sat}-\gamma_w)/\gamma_b + \gamma_w] H^2$ = 0.19 kips

Where h = 5.0833 ft, H = -2.0833 ft

$H_s = w_s P_a (H_T + H_B + h_f)/\gamma_b$	=	0.35	kips
$H_p = 0.5\,P_p (h_p + h_f + h_k)^2$	=	1.04	kips
$W_s = w_s (L_H + t_b - t_t)$	=	0.60	kips
$W_b = W_{b1} + W_{b2}$	=	1.16	kips
Where W_{b1} = 1.52 kips, W_{b2} =		-0.37	kips
$W_f = h_f (L_H + t_b + L_T)\gamma_c$	=	0.65	kips
$W_k = h_k t_b \gamma_c$	=	0.00	kips
$W_{w,t} = t_t H_T \gamma_c$	=	0.15	kips
$W_{w,b} = t_b H_B \gamma_c$	=	0.15	kips
$P = ((R_{dead} + R_{live}) * N_{beam})/B$	=	12.47	kips



FACTORED LOADS

γH_b = 1.6 H_b	=	0.30	kips
γH_s = 1.6 H_s	=	0.56	kips
γW_s = 1.6 W_s	=	0.96	kips
γW_b = 1.2 W_b	=	1.39	kips
γW_f = 1.2 W_f	=	0.78	kips
γW_k = 1.2 W_k	=	0.00	kips
$\gamma W_{w,t}$ = 1.2 $W_{w,t}$	=	0.17	kips
$\gamma W_{w,b}$ = 1.2 $W_{w,b}$	=	0.17	kips
γP = 1.75 P	=	21.81	kips

OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_b	0.19	0.30	-0.83	-0.16	-0.25
H_s	0.35	0.56	1.50	0.53	0.85
Σ	0.54	0.86		0.37	0.60
	(kip)	(kip)	(ft)	(kip-ft)	(kip-ft)

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.60	0.96	3.25	1.95	3.12
W_b	1.16	1.39	3.25	3.75	4.51
W_f	0.65	0.78	2.25	1.47	1.76
W_k	0.00	0.00	1.50	0.00	0.00
$W_{w,t}$	0.15	0.17	1.50	0.22	0.26
$W_{w,b}$	0.15	0.17	1.50	0.22	0.26
P	12.47	21.81	1.50	18.70	32.72
Σ	15.16	25.29		26.31	42.63
	(kip)	(kip)	(ft)	(kip-ft)	(kip-ft)

OVERTURNING FACTOR OF SAFETY

$$SF = \frac{\Sigma Wx}{\Sigma Hy} = 70.29 > 1.5$$

PASS

CHECK SOIL BEARING CAPACITY (ACI 318-02 SEC.15.2.2)

$$L = L_T + t_b + L_H = 4.50 \text{ ft}$$

$$e = \frac{L}{2} - \frac{\Sigma Wx - \Sigma Hy}{\Sigma W} = 0.54 \text{ ft}$$

$$q_{MAX} = \begin{cases} \dfrac{\Sigma W\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{\Sigma W}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases} = 0.58 \text{ ksf} < Q_a = 1.5 \text{ ksf}$$

PASS

CHECK FLEXURE CAPACITY, $A_{s,1}$ & $A_{s,2}$, FOR STEM (ACI318-02 SEC.10.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

h = 5.0833 ft, H' = -3.0833 ft

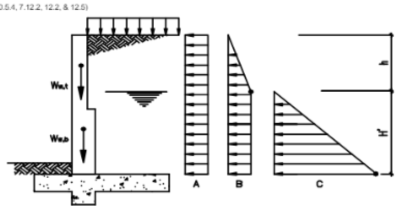
$A = w_s P_a/\gamma_b$	=	118	plf
$B = h P_a$	=	299	plf
$C = [P_a(\gamma_{sat}-\gamma_w)/\gamma_b + \gamma_w] H'$	=	-71	plf

At base of top stem

M_u =	0.18	ft-kips
V_u =	0.49	kips
P_u =	22.16	kips

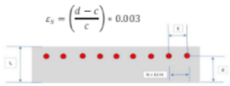
At base of bottom stem

M_u =	0.18	ft-kips
V_u =	0.49	kips
P_u =	22.16	kips



$$\phi M_n = \phi\left[A_s f_y\left(d - \frac{A_s f_y - P_u}{1.7 b f_c'}\right)\right]$$

$$a = \frac{A_s f_y}{0.85 * b * f_c'} \qquad c = \frac{a}{\beta_1} \qquad \varepsilon_s = \left(\frac{d-c}{c}\right) * 0.003$$



		At top stem		At base of bottom stem			
ϕM_n	=	13.30	ft-kips	13.30	ft-kips	>	M_u
		PASS		PASS			
a	=	0.58	in	0.58	in		
c	=	0.68	in	0.68	in		
d	=	10.00	in	10.00	in		
ε_s	=	0.041		0.041			
		YIELDED		YIELDED			
b	=	12	in	12	in		
φ	=	0.9		0.9			
A_s	=	0.44	in²	0.44	in²		
ρ	=	0.003		0.003			
β_1	=	0.85		0.85			
ρ_{MAX}	=	0.028		0.028		>	ρ
		PASS		PASS			
ρ_{MIN}	=	0.002		0.002		<	ρ
		PASS		PASS			

$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f_c'}{f_y}\frac{87}{87+f_y}\right) \qquad \rho_{MIN} = 0.0018\frac{t}{d}$$

**set level bridge surface, bridge ground elevation may vary, client will likely need to add fill to trail to connect

Bridge Length = 40 ft

2 span bridge

Act. Earth Press. Coeff:	K_a =	$\tan^2(45-(\phi/2))$ =		K_o :	
Active Earth Pressure:	P_a =	$K_a * \gamma_{soil}$ =		<1	K_a = D/B
Pass. Earth Press. Coeff:	K_p =	$\tan^2(45+(\phi/2))$ =		>1	$K_o = \tan^{-1}(D/B)$
Passive Earth Pressure:	P_p =	$K_p * \gamma_{soil}$ =			

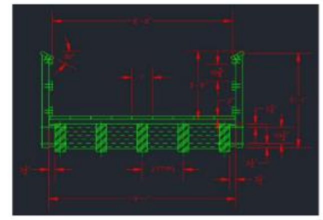


TABLE 1806.2 PRESUMPTIVE LOAD-BEARING VALUES

CLASS OF MATERIALS	VERTICAL FOUNDATION PRESSURE (psf)	LATERAL BEARING PRESSURE (psf/ft below natural grade)	LATERAL SLIDING RESISTANCE: Coefficient of friction[a]	LATERAL SLIDING RESISTANCE: Cohesion (psf)[b]
1. Crystalline bedrock	12,000	1,200	0.70	—
2. Sedimentary and foliated rock	4,000	400	0.35	—
3. Sandy gravel and gravel (GW and GP)	3,000	200	0.35	—
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC)	2,000	150	0.25	—
5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)	1,500	100	—	130

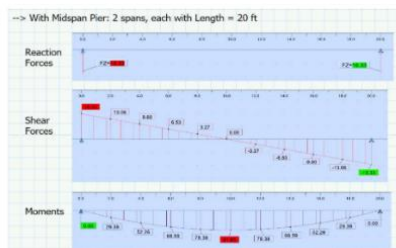
For SI: 1 pound per square foot = 0.0479kPa, 1 pound per square foot per foot = [illegible] kPa/m.
a. Coefficient to be multiplied by the dead load.
b. Cohesion value to be multiplied by the contact area, as limited by Section [illegible].

Table 5.17. Typical compacted densities and optimum moisture contents for USCS soil types (after Carter and Bentley, 1991).

Soil Description	USCS Class	Compacted Dry Unit Weight (lb/ft3)	Compacted Dry Unit Weight (kN/m3)	Optimum Moisture Content (%)
Gravel/sand mixtures:				
well-graded, clean	GW	125-134	19.6-21.1	8-11
poorly-graded, clean	GP	115-125	18.1-19.6	11-14
well-graded, small silt content	GM	119-134	18.6-21.1	8-12
well-graded, small clay content	GC	115-125	18.1-19.6	9-14
Sands and sandy soils:				
well-graded, clean	SW	109-131	17.2-20.6	9-16
poorly-graded, small silt content	SP	94-119	15.7-18.6	12-21
well-graded, small silt content	SM	109-125	17.2-19.6	11-16
well-graded, small clay content	SC	106-125	16.7-19.6	11-19
Fined-grained soils of low plasticity:				
silts	ML	94-119	14.7-18.6	12-24
clays	CL	94-119	14.7-18.6	12-24
organic silts	OL	81-100	12.7-15.7	21-33
Fine-grained soils of high plasticity:				
silts	MH	69-94	10.8-14.7	24-40
clays	CH	81-106	12.7-16.6	19-36
organic clays	OH	66-100	10.3-15.7	21-45

1. Overturning Analysis

Combined Loading: --> With Midspan Pier: 2 spans, each with Length = 20 ft



HS Truck Loading: --> With Midspan Pier: 2 spans, each at Length = 20 ft

1. Overturning Analysis

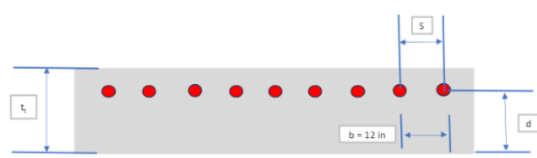
2. Bearing Capacity Analysis

2. Bearing Capacity Analysis

3. Flexure Capacity Analysis

$$M_u = (A*H)(0.5*H) + (B*H')(0.5*H') + (0.5*B*h)\left(H' + \frac{1}{3}h\right) + (0.5*C*H')\left(\frac{1}{3}H'\right)$$

$$V_u = (A*H) + (B*H') + (0.5*B*h) + (0.5*C*H')$$



$\beta_1 = 0.85$	for $0 < f_c' \le 4000$ psi
$\beta_1 = 0.85 - 0.05(f_c' - 4000)/1000$	for 4000 psi $< f_c' \le 8000$ psi
$\beta_1 = 0.65$	for $f_c' > 8000$ psi

3. Flexure Capacity Analysis

CHECK SHEAR CAPACITY FOR STEM (ACI 318-02 SEC.15.5.2, 11.1.3.1, & 11.3)

		At top stem		At base of bottom stem			
$V_{allowable} = 2\phi bd\sqrt{f'_c}$	=	9.86	kips ,	9.86	kips	>	V_u
		PASS		PASS			

where ϕ = 0.75 (ACI 318-02, Section 9.3.2.3) ϕ = 0.75

4. Shear Capacity Analysis

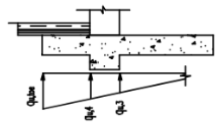
4. Shear Capacity Analysis

5. Heel Flexure Capacity Analysis

CHECK HEEL FLEXURE CAPACITY A.. FOR FOOTING (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f'_c}{f_y}\frac{87}{87+f_y}\right) = 0.0278 \qquad \rho_{MIN} = \frac{0.0018}{2}\frac{h_f}{d} = 0.0054$$

$$M_{u,3} = \begin{cases} \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{(q_{u,3}+2q_{u,heel})bL_H^2}{6}, & for \quad e_u \le \frac{L}{6} \\ \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{q_{u,3}bS^2}{6}, & for \quad e_u > \frac{L}{6} \end{cases} = 2.08 \text{ ft-kips}$$



$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\frac{M_{u,3}}{0.383bd^2 f'_c}}\right)}{f_y} = 0.0166$$

PASS PASS

where							
d	=	2.00	in	$q_{u,toe}$	=	0.74	ksf
b	=	12.00	in	$q_{u,heel}$	=	0.38	ksf
e_u	=	0.54	ft	$q_{u,3}$	=	0.58	ksf
S	=	16.00	in	ϕ	=	0.9	

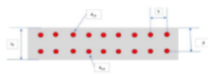
$$q_{u,toe} = \frac{P}{BL}(1+\frac{6e}{B})$$

$$q_{u,heel} = \frac{P}{BL}(1-\frac{6e}{B})$$

$$\phi M_n = \phi\left[A_S f_y\left(d - \frac{A_S f_y - P_u}{1.7bf'_c}\right)\right] = 2.12 \text{ ft-kips}$$

$(A_{S,3})_{required}$ = 0.33 in²/ft < $A_{S,3}$ = 0.44 in²/ft

PASS



5. Heel Flexure Capacity Analysis

6. Toe Flexure Capacity Analysis

CHECK TOE FLEXURE CAPACITY A.. FOR FOOTING (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f'_c}{f_y}\frac{87}{87+f_y}\right) = 0.028 \qquad \rho_{MIN} = MIN\left(\frac{4}{3}\rho, \ \frac{0.0018}{2}\frac{h_f}{d}\right) = 0.0001$$

$$M_{u,4} = \frac{(q_{u,4}+2q_{u,toe})bL_T^2}{6} - \frac{L_T^2}{2L}\gamma w_f = 0.27 \text{ ft-kips}$$

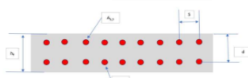
$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\frac{M_{u,4}}{0.383bd^2 f'_c}}\right)}{f_y} = 0.0001$$

where			
d	=	10.00	in
$q_{u,4}$	=	0.66	ksf
b	=	12.00	in
A_s	=	0.44	in²
ϕ	=	0.9	
ρ	=	0.0031	
		PASS	PASS

$$\phi M_n = \phi\left[A_S f_y\left(d - \frac{A_S f_y - P_u}{1.7bf'_c}\right)\right] = 0.31 \text{ ft-kips}$$

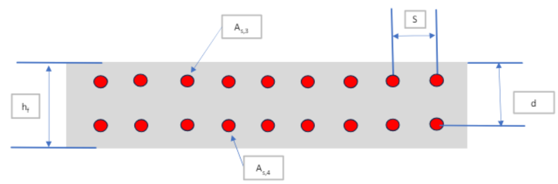
$(A_{S,4})_{required}$ = 0.01 in²/ft < $A_{S,4}$ = 0.44 in²/ft

PASS



toe - tension bottom

heel - tension top



6. Toe Flexure Capacity Analysis

7. Key Capacity Analysis

CHECK KEY CAPACITY FOR FOOTING

1.5 (H_b + H_s) = 0.81 kips < $H_p + \mu\Sigma W$ = 5.59 kips

PASS

Techincal References:

1. Alan Williams: "Structuiral Engineering Reference Manual", Professional Publications, Inc, 2001.
2. Alan Williams: "Structuiral Engineering License Review Problems and Solutions", Oxford University Press, 2003.

Reference 13: Bridge 2 Abutment Calculations without Water

Retaining Wall Design Based on ACI 318-02

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	3	ksi
REBAR YIELD STRESS	f_y	=	40	ksi
LATER SOIL PRESSURE	P_a	=	58.83	pcf (equivalent fluid pressure)
PASSIVE PRESSURE	P_p	=	244.75	pcf
ACTIVE EARTH PRESSURE COEFF.	K_a	=	0.490	
PASSIVE EARTH PRESSURE COEFF.	K_p	=	2.040	
FRICTION ANGLE	ϕ	=	20	deg
SURCHARGE WEIGHT	w_s	=	240	psf
FRICTION COEFFICIENT	μ	=	0.3	
ALLOW SOIL PRESSURE	Q_a	=	1.5	ksf IBC Table 1806.2
THICKNESS OF TOP STEM	t_t	=	12	in
THICKNESS OF KEY & STEM	t_b	=	12	in
TOE WIDTH	L_T	=	1	ft
HEEL WIDTH	L_H	=	2.5	ft
HEIGHT OF TOP STEM	H_T	=	1	ft
HEIGHT OF BOT. STEM	H_B	=	1	ft
FOOTING THICKNESS	h_f	=	12	in
KEY DEPTH	h_k	=	0	in
SOIL OVER TOE	h_p	=	23.00	in
	y	=	2	ft
	L	=	4.50	ft
	B	=	10	ft
UNIT WEIGHT OF SOIL	γ_b	=	120	pcf
UNIT WEIGHT OF CONCRETE	γ_c	=	145	pcf

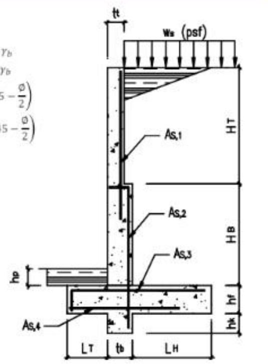
TOP STEM REINF. ($A_{s,1}$)	1	#	6	@	16	in o.c., at middle	A_{bar} =	0.44	in^2
BOT. STEM REINF. ($A_{s,2}$)	2	#	6	@	16	in o.c., at each face	A_{bar} =	0.44	in^2
TOP REINF.OF FOOTING ($A_{s,3}$)		#	6	@	16	in	A_{bar} =	0.44	in^2
BOT. REINF.OF FOOTING ($A_{s,4}$)		#	6	@	16	in	A_{bar} =	0.44	in^2

$P_a = K_a * \gamma_b$

$P_p = K_p * \gamma_b$

$K_a = \tan^2\left(45 - \frac{\phi}{2}\right)$

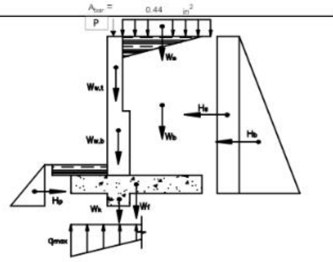
$K_p = \tan^2\left(45 - \frac{\phi}{2}\right)$



ANALYSIS

SERVICE LOADS

$H_b = 0.5\,P_a\,(H_T + H_B + h_f)^2$	=	0.26	kips
$H_s = w_s\,P_a\,(H_T + H_B + h_f)\,/\,\gamma_b$	=	0.35	kips
$H_p = 0.5\,P_p\,(h_p + h_f + h_k)^2$	=	1.04	kips
$W_s = w_s\,(L_H + t_b - t_t)$	=	0.60	kips
$W_b = [H_T\,(L_H + t_b - t_t) + H_B\,L_H]\,\gamma_b$	=	0.60	kips
$W_f = h_f\,(L_H + t_b + L_T)\,\gamma_c$	=	0.65	kips
$W_k = h_k\,t_b\,\gamma_c$	=	0.00	kips
$W_{w,t} = t_t\,H_T\,\gamma_c$	=	0.15	kips
$W_{w,b} = t_b\,H_B\,\gamma_c$	=	0.15	kips
$P = R_{comb} + R_{HS}$	=	12.47	kips



FACTORED LOADS

γH_b = 1.6 H_b	=	0.42	kips
γH_s = 1.6 H_s	=	0.56	kips
γW_s = 1.6 W_s	=	0.96	kips
γW_b = 1.2 W_b	=	0.72	kips
γW_f = 1.2 W_f	=	0.78	kips
γW_k = 1.2 W_k	=	0.00	kips
$\gamma W_{w,t}$ = 1.2 $W_{w,t}$	=	0.17	kips
$\gamma W_{w,b}$ = 1.2 $W_{w,b}$	=	0.17	kips
γP = 1.75 P	=	21.81	kips

OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_b	0.26	0.42	1.00	0.26	0.42
H_s	0.35	0.56	1.50	0.53	0.85
Σ	0.62	0.99		**0.79**	1.27
	(kip)	(kip)	(ft)	(kip-ft)	(kip-ft)

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.60	0.96	3.25	1.95	3.12
W_b	0.60	0.72	3.25	1.95	2.34
W_f	0.65	0.78	2.25	1.47	1.76
W_k	0.00	0.00	1.50	0.00	0.00
$W_{w,t}$	0.15	0.17	1.50	0.22	0.26
$W_{w,b}$	0.15	0.17	1.50	0.22	0.26
P	12.47	21.81	1.50	18.70	32.72
Σ	14.61	24.62		**24.50**	40.46
	(kip)	(kip)	(ft)	(kip-ft)	(kip-ft)

OVERTURNING FACTOR OF SAFETY

$SF = \frac{\Sigma Wx}{\Sigma Hy}$ = 30.85 > 1.5

PASS

CHECK SOIL BEARING CAPACITY (ACI 318-02 SEC.15.2.2)

$L = L_T + t_b + L_H$ = 4.50 ft

$e = \frac{L}{2} - \frac{\Sigma Wx - \Sigma Hy}{\Sigma W}$ = 0.63 ft

$$q_{MAX} = \begin{cases} \frac{\Sigma W\left(1+\frac{6e}{L}\right)}{BL}, & for \quad e \le \frac{L}{6} \\ \frac{\Sigma W}{3B(0.5L-e)}, & for \quad e > \frac{L}{6} \end{cases}$$

= 0.60 ksf < Q_a = 1.5 ksf

PASS

CHECK FLEXURE CAPACITY, $A_{s,1}$ & $A_{s,2}$, FOR STEM (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$M_u = \left(\frac{w_s P_a}{\gamma_b} * H\right)(0.5 * H) + (0.5 * Pa * H^2)(\frac{1}{3} * H)$$ = 0.31 ft-kips

P_u = 22.16 kips

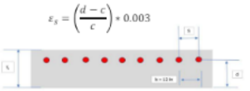
$$\phi M_n = \phi\left[A_s f_y\left(d - \frac{A_s f_y - P_u}{1.7bf_c'}\right)\right]$$

		At top stem		At base of bottom stem			
ϕM_n	=	13.30 PASS	ft-kips ,	13.30 PASS	ft-kips	>	M_u

$a = \frac{A_s f_y}{0.85 * b * f_c'}$ $c = \frac{a}{\beta_1}$ where

$\varepsilon_s = \left(\frac{d-c}{c}\right) * 0.003$

		At top stem		At base of bottom stem	
a	=	0.58	in ,	0.58	in
c	=	0.68	in ,	0.68	in
d	=	10.00	in ,	10.00	in
ε_s	=	0.041 YIELDED		0.041 YIELDED	
b	=	12	in ,	12	in
ϕ	=	0.9	,	0.9	
A_s	=	0.44	in^2 ,	0.44	in^2
ρ	=	0.003		0.003	
β_1	=	0.85		0.85	



$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f_c'}{f_y}\frac{87}{87+f_y}\right)$$ = 0.028 PASS, 0.028 PASS > ρ

$$\rho_{MIN} = 0.0018\frac{t}{d}$$ = 0.002 PASS, 0.002 PASS < ρ

Act. Earth Press. Coeff: K_a = $\tan^2(45-(\phi/2))$ =

Active Earth Pressure: P_a = $K_a * \gamma_{soil}$ =

Pass. Earth Press. Coeff: K_p = $\tan^2(45+(\phi/2))$ =

Passive Earth Pressure: P_p = $K_p * \gamma_{soil}$ =

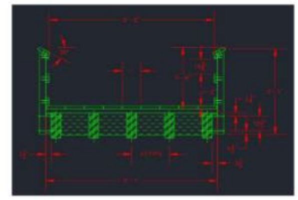


TABLE 1806.2 PRESUMPTIVE LOAD-BEARING VALUES

CLASS OF MATERIALS	VERTICAL FOUNDATION PRESSURE (psf)	LATERAL BEARING PRESSURE (psf/ft below natural grade)	LATERAL SLIDING RESISTANCE: Coefficient of friction[a]	Cohesion (psf)[b]
1. Crystalline bedrock	12,000	1,200	0.70	—
2. Sedimentary and foliated rock	4,000	400	0.35	—
3. Sandy gravel and gravel (GW and GP)	3,000	200	0.35	—
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC)	2,000	150	0.25	—
5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)	1,500	100	—	130

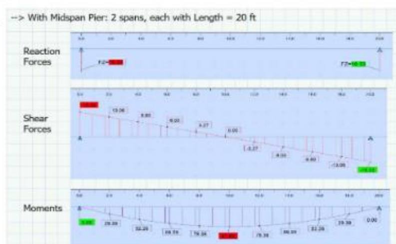
For SI: 1 pound per square foot = 0.0479kPa, 1 pound per square foot per foot = 0.157 kPa/m.
a. Coefficient to be multiplied by the dead load.
b. Cohesion value to be multiplied by the contact area, as limited by Section 1806.3.2.

Table 5.11. Typical compacted densities and optimum moisture contents for USCS soil types (after Carter and Bentley, 1991).

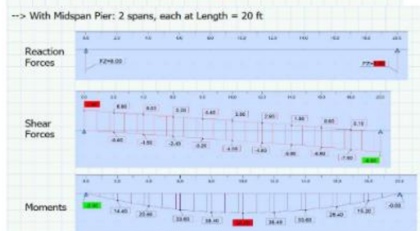
Soil Description	USCS Class	Compacted Dry Unit Weight (lb/ft3)	(kN/m3)	Optimum Moisture Content (%)
Gravel/sand mixtures:				
well graded, clean	GW	125-134	19.6-21.1	8-11
poorly graded, clean	GP	115-125	18.1-19.6	11-14
well graded, small silt content	GM	119-134	18.6-21.1	8-12
well graded, small clay content	GC	115-125	18.1-19.6	9-14
Sands and sandy soils:				
well graded, clean	SW	109-131	17.2-20.6	9-16
poorly graded, small silt content	SP	94-119	15.7-18.6	12-21
well graded, small silt content	SM	109-125	17.2-19.6	11-16
well graded, small clay content	SC	106-125	16.7-19.6	11-19
Fined-grained soils of low plasticity:				
silts	ML	94-119	14.7-18.6	12-24
clays	CL	94-119	14.7-18.6	12-24
organic silts	OL	81-100	12.7-15.7	21-33
Fine grained soils of high plasticity:				
silts	MH	69-94	10.8-14.7	24-40
clays	CH	81-106	12.7-16.6	19-36
organic clays	OH	66-100	10.3-15.7	21-45

1. Overturning Analysis

Combined Loading: --> With Midspan Pier: 2 spans, each with Length = 20 ft



HS Truck Loading: --> With Midspan Pier: 2 spans, each at Length = 20 ft

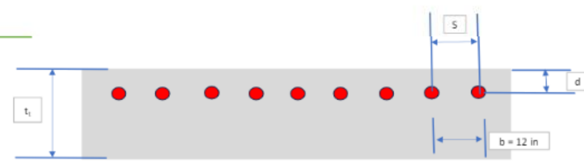


1. Overturning Analysis

2. Bearing Capacity Analysis

2. Bearing Capacity Analysis

3. Flexure Capacity Analysis



β_1 = 0.85	for $0 < f'_c \le 4000$ psi
β_1 = $0.85 - 0.05(f'_c - 4000)/1000$	for 4000 psi $< f'_c \le 8000$ psi
β_1 = 0.65	for $f'_c > 8000$ psi

$$M_u = \gamma\left(\frac{P_a y^3}{6} + \frac{P_a y^2 w_s}{2\gamma_b}\right)$$

$$P_u = \gamma W_w$$

$$V = \gamma\left(\frac{P_a y^2}{2} + \frac{w_s P_a y}{\gamma_b}\right)$$

3. Flexure Capacity Analysis

4. Shear Capacity Analysis

CHECK SHEAR CAPACITY FOR STEM (ACI 318-02 SEC.15.5.2, 11.1.3.1, & 11.3)

		At top stem		At base of bottom stem			
$V_u = \left(\frac{w_s P_a}{\gamma_b} * H\right) + (0.5 * Pa * H^2)$	=	0.35	kips ,	0.35	kips		
$V_{allowable} = 2\phi bd\sqrt{f'_c}$	=	9.86 PASS	kips ,	9.86 PASS	kips	>	V

where ϕ = 0.75 (ACI 318-02, Section 9.3.2.3) ϕ = 0.75

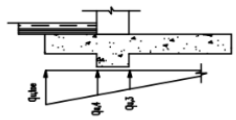
4. Shear Capacity Analysis

5. Heel Flexure Capacity Analysis

CHECK HEEL FLEXURE CAPACITY. $A_{s,3}$ FOR FOOTING (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2. & 12.5)

$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f'_c}{f_y}\frac{87}{87+f_y}\right) = 0.0278 \qquad \rho_{MIN} = \frac{0.0018}{2}\frac{h_f}{d} = 0.0054$$

$$M_{u,3} = \begin{cases} \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{(q_{u,3}+2q_{u,heel})bL_H^2}{6}, & for \quad e_u \le \frac{L}{6} \\ \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{q_{u,3}bS^2}{6}, & for \quad e_u > \frac{L}{6} \end{cases} = 1.339 \text{ ft-kips}$$



$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\frac{M_{u,3}}{0.383bd^2 f'_c}}\right)}{f_y} = 0.0101$$

PASS PASS

where							
d_{heel}	=	2.00	in	$q_{u,toe}$	=	0.75	ksf
b	=	12.00	in	$q_{u,heel}$	=	0.34	ksf
e_u	=	0.63	ft	$q_{u,3}$	=	0.57	ksf
S	=	16.00	ft	ϕ	=	0.9	

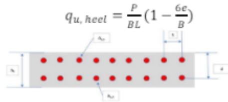
$$q_{u,toe} = \frac{P}{BL}(1+\frac{6e}{B})$$

$$q_{u,heel} = \frac{P}{BL}(1-\frac{6e}{B})$$

$$\phi M_n = \phi\left[A_S f_y\left(d - \frac{A_S f_y - P_u}{1.7bf'_c}\right)\right] = 1.34 \text{ ft-kips}$$

$(A_{S,3})_{required}$ = 0.20 in^2 / ft < $A_{S,3}$ = 0.44 in^2 / ft

PASS



5. Heel Flexure Capacity Analysis

6. Toe Flexure Capacity Analysis

CHECK TOE FLEXURE CAPACITY. $A_{s,4}$ FOR FOOTING (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f'_c}{f_y}\frac{87}{87+f_y}\right) = 0.0278 \qquad \rho_{MIN} = MIN\left(\frac{4}{3}\rho, \quad \frac{0.0018}{2}\frac{h_f}{d}\right) = 0.0001$$

$$M_{u,4} = \frac{(q_{u,4}+2q_{u,toe})bL_T^2}{6} - \frac{L_T^2}{2L}\gamma w_f = 0.27 \text{ ft-kips}$$

$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\frac{M_{u,4}}{0.383bd^2 f'_c}}\right)}{f_y} = 0.0001$$

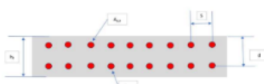
where			
d_{toe}	=	10.00	in
$q_{u,4}$	=	0.66	ksf
b	=	12.00	in
A_s	=	0.44	in^2
ϕ	=	0.9	
ρ	=	0.0031	

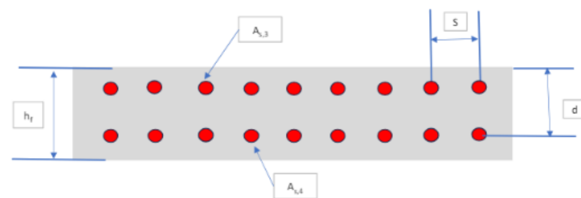
PASS PASS

$$\phi M_n = \phi\left[A_S f_y\left(d - \frac{A_S f_y - P_u}{1.7bf'_c}\right)\right] = 0.31 \text{ ft-kips}$$

$(A_{S,4})_{required}$ = 0.01 in^2 / ft < $A_{S,4}$ = 0.44 in^2 / ft

PASS





6. Toe Flexure Capacity Analysis

7. Key Capacity Analysis

CHECK KEY CAPACITY FOR FOOTING

1.5 (H_b + H_s) = 0.93 kips < H_p + μ ΣW = 5.42 kips

PASS

Techincal References:

1. Alan Williams: "Structuiral Engineering Reference Manual", Professional Publications, Inc, 2001.
2. Alan Williams: "Structuiral Engineering License Review Problems and Solutions", Oxford University Press, 2003.

Reference 14: Bridge 2 Pier Calculations with Water

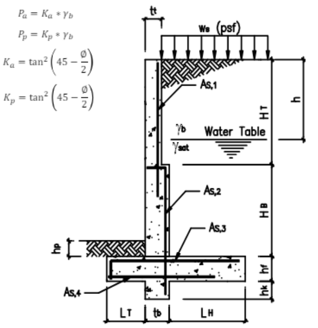
Retaining Wall Design Based on ACI 318-02

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	3	ksi
REBAR YIELD STRESS	f_y	=	40	ksi
LATER SOIL PRESSURE	$P_a=k_a\gamma_b$	=	58.83	pcf (equivalent fluid pressure)
ACTIVE EARTH PRESSURE COEFF.	K_a	=	0.490	
PASSIVE EARTH PRESSURE COEFF.	K_p	=	2.040	
FRICTION ANGLE	ϕ	=	20	deg
BACKFILL SPECIFIC WEIGHT	γ_b	=	120	pcf
SATURATED SPECIFIC WEIGHT	γ_{sat}	=	133.4	pcf
WATER SPECIFIC WEIGHT	γ_w	=	62.4	pcf
CONCRETE UNIT WEIGHT	γ_c	=	145	pcf
WATER TABLE	h	=	5.08	ft
PASSIVE PRESSURE	P_p	=	244.75	pcf
SURCHARGE WEIGHT	w_s	=	240	psf
FRICTION COEFFICIENT	μ	=	0.3	
ALLOW SOIL PRESSURE	Q_a	=	1.5	ksf IBC Table 1806.2
THICKNESS OF TOP STEM	t_t	=	12	in
THICKNESS OF KEY & STEM	t_b	=	12	in
TOE WIDTH	L_T	=	2	ft
HEEL WIDTH	L_H	=	3	ft
HEIGHT OF TOP STEM	H_T	=	1	ft
HEIGHT OF BOT. STEM	H_B	=	1	ft
FOOTING THICKNESS	h_f	=	12	in
KEY DEPTH	h_k	=	0	in
SOIL OVER TOE	h_p	=	23.00	in
	y	=	2	ft
	L	=	6.00	ft
	B	=	10.00	ft

Bridge Surface = 746.00 ft
Top of abutment = 743.08 ft
Water surface elevation = 738.00 ft
Under bridge ground elevation = 743.00 ft

$$P_a = K_a * \gamma_b$$
$$P_p = K_p * \gamma_b$$
$$K_a = \tan^2\left(45 - \frac{\emptyset}{2}\right)$$
$$K_p = \tan^2\left(45 - \frac{\emptyset}{2}\right)$$



TOP STEM REINF. ($A_{s,1}$)	#	6	@	16	in o.c., (Caution > 18"o.c. max. ACI 14.3.5)	A_{bar} =	0.44	in^2
$A_{s,1}$ LOCATION (0=at soil face, 1=at middle)		0	at soil face					
BOT. STEM REINF. ($A_{s,2}$)	#	6	@	16	in o.c.	A_{bar} =	0.44	in^2
$A_{s,2}$ LOCATION (0=at soil face, 1=at middle)		0	at soil face					
TOP REINF. OF FOOTING ($A_{s,3}$)	#	6	@	16	in o.c., (Caution > 18"o.c. max. ACI 7.12.2.2)	A_{bar} =	0.44	in^2
BOT. REINF. OF FOOTING ($A_{s,4}$)	#	6	@	16	in	A_{bar} =	0.44	in^2

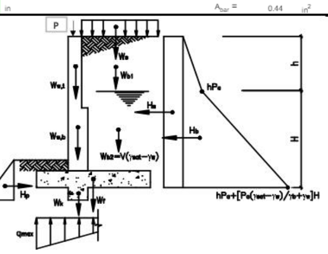
ANALYSIS

SERVICE LOADS

$H_b = 0.5\,P_a h^2 + h P_a H + 0.5\,[P_a(\gamma_{sat}-\gamma_w)/\gamma_b + \gamma_w]\,H^2$ = 0.19 kips

Where h = 5.0833 ft , H = -2.0833 ft

$H_s = w_s P_a (H_T + H_B + h_f)/\gamma_b$	=	0.35	kips
$H_p = 0.5\,P_p (h_p + h_f + h_k)^2$	=	1.04	kips
$W_s = w_s (L_H + t_b - t_t)$	=	0.72	kips
$W_b = W_{b1} + W_{b2}$	=	1.39	kips
Where W_{b1} = 1.83 kips , W_{b2}	=	-0.44	kips
$W_f = h_f (L_H + t_b + L_T)\gamma_c$	=	0.87	kips
$W_k = h_k\, t_b\, \gamma_c$	=	0.00	kips
$W_{w,t} = t_t\, H_T\, \gamma_c$	=	0.15	kips
$W_{w,b} = t_b\, H_B\, \gamma_c$	=	0.15	kips
$P = ((R_{... } + R_{...}) * N_{...}) / B$	=	24.93	kips



FACTORED LOADS

$\gamma H_b = 1.6\,H_b$	=	0.30	kips
$\gamma H_s = 1.6\,H_s$	=	0.56	kips
$\gamma W_s = 1.6\,W_s$	=	1.15	kips
$\gamma W_b = 1.2\,W_b$	=	1.66	kips
$\gamma W_f = 1.2\,W_f$	=	1.04	kips
$\gamma W_k = 1.2\,W_k$	=	0.00	kips
$\gamma W_{w,t} = 1.2\,W_{w,t}$	=	0.17	kips
$\gamma W_{w,b} = 1.2\,W_{w,b}$	=	0.17	kips
$\gamma P = 1.75\,P$	=	43.63	kips

OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_b	0.19	0.30	-0.83	-0.16	-0.25
H_s	0.35	0.56	1.50	0.53	0.85
Σ	0.54	0.86		**0.37**	0.60
	(kip)	(kip)	(ft)	(kip-ft)	(kip-ft)

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.72	1.15	4.50	3.24	5.18
W_b	1.39	1.66	4.50	6.24	7.49
W_f	0.87	1.04	3.00	2.61	3.13
W_k	0.00	0.00	2.50	0.00	0.00
$W_{w,t}$	0.15	0.17	2.50	0.36	0.44
$W_{w,b}$	0.15	0.17	2.50	0.36	0.44
P	24.93	43.63	2.50	62.33	109.07
Σ	28.20	47.83		**75.14**	125.74
	(kip)	(kip)	(ft)	(kip-ft)	(kip-ft)

OVERTURNING FACTOR OF SAFETY

$$SF = \frac{\Sigma Wx}{\Sigma Hy} = 200.78 > 1.5$$

PASS

CHECK SOIL BEARING CAPACITY (ACI 318-02 SEC.15.2.2)

$$L = L_T + t_b + L_H = 6.00 \text{ ft}$$

$$e = \frac{L}{2} - \frac{\Sigma Wx - \Sigma Hy}{\Sigma W} = 0.35 \text{ ft}$$

$$q_{MAX} = \begin{cases} \dfrac{\Sigma W\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{\Sigma W}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases} = 0.63 \text{ ksf} < Q_a = 1.5 \text{ ksf}$$

PASS

CHECK FLEXURE CAPACITY, $A_{s,1}$ & $A_{s,2}$, FOR STEM (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

h = 5.0833 ft , H' = -3.0833 ft

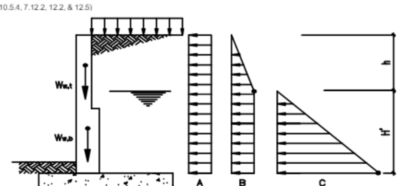
$A = w_s P_a / \gamma_b$ =	118	plf
$B = h P_a$ =	299	plf
$C = [P_a(\gamma_{sat}-\gamma_w)/\gamma_b + \gamma_w]\,H'$ =	-71	plf

At base of top stem

M_u =	0.18	ft-kips
V_u =	0.49	kips
P_u =	43.98	kips

At base of bottom stem

M_u =	0.18	ft-kips
V_u =	0.49	kips
P_u =	43.98	kips



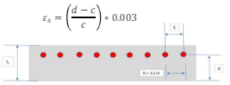
$$\phi M_n = \phi\left[A_s f_y\left(d - \frac{A_s f_y - P_u}{1.7 b f'_c}\right)\right]$$

$$a = \frac{A_s f_y}{0.85 * b * f_c'} \qquad c = \frac{a}{\beta_1}$$

$$\varepsilon_s = \left(\frac{d-c}{c}\right) * 0.003$$

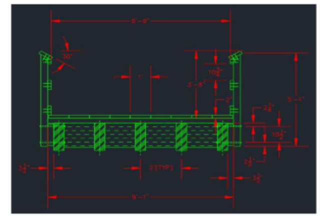
$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f'_c}{f_y}\frac{87}{87+f_y}\right)$$

$$\rho_{MIN} = 0.0018\frac{t}{d}$$



		At top stem		At base of bottom stem			
ϕM_n	=	13.77 PASS	ft-kips ,	13.77 PASS	ft-kips	>	M_u
where a	=	0.58	in ,	0.58	in		
c	=	0.68	in ,	0.68	in		
d	=	10.00	in ,	10.00	in		
ε_s	=	0.041 YIELDED		0.041 YIELDED			
b	=	12	in ,	12	in		
ϕ	=	0.9	,	0.9			
A_s	=	0.44	in^2 ,	0.44	in^2		
ρ	=	0.003		0.003			
β_1	=	0.85		0.85			
ρ_{MAX}	=	0.028 PASS		0.028 PASS		>	ρ
ρ_{MIN}	=	0.002 PASS		0.002 PASS		<	ρ

Pier @ 20 ft



Act. Earth Press. Coeff:	K_a =	$\tan^2(45-(\phi/2))$ =	K_a :	
Active Earth Pressure:	P_a =	$K_a * \gamma_{soil}$ =	<1	$K_a = D/B$
Pass. Earth Press. Coeff:	K_p =	$\tan^2(45+(\phi/2))$ =	>1	$K_a = \tan^{-1}(D/B)$
Passive Earth Pressure:	P_p =	$K_p * \gamma_{soil}$ =		

TABLE 1806.2 PRESUMPTIVE LOAD-BEARING VALUES

CLASS OF MATERIALS	VERTICAL FOUNDATION PRESSURE (psf)	LATERAL BEARING PRESSURE (psf/ft below natural grade)	LATERAL SLIDING RESISTANCE: Coefficient of friction[a]	LATERAL SLIDING RESISTANCE: Cohesion (psf)[b]
1. Crystalline bedrock	12,000	1,200	0.70	—
2. Sedimentary and foliated rock	4,000	400	0.35	—
3. Sandy gravel and gravel (GW and GP)	3,000	200	0.35	—
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC)	2,000	150	0.25	—
5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)	1,500	100	—	130

For SI: 1 pound per square foot = 0.0479kPa, 1 pound per square foot per foot = 0.157 kPa/m.
a. Coefficient to be multiplied by the dead load.
b. Cohesion value to be multiplied by the contact area, as limited by Section 1806.3.2.

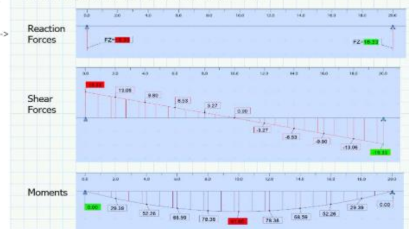
Table 5.17. Typical compacted densities and optimum moisture contents for USCS soil types (after Carter and Bentley, 1991).

Soil Description	USCS Class	Compacted Dry Unit Weight (lb/ft3)	Compacted Dry Unit Weight (kN/m3)	Optimum Moisture Content (%)
Gravel/sand mixtures:				
well-graded, clean	GW	125-134	19.6-21.1	8-11
poorly-graded, clean	GP	115-125	18.1-19.6	11-14
well-graded, small silt content	GM	119-134	18.6-21.1	8-12
well-graded, small clay content	GC	115-125	18.1-19.6	9-14
Sands and sandy soils:				
well-graded, clean	SW	109-131	17.2-20.6	9-16
poorly-graded, small silt content	SP	94-119	15.7-18.6	12-21
well-graded, small silt content	SM	109-125	17.2-19.6	11-16
well-graded, small clay content	SC	106-125	16.7-19.6	11-19
Fined-grained soils of low plasticity:				
silts	ML	94-119	14.7-18.6	12-24
clays	CL	94-119	14.7-18.6	12-24
organic silts	OL	81-100	12.7-15.7	21-33
Fine-grained soils of high plasticity:				
silts	MH	69-94	10.8-14.7	24-40
clays	CH	81-106	12.7-18.6	19-36
organic clays	OH	66-100	10.3-16.7	21-45

1. Overturning Analysis

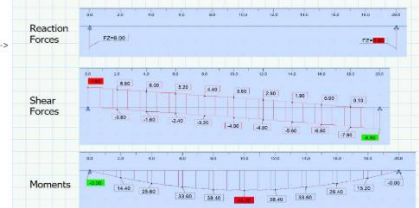
Combined Loading: --> With Midspan Pier: 2 spans, each with Length = 20 ft

**Multiply by 2 -->



HS Truck Loading: --> With Midspan Pier: 2 spans, each at Length = 20 ft

**Multiply by 2 -->



1. Overturning Analysis

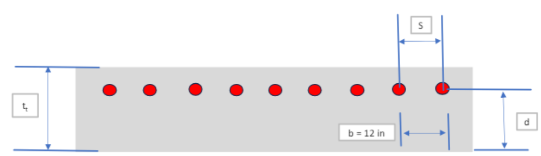
2. Bearing Capacity Analysis

2. Bearing Capacity Analysis

3. Flexure Capacity Analysis

$$M_u = (A*H)(0.5*H) + (B*H')(0.5*H') + (0.5*B*h)\left(H' + \frac{1}{3}h\right) + (0.5*C*H')\left(\frac{1}{3}H'\right)$$

$$V_u = (A*H) + (B*H') + (0.5*B*h) + (0.5*C*H')$$



$\beta_1 = 0.85$	for $0 < f'_c \le 4000$ psi
$\beta_1 = 0.85 - 0.05(f'_c - 4000)/1000$	for 4000 psi $< f'_c \le 8000$ psi
$\beta_1 = 0.65$	for $f'_c > 8000$ psi

3. Flexure Capacity Analysis

4. Shear Capacity Analysis

CHECK SHEAR CAPACITY FOR STEM (ACI 318-02 SEC.15.5.2, 11.1.3.1, & 11.3)

	At top stem		At base of bottom stem			
$V_{allowable} = 2\phi bd\sqrt{f'_c}$ =	9.86	kips ,	9.86	kips	>	V_u
	PASS		PASS			

where ϕ = 0.75 (ACI 318-02, Section 9.3.2.3) ϕ = 0.75

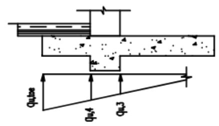
4. Shear Capacity Analysis

5. Heel Flexure Capacity Analysis

CHECK HEEL FLEXURE CAPACITY $A_{s,3}$ FOR FOOTING (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f'_c}{f_y}\frac{87}{87+f_y}\right) = 0.0278 \qquad \rho_{MIN} = \frac{0.0018}{2}\frac{h_f}{d} = 0.0054$$

$$M_{u,3} = \begin{cases} \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{(q_{u,3}+2q_{u,heel})bL_H^2}{6}, & for \quad e_u \le \frac{L}{6} \\ \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{q_{u,3}bS^2}{6}, & for \quad e_u > \frac{L}{6} \end{cases} = 1.92 \text{ ft-kips}$$



$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\frac{M_{u,3}}{0.383bd^2 f'_c}}\right)}{f_y} = 0.0151$$

PASS PASS

where	d	=	2.00	in	$q_{u,toe}$	=	0.96	ksf
	b	=	12.00	in	$q_{u,heel}$	=	0.63	ksf
	e_u	=	0.35	ft	$q_{u,3}$	=	0.80	ksf
	S	=	16.00	in	ϕ	=	0.9	

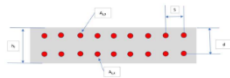
$$q_{u,toe} = \frac{P}{BL}(1+\frac{6e}{B})$$

$$q_{u,heel} = \frac{P}{BL}(1-\frac{6e}{B})$$

$$\phi M_n = \phi\left[A_s f_y\left(d - \frac{A_s f_y - P_u}{1.7bf'_c}\right)\right] = 1.99 \text{ ft-kips}$$

$(A_{s,3})_{required}$ = 0.26 in^2 / ft < $A_{s,3}$ = 0.44 in^2 / ft

PASS



5. Heel Flexure Capacity Analysis

6. Toe Flexure Capacity Analysis

CHECK TOE FLEXURE CAPACITY $A_{s,4}$ FOR FOOTING (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f'_c}{f_y}\frac{87}{87+f_y}\right) = 0.028 \qquad \rho_{MIN} = MIN\left(\frac{4}{3}\rho, \ \frac{0.0018}{2}\frac{h_f}{d}\right) = 0.0006$$

$$M_{u,4} = \frac{(q_{u,4}+2q_{u,toe})bL_T^2}{6} - \frac{L_T^2}{2L}\gamma w_f = 1.51 \text{ ft-kips}$$

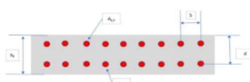
where	d	=	10.00	in
	$q_{u,4}$	=	0.85	ksf
	b	=	12.00	in
	A_s	=	0.44	in^2
	ϕ	=	0.9	
	ρ	=	0.0031	
			PASS	PASS

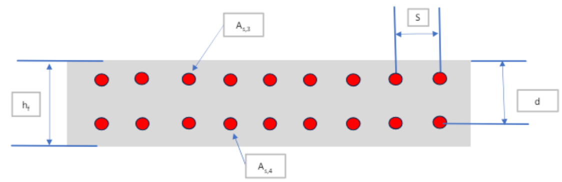
$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\frac{M_{u,4}}{0.383bd^2 f'_c}}\right)}{f_y} = 0.0004$$

$$\phi M_n = \phi\left[A_s f_y\left(d - \frac{A_s f_y - P_u}{1.7bf'_c}\right)\right] = 1.60 \text{ ft-kips}$$

$(A_{s,4})_{required}$ = 0.05 in^2 / ft < $A_{s,4}$ = 0.44 in^2 / ft

PASS





toe - tension bottom

heel - tension top

6. Toe Flexure Capacity Analysis

7. Key Capacity Analysis

CHECK KEY CAPACITY FOR FOOTING

1.5 (H_p + H_s) = 0.81 kips < $H_p + \mu \Sigma W$ = 9.50 kips

PASS

Techincal References:

1. Alan Williams: "Structuiral Engineering Reference Manual", Professional Publications, Inc, 2001.
2. Alan Williams: "Structural Engineering License Review Problems and Solutions", Oxford University Press, 2003.

Reference 15: Bridge 2 Pier Calculations without Water

Retaining Wall Design Based on ACI 318-02

INPUT DATA & DESIGN SUMMARY

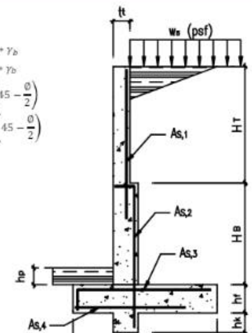
CONCRETE STRENGTH	f_c'	=	3	ksi			
REBAR YIELD STRESS	f_y	=	40	ksi			
LATER SOIL PRESSURE	P_a	=	58.83	pcf (equivalent fluid pressure)			
PASSIVE PRESSURE	P_p	=	244.75	pcf			
ACTIVE EARTH PRESSURE COEFF.	K_a	=	0.490				
PASSIVE EARTH PRESSURE COEFF.	K_p	=	2.040				
FRICTION ANGLE	ϕ	=	20	deg			
SURCHARGE WEIGHT	w_s	=	240	psf			
FRICTION COEFFICIENT	μ	=	0.3				
ALLOW SOIL PRESSURE	Q_a	=	1.5	ksf	IBC Table 1806.2		
THICKNESS OF TOP STEM	t_t	=	12	in			
THICKNESS OF KEY & STEM	t_b	=	12	in			
TOE WIDTH	L_T	=	2	ft			
HEEL WIDTH	L_H	=	3	ft			
HEIGHT OF TOP STEM	H_T	=	1	ft			
HEIGHT OF BOT. STEM	H_B	=	1	ft			
FOOTING THICKNESS	h_f	=	12	in			
KEY DEPTH	h_k	=	0	in			
SOIL OVER TOE	h_p	=	23.00	in			
	y	=	2	ft			
	L	=	6	ft			
	B	=	10	ft			
UNIT WEIGHT OF SOIL	γ_b	=	120	pcf			
UNIT WEIGHT OF CONCRETE	γ_c	=	145	pcf			
TOP STEM REINF. ($A_{s,1}$)	1	#	6	@	16	in o.c., at middle	A_{bar} = 0.44 in²
BOT. STEM REINF. ($A_{s,2}$)	2	#	6	@	16	in o.c., at each face	A_{bar} = 0.44 in²
TOP REINF.OF FOOTING ($A_{s,3}$)		#	6	@	16	in	A_{bar} = 0.44 in²
BOT. REINF.OF FOOTING ($A_{s,4}$)		#	6	@	16	in	A_{bar} = 0.44 in²

$P_a = K_a * \gamma_b$

$P_p = K_p * \gamma_b$

$K_a = \tan^2\left(45 - \frac{\varnothing}{2}\right)$

$K_p = \tan^2\left(45 - \frac{\varnothing}{2}\right)$



Act. Earth Press. Coeff: K_a = $\tan^2(45-(\phi/2))$ =

Active Earth Pressure: P_a = $K_a * \gamma_{soil}$ =

Pass. Earth Press. Coeff: K_p = $\tan^2(45+(\phi/2))$ =

Passive Earth Pressure: P_p = $K_p * \gamma_{soil}$ =

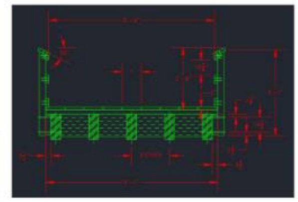


TABLE 1806.2 PRESUMPTIVE LOAD-BEARING VALUES

CLASS OF MATERIALS	VERTICAL FOUNDATION PRESSURE (psf)	LATERAL BEARING PRESSURE (psf/ft below natural grade)	LATERAL SLIDING RESISTANCE: Coefficient of friction[a]	LATERAL SLIDING RESISTANCE: Cohesion (psf)[b]
1. Crystalline bedrock	12,000	1,200	0.70	—
2. Sedimentary and foliated rock	4,000	400	0.35	—
3. Sandy gravel and gravel (GW and GP)	3,000	200	0.35	—
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC)	2,000	150	0.25	—
5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)	1,500	100	—	130

For SI: 1 pound per square foot = 0.0479kPa, 1 pound per square foot per foot = 0.157 kPa/m.
a. Coefficient to be multiplied by the dead load.
b. Cohesion value to be multiplied by the contact area, as limited by Section 1806.3.2.

Table 5.17. Typical compacted densities and optimum moisture contents for USCS soil types (after Carter and Bentley 1991).

Soil Description	USCS Class	Compacted Dry Unit Weight (lb/ft³)	Compacted Dry Unit Weight (kN/m³)	Optimum Moisture Content (%)
Gravel/sand mixtures:				
[illegible]	GW	[illegible]	[illegible]	[illegible]
[illegible]	GP	[illegible]	[illegible]	[illegible]
[illegible]	GM	[illegible]	[illegible]	[illegible]
[illegible]	GC	[illegible]	[illegible]	[illegible]
Sands and sandy soils:				
[illegible]	SW	[illegible]	[illegible]	[illegible]
[illegible]	SP	[illegible]	[illegible]	[illegible]
[illegible]	SM	[illegible]	[illegible]	[illegible]
[illegible]	SC	[illegible]	[illegible]	[illegible]
Fine-grained soils of low plasticity:				
silts	ML	[illegible]	[illegible]	[illegible]
clays	CL	[illegible]	[illegible]	[illegible]
organic silts	OL	[illegible]	[illegible]	[illegible]
Fine-grained soils of high plasticity:				
silts	MH	[illegible]	[illegible]	[illegible]
clays	CH	[illegible]	[illegible]	[illegible]
organic clays	OH	[illegible]	[illegible]	[illegible]

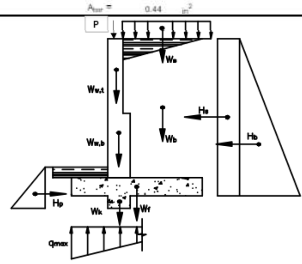
ANALYSIS

SERVICE LOADS

$H_b = 0.5\,P_a\,(H_T + H_B + h_f)^2$	=	0.26	kips
$H_s = w_s\,P_a\,(H_T + H_B + h_f) / \gamma_b$	=	0.35	kips
$H_p = 0.5\,P_p\,(h_p + h_f + h_k)^2$	=	1.04	kips
$W_s = w_s\,(L_H + t_b - t_t)$	=	0.72	kips
$W_b = [H_T\,(L_H + t_b - t_t) + H_B\,L_H]\,\gamma_b$	=	0.72	kips
$W_f = h_f\,(L_H + t_b + L_T)\,\gamma_c$	=	0.87	kips
$W_k = h_k\ t_b\ \gamma_c$	=	0.00	kips
$W_{w,t} = t_t\ H_T\ \gamma_c$	=	0.15	kips
$W_{w,b} = t_b\ H_B\ \gamma_c$	=	0.15	kips
$P = R_{comb} + R_{H5}$	=	24.93	kips

FACTORED LOADS

$\gamma H_b = 1.6\,H_b$	=	0.42	kips
$\gamma H_s = 1.6\,H_s$	=	0.56	kips
$\gamma W_s = 1.6\,W_s$	=	1.15	kips
$\gamma W_b = 1.2\,W_b$	=	0.86	kips
$\gamma W_f = 1.2\,W_f$	=	1.04	kips
$\gamma W_k = 1.2\,W_k$	=	0.00	kips
$\gamma W_{w,t} = 1.2\,W_{w,t}$	=	0.17	kips
$\gamma W_{w,b} = 1.2\,W_{w,b}$	=	0.17	kips
$\gamma P = 1.75\,P$	=	43.63	kips



OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_b	0.26	0.42	1.00	0.26	0.42
H_s	0.35	0.56	1.50	0.53	0.85
Σ	0.62	0.99		**0.79**	1.27
	(kip)	(kip)	(ft)	(kip-ft)	(kip-ft)

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.72	1.15	4.50	3.24	5.18
W_b	0.72	0.86	4.50	3.24	3.89
W_f	0.87	1.04	3.00	2.61	3.13
W_k	0.00	0.00	2.50	0.00	0.00
$W_{w,t}$	0.15	0.17	2.50	0.36	0.44
$W_{w,b}$	0.15	0.17	2.50	0.36	0.44
P	24.93	43.63	2.50	62.33	109.07
Σ	27.53	47.04		**72.14**	122.14
	(kip)	(kip)	(ft)	(kip-ft)	(kip-ft)

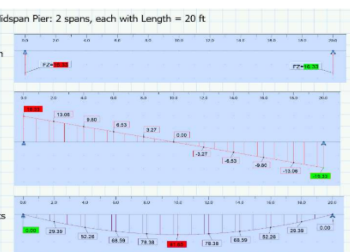
OVERTURNING FACTOR OF SAFETY

$SF = \frac{\Sigma Wx}{\Sigma Hy}$ = 90.83 PASS > 1.5

1. Overturning Analysis

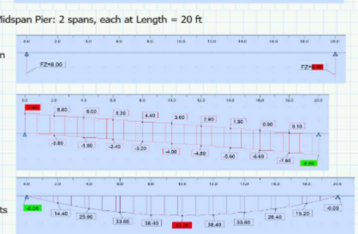
Combined Loading: --> With Midspan Pier: 2 spans, each with Length = 20 ft

**Multiply by 2 -->



H5 Truck Loading: --> With Midspan Pier: 2 spans, each at Length = 20 ft

**Multiply by 2 -->



1. Overturning Analysis

2. Bearing Capacity Analysis

CHECK SOIL BEARING CAPACITY (ACI 318-02 SEC.15.2.2)

$L = L_T + t_b + L_H$ = 6.00 ft

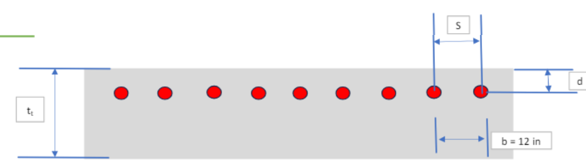
$e = \frac{L}{2} - \frac{\Sigma Wx - \Sigma Hy}{\Sigma W}$ = 0.41 ft

$$q_{MAX} = \begin{cases} \frac{\Sigma W\left(1 + \frac{6e}{L}\right)}{BL}, & for \quad e \le \frac{L}{6} \\ \frac{\Sigma W}{3B(0.5L - e)}, & for \quad e > \frac{L}{6} \end{cases}$$

= 0.65 ksf PASS < Q_a = 1.5 ksf

2. Bearing Capacity Analysis

3. Flexure Capacity Analysis



CHECK FLEXURE CAPACITY, $A_{S,1}$ & $A_{S,2}$, FOR STEM (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$M_u = \left(\frac{w_s P_a}{\gamma_b} * H\right)(0.5 * H) + (0.5 * Pa * H^2)(\frac{1}{3} * H)$ = 0.31 ft-kips

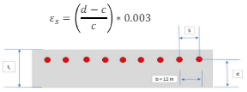
P_u = 43.98 kips

$$\phi M_n = \phi\left[A_S f_y\left(d - \frac{A_S f_y - P_u}{1.7bf_c'}\right)\right]$$

		At top stem		At base of bottom stem			
ϕM_n	=	13.77 PASS	ft-kips ,	13.77 PASS	ft-kips	>	M_u

$a = \frac{A_s f_y}{0.85 * b * f_c'}$ $c = \frac{a}{\beta_1}$ where

$\varepsilon_s = \left(\frac{d-c}{c}\right) * 0.003$



		At top stem		At base of bottom stem	
a	=	0.58	in ,	0.58	in
c	=	0.68	in ,	0.68	in
d	=	10.00	in ,	10.00	in
ε_s	=	0.041 YIELDED		0.041 YIELDED	
b	=	12	in ,	12	in
ϕ	=	0.9	,	0.9	
A_s	=	0.44	in² ,	0.44	in²
ρ	=	0.003		0.003	
β_1	=	0.85		0.85	

$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f_c'}{f_y}\frac{87}{87 + f_y}\right)$ = 0.028 PASS , 0.028 PASS > ρ

$\rho_{MIN} = 0.0018\frac{t}{d}$ = 0.002 PASS , 0.002 PASS < ρ

β_1 = 0.85	for $0 < f'_c \le 4000$ psi
β_1 = 0.85 – 0.05(f'_c – 4000)/1000	for 4000 psi < $f'_c \le 8000$ psi
β_1 = 0.65	for $f'_c > 8000$ psi

$$M_u = \gamma\left(\frac{P_a y^3}{6} + \frac{P_a y^2 w_s}{2\gamma_b}\right)$$

$$P_u = \gamma W_w$$

$$V = \gamma\left(\frac{P_a y^2}{2} + \frac{w_s P_a y}{\gamma_b}\right)$$

3. Flexure Capacity Analysis

4. Shear Capacity Analysis

CHECK SHEAR CAPACITY FOR STEM (ACI 318-02 SEC.15.5.2, 11.1.3.1, & 11.3)

		At top stem		At base of bottom stem			
$V_u = \left(\frac{w_s P_a}{\gamma_b} * H\right) + (0.5 * Pa * H^2)$	=	0.35	kips ,	0.35	kips		
$V_{allowable} = 2\phi b d\sqrt{f'_c}$	=	9.86	kips ,	9.86	kips	>	V
		PASS		PASS			

where ϕ = 0.75 (ACI 318-02, Section 9.3.2.3) ϕ = 0.75

4. Shear Capacity Analysis

5. Heel Flexure Capacity Analysis

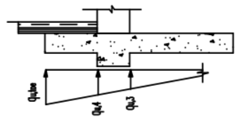
CHECK HEEL FLEXURE CAPACITY. $A_{s,3}$ FOR FOOTING (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f'_c}{f_y}\frac{87}{87+f_y}\right) = 0.0278$$

$$\rho_{MIN} = \frac{0.0018}{2}\frac{h_f}{d} = 0.0054$$

$$M_{u,3} = \begin{cases} \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{(q_{u,3}+2q_{u,heel})bL_H^2}{6}, & for\ e_u \le \frac{L}{6} \\ \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{q_{u,3}bS^2}{6}, & for\ e_u > \frac{L}{6} \end{cases}$$

= 0.856 ft-kips



$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\frac{M_{u,3}}{0.383bd^2 f'_c}}\right)}{f_y} = 0.0062$$

PASS PASS

where							
d_{heel}	=	2.00	in	$q_{u,toe}$	=	0.98	ksf
b	=	12.00	in	$q_{u,heel}$	=	0.59	ksf
e_u	=	0.41	ft	$q_{u,3}$	=	0.78	ksf
S	=	16.00	ft	ϕ	=	0.9	

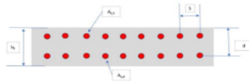
$$q_{u,toe} = \frac{P}{BL}\left(1+\frac{6e}{B}\right)$$

$$q_{u,heel} = \frac{P}{BL}\left(1-\frac{6e}{B}\right)$$

$$\phi M_n = \phi\left[A_S f_y\left(d - \frac{A_S f_y - P_u}{1.7bf'_c}\right)\right] = 0.87 \text{ ft-kips}$$

$(A_{S,3})_{required}$ = 0.11 in^2 / ft < $A_{S,3}$ = 0.44 in^2 / ft

PASS



5. Heel Flexure Capacity Analysis

6. Toe Flexure Capacity Analysis

CHECK TOE FLEXURE CAPACITY. $A_{s,4}$ FOR FOOTING (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f'_c}{f_y}\frac{87}{87+f_y}\right) = 0.0278$$

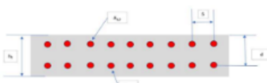
$$\rho_{MIN} = MIN\left(\frac{4}{3}\rho,\ \frac{0.0018}{2}\frac{h_f}{d}\right) = 0.0006$$

$$M_{u,4} = \frac{(q_{u,4}+2q_{u,toe})bL_T^2}{6} - \frac{L_T^2}{2L}\gamma w_f = 1.52 \text{ ft-kips}$$

where			
d_{toe}	=	10.00	in
$q_{u,4}$	=	0.85	ksf
b	=	12.00	in
A_s	=	0.44	in^2
ϕ	=	0.9	
ρ	=	0.0031	
		PASS	PASS

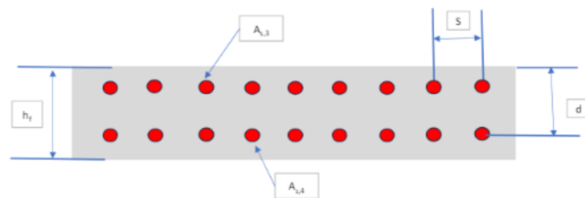
$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\frac{M_{u,4}}{0.383bd^2 f'_c}}\right)}{f_y} = 0.0004$$

$$\phi M_n = \phi\left[A_S f_y\left(d - \frac{A_S f_y - P_u}{1.7bf'_c}\right)\right] = 1.60 \text{ ft-kips}$$



$(A_{S,4})_{required}$ = 0.05 in^2 / ft < $A_{S,4}$ = 0.44 in^2 / ft

PASS



6. Toe Flexure Capacity Analysis

7. Key Capacity Analysis

CHECK KEY CAPACITY FOR FOOTING

1.5 (H_a + H_s) = 0.93 kips < H_p + μ ΣW = 9.30 kips

PASS

Techincal References:

1. Alan Williams: "Structural Engineering Reference Manual", Professional Publications, Inc. 2001.
2. Alan Williams: "Structural Engineering License Review Problems and Solutions", Oxford University Press, 2003.

Reference 16: Bridge 3 Abutment Calculations with Water

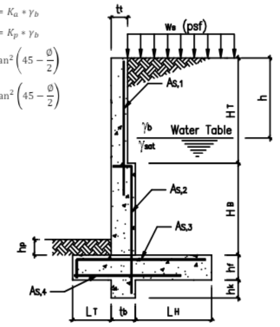
Retaining Wall Design Based on ACI 318-02

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	3	ksi
REBAR YIELD STRESS	f_y	=	40	ksi
LATER SOIL PRESSURE	$P_a = k_a \gamma_b$	=	58.83	pcf (equivalent fluid pressure)
ACTIVE EARTH PRESSURE COEFF.	K_a	=	0.490	
PASSIVE EARTH PRESSURE COEFF.	K_p	=	2.040	
FRICTION ANGLE	ϕ	=	20	deg
BACKFILL SPECIFIC WEIGHT	γ_b	=	120	pcf
SATURATED SPECIFIC WEIGHT	γ_{sat}	=	133.4	pcf
WATER SPECIFIC WEIGHT	γ_w	=	62.4	pcf
CONCRETE UNIT WEIGHT	γ_c	=	145	pcf
WATER TABLE	h	=	2.08	ft
PASSIVE PRESSURE	P_p	=	244.75	pcf
SURCHARGE WEIGHT	w_s	=	240	psf
FRICTION COEFFICIENT	μ	=	0.3	
ALLOW SOIL PRESSURE	Q_a	=	1.5	ksf IBC Table 1806.2
THICKNESS OF TOP STEM	t_t	=	12	in
THICKNESS OF KEY & STEM	t_b	=	12	in
TOE WIDTH	L_T	=	2	ft
HEEL WIDTH	L_H	=	3	ft
HEIGHT OF TOP STEM	H_T	=	1	ft
HEIGHT OF BOT. STEM	H_B	=	1	ft
FOOTING THICKNESS	h_f	=	12	in
KEY DEPTH	h_k	=	0	in
SOIL OVER TOE	h_p	=	47.00	in
	y	=	2	ft
	L	=	6.00	ft
	B	=	10.00	ft

Bridge Surface =	743.00 ft
Top of abutment =	740.08 ft
Water surface elevation =	738.00 ft
Under bridge ground elevation =	742.00 ft

$$P_a = K_a * \gamma_b$$
$$P_p = K_p * \gamma_b$$
$$K_a = \tan^2\left(45 - \frac{\emptyset}{2}\right)$$
$$K_p = \tan^2\left(45 - \frac{\emptyset}{2}\right)$$



TOP STEM REINF. ($A_{s,1}$)	#	6	@	16	in o.c., (Caution > 18"o.c. max. ACI 14.3.5)	A_{bar} =	0.44 in^2
$A_{s,1}$ LOCATION (0=at soil face, 1=at middle)		0	at soil face				
BOT. STEM REINF. ($A_{s,2}$)	#	6	@	16	in o.c.	A_{bar} =	0.44 in^2
$A_{s,2}$ LOCATION (0=at soil face, 1=at middle)		0	at soil face				
TOP REINF. OF FOOTING ($A_{s,3}$)	#	5	@	16	in o.c., (Caution > 18"o.c. max. ACI 7.12.2.2)	A_{bar} =	0.31 in^2
BOT. REINF. OF FOOTING ($A_{s,4}$)	#	5	@	16	in	A_{bar} =	0.31 in^2

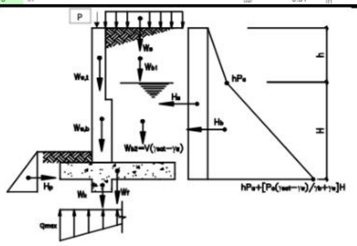
ANALYSIS

SERVICE LOADS

$H_b = 0.5 P_a h^2 + h P_a H + 0.5 [P_a (\gamma_{sat} - \gamma_w) / \gamma_b + \gamma_w] H^2$ = 0.25 kips

Where h = 2.0833 ft , H = 0.9167 ft

$H_s = w_s P_a (H_T + H_B + h_f) / \gamma_b$	=	0.35	kips
$H_p = 0.5 P_p (h_p + h_f + h_k)^2$	=	2.96	kips
$W_s = w_s (L_H + t_b - t_t)$	=	0.72	kips
$W_b = W_{b1} + W_{b2}$	=	0.95	kips
Where W_{b1} = 0.75 kips , W_{b2} =		0.20	kips
$W_f = h_f (L_H + t_b + L_T) \gamma_c$	=	0.87	kips
$W_k = h_k\ t_b\ \gamma_c$	=	0.00	kips
$W_{w,t} = t_t\ H_T\ \gamma_c$	=	0.15	kips
$W_{w,b} = t_b\ H_B\ \gamma_c$	=	0.15	kips
$P = ((R_{truck} + R_{DL}) * N_{beams}) / B$	=	24.31	kips



FACTORED LOADS

$\gamma H_b = 1.6 H_b$	=	0.40	kips
$\gamma H_s = 1.6 H_s$	=	0.56	kips
$\gamma W_s = 1.6 W_s$	=	1.15	kips
$\gamma W_b = 1.2 W_b$	=	1.13	kips
$\gamma W_f = 1.2 W_f$	=	1.04	kips
$\gamma W_k = 1.2 W_k$	=	0.00	kips
$\gamma W_{w,t} = 1.2 W_{w,t}$	=	0.17	kips
$\gamma W_{w,b} = 1.2 W_{w,b}$	=	0.17	kips
$\gamma P = 1.75 P$	=	42.53	kips

OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_b	0.25	0.40	0.81	0.20	0.32
H_s	0.35	0.56	1.50	0.53	0.85
Σ	0.60	0.96		**0.73**	1.17
	(kip)	(kip)	(ft)	(kip-ft)	(kip-ft)

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.72	1.15	4.50	3.24	5.18
W_b	0.95	1.13	4.50	4.25	5.10
W_f	0.87	1.04	3.00	2.61	3.13
W_k	0.00	0.00	2.50	0.00	0.00
$W_{w,t}$	0.15	0.17	2.50	0.36	0.44
$W_{w,b}$	0.15	0.17	2.50	0.36	0.44
P	24.31	42.53	2.50	60.76	106.33
Σ	27.13	46.21		**71.59**	120.62
	(kip)	(kip)	(ft)	(kip-ft)	(kip-ft)

OVERTURNING FACTOR OF SAFETY

$$SF = \frac{\Sigma Wx}{\Sigma Hy} = 97.72 > 1.5$$

PASS

CHECK SOIL BEARING CAPACITY (ACI 318-02 SEC.15.2.2)

$$L = L_T + t_b + L_H = 6.00 \text{ ft}$$

$$e = \frac{L}{2} - \frac{\Sigma Wx - \Sigma Hy}{\Sigma W} = 0.39 \text{ ft}$$

$$q_{MAX} = \begin{cases} \dfrac{\Sigma W\left(1+\dfrac{6e}{L}\right)}{BL}, & for\ \ e \le \dfrac{L}{6} \\ \dfrac{\Sigma W}{3B(0.5L-e)}, & for\ \ e > \dfrac{L}{6} \end{cases} = 0.63 \text{ ksf} < Q_a = 1.5 \text{ ksf}$$

PASS

CHECK FLEXURE CAPACITY, $A_{s,1}$ & $A_{s,2}$, FOR STEM (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

h = 2.0833 ft , H' = -0.0833 ft

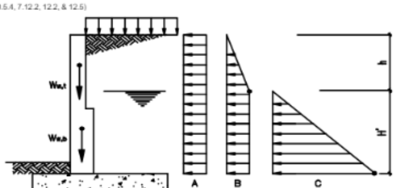
$A = w_s P_a / \gamma_b$ =	118	plf
$B = h P_a$ =	123	plf
$C = [P_a (\gamma_{sat} - \gamma_w) / \gamma_b + \gamma_w]\ H'$ =	-2	plf

At base of top stem

M_u =	0.35	ft-kips
V_u =	0.31	kips
P_u =	42.88	kips

At base of bottom stem

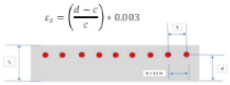
M_u =	0.35	ft-kips
V_u =	0.31	kips
P_u =	42.88	kips



$$\phi M_n = \phi\left[A_s f_y \left(d - \frac{A_s f_y - P_u}{1.7 b f_c'}\right)\right]$$

$$a = \frac{A_s f_y}{0.85 * b * f_c'} \qquad c = \frac{a}{\beta_1} \qquad \text{where}$$

$$\varepsilon_s = \left(\frac{d-c}{c}\right) * 0.003$$

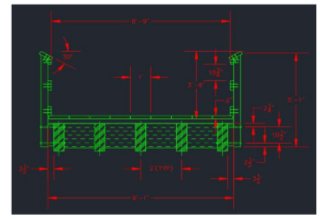


$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f_c'}{f_y}\frac{87}{87+f_y}\right)$$

$$\rho_{MIN} = 0.0018\frac{t}{d}$$

		At top stem		At base of bottom stem			
ϕM_n	=	13.75 PASS	ft-kips ,	13.75 PASS	ft-kips	>	M_u
a	=	0.58	in ,	0.58	in		
c	=	0.68	in ,	0.68	in		
d	=	10.00	in ,	10.00	in		
ε_s	=	0.041 YIELDED		0.041 YIELDED			
b	=	12	in ,	12	in		
ϕ	=	0.9	,	0.9			
A_s	=	0.44	in^2 ,	0.44	in^2		
ρ	=	0.003		0.003			
β_1	=	0.85		0.85			
ρ_{MAX}	=	0.028 PASS		0.028 PASS		>	ρ
ρ_{MIN}	=	0.002 PASS		0.002 PASS		<	ρ

**raising bridge up for constructability, grade to bridge deck
**small foundation needed
**recommend leaving this bridge as is



Act. Earth Press. Coeff:	K_a =	$\tan^2(45-(\phi/2))$ =	K_a :	
Active Earth Pressure:	P_a =	$K_a * \gamma_{soil}$ =	<1	K_a = D/B
Pass. Earth Press. Coeff:	K_p =	$\tan^2(45+(\phi/2))$ =	>1	$K_a = \tan^{-1}(D/B)$
Passive Earth Pressure:	P_p =	$K_p * \gamma_{soil}$ =		

TABLE 1806.2 PRESUMPTIVE LOAD-BEARING VALUES

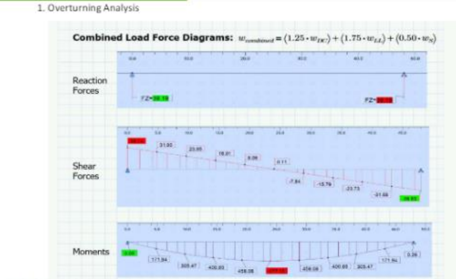
CLASS OF MATERIALS	VERTICAL FOUNDATION PRESSURE (psf)	LATERAL BEARING PRESSURE (psf/ft below natural grade)	LATERAL SLIDING RESISTANCE	
			Coefficient of friction[a]	Cohesion (psf)[b]
1. Crystalline bedrock	12,000	1,200	0.70	—
2. Sedimentary and foliated rock	4,000	400	0.35	—
3. Sandy gravel and gravel (GW and GP)	3,000	200	0.35	—
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC)	2,000	150	0.25	—
5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)	1,500	100	—	130

For SI: 1 pound per square foot = 0.0479kPa, 1 pound per square foot per foot = 0.157 kPa/m.
a. Coefficient to be multiplied by the dead load.
b. Cohesion value to be multiplied by the contact area, as limited by Section 1806.3.2.

Table 5.17. Typical compacted densities and optimum moisture contents for USCS soil types (after Carter and Bentley, 1991).

Soil Description	USCS Class	Compacted Dry Unit Weight (lb/ft3)	Compacted Dry Unit Weight (kN/m3)	Optimum Moisture Content (%)
Gravel/sand mixtures:				
well-graded, clean	GW	125-134	19.6-21.1	8-11
poorly-graded, clean	GP	115-125	18.1-19.6	11-14
well-graded, small silt content	GM	119-134	18.6-21.1	8-12
well-graded, small clay content	GC	115-125	18.1-19.6	9-14
Sands and sandy soils:				
well-graded, clean	SW	109-131	17.2-20.6	9-16
poorly-graded, small silt content	SP	94-119	15.7-18.6	12-21
well-graded, small silt content	SM	109-125	17.2-19.6	11-16
well-graded, small clay content	SC	106-125	16.7-19.6	11-19
Fined-grained soils of low plasticity:				
silts	ML	94-119	14.7-18.6	12-24
clays	CL	94-119	14.7-18.6	12-24
organic silts	OL	81-100	12.7-15.7	21-33
Fine-grained soils of high plasticity:				
silts	MH	69-94	10.8-14.7	24-40
clays	CH	81-106	12.7-18.6	19-36
organic clays	OH	66-100	10.3-15.7	21-45

1. Overturning Analysis



1. Overturning Analysis

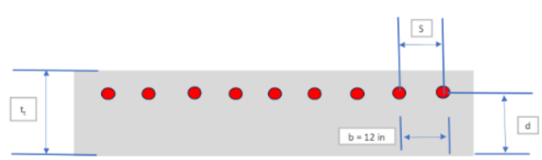
2. Bearing Capacity Analysis

2. Bearing Capacity Analysis

3. Flexure Capacity Analysis

$$M_u = (A*H)(0.5*H) + (B*H')(0.5*H') + (0.5*B*h)\left(H' + \frac{1}{3}h\right) + (0.5*C*H')\left(\frac{1}{3}H'\right)$$

$$V_u = (A*H) + (B*H') + (0.5*B*h) + (0.5*C*H')$$



$\beta_1 = 0.85$	for $0 < f'_c \le 4000$ psi
$\beta_1 = 0.85 - 0.05(f'_c - 4000)/1000$	for 4000 psi $< f'_c \le 8000$ psi
$\beta_1 = 0.65$	for $f'_c > 8000$ psi

3. Flexure Capacity Analysis

4. Shear Capacity Analysis

CHECK SHEAR CAPACITY FOR STEM (ACI 318-02 SEC.15.5.2, 11.1.3.1, & 11.3)

	At top stem		At base of bottom stem			
$V_{allowable} = 2\phi bd\sqrt{f'_c}$ =	9.86	kips ,	9.86	kips	>	V_u
	PASS		PASS			

where ϕ = 0.75 (ACI 318-02, Section 9.3.2.3) ϕ = 0.75

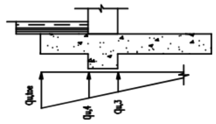
4. Shear Capacity Analysis

5. Heel Flexure Capacity Analysis

CHECK HEEL FLEXURE CAPACITY $A_{s,3}$ FOR FOOTING (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f'_c}{f_y}\frac{87}{87+f_y}\right) = 0.0278 \qquad \rho_{MIN} = \frac{0.0018}{2}\frac{h_f}{d} = 0.0054$$

$$M_{u,3} = \begin{cases} \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{(q_{u,3}+2q_{u,heel})bL_H^2}{6}, & for \quad e_u \le \frac{L}{6} \\ \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{q_{u,3}bS^2}{6}, & for \quad e_u > \frac{L}{6} \end{cases} = 1.28 \text{ ft-kips}$$



$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\frac{M_{u,3}}{0.383bd^2 f'_c}}\right)}{f_y} = 0.0096$$

PASS PASS

where							
d	=	2.00	in	$q_{u,toe}$	=	0.95	ksf
b	=	12.00	in	$q_{u,heel}$	=	0.59	ksf
e_u	=	0.39	ft	$q_{u,3}$	=	0.77	ksf
S	=	16.00	in	ϕ	=	0.9	

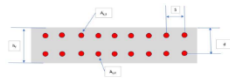
$$q_{u,toe} = \frac{P}{BL}\left(1+\frac{6e}{B}\right)$$

$$q_{u,heel} = \frac{P}{BL}\left(1-\frac{6e}{B}\right)$$

$$\phi M_n = \phi\left[A_S f_y\left(d - \frac{A_S f_y - P_u}{1.7bf'_c}\right)\right] = 1.32 \text{ ft-kips}$$

$(A_{S,3})_{required}$ = 0.17 in²/ft < $A_{S,3}$ = 0.31 in²/ft

PASS



5. Heel Flexure Capacity Analysis

6. Toe Flexure Capacity Analysis

CHECK TOE FLEXURE CAPACITY $A_{s,4}$ FOR FOOTING (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f'_c}{f_y}\frac{87}{87+f_y}\right) = 0.028 \qquad \rho_{MIN} = MIN\left(\frac{4}{3}\rho,\ \frac{0.0018}{2}\frac{h_f}{d}\right) = 0.0005$$

$$M_{u,4} = \frac{(q_{u,4}+2q_{u,toe})bL_T^2}{6} - \frac{L_T^2}{2L}\gamma w_f = 1.47 \text{ ft-kips}$$

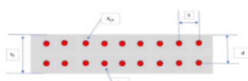
where			
d	=	10.00	in
$q_{u,4}$	=	0.83	ksf
b	=	12.00	in
A_s	=	0.31	in²
ϕ	=	0.9	
ρ	=	0.0022	
		PASS	PASS

$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\frac{M_{u,4}}{0.383bd^2 f'_c}}\right)}{f_y} = 0.0004$$

$$\phi M_n = \phi\left[A_S f_y\left(d - \frac{A_S f_y - P_u}{1.7bf'_c}\right)\right] = 1.60 \text{ ft-kips}$$

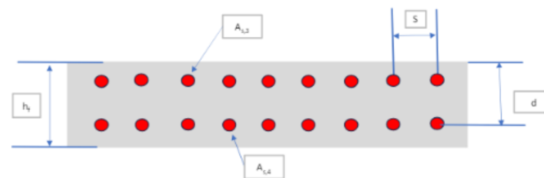
$(A_{S,4})_{required}$ = 0.05 in²/ft < $A_{S,4}$ = 0.31 in²/ft

PASS



toe - tension bottom

heel - tension top



6. Toe Flexure Capacity Analysis

7. Key Capacity Analysis

CHECK KEY CAPACITY FOR FOOTING

1.5 (H_b + H_s) = 0.90 kips < H_b + μ ΣW = 11.10 kips

PASS

Techincal References:

1. Alan Williams: "Structural Engineering Reference Manual", Professional Publications, Inc, 2001.
2. Alan Williams: "Structural Engineering License Review Problems and Solutions", Oxford University Press, 2003.

Reference 17: Bridge 3 Abutment Calculations without Water

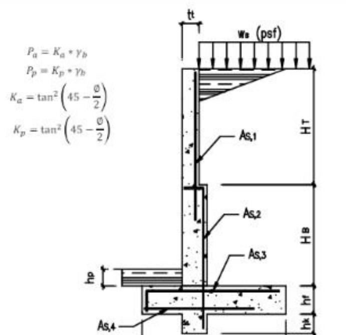
Retaining Wall Design Based on ACI 318-02

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	3	ksi			
REBAR YIELD STRESS	f_y	=	40	ksi			
LATER SOIL PRESSURE	P_a	=	58.83	pcf (equivalent fluid pressure)			
PASSIVE PRESSURE	P_p	=	244.75	pcf			
ACTIVE EARTH PRESSURE COEFF.	K_a	=	0.490				
PASSIVE EARTH PRESSURE COEFF.	K_p	=	2.040				
FRICTION ANGLE	ϕ	=	20	deg			
SURCHARGE WEIGHT	w_s	=	240	psf			
FRICTION COEFFICIENT	μ	=	0.3				
ALLOW SOIL PRESSURE	Q_a	=	1.5	ksf	IBC Table 1806.2		
THICKNESS OF TOP STEM	t_t	=	12	in			
THICKNESS OF KEY & STEM	t_b	=	12	in			
TOE WIDTH	L_T	=	2	ft			
HEEL WIDTH	L_H	=	3	ft			
HEIGHT OF TOP STEM	H_T	=	1	ft			
HEIGHT OF BOT. STEM	H_B	=	1	ft			
FOOTING THICKNESS	h_f	=	12	in			
KEY DEPTH	h_k	=	0	in			
SOIL OVER TOE	h_p	=	47.00	in			
	y	=	2	ft			
	L	=	6	ft			
	B	=	10	ft			
UNIT WEIGHT OF SOIL	γ_b	=	120	pcf			
UNIT WEIGHT OF CONCRETE	γ_c	=	145	pcf			

TOP STEM REINF. ($A_{s,1}$)	1	#	6	@	16	in o.c., at middle	A_{bar} =	0.44	in^2
BOT. STEM REINF. ($A_{s,2}$)	2	#	6	@	16	in o.c., at each face	A_{bar} =	0.44	in^2
TOP REINF.OF FOOTING ($A_{s,3}$)		#	5	@	16	in	A_{bar} =	0.31	in^2
BOT. REINF.OF FOOTING ($A_{s,4}$)		#	5	@	16	in	A_{bar} =	0.31	in^2

$$P_a = K_a * \gamma_b$$
$$P_p = K_p * \gamma_b$$
$$K_a = \tan^2\left(45 - \frac{\emptyset}{2}\right)$$
$$K_p = \tan^2\left(45 - \frac{\emptyset}{2}\right)$$



Act. Earth Press. Coeff: $K_a = \tan^2(45-(\phi/2)) =$

Active Earth Pressure: $P_a = K_a{}^*\gamma_{soil} =$

Pass. Earth Press. Coeff: $K_p = \tan^2(45+(\phi/2)) =$

Passive Earth Pressure: $P_p = K_p{}^*\gamma_{soil} =$

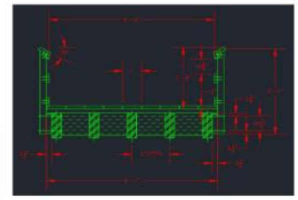


TABLE 1806.2 PRESUMPTIVE LOAD-BEARING VALUES

CLASS OF MATERIALS	VERTICAL FOUNDATION PRESSURE (psf)	LATERAL BEARING PRESSURE (psf/ft below natural grade)	LATERAL SLIDING RESISTANCE: Coefficient of friction[a]	LATERAL SLIDING RESISTANCE: Cohesion (psf)[b]
1. Crystalline bedrock	12,000	1,200	0.70	—
2. Sedimentary and foliated rock	4,000	400	0.35	—
3. Sandy gravel and gravel (GW and GP)	3,000	200	0.35	—
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC)	2,000	150	0.25	—
5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)	1,500	100	—	130

For SI: 1 pound per square foot = 0.0479kPa, 1 pound per square foot per foot = 0.157 kPa/m.
a. Coefficient to be multiplied by the dead load.
b. Cohesion value to be multiplied by the contact area, as limited by Section 1806.3.2.

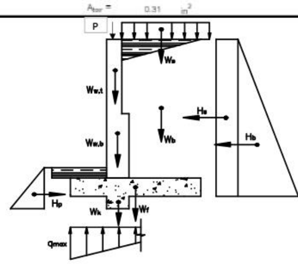
Table 5.17. Typical compacted densities and optimum moisture contents for USCS soil types (after Carter and Bentley, 1991).

Soil Description	USCS Class	Compacted Dry Unit Weight (lb/ft3)	Compacted Dry Unit Weight (kN/m3)	Optimum Moisture Content (%)
Gravel/sand mixtures				
well-graded, clean	GW	125-134	19.6-21.1	8-11
poorly-graded, clean	GP	115-125	18.1-19.6	11-14
well-graded, small silt content	GM	119-134	18.6-21.1	8-12
well-graded, small clay content	GC	115-125	18.1-19.6	9-14
Sands and sandy soils				
well-graded, clean	SW	109-131	17.2-20.6	9-16
poorly graded, small silt content	SP	94-115	16.7-18.6	12-21
well-graded, small silt content	SM	109-125	17.2-19.6	11-16
well-graded, small clay content	SC	106-125	16.7-19.6	11-19
Fine-grained soils of low plasticity				
silts	ML	94-119	14.7-18.6	12-24
clays	CL	94-119	14.7-18.6	12-24
organic silts	OL	81-100	12.7-15.7	21-33
Fine-grained soils of high plasticity				
silts	MH	69-94	10.8-14.7	24-40
clays	CH	81-106	12.7-16.6	19-36
organic clays	OH	66-100	10.3-15.7	21-45

ANALYSIS

SERVICE LOADS

$H_b = 0.5 P_a (H_T + H_B + h_f)^2$	=	0.26	kips
$H_s = w_s P_a (H_T + H_B + h_f) / \gamma_b$	=	0.35	kips
$H_p = 0.5 P_p (h_p + h_f + h_k)^2$	=	2.96	kips
$W_s = w_s (L_H + t_b - t_t)$	=	0.72	kips
$W_b = [H_T (L_H + t_b - t_t) + H_B L_H] \gamma_b$	=	0.72	kips
$W_f = h_f (L_H + t_b + L_T) \gamma_c$	=	0.87	kips
$W_k = h_k\ t_b\ \gamma_c$	=	0.00	kips
$W_{w,t} = t_t\ H_T\ \gamma_c$	=	0.15	kips
$W_{w,b} = t_b\ H_B\ \gamma_c$	=	0.15	kips
$P = R_{comb} + R_{H5}$	=	24.31	kips



FACTORED LOADS

$\gamma H_b = 1.6 H_b$	=	0.42	kips
$\gamma H_s = 1.6 H_s$	=	0.56	kips
$\gamma W_s = 1.6 W_s$	=	1.15	kips
$\gamma W_b = 1.2 W_b$	=	0.86	kips
$\gamma W_f = 1.2 W_f$	=	1.04	kips
$\gamma W_k = 1.2 W_k$	=	0.00	kips
$\gamma W_{w,t} = 1.2 W_{w,t}$	=	0.17	kips
$\gamma W_{w,b} = 1.2 W_{w,b}$	=	0.17	kips
$\gamma P = 1.75 P$	=	42.53	kips

OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_b	0.26	0.42	1.00	0.26	0.42
H_s	0.35	0.56	1.50	0.53	0.85
Σ	0.62	0.99		**0.79**	1.27
	(kip)	(kip)	(ft)	(kip-ft)	(kip-ft)

RESISTING MOMENT

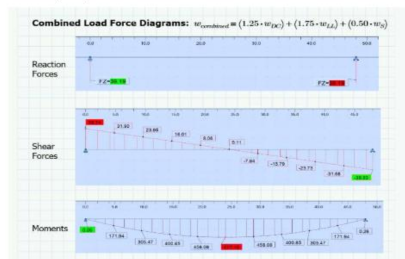
	W	γW	x	W x	γW x
W_s	0.72	1.15	4.50	3.24	5.18
W_b	0.72	0.86	4.50	3.24	3.89
W_f	0.87	1.04	3.00	2.61	3.13
W_k	0.00	0.00	2.50	0.00	0.00
$W_{w,t}$	0.15	0.17	2.50	0.36	0.44
$W_{w,b}$	0.15	0.17	2.50	0.36	0.44
P	24.31	42.53	2.50	60.76	106.33
Σ	26.91	45.94		**70.58**	119.41
	(kip)	(kip)	(ft)	(kip-ft)	(kip-ft)

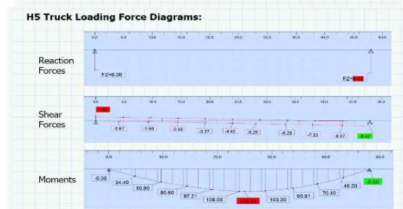
OVERTURNING FACTOR OF SAFETY

$$SF = \frac{\Sigma Wx}{\Sigma Hy} = 88.86 > 1.5$$

PASS

1. Overturning Analysis





1. Overturning Analysis

2. Bearing Capacity Analysis

CHECK SOIL BEARING CAPACITY (ACI 318-02 SEC.15.2.2)

$$L = L_T + t_b + L_H = 6.00 \text{ ft}$$

$$e = \frac{L}{2} - \frac{\Sigma Wx - \Sigma Hy}{\Sigma W} = 0.41 \text{ ft}$$

$$q_{MAX} = \begin{cases} \dfrac{\Sigma W\left(1+\frac{6e}{L}\right)}{BL}, & for\ \ e \le \frac{L}{6} \\ \dfrac{\Sigma W}{3B(0.5L-e)}, & for\ \ e > \frac{L}{6} \end{cases} = 0.63 \text{ ksf} < Q_a = 1.5 \text{ ksf}$$

PASS

2. Bearing Capacity Analysis

3. Flexure Capacity Analysis

CHECK FLEXURE CAPACITY, $A_{S,1}$ & $A_{S,2}$, FOR STEM (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

$$M_u = \left(\frac{w_s P_a}{\gamma_b} * H\right)(0.5 * H) + (0.5 * Pa * H^2)\left(\frac{1}{3} * H\right) = 0.31 \text{ ft-kips}$$

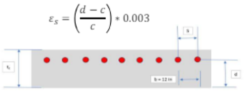
P_u = 42.88 kips

$$\phi M_n = \phi\left[A_S f_y\left(d - \frac{A_S f_y - P_u}{1.7 b f_c'}\right)\right]$$

At top stem = 13.75 ft-kips, PASS; At base of bottom stem = 13.75 ft-kips, PASS; > M_u

$$a = \frac{A_s f_y}{0.85 * b * f_c'} \qquad c = \frac{a}{\beta_1} \qquad \text{where}$$

$$\varepsilon_s = \left(\frac{d-c}{c}\right) * 0.003$$



		At top stem		At base of bottom stem	
a	=	0.58	in ,	0.58	in
c	=	0.68	in ,	0.68	in
d	=	10.00	in ,	10.00	in
ε_s	=	0.041		0.041	
		YIELDED		YIELDED	
b	=	12	in ,	12	in
ϕ	=	0.9	,	0.9	
A_s	=	0.44	in^2 ,	0.44	in^2
ρ	=	0.003		0.003	
β_1	=	0.85		0.85	

$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f_c'}{f_y}\frac{87}{87+f_y}\right) = 0.028 \text{ (PASS)},\ 0.028 \text{ (PASS)} > \rho$$

$$\rho_{MIN} = 0.0018\frac{t}{d} = 0.002 \text{ (PASS)},\ 0.002 \text{ (PASS)} < \rho$$

β_1 = 0.85		for $0 < f'_c \le 4000$ psi
β_1 = 0.85 – 0.05(f'_c – 4000)/1000		for 4000 psi < $f'_c \le 8000$ psi
β_1 = 0.65		for $f'_c > 8000$ psi

$$M_u = \gamma\left(\frac{P_a y^3}{6} + \frac{P_a y^2 w_s}{2\gamma_b}\right)$$

$$P_u = \gamma W_w$$

$$V = \gamma\left(\frac{P_a y^2}{2} + \frac{w_s P_a y}{\gamma_b}\right)$$

3. Flexure Capacity Analysis

CHECK SHEAR CAPACITY FOR STEM (ACI 318-02 SEC.15.5.2, 11.1.3.1, & 11.3)

4. Shear Capacity Analysis

		At top stem		At base of bottom stem			
$V_u = \left(\frac{w_s P_a}{\gamma_b} * H\right) + (0.5 * Pa * H^2)$	=	0.35	kips ,	0.35	kips		
$V_{allowable} = 2\phi b d\sqrt{f'_c}$	=	9.86 PASS	kips ,	9.86 PASS	kips	>	V

where ϕ = 0.75 (ACI 318-02, Section 9.3.2.3) ϕ = 0.75

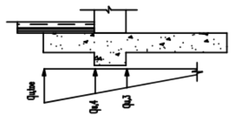
4. Shear Capacity Analysis

CHECK HEEL FLEXURE CAPACITY, $A_{s,3}$, FOR FOOTING (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

5. Heel Flexure Capacity Analysis

$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f'_c}{f_y}\frac{87}{87+f_y}\right) = 0.0278 \qquad \rho_{MIN} = \frac{0.0018}{2}\frac{h_f}{d} = 0.0054$$

$$M_{u,3} = \begin{cases} \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{(q_{u,3} + 2q_{u,heel})bL_H^2}{6}, & for \quad e_u \le \frac{L}{6} \\ \frac{L_H}{2}\left(\gamma w_s + \gamma w_b + \frac{L_H}{L}\gamma w_f\right) - \frac{q_{u,3}bS^2}{6}, & for \quad e_u > \frac{L}{6} \end{cases} = 0.921 \text{ ft-kips}$$



$$\rho = \frac{0.85 f'_c\left(1 - \sqrt{1 - \frac{M_{u,3}}{0.383bd^2 f'_c}}\right)}{f_y} = 0.0067$$

PASS PASS

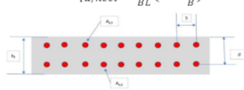
where							
d_{heel}	=	2.00	in	$q_{u,toe}$	=	0.95	ksf
b	=	12.00	in	$q_{u,heel}$	=	0.58	ksf
e_u	=	0.41	ft	$q_{u,3}$	=	0.77	ksf
S	=	16.00	ft	ϕ	=	0.9	

$$q_{u,toe} = \frac{P}{BL}\left(1 + \frac{6e}{B}\right)$$

$$q_{u,heel} = \frac{P}{BL}\left(1 - \frac{6e}{B}\right)$$

$$\phi M_n = \phi\left[A_S f_y\left(d - \frac{A_S f_y - P_u}{1.7bf'_c}\right)\right] = 0.94 \text{ ft-kips}$$

$(A_{S,3})_{required}$ = 0.12 in^2 / ft PASS < $A_{S,3}$ = 0.31 in^2 / ft



5. Heel Flexure Capacity Analysis

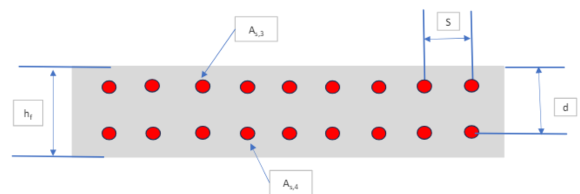
CHECK TOE FLEXURE CAPACITY, $A_{s,4}$, FOR FOOTING (ACI 318-02 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

6. Toe Flexure Capacity Analysis

$$\rho_{MAX} = 0.75\left(\frac{0.85\beta_1 f'_c}{f_y}\frac{87}{87+f_y}\right) = 0.0278 \qquad \rho_{MIN} = MIN\left(\frac{4}{3}\rho, \ \frac{0.0018}{2}\frac{h_f}{d}\right) = 0.0005$$

$$M_{u,4} = \frac{(q_{u,4} + 2q_{u,toe})bL_T^2}{6} - \frac{L_T^2}{2L}\gamma w_f = 1.47 \text{ ft-kips}$$

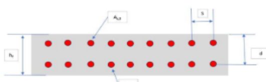
where			
d_{toe}	=	10.00	in
$q_{u,4}$	=	0.83	ksf
b	=	12.00	in
A_s	=	0.31	in^2
ϕ	=	0.9	
ρ	=	0.0022	
		PASS	PASS



$$\rho = \frac{0.85 f'_c\left(1 - \sqrt{1 - \frac{M_{u,4}}{0.383bd^2 f'_c}}\right)}{f_y} = 0.0004$$

$$\phi M_n = \phi\left[A_S f_y\left(d - \frac{A_S f_y - P_u}{1.7bf'_c}\right)\right] = 1.60 \text{ ft-kips}$$

$(A_{S,4})_{required}$ = 0.05 in^2 / ft PASS < $A_{S,4}$ = 0.31 in^2 / ft



6. Toe Flexure Capacity Analysis

CHECK KEY CAPACITY FOR FOOTING

7. Key Capacity Analysis

1.5 (H_p + H_s) = 0.93 kips PASS < $H_p + \mu \Sigma W$ = 11.03 kips

Techincal References:

1. Alan Williams: "Structuiral Engineering Reference Manual", Professional Publications, Inc, 2001.
2. Alan Williams: "Structuiral Engineering License Review Problems and Solutions", Oxford University Press, 2003.

Appendix E: Roadways

I.M. 3.210
March 15, 2023

AASHTO Guidelines

For Rural Local Roads

These "Guidelines" are a composite of the AASHTO recommendations from Chapter 5 of the Green Book (2018) and the Guidelines for Geometric Design of Low-Volume Roads (2019). The values in the first three columns are based on the Green Book. The values in the last column (Agricultural Access) are based on the Guidelines for Geometric Design of Low-Volume Roads. These guidelines are presented to help in the design of new construction or reconstruction projects on rural local roads. For Federal-aid projects, design values below those shown in this table may be used on a project-by-project basis, provided that a design exception or justification is approved by the Iowa DOT Administering Bureau, as per I.M. 3.260, Design Exception Process.

Design Elements		**All Local Roads**							
Design Volume (ADT)	Green Book reference	Over 2000		2000 – 400		Under 400		Under 400 Agricultural Access (10)	
Terrain (1)		Level	Rolling	Level	Rolling	Level	Rolling	Level	Rolling
Design Speed (mph)	Table 5-1	50	40	50	40	40	30	30	20
Stopping Sight Distance (ft) (2)	Tables 3-1 & 5-3	425	305	425	305	305	200	165	95
Minimum K for Crest/Sag Vertical Curves	Tables 3-35, 3-37, & 5-3	84/96	44/64	84/96	44/64	44/64	19/37	13	5
Minimum Horizontal Curve Radius (ft) (3)	Table 3-7	758	444	758	444	444	214	135	75
Maximum Gradient (%) (4)	Table 5-2	6	10	6	10	7	10	--	--
Traveled Way (ft) (5)	Table 5-5	22	22	22	20	18	18	18	18
Shoulder Width (ft)	Table 5-5	6	6	3	3	2	2	2	2
New Bridge Roadway Width (ft) (6)	Table 5-6	34	34	28	26	22	22	TW+2'	TW+2'
Existing Bridge Roadway Width (ft) (7)	Table 5-7 (2011 Green Book)	28	28	24	24	22	22	UAC	UAC
Foreslope (8)	Page 5-11	2:1*	2:1*	2:1*	2:1*	2:1*	2:1*	UAC*	UAC*
Clear Zone Distance (ft)		See note (9)							

NOTES:

(1) AASHTO "Mountainous" terrain design guides may be used on Federal-aid projects only with Iowa DOT concurrence. Note (1) in the Design Aids Table provides definitions for Level and Rolling.

(2) Stopping Sight Distance is based on level roadways for all situations shown. For downgrades and upgrades, consult Table 3-2 in the Green Book.

(3) Based on a maximum superelevation (e) of 0.08.

(4) a. Short lengths of grade (less than 500 feet) and grades on low-volume rural collectors (<2000 vpd) may be steepened by 2%.
b. No values are shown in the Agricultural Access column because there are no criteria for maximum gradient in the Guidelines for Geometric Design of Low-Volume Roads.

(5) For Design Volumes of 2000 ADT or greater, consider using Traveled Way width of 24ft where substantial truck volumes are present or agricultural equipment frequently uses the road.

(6) a. Where the Approach Roadway Width (Traveled Way plus shoulders) is surfaced, that surface width should be carried across the structure.
b. Minimum clear roadway width for bridges is Traveled way + 2ft (each side) for Design Volumes of under 400 ADT, Traveled way + 3ft (each side) for Design Volumes of 400-2000 ADT, and Approach Roadway Width for Design Volumes over 2000 ADT.
c. For Design Volumes of 2000 ADT or greater, for bridges over 100 feet long, the width may be the Traveled Way plus 6 feet (3 feet on each side).
d. Design Loading shall be at least HL-93.
e. Refer to I.M. 3.230, Traffic Barriers (Guardrail and Bridge Barrier Rail), for information on when to install or upgrade guardrail and/or bridge barrier rail.

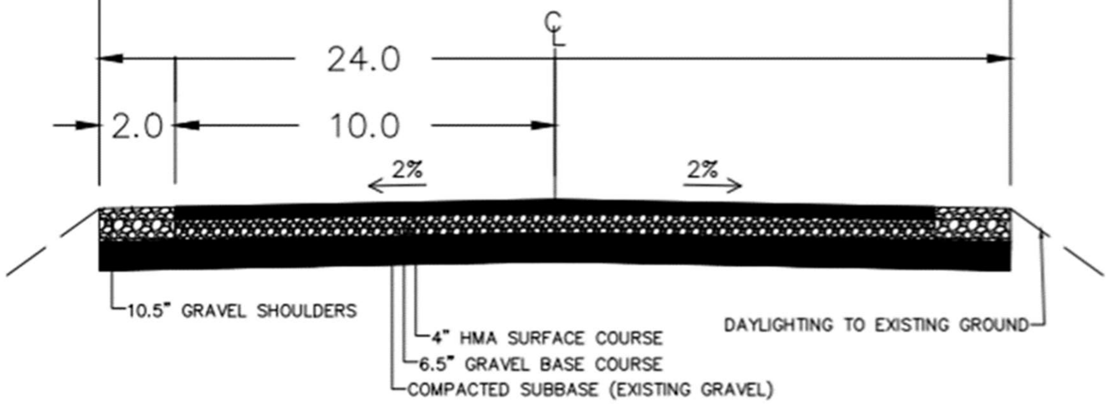
(7) a. Applies to bridges less than 100 feet in length. Bridges over 100 feet will be analyzed individually.
b. Design loading shall be at least HS-20. Refer to I.M. 1.100, Highway Bridge Programs for Cities and Counties for requirements on bridge rehabilitation projects.
c. 20 foot minimum clear roadway width is acceptable for Design Volumes from 0 – 250 ADT.
d. Existing Bridge Roadway Width should be greater than or equal to the Traveled Way width, unless a design exception has been approved.
e. Refer to I.M. 3.230, Traffic Barriers (Guardrail and Bridge Barrier Rail), for information on when to install or upgrade guardrail and/or bridge barrier rail.

(8) * If slopes steeper than 3:1 are used within the recommended clear zone distance, they should be reviewed for shielding with a traffic barrier, as per I.M. 3.240, Clear Zone Guidelines.

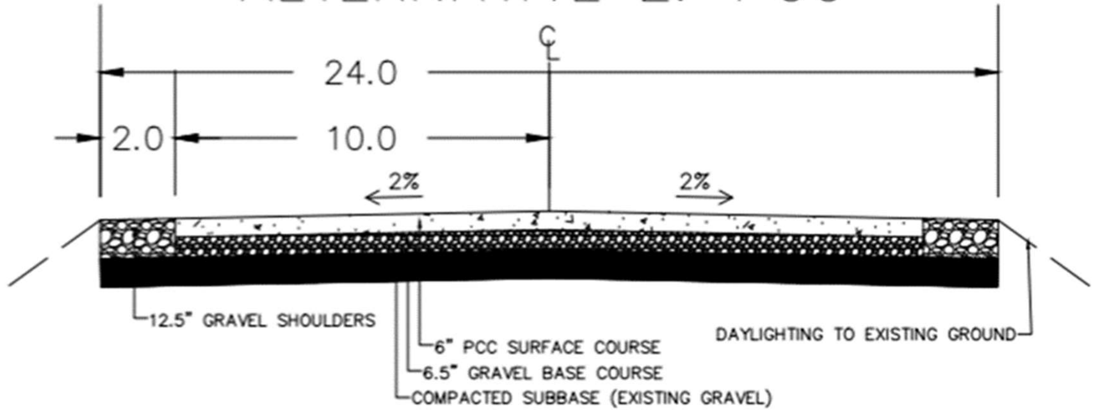
(9) The recommended clear zone distance is a function of Design Speed, Design Volume, horizontal curvature, and roadside geometry. To determine the recommended clear zone distance, refer to I.M. 3.240, Clear Zone Guidelines.

(10) Values in this column are taken from the Guidelines for Geometric Design of Low-Volume Roads unless specified otherwise below.
a. Design Speed is taken from the Green Book using a Design Volume of under 50vpd.
b. While the Guidelines for the Geometric Design of Low-Volume Roads allow for a lesser width, for construction projects, Iowa Code 309.39 states, "…and no traveled roadway shall be less than twenty-two feet from shoulder to shoulder."
** The Guidelines for the Geometric Design of Low-Volume Roads specify a minimum Total Roadway Width (Traveled Way plus shoulders) of 24 feet for the Design Speeds listed.
c. Page 4-6 of the Guidelines for the Geometric Design of Low-Volume Roads states, "Existing bridges can remain in place without widening unless there is evidence of a site-specific crash pattern related to the width of the bridge. However, Iowa Code 309.74 states, "All culverts shall have a clear width of roadway of at least twenty feet. Bridges shall have a clear width of roadway of at least sixteen feet."

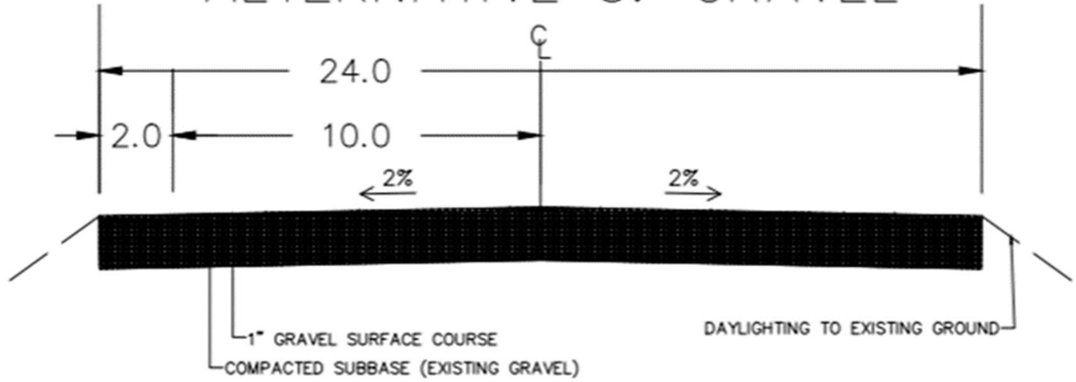
ALTERNATIVE 1: HMA



ALTERNATIVE 2: PCC



ALTERNATIVE 3: GRAVEL



<u>Inputs into Pave Xpress</u>

Asphalt:

Design life: 20 yr

Reliability: 80%

S_0: 0.35

p_i: 4.5

p_t: 2

Δpsi: 2.5

ESAL (http://www.apps.acpa.org/apps/ESAL.aspx): 7538

ADT: 50

% trucks: 2%

Layer coefficient: 0.44

Drainage coefficient: 1

Min thickness: 3 in

Soil type: Varies

CBR: 3

Resilient Modulus (PSI): 5161.17

Base Layer Type: Aggregate

Base modulus: 15000

Base thickness: 6.5 in

Drainage Factor: 1.2

Layer Coefficient: 0.44

<u>Asphalt Thickness: 4in</u>

<u>Aggregate Base Thickness: 6.5”</u>

Concrete:

Design life: 20 yr

Reliability: 80%

S_0: 0.35

p_i: 4.5

p_t: 2

Δpsi: 2.5

Closest City: Des Moines

ESAL (http://www.apps.acpa.org/apps/ESAL.aspx): 7538

ADT: 50

% trucks: 2%

Modulus of Rupture: 800 psi (typical)

Modulus of Elasticity: 4,000,000 psi (typical)

Poisson’s ratio: 0.3 (typical)

Joint spacing: 170 in (typical)

Load Transfer Coefficient: 3 (typical)
Edge Support: 1.01 (typical)
Base Layer Type: Aggregate
Base modulus: 15000
Base thickness: 6.5 in
Drainage Factor: 1.2
Slab Friction Coefficient: 1.4
EFF Modulus of Subgrade Reaction: 2720
PCC Thickness: 6"
Aggregate Base Thickness: 6.5"

Design Criteria Used Based on I.M. 3.210

Lane Width: 10 ft
Shoulder Width: 2 ft
Slopes: 3:1 H:V
SSD: 95 ft
Min R: 75 ft
ROVC Crest K: 5
ROVC Sag K: 5
Min Grade: 0.5%
Max Grade: 12%

Table 4.A: Roadway Cost Estimates Alternative 1 (HMA)

Project:	**White Oaks Nature Conservation Road Design**						
Item	**Item Name (BID TABS)**	**Item Code**	**Unit**	**Unit Cost**	**Quantity**	**Total Cost**	**Rounded Cost**
Excavation - Class 10							
Cut/Fill	EXCAVATION, CLASS 10, ROADWAY AND BORROW	2102-2710070	CY	$ 4.36	1278.75	$ 5,575.35	$ 5,575.00
Backfill Placing	GRANULAR BACKFILL	2402-0425031	TON	$ 35.17	1516.2795	$ 53,327.55	$ 53,500.00
1" Subbase Compaction	COMPACTION WITH MOISTURE AND DENSITY CONTROL	2107-0875000	CY	$ 1.28	245.6844444	$ 314.48	$ 315.00
6.5" Granular Base	GRANULAR SURFACING ON ROAD, CLASS A CRUSHED STONE	2312-8260051	TON	$ 39.91	1889.722852	$ 75,418.84	$ 75,500.00
Pavement							
4" HMA Pavement	HOT MIX ASPHALT MIXTURE, COMMERICAL MIX (INCLUDES ASPHALT BINDER), AS PER PLAN	2303-0000100	TON	$ 144.71	1658.37	$ 239,982.72	$ 240,000.00
10.5" Gravel Shoulders	GRANULAR SHOULDERS, TYPE A	2121-7425010	TON	$ 26.55	610.5258444	$ 16,209.46	$ 16,200.00
4" Top Soil	TOPSOIL, STRIP, SALVAGE AND SPREAD	2105-8425015	CY	$ 6.17	185.6239037	$ 1,145.30	$ 1,150.00
Hydraulic Seeding	HYDRAULIC SEEDING	2601-2636070	ACRE	$ 4,199.05	0.345168416	$ 1,449.38	$ 1,450.00
Pavement Markings	PAINTED PAVEMENT MARKING, WATERBORNE OR SOLVENT-BASED	2527-9263109	STA	$ 21.94	13+30.790741	$ 291.98	$ 290.00
Total Cost							
Option 1 HMA						$ 393,715.05	$ 393,980.00

Table 4.B: Roadway Cost Estimates Alternative 2 (PCC)

Project:	**White Oaks Nature Conservation Road Design**						
Item	**Item Name (BID TABS)**	**Item Code**	**Unit**	**Unit Cost**	**Quantity**	**Total Cost**	**Rounded Cost**
Excavation - Class 10							
Cut/Fill	EXCAVATION, CLASS 10, ROADWAY AND BORROW	2102-2710070	CY	$ 4.36	1277.19	$ 5,568.55	$ 5,575.00
Backfill Placing	GRANULAR BACKFILL	2402-0425031	TON	$ 35.17	1469.3924	$ 51,678.53	$ 51,500.00
1" Subbase Compaction	COMPACTION WITH MOISTURE AND DENSITY CONTROL	2107-0875000	CY	$ 1.28	245.6844444	$ 314.48	$ 315.00
6.5" Granular Base	GRANULAR SURFACING ON ROAD, CLASS A CRUSHED STONE	2312-8260051	TON	$ 39.91	1889.722852	$ 75,418.84	$ 75,500.00
Pavement							
6" PCC Pavement	STANDARD OR SLIP FORM PORTLAND CEMENT CONCRETE PAVEMENT, CLASS S, CLASS 2 DURABILITY, 6 IN.	2301-1032060	SY	$ 49.00	7370.533333	$ 361,156.13	$ 361,000.00
12.5" Gravel Shoulders	GRANULAR SHOULDERS, TYPE A	2121-7425010	TON	$ 26.55	2180.449444	$ 57,890.93	$ 58,000.00
4" Top Soil	TOPSOIL, STRIP, SALVAGE AND SPREAD	2105-8425015	CY	$ 6.17	185.6239037	$ 1,145.30	$ 1,150.00
Hydraulic Seeding	HYDRAULIC SEEDING	2601-2636070	ACRE	$ 4,199.05	0.345168416	$ 1,449.38	$ 1,450.00
Pavement Markings	PAINTED PAVEMENT MARKING, WATERBORNE OR SOLVENT-BASED	2527-9263109	STA	$ 21.94	13+30.790741	$ 291.98	$ 290.00
Total Cost							
Option 2 PCC						$ 554,914.11	$ 554,780.00

Table 4.C: Roadway Cost Estimates Alternative 3 (Gravel)

Project:	**White Oaks Nature Conservation Road Design**						
Item	**Item Name (BID TABS)**	**Item Code**	**Unit**	**Unit Cost**	**Quantity**	**Total Cost**	**Rounded Cost**
Excavation - Class 10							
Cut/Fill	EXCAVATION, CLASS 10, ROADWAY AND BORROW	2102-2710070	CY	$ 4.36	0	$ -	$ -
Backfill Placing	GRANULAR BACKFILL	2402-0425031	TON	$ 35.17	0	$ -	$ -
1" Subbase Compaction	COMPACTION WITH MOISTURE AND DENSITY CONTROL	2107-0875000	CY	$ 1.28	245.6844444	$ 314.48	$ 315.00
Pavement							
1" Gravel Pavement Surfacing	GRANULAR SURFACING ON ROAD, CLASS A CRUSHED STONE	2312-8260051	TON	$ 39.91	290.7265926	$ 11,602.90	$ 11,600.00
1" Gravel Shoulders Surfacing	GRANULAR SHOULDERS, TYPE A	2121-7425010	TON	$ 26.55	58.14531852	$ 1,543.76	$ 1,550.00
4" Top Soil	TOPSOIL, STRIP, SALVAGE AND SPREAD	2105-8425015	CY	$ 6.17	0	$ -	$ -
Hydraulic Seeding	HYDRAULIC SEEDING	2601-2636070	ACRE	$ 4,199.05	0	$ -	$ -
Total Cost							
Option 3 Existing Gravel						$ 13,461.13	$ 13,465.00

Table 4.D: Roadway Cost Estimates Phase 2 (HMA)

Project:	**White Oaks Nature Conservation Road Design**						
Item	**Item Name (BID TABS)**	**Item Code**	**Unit**	**Unit Cost**	**Quantity**	**Total Cost**	**Rounded Cost**
Excavation - Class 10							
Cut/Fill	EXCAVATION, CLASS 10, ROADWAY AND BORROW	2102-2710070	CY	$ 4.36	327.47	$ 1,427.77	$ 1,425.00
Backfill Placing	GRANULAR BACKFILL	2402-0425031	TON	$ 35.17	298.2284	$ 10,488.69	$ 10,500.00
1" Subbase Compaction	COMPACTION WITH MOISTURE AND DENSITY CONTROL	2107-0875000	CY	$ 1.28	72.73703704	$ 93.10	$ 93.00
6.5" Granular Base	GRANULAR SURFACING ON ROAD, CLASS A CRUSHED STONE	2312-8260051	TON	$ 39.91	559.4690432	$ 22,328.41	$ 22,300.00
Pavement							
4" HMA Pavement	HOT MIX ASPHALT MIXTURE, COMMERICAL MIX (INCLUDES ASPHALT BINDER), AS PER PLAN	2303-0000100	TON	$ 144.71	490.975	$ 71,048.99	$ 71,000.00
10.5" Gravel Shoulders	GRANULAR SHOULDERS, TYPE A	2121-7425010	TON	$ 26.55	180.751537	$ 4,798.95	$ 4,800.00
4" Top Soil	TOPSOIL, STRIP, SALVAGE AND SPREAD	2105-8425015	CY	$ 6.17	43.60473086	$ 269.04	$ 270.00
Hydraulic Seeding	HYDRAULIC SEEDING	2601-2636070	ACRE	$ 4,199.05	0.081083177	$ 340.47	$ 340.00
Pavement Markings	PAINTED PAVEMENT MARKING, WATERBORNE OR SOLVENT-BASED	2527-9263109	STA	$ 21.94	9+81.95	$ 21,543.98	$ 21,500.00
Total Cost							
Option 1 HMA						$ 132,339.42	$ 132,228.00

Appendix F: Parking Lots

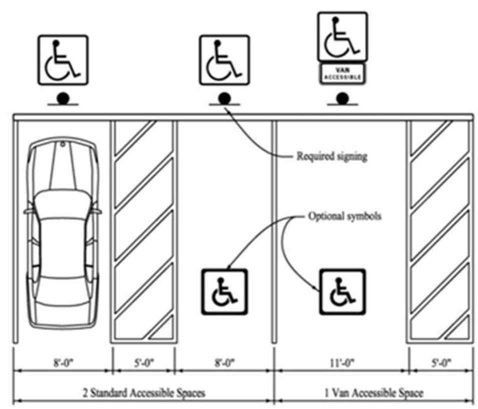
Table 8B-1.02: Minimum Parking Dimensions

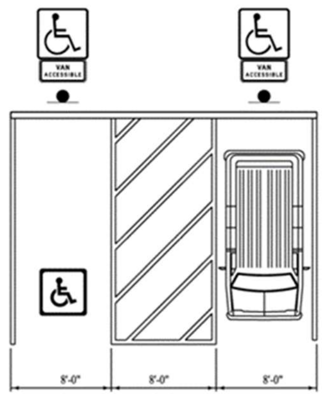
Parking Lot Dimension		Parking Angle (θ)				
		Two-way Aisle			**One-way Aisle**	
		90°	**60°**	**45°**	**60°**	**45°**
Stall Projection	SP	18'-0"	15'-7"	12'-9"	15'-7"	12'-9"
Aisle Width	A	24'-0"	25'-10"	29'-8"	20'-4"	21'-6"
Base Module	M_1	60'-0"	57'-0"	55'-2"	51'-6"	47'-0"
Single Loaded Module	M_2	42'-0"	39'-0"	37'-7"	32'-6"	29'-5"
Wall to Interlock	M_3	60'-0"	55'-10"	52'-2"	49'-4"	44'-0"
Interlock to Interlock	M_4	60'-0"	53'-8"	49'-2"	47'-2"	41'-0"
Overhang	o	2'-6"	2'-2"	1'-9"	2'-2"	1'-9"
Stall Width — *8'-6"* — Width Projection	WP	8'-6"	9'-10"	12'-0"	9'-10"	12'-0"
Stall Width — *8'-6"* — Interlock	i	0'-0"	2'-2"	3'-0"	2'-2"	3'-0"
Stall Width — *9'-0"* — Width Projection	WP	9'-0"	10'-5"	12'-9"	10'-5"	12'-9"
Stall Width — *9'-0"* — Interlock	i	0'-0"	2'-3"	3'-2"	2'-3"	3'-2"

Notes:
1. Aisle width may be increased up to 3 feet to provide a higher level of comfort.
2. In lots where at least 30% of stalls have curbs, aisle width may be reduced by 1'-0".
3. Light poles and columns may protrude a maximum of 2 feet into a parking module as long as they do not encroach on more than 30% of the stalls. When more than 30% of the stalls are encroached, interlock reductions cannot be taken.
4. For additional parking angles, refer to The Dimensions of Parking, ULI, NPA

Source: Adapted from Urban Land Institute, National Parking Association

Figure 8B-1.02: Accessible Space Dimensions





Alternate Van Accessible Parking Dimensions

Table 8B-1.03: Pavement Thickness for Light Loads
(Parking lots with 200 or less cars/day and/or 2 or less trucks/day or equivalent axle loads)

Subgrade CBR	**Surface Material**	**On 12"of Prepared Subgrade**		**On 12" of Prepared Subgrade with 4" Granular Subbase**	
		Minimum	***Desirable***	***Minimum***	***Desirable***
9	Rigid	5"	6"	4"	5"
	Flexible	5"	6"	4"	5"
6	Rigid	5"	6"	4"	5"
	Flexible	5"	6"	4"	5"
3	Rigid	5"	6"	4"	5"
	Flexible	6"	6"	5"	5"

Table 8B-1.04: Pavement Thickness for Moderate Loads
(Parking areas, entrances, perimeter travel lanes, and frontage roads subject to 201 to 700 cars/day and/or 3 to 50 trucks/day or equivalent axle loads)

Subgrade CBR	Surface Material	On 12"of Prepared Subgrade		On 12" of Prepared Subgrade with Granular Subbase		
		Minimum	***Desirable***	***Thickness of Granular Subbase***	***Minimum***	***Desirable***
9	Rigid	5"	6"	4"	4"	5"
	Flexible	5"	6"	6"	4"	5"
6	Rigid	5"	6"	6"	4.5"	5"
	Flexible	6"	6"	8"	5"	5"
3	Rigid	5.5"	6"	6"	5"	5"
	Flexible	6"	7"	8"	6"	6"

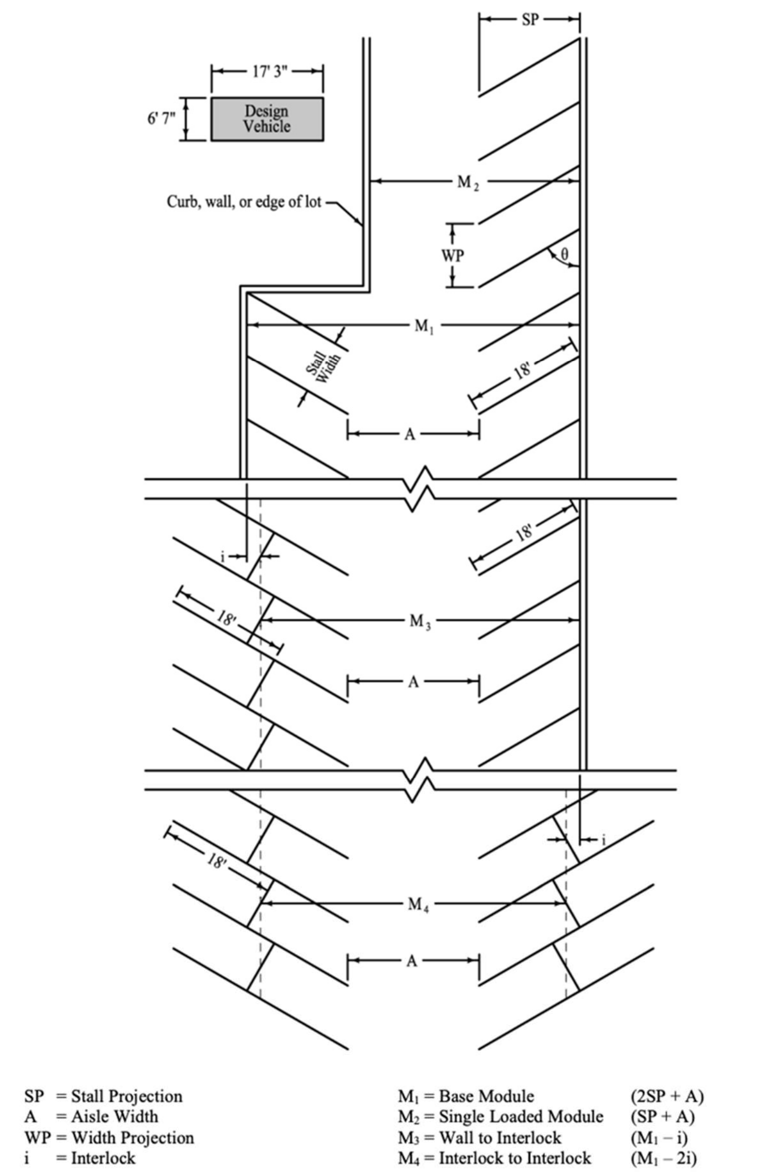
The portions of the parking facility serving truck traffic such as entrances, perimeter travel lanes, trash dumpster sites, and delivery truck routes must be designed to accommodate heavier loads. The number, type, and weight of delivery vehicles can usually be predicted with a fair level of accuracy. With this information, ESAL values and pavement thicknesses can be determined using the methodology described in Chapter 5 - Roadway Design.

If the parking lot is to service an industrial area, such as a truck stop or manufacturing facility, the volume of truck traffic and the associated ESALs should be determined and an independent pavement thickness determination completed to ensure meeting the 20 year design life needs of the project.

Table 8C-1.02: Minimum Accessible Parking Ratios

Total Number of Spaces Provided	Minimum Number of Accessible Spaces
1 to 25	1
26 to 50	2
51 to 75	3
76 to 100	4
101 to 150	5
151 to 200	6
201 to 300	7
301 to 400	8
401 to 500	9
501 to 1,000	2% of total
1,001 and over	20, plus 1 for each 100, or fraction thereof, over 1,000

Figure 8B-1.01: Parking Dimensions



For Pavement Thickness Calculations, See Appendix E

<u>Access (SUDAS Section 8B-1 A)</u>

Width: **Where separate entrances and exits cannot be provided, the driveway to the parking lot should be at least 24 feet wide to provide two 12 foot lanes**

<u>Normal Parking Stalls: (SUDAS Section 8B-1 C)</u>

Recommended Stall Width: **9’**

Recommended Stall Length (Non-Trailers): **18’**

ADA (SUDAS Section 8B-1 D)

ADA Car Minimum Width: **8’**
ADA Van Minimum Width: **11’**
Car Access Aisle Width: **5’, if made 8’ then width of adjacent van spot can be 8’**
Van Access Aisle Width: **5’, if made 8’ then width of adjacent van spot can be 8’**
ADA Lane Lengths: **Use typical 18’**

Parking lot grades: (SUDAS Section 8B-1 E)

Typical: **Slopes of 1.5% should be used to ensure proper drainage and eliminate standing water and icy conditions.**

(Possibly by boat ramp): **Minimum pavement slopes of 0.6% may be used, however since the potential for flat areas is greater, additional measures to address drainage, such as slotted drains or pervious pavement, may be necessary.**

ADA: **Slopes greater than 2% in areas between the parking lot destination and the accessible parking stalls should be avoided as they create a situation where constructing an accessible route is difficult. Slopes greater than 5% are discouraged.**

Pavement: (SUDAS Section 8B-1 F)

Design Life: **20 years**
Pavement Thickness Cars only: **Table 8B-1.03**
Pavement Thickness Trucks and Trailers: **Table 8B-1.04**

Spots: (SUDAS Section 8C-1B)

Spots Required: ???
ADA Car Spots Required: **Table 8C-1.02**
ADA Van Spots Required: **1 per 6 car spots, if only one ADA spot on site make it van**

Table 5.A Parking Lot Cost Estimate Alternative 1 (HMA)

Project	**White Oak Nature Conservation Parking Lot Design**						
Item	**Item Name (BID TABS)**	**Item Code**	**Unit**	**Unit Cost**	**Quantity**	**Total Cost**	**Rounded Cost**
General							
Tree Relocation	TREE, TRANSPLANTING	2610-0000150	EACH	$ 2,500.00	1	$ 2,500.00	$ 2,500.00
Excavation - Class 10							
Cut/Fill	EXCAVATION, CLASS 10, ROADWAY AND BORROW	2102-2710070	CY	$4.36	1914.6215	$8,347.75	$8,300.00
Backfill Placing	GRANULAR BACKFILL	2402-0425031	TON	$35.17	2718.76253	$95,618.88	$95,500.00
1" Subbase Compaction	COMPACTION WITH MOISTURE AND DENSITY CONTROL	2107-0875000	CY	$1.28	135.6282463	$173.60	$175.00
6.5" Granular Base	GRANULAR SURFACING ON ROAD, CLASS A CRUSHED STONE	2312-8260051	TON	$39.91	1251.848713	$49,961.28	$50,000.00
Pavement							
4" HMA Pavement	HOT MIX ASPHALT MIXTURE, COMMERICAL MIX (INCLUDES ASPHALT BINDER), AS PER PLAN	2303-0000100	TON	$144.71	1065.948078	$154,253.35	$154,500.00
4" Top Soil	TOPSOIL, STRIP, SALVAGE AND SPREAD	2105-8425015	CY	$6.17	95.6567716	$590.20	$590.00
Hydraulic Seeding	HYDRAULIC SEEDING	2601-2636070	ACRE	$4,199.05	0.177874532	$746.90	$745.00
Pavement Markings	PAINTED PAVEMENT MARKING, WATERBORNE OR SOLVENT-BASED	2527-9263109	STA	$21.94	3.6	$78.98	$79.00
ADA SYMBOLS	PAINTED SYMBOLS AND LEGENDS, WATERBORNE OR SOLVENT-BASED	2527-9263137	EACH	$ 116.10	3	$ 348.30	$ 350.00
Sidewalk							
6" PCC Sidewalk	SIDEWALK, P.C. CONCRETE, 6 IN.	2511-7526006	SY	$ 97.41	145.0698556	$ 14,131.25	$ 14,100.00
Total Cost							
HMA Parking Lot						$ 326,750.51	$ 326,839.00

Table 5.B Parking Lot Cost Estimate Alternative 2 (PCC)

Project	**White Oak Nature Conservation Parking Lot Design**						
Item	**Item Name (BID TABS)**	**Item Code**	**Unit**	**Unit Cost**	**Quantity**	**Total Cost**	**Rounded Cost**
General							
Tree Relocation	TREE, TRANSPLANTING	2610-0000150	EACH	$ 2,500.00	1	$ 2,500.00	$ 2,500.00
Excavation - Class 10							
Cut/Fill	EXCAVATION, CLASS 10, ROADWAY AND BORROW	2102-2710070	CY	$4.36	1914.6215	$8,347.75	$8,300.00
Backfill Placing	GRANULAR BACKFILL	2402-0425031	TON	$35.17	2718.76253	$95,618.88	$95,500.00
1" Subbase Compaction	COMPACTION WITH MOISTURE AND DENSITY CONTROL	2107-0875000	CY	$1.28	135.6282463	$173.60	$175.00
6.5" Granular Base	GRANULAR SURFACING ON ROAD, CLASS A CRUSHED STONE	2312-8260051	TON	$39.91	1251.848713	$49,961.28	$50,000.00
Pavement							
6" PCC Pavement	STANDARD OR SLIP FORM PORTLAND CEMENT CONCRETE PAVEMENT, CLASS S, CLASS 2 DURABILITY, 6 IN.	2301-1032060	SY	$49.00	4737.547011	$232,139.80	$232,000.00
4" Top Soil	TOPSOIL, STRIP, SALVAGE AND SPREAD	2105-8425015	CY	$6.17	95.6567716	$590.20	$590.00
Hydraulic Seeding	HYDRAULIC SEEDING	2601-2636070	ACRE	$4,199.05	0.177874532	$746.90	$745.00
Pavement Markings	PAINTED PAVEMENT MARKING, WATERBORNE OR SOLVENT-BASED	2527-9263109	STA	$21.94	3.6	$78.98	$79.00
ADA SYMBOLS	PAINTED SYMBOLS AND LEGENDS, WATERBORNE OR SOLVENT-BASED	2527-9263137	EACH	$ 116.10	3	$348.30	$350.00
Sidewalk							
6" PCC Sidewalk	SIDEWALK, P.C. CONCRETE, 6 IN.	2511-7526006	SY	$ 97.41	145.0698556	$ 14,131.25	$ 14,100.00
Total Cost							
PCC Parking Lot						$ 404,636.96	$ 404,339.00

Table 5.C Parking Lot Cost Estimate Alternative 3 (Gravel)

Project	White Oak Nature Conservation Parking Lot Design						
Item	**Item Name (BID TABS)**	**Item Code**	**Unit**	**Unit Cost**	**Quantity**	**Total Cost**	**Rounded Cost**
General							
Tree Relocation	TREE, TRANSPLANTING	2610-0000150	EACH	$ 2,500.00	1	$ 2,500.00	$ 2,500.00
Excavation - Class 10							
Cut/Fill	EXCAVATION, CLASS 10, ROADWAY AND BORROW	2102-2710070	CY	$4.36	1914.6215	$8,347.75	$8,300.00
Backfill Placing	GRANULAR BACKFILL	2402-0425031	TON	$35.17	2718.76253	$95,618.88	$95,500.00
1" Subbase Compaction	COMPACTION WITH MOISTURE AND DENSITY CONTROL	2107-0875000	CY	$1.28	135.6282463	$173.60	$175.00
Pavement							
1" Gravel Pavement Surfacing	GRANULAR SURFACING ON ROAD, CLASS A CRUSHED STONE	2312-8260051	TON	$39.91	257.933115	$10,294.11	$10,300.00
4" Top Soil	TOPSOIL, STRIP, SALVAGE AND SPREAD	2105-8425015	CY	$6.17	95.6567716	$590.20	$590.00
Hydraulic Seeding	HYDRAULIC SEEDING	2601-2636070	ACRE	$4,199.05	0.177874532	$746.90	$745.00
Parking Stops	RUBBER PARKING STOPS (https://www.uline.com/BL_1062/Parking-Stops)	H-4608	EACH	$65.00	22	$1,430.00	$1,425.00
ADA SYMBOLS	PAINTED SYMBOLS AND LEGENDS, WATERBORNE OR SOLVENT-BASED	2527-9263137	EACH	$ 116.10	3	$348.30	$350.00
Sidewalk							
1" Gravel Sidewalk	GRANULAR SURFACING ON ROAD, CLASS A CRUSHED STONE	2312-8260051	TON	$ 39.91	5.722199858	$ 228.37	$ 230.00
Total Cost							
Gravel Parking Lot						$ 120,278.12	$ 120,115.00

Table 5.D Parking Lot Cost Estimate Phase 2 (HMA)

Project	White Oak Nature Conservation Parking Lot Design						
Item	**Item Name (BID TABS)**	**Item Code**	**Unit**	**Unit Cost**	**Quantity**	**Total Cost**	**Rounded Cost**
General							
Tree Relocation	TREE, TRANSPLANTING	2610-0000150	EACH	$ 2,500.00	0	$ -	$ -
Excavation - Class 10							
Cut/Fill	EXCAVATION, CLASS 10, ROADWAY AND BORROW	2102-2710070	CY	$4.36	590.9359	$ 2,576.48	$ 2,575.00
Backfill Placing	GRANULAR BACKFILL	2402-0425031	TON	$35.17	839.128978	$ 29,512.17	$ 29,500.00
1" Subbase Compaction	COMPACTION WITH MOISTURE AND DENSITY CONTROL	2107-0875000	CY	$1.28	49.24465833	$ 63.03	$ 63.00
6.5" Granular Base	GRANULAR SURFACING ON ROAD, CLASS A CRUSHED STONE	2312-8260051	TON	$39.91	454.5281964	$ 18,140.22	$ 18,100.00
Pavement							
4" HMA Pavement	HOT MIX ASPHALT MIXTURE, COMMERICAL MIX (INCLUDES ASPHALT BINDER), AS PER PLAN	2303-0000100	TON	$144.71	398.8817325	$ 57,722.18	$ 57,500.00
4" Top Soil	TOPSOIL, STRIP, SALVAGE AND SPREAD	2105-8425015	CY	$6.17	0	$ -	$ -
Hydraulic Seeding	HYDRAULIC SEEDING	2601-2636070	ACRE	$4,199.05	0	$ -	$ -
Pavement Markings	PAINTED PAVEMENT MARKING, WATERBORNE OR SOLVENT-BASED	2527-9263109	STA	$21.94	0	$ -	$ -
ADA SYMBOLS	PAINTED SYMBOLS AND LEGENDS, WATERBORNE OR SOLVENT-BASED	2527-9263137	EACH	$ 116.10	0	$ -	$ -
Sidewalk							
6" PCC Sidewalk	SIDEWALK, P.C. CONCRETE, 6 IN.	2511-7526006	SY	$ 97.41	0	$ -	$ -
Total Cost							
HMA Parking Lot						$ 108,014.08	$ 107,738.00

Appendix G: Boat Ramp

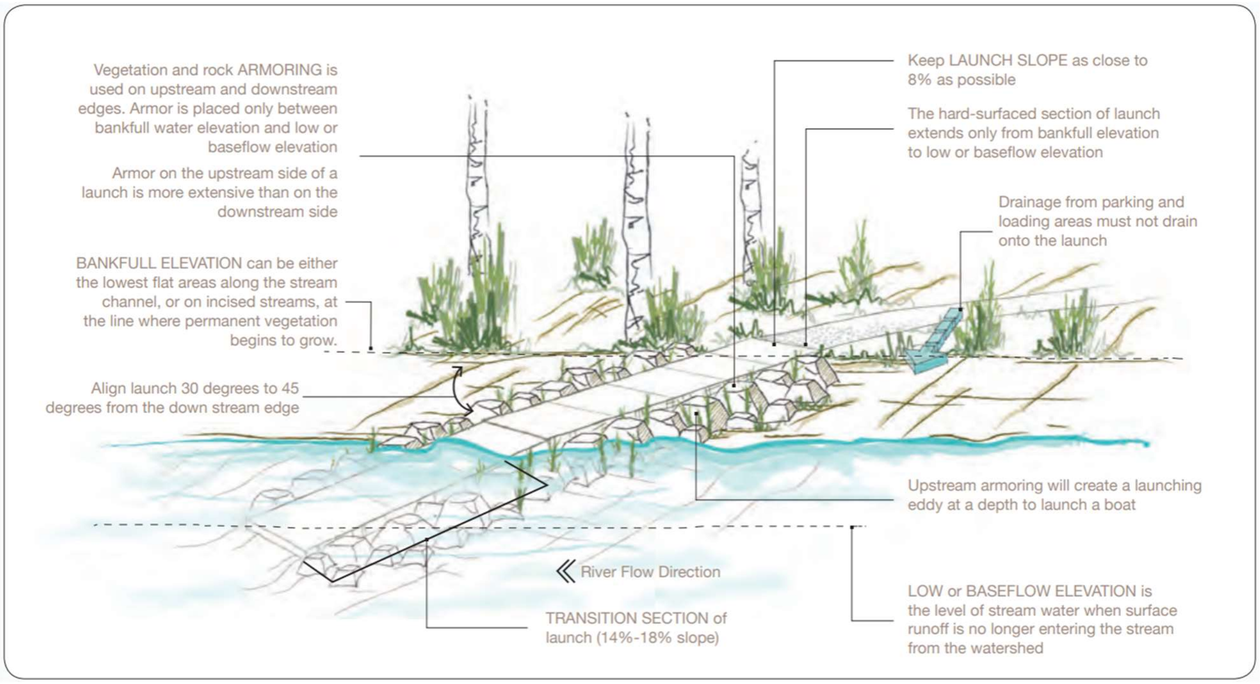


Figure G1: U.S. Army Corps of Engineers Boat Ramp Design Standards

Figure G2: Boat Ramp Plan

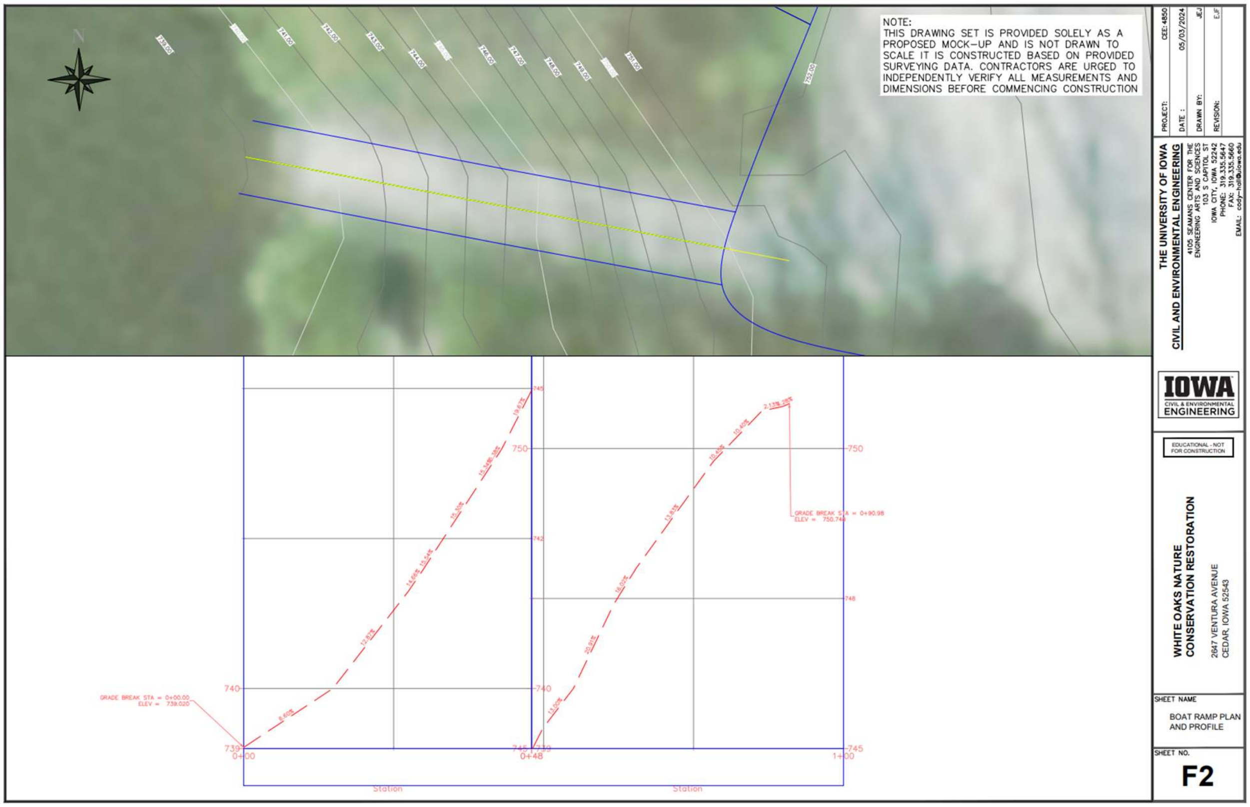


Figure G3: Boat Ramp Plan and Profile

Appendix I: Playgrounds

Reference 1: Prefabricated Concrete Cornhole

CORNHOLE / BAG TOSS

CONCRETE CORNHOLE / BAG TOSS

Size: 55" L x 31" W

Weight: 633 lbs. each, 1,266 lbs. per set

Optional Features

Custom Logos

Bike Deterrent Blocks

Water Resistant Bags

Bag Throwers Area Blocks

Skateboard Deterrent Bars

Reference 2: Prefabricated Concrete Ladder Toss

LADDER TOSS

CONCRETE LADDER TOSS
Regulation Size: 42" H x 32" W, base width 24"
Weight: 280 lbs.

Optional Features
Support Base
Throwers Base *(pictured in photograph)*

STEEL LADDER TOSS
Regulation Size: 42" H x 32" W, base width 24"
Weight: 425 lbs.

Optional Features
Support Base
Throwers Base

Appendix J: Bibliography

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