The University of Iowa | Office of Outreach and Engagement

## FINAL DELIVERABLE

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| **Title** | Redbridge Relocation and Quarry Springs Park Design |
| **Completed By** | Adam Tuchscherer, Bryton Meyer, Damir-Alexandre Von Rohrbach |
| **Date Completed** | December 2018 |
| **UI Department** | Civil and Environmental Engineering |
| **Course Name** | CEE:4850:0001 Senior Design |
| **Instructor** | Paul Hanley |
| **Community Partners** | Colfax, IA |

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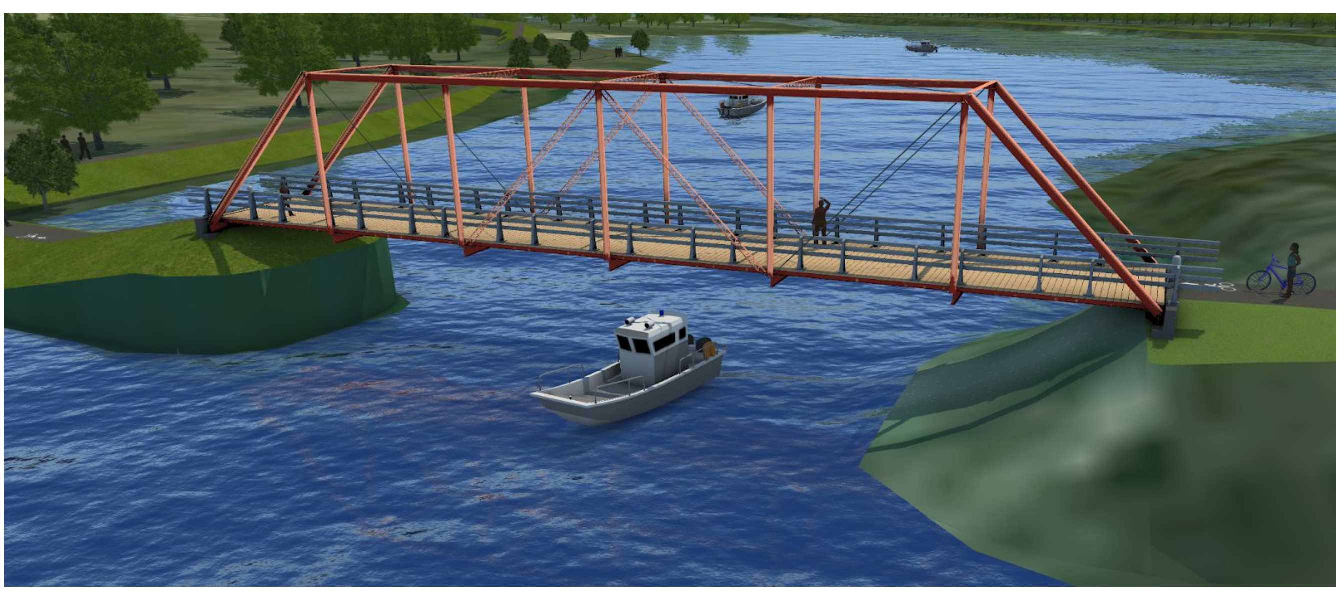
# Red Bridge Relocation and Quarry Springs Park

**DEPARTMENT OF CIVIL & ENVIRONMENTAL ENGINEERING**

**Project Design & Management**

**Hawkeye Engineering Inc.**

**Client: City of Colfax with Jeff Davidson as Liaison**

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# Section I. Executive Summary

## 1. Executive Summary

A team from Hawkeye Engineering Inc., consisting of, University of Iowa students, Damir Alexandre Von Rohrbach, Bryton Meyer, and Adam Tuchscherer, was given the task of preparing the site location within Quarry Springs Park for the relocated Red Bridge. Along with the site prep, the team from the University of Iowa also designed abutments for the relocated Red Bridge, calculated cut and fill quantities for the Quarry Springs pedestrian trail system, designed a sidewalk extension for State Highway 117, designed a trail route from the Red Bridge to the downtown district, provided restoration designs for the relocated Red Bridge, and compiled a cost estimation. The Red Bridge holds significant historical significance to the community of Colfax and many others within Jasper County. The Red Bridge is the last Warren Truss structure in the county. Other Warren Truss structures in Jasper County were either disassembled or damaged beyond repair due to the elements of Iowa's weather. It was imperative that the Red Bridge be relocated and restored before the structure was damaged beyond repair.

Due to rain, a site visit to the Red Bridge was not permissible; however, our team was able to access the Quarry Springs Park and the downtown area. Because the site visit to the Red Bridge did not take place, the Red Bridge was to be assumed structurally sound for the purpose of the University of Iowa's Capstone Design class.

The scope of the project consisted of determining a location for the Warren Truss portion of the current Red Bridge after relocation from its location over the South Skunk River, to a location over a body of water within Quarry Springs Park, North of Colfax. The Pony Truss portion of the current structure was not included in the relocated design due to the community's chosen alternative. At the new location, the structure will be a fully functioning pedestrian and bicycle bridge with illumination features, new abutments, a new deck, and a pedestrian railing system that meets standards AASHTO standards.

Also included in the relocated structure's design is a pedestrian and bicycle trail connecting the Red Bridge, within Quarry Springs Park, to the downtown area, offering convenient access to local businesses. The implementation of this segment of the trail will make it easy for Colfax residents to access the park and for park users to access downtown businesses. A general cross section of the sidewalk trail was generated using Autodesk Civil 3D, along with cut and fill quantities. Potential utility obstructions were also noted.

On the direct path from the downtown district to the park, there is an existing bridge located on State Highway 117. The community requested that the sidewalk of this structure be widened for ease of use for pedestrians and bicyclists. Hawkeye Engineering Inc. has created a sidewalk design for the Highway 117 bridge that offers 10'-1" of travelway for trail users as opposed to the previous 5'-0". The highway bridge will also be equipped with a new railing that will depict the rich history of colfax. Each handrail panel will have custom cut steel panel that showcases the two main industries that Colfax was built on: coal mining and mineral springs. The new State Highway 117 bridge's railing is also in compliance with all AASHTO standards for bike trails. The expansion of the sidewalk was analyzed using Autodesk Robot, and ANSYS, a finite element analysis tool. After much consideration, the expansion design was created that is the most cost effective and easy to instal.

The pedestrian and bicycle trail system within Quarry Springs Park was modeled using Autodesk Civil 3D. The model was generated using the layout of an existing site plan created by the Colfax Parks Board. Using SUDAS standards, a typical cross section of the park's trail system was created. Calculated cut, fill, and material quantities were used to compute a cost estimate for the park's internal trail system. Cut and fill calculations were also based on current contours of the Colfax area provided by the Iowa Department of Natural Resources. Final design calculations should be based on a field survey. A typical cross section of the internal trail system was created for the Park Board's future use. It is recommended that the construction phasing of the Quarry Springs Park trail system be broken into two phases. The first phase includes the updated downtown sidewalk, the State Highway 117 sidewalk expansion, and a trail segment connecting the north side of the 117 bridge to the south side of the Red Bridge at its new location. The first construction phase also includes a segment extending from the north side of the relocated Red Bridge to the existing gravel road within Quarry Springs Park. This will allow pedestrians and bicyclists to use the park, and the trail, while having direct access to downtown.

Aesthetic features within the park and bridge structures were also considered in the Red Bridge Relocation design. The surface area of the Red Bridge was determined to estimate paint and primer quantities. Due to the age of the Red Bridge, and not being able to access the structure, it is assumed that the existing paint on the Red Bridge is lead based. It is recommended that the Red Bridge be transported to a local facility to be sandblasted and painted; however, the structure can be relocated, sandblasted, and painted on-site at additional cost. The cost estimate compiled by Hawkeye Engineering Inc. assumes the structure is blasted and painted off-site and prior to placement at its final location.

Together, all members combined their design and construction costs to determine a total cost estimate. The Red Bridge Relocation and design is estimated to cost the community of Colfax $2,344,610 prior to any grants or donations. This cost estimate includes excavation, fill, and material costs for the first phase of the Quarry Springs Park trail system, the expansion of State Hwy 117 bridge sidewalk, the new Red Bridge abutments, and the updated sidewalk design in the downtown area. It also includes all cutting and grubbing of trees, erosion control devices, paint and primer, and any needed traffic control devices or signage associated with each element of the project. Included in the stated cost estimate s also a $70,000 spot repair contingency. The included cost estimate does not include the second phase of the Quarry Springs Park construction, the transportation of the Red Bridge, and any major structural repairs on the Red Bridge.

## Section II. Organization Qualifications and Experience

### 1. Name of Organization

**Hawkeye Engineering Inc.**

### 2. Organization Location and Contact Information

Structural Engineers in Training:

**Damir Alexandre Von Rohrbach:**

Cell: (941)-451-4262

Email: damiralexandre-vonrohrbach@uiowa.edu

**Adam Tuchscherer:**

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Civil Engineer in Training:

**Bryton Meyer:**

Cell: (563)-380-7266

Email: bryton-meyer@uiowa.edu

Our main office is located at 4105 Seamans Center for the Engineering Arts and Sciences Iowa in Iowa City, IA, 52242. This was the location in which the contract will be managed and communication was organized.

### 3. Organization and Design Team Description

As a group of talented students enrolled in the University of Iowa's Capstone Design class, this team was assigned to the Red Bridge relocation project. Hawkeye Engineering Inc. consists of two Structural Engineering students, Damir Von Rohrbach and Adam Tuchscherer, and a Civil Engineering student, Bryton Meyer.

Damir completed all structural analysis and railing design for the Highway 117 Bridge. Adam completed the design of the trail system from the downtown area to the Red Bridge at its new location, developed a lighting system and designed a railing for the Red Bridge. Adam also located all potential utilities that may affect construction tasks. Bryton designed the Red Bridge abutments, designed a new deck surface for the relocated structure, and estimated all paint and primer estimates and costs. Bryton also modeled the pedestrian trail system within Quarry Springs Park.

Damir and Adam have adequate experience in bridge design, concrete design, and transportation design and  Bryton shows proficiency in bridge design, concrete design, transportation design, and formal bridge inspections. Hawkeye Engineering Inc., was organized based off of similar engineering qualifications. Members of Hawkeye Engineering Inc., all showed proficiency in structural and transportation design strategies. Expertise also includes the inspection of bridges of various sizes, abutment and pier design using Civil 3D and structural bridge analysis using ROBOT and ANSYS.

## Section III. Design Services

### 1. Project Scope

The scope of this project called for a pedestrian trail system within Quarry Springs Park be modeled with cut and fill calculations determined, abutments be designed for the relocated Red Bridge, the sidewalk of the existing State Highway 117 bridge be expanded, a trail segment from downtown colfax to the Red Bridge be designed, and the Red Bridge to be refurbished so it can be a fully functioning pedestrian bridge within Quarry Springs Park. Lighting fixtures for the relocated Red Bridge were also designed and aesthetic features depicting the community's rich history were depicted in the Highway 117 bridge railing.

### 2. Work Plan

Hawkeye Engineering Inc. laid out a work plan that was implemented for the duration of the Red Bridge design project. Firstly, our team identified all permits and required building codes. Next we obtained any necessary dimensions for analysis. Then we began structurally analyzing Highway 117 sidewalk expansion. The relocated Red Bridge abutments were designed and the Quarry Springs Park trail system was then mapped and modeled. Our team then conducted a social and environmental study, focusing on the impact our design had on the community and its surroundings. Once all of the listed tasks were completed, we then completed a cost analysis of each task and found an overall cost estimate of the Red Bridge Relocation project.

# Section IV. Constraints, Challenges and Impacts

## 1. Constraints

There were not many constraints in the design of the relocated Red Bridge. The biggest constraint the project offered was location. During our design process, we proposed three alternatives for the Colfax Park Board to choose from. They picked a location on the South side of Quarry Springs Park, which is our final design location. We had to design for that particular location.

Another constraint was related to the Red Bridge's new design. Because the Red Bridge is listed on the National Register of Historic Places, the materials or methods used to rehab the Red Bridge in the final design has to be original to the bridge. This played a factor in the new deck surface and the railing system meeting all updated codes.

The Red Bridge is a Pin/Rigid-Connected Warren Through Truss, so the structure could not be disassembled and reassembled at a later date. Because as built plans were not available or discovered and a site visit to the Red Bridge was not permissible our team assumed the structure was structurally sound. In doing so, we assumed the structure was stable enough to be lifted by crane and transported to the new site location.

## 2. Challenges

Throughout the course of our work we faced some issues that posed as a challenge. Firstly, we considered the size of Colfax a challenge. Because they are such a small town, we tried to design the most feasible alternative possible. Of the three alternative locations we provided, the option the Colfax Park Board was the most feasible for the community.

The next challenge we faced was the expansion process of the existing Highway 117 bridge. Because the bridge did not meet SUDAS standards for a pedestrian and bike trail width, it needed to be either widened or signage needed to be posted. The community chose to widen the sidewalk for the final design.

Lastly, the original paint finish of the structure posed a construction phasing challenge. Because the structure was constructed prior to 1977, the paint was assumed to be lead-based. Many questions arose such as: when would it be most cost effective to sandblast and re-paint the relocated structure? We concluded that the most cost-effective way to sandblast and paint the bridge, would be to do so during the relocation process, instead of lifting and moving the structure twice or paying more to enclose the structure over the water.

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

Jasper County residents have shown support for the relocation of the Red Bridge to Quarry Springs Park. The relocation of the structure to Quarry Springs Park allows residents and tourists to admire and use the historic bridge on a regular basis. After relocation of the Red Bridge and providing a full functional trail system, we expect the City of Colfax to use the historic bridge as a key access point to Quarry Springs Park. We hope that this project will positively impact the community by allowing a convenient access point to the park and biking or walking trail system. The bridge and its new location will attract non-residents and generate revenue for the City of Colfax. Advertisements for the historic bridge could potentially be made on the I-80 interstate that passes North of Colfax, attracting more people to the town.

Our firm considered the list of alternatives that would improve the quality of life and the community's aesthetic appeal, while considering economic alternatives for the City and County. Preliminary pedestrian/bike trail designs throughout the South end of Quarry Springs Park were analyzed and all provided additional extra curricular activities for the community's enjoyment.

Decreasing the amount of negative environmental impacts in the area was also a consideration during the design process. The use of minimal water pollution practices were used in the abutment design by implementing mesh netting on the banks to prevent soil erosion. A negative environmental impact that this project will cause is construction inconvenience to the residence of Colfax, especially that the Highway 117 bridge connects Colfax to I-80. In order to reduce the delay of traffic, efficient construction and detouring must be implemented, however since Colfax is a small town congestion due to construction should not be a major issue.

## Section V. Alternative Solutions That Were Considered

During our design process, there were many alternatives that were considered in various aspects of the project. The primary alternatives that were considered included: Red Bridge location, girder connection for the Highway 117 sidewalk expansion, and the approach on the North side of the Red Bridge's final location.

There were three location alternatives considered in our initial design. The first was the community's original, desired location on the South side of Quarry Springs Park, spanning the South Skunk River. The second spanned the lake on the South side of the peninsula at the center of the park. The final alternative spanned the lake on the North side of the peninsula. The first alternative required the design of abutments and piers, while the other two only required the design of piers. Constructing piers would have increased construction and material costs significantly. The first alternative would also require the design of two additional spans so the structure was long enough to span the South Skunk River. This too would have increased construction and material costs significantly. The second alternative required the design of abutments, and some excavation and fill on the North side of the structure; however, there would be large amounts of fill at this location. The final alternative also required abutment design, but no pier design. This location also required excavation and fill operations to create enough freeboard on the existing dike to allow kayakers and canoers to travel underneath the structure. The Colfax community decided to proceed with alternative number two.

Two alternative designs were considered for the Highway 117 bridge's sidewalk expansion. The expansion required the design and installation of a new steel girder to support the weight of the extended sidewalk. Hawkeye Engineering Inc. was faced with the challenge of designing a support that would hold the girder in place and connect back to the piers of the bridge. One of the alternatives, was to design a bracket that would attach to the pier by drilling through the pier to the other side. then placing a tensioning cable through the hole and securing it to the bracket. The cable would then be tensioned and the bracket would be held in place through friction. The second alternative was to fabricate two identical steel plates and "sandwich" the pier on each side. These plates would then be further secured by a top plate, which would act as load bearing plate.

On the North side of the Red Bridges relocated location, there were large amounts of fill required for the approach to the North abutment. In our final design, we simply added enough fill at a 5% slope to tie into existing ground with the approach; however a suspended approach slab may also be beneficial to consider before proceeding with construction. If a suspended approach slab is not considered, sheet piles should be considered on the sides of the approach to ensure stability and prevent any washout that may occur.

## Section VI. Final Design Details

The Red Bridge abutment design process began with trying to identify the appropriate lengths for the approach elevation, elevation of bottom of foundation, foundation depth, toe width, heel width, height of footing, and width needed for supports. In order to determine these variables, we created a formula sheet that calculated the overturning factor of safety, sliding factor of safety, and bearing capacity factor of safety. Checks were then performed, using these factors of safety, to determine if the abutment was structurally adequate. We also checked for uplift. Each check was satisfied in our calculations, shown in Appendix A.3. The same variables needed to be checked for the integrated wing wall design. Using the same process, we checked the factor of safeties for overturning, sliding, bearing capacity, and uplift. Such calculations are located in Appendix A.3. The bottom of the deck is placed at the fifty year rainfall event elevation. Next, we calculated the decking quantity for the bridge. From the calculations in Appendix A.7, we were able to get the amount of lumber required for the deck. Finally, a railing for the relocated Red Bridge was designed using AASHTO standards. Because Hawkeye Engineering Inc., was not able to contact the National Register of Historic Places, other railing designs should be considered to satisfy the National Historic Register restoration standards.

Utilizing Civil 3D, we were able to model the trail park system within Quarry Springs Park. Prior to calculating cut and fill quantities for the trail system, the area of clearing and grubbing was calculated. Such calculations can be found in Appendix A.6. Within Civil 3D, we created a cross section for the trail model. The assembly included appropriate subbase and pavement thicknesses, as well as the trail width. We decided to use a concrete surface with a thickness of 6" and a 1.5% cross slope. The model included a subbase thickness of 6". The width of the trail remained constant at 10'. We also have 2' shoulders and a daylight on each side of the trail with a 6:1 slope to existing ground. All of these values were chosen and meet requirements found in SUDAS. After modeling the trail system in Civil 3D, we were able to obtain the cut and fill quantities associated with trail design. We hand calculated the cut and fill quantities for the the abutment approaches on either end of the Red Bridge. These calculations can be located in Appendix A.4. PCC pavement was used as the pavement material so the downtown trail surface would be consistent of that within Quarry Springs Park. Asphalt could be used as an alternative to reduce cost; however, a new cross section design would need to be created.

For the sidewalk design it was a similar process as the trail system. We used Civil 3D again to assemble the sidewalk. The sidewalk was split  up into 3 segments. The first 2 segments of the sidewalk was designed with a width of 8' and thickness of 6" and a slope of 1.5%. These segments also had a 4' buffer to separate vehicle traffic from pedestrian and bike traffic on the sidewalk. The third segment has the same dimensions as the first two segments, but the third segment does not have a buffer. These values were used to meet SUDAS requirements. After that we generated the cut and fill for each segment of the sidewalk which had a net of fill 410 CY. Also any electrical utilities that are in the right of way may need to be moved by the electrical company.

The first step in expanding the State Highway 117 Bridge was finding all the active loads acting on the structure. Using the LRFD Bridge design manual the proper requirements were found. Using ACI and AISC, appropriate design stresses for reinforcing steel, deck concrete, substructure concrete, and structural steel were found. To get a better understanding of what was needed to design the expansion, concept drawings were created. Load calculations acting on the new sidewalk were then calculated. Once the load calculations were calculated, Autodesk Robot was used to calculate shear and bending moment diagrams for a continuous Girder. After the correct load combinations for maximum shear and maximum moment, the deflection was calculated using an unfactored live load per AISC code. Next, a checked was performed to analyze the assumed W24x306 section. The assumption was correct and the assumed section had the required DCR of deflection, flexural strength, shear strength, and yielding. Since the span of the State Highway 117 bridge is 320ft the girder requires splices at the point of inflection. At the splices, moment splice connections were designed. The calculations can be found in Appendix A.2. From there, another check was performed to determine the adequacy of the web and flange connections. Shear studs were designed based on the AIC manual the spacings for the shear studs were 7" and 9" depending on the span between piers, at the 70 ft spans 7" spacings were used using ¾"x8" Nelson shear stud, and for the 90 ft span 9" spacings were used using ¾"x8" Nelson shear stud. The girder needed an extra two feet to connect to the steel pier expansion. A W18X211 was used to match the dimension of the W24X306 girder to increase workability by providing enough space for a connection.

The W18x211 was the placed on a roller/fixed support (depending on the pier or foundation, we have four rollers and one fixed support). The roller then rests on a steel pier expansion bracket that transfers all the dead and live loads acting on the structure into the existing pier. The analysis of the pier was performed using ANSYS finite element software. All of the previous stated calculations can be found in appendix A.2.

## Section VII. Engineer's Cost Estimate

The total construction cost for our project was estimated to be $2,344,610, as referenced in Appendix B.  To obtain this data, reference data from the RS Means Heavy Construction Cost Data handbook. The quantities were obtained from the models and software previously stated.we used the reference data from the RS Means Heavy Construction Cost Data. The final cost estimate includes excavation, fill, and material costs for the first construction phase of the Quarry Springs Park trail, the expansion of State Hwy 117 bridge sidewalk, the new Red Bridge abutments, and the updated sidewalk design in the downtown area. It also includes all cutting and grubbing of trees, erosion control devices, paint and primer, and any needed traffic control devices or signage. A $70,000 spot repair contingency was included in the provided construction cost estimate. The included cost estimate *does not* include the second construction phase of the Quarry Springs Park trail system or the transportation of the Red Bridge.

[ $2,344,610 ]

# Section VIII. References

## 1. References

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Office of Design Design Manual. (n.d.). Retrieved November 28, 2018, from https://iowadot.gov/design/design-manual

*SUDAS Design Manual* [PDF]. (2018).

*The Reinforced Concrete Design Handbook Design Aid* [PDF]. (2015).

## APPENDIX . A Calculations



Excel sheet included at the end for detailed connection design

**Structural Steel Notes:**
Design of structural steel is based on the following:

1. Pedestrian Live Load 85psf based on LRFD Bridge design Specifications.

2. Primary memeber flexural capacity is based on Elastic Section Properties (S) not Plastic Section Properties (Z).

3. Live Load deflection Limit: L/800 (based on LRFD design for vehicle and pedestrian load.)

4.Live load Distribution factor for defelction: equally distributed to all beams.

**Specifications:**
Design: AASHTO LRFD 4th edition, series of 2007 up to 2008.
Construction: Iowa Department of Trasportationstandard specifications for highway and bridge constrcution, series 2009.
Welding: AASHTO/AWS D1.5 as specified and modified by the standard specifications and current suplemental specifications.

**Design Stresses:**
Design stresses for the following meterials are in accordinance with the AASHTO LRFD 4th edition, series 2007 up to 2008.

Reinforcing steel in Accordinance with section 5, Grade 60.

Deck concrete in accordinace with section 5, fc'=3500psi.
substructure concrete in accordinance with section 5, fc'=3500psi

Structural steel in accordinace with section 6, ASTM A709, Grade 50W and Grade 36.

**Steel Notes:**
All structural steel, except as noted, shall confront to ASTM A709 grade 50. the minimum yelid point for grade 50 structural steel is 50ksi for plates 4" and under in thickness, and structural shapes.

All structural steel pieces comprising the abutment and pier bearings shall comply with the requirements as states in the notes.

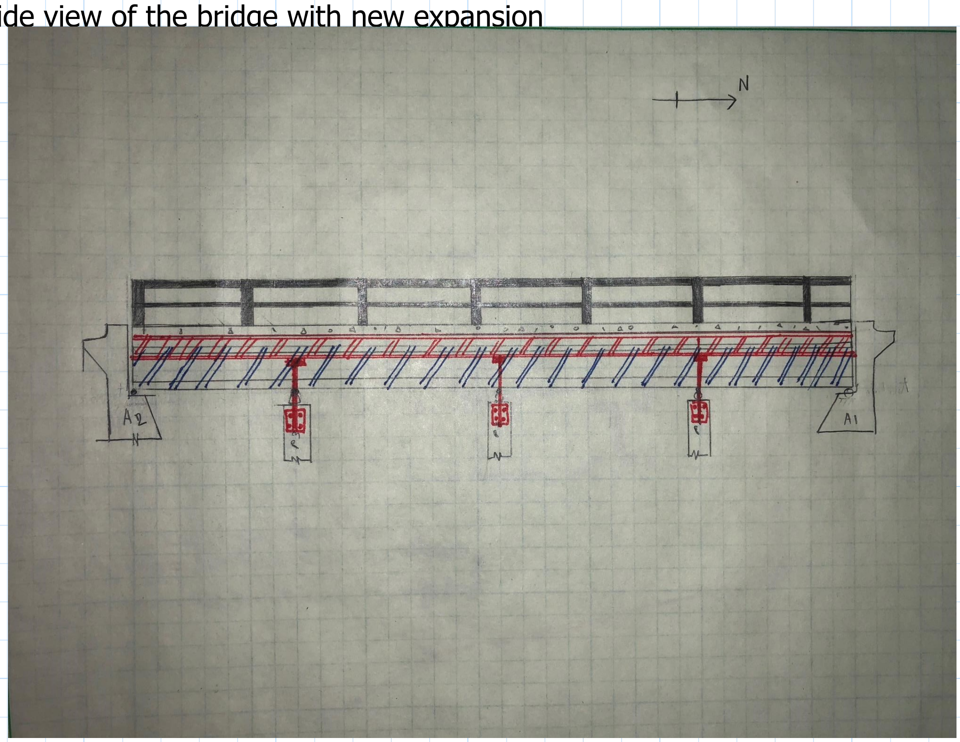
Shear studs are to be of an approved type listed in materials I.M. 453.1, Appendix A

The steel shall be for the beam shall be the same type and from the same source as existing members

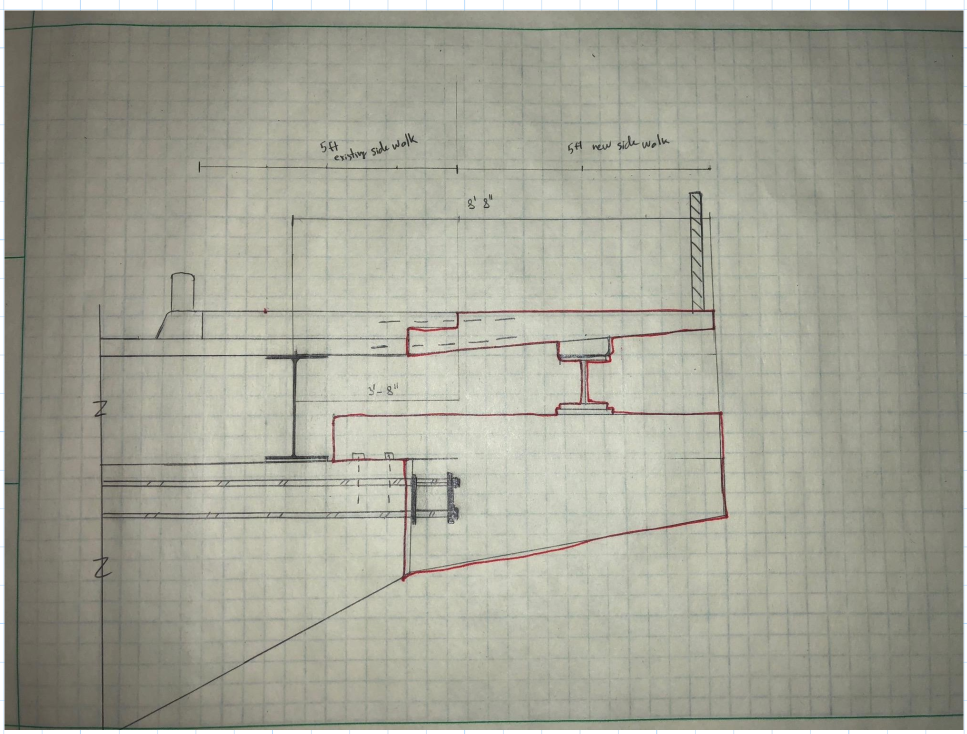
## PEDESTRIAN SIDEWALK EXPANSION

### Concept Drawings

Side view of the bridge with new expansion




Cross-section view of the bridge with new expansion



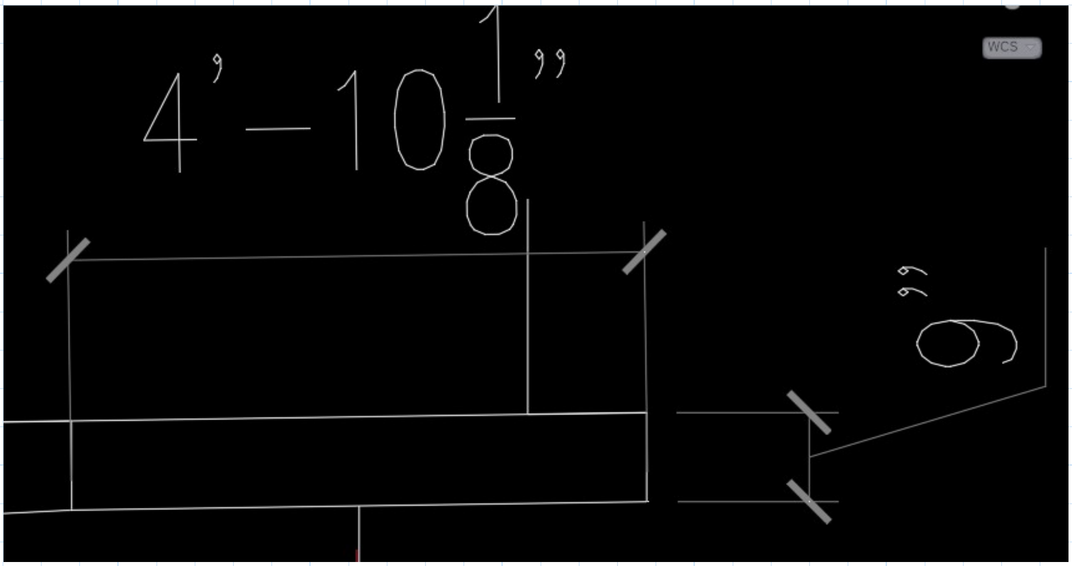
## Load calculations

Geometry

sidewalk is only between the end of barrier and start of rail

$L_{newside} := 4\ ft + 10.125\ in$ Length of new slab

$t_{conc} := 9\ in$ concrete thickness




Calculate Loads

Since the existing structure is stable, we will copy the slab design and use their values for steel reinforcemnet, concrete thickness and clearcover.

Pedestrain load

$Ll := 85\ psf$ AASHTO LRFD manula

Self weight of concrete, Dead load

$\gamma_c := 150\ pcf$

$$D_{deck} := \gamma_c \cdot \left(L_{newside} \cdot t_{conc} + 6\ in \cdot 16\ in\right) = 0.645\ \frac{kip}{ft}$$

$$D_{rail} := 50\ \frac{lbf}{ft}$$

$$D := D_{deck} + D_{rail} = 0.695\ \frac{kip}{ft}$$

Calculate Loads on girder with LRFD combinations

$$D_{tot} := D = 0.695\ \frac{kip}{ft}$$

$$L_{tot} := Ll \cdot \left(L_{newside} - 1\ ft\right) = 0.327\ \frac{kip}{ft}$$

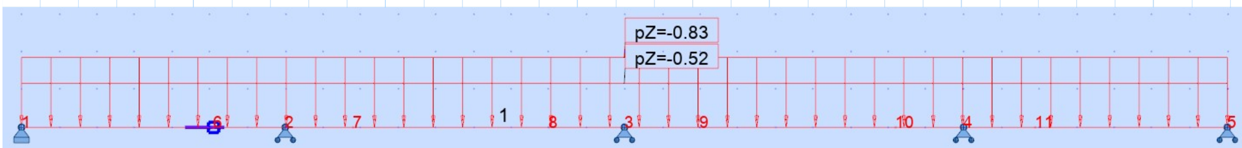
live load is only from the walking area, not in including the rail

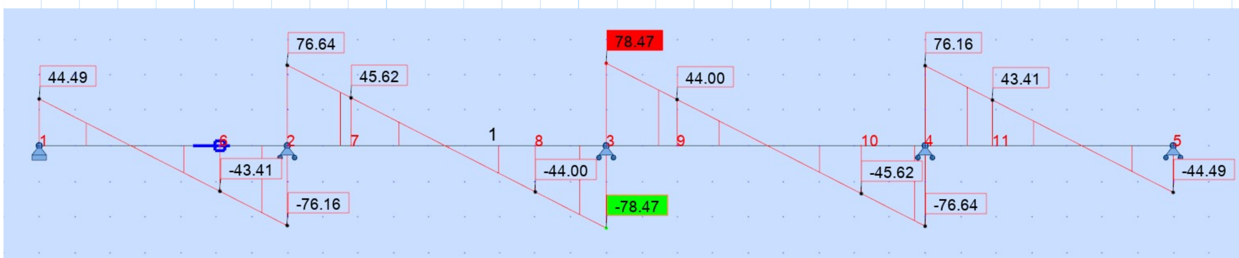
W24x306

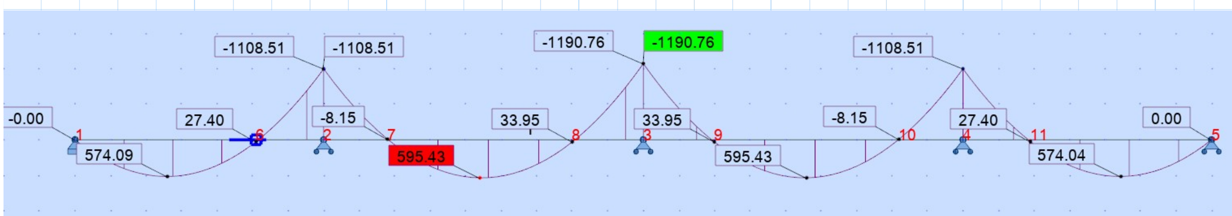
## STRUCTURAL ANALYSIS

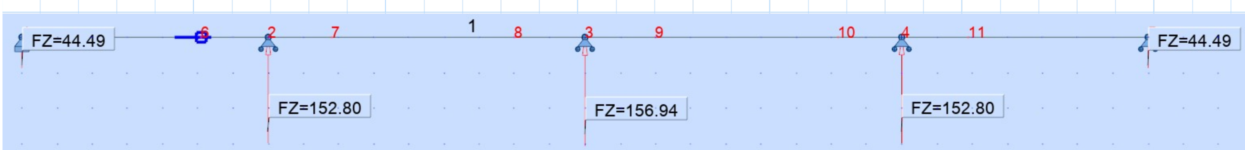
Max moment and Shear using uniform, adjacent, and alternating loadings

Uniform loading



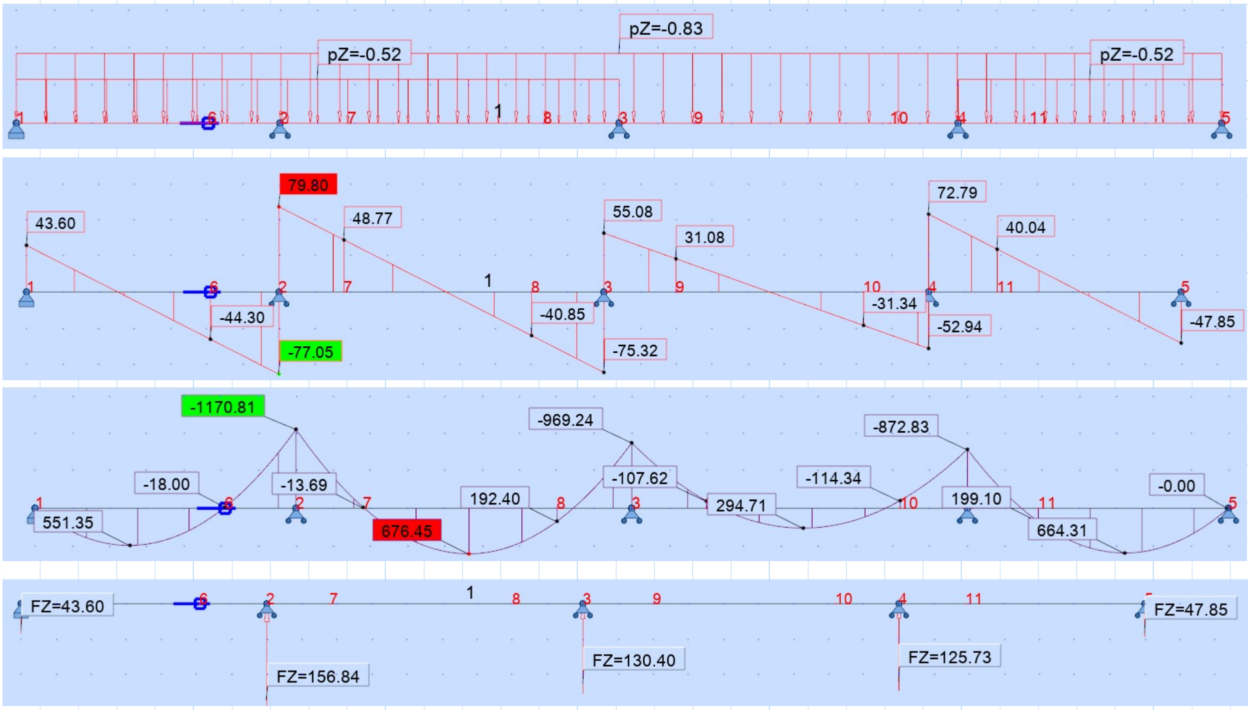

$$V1 := 78.47\ kip$$

$$M1_{pos} := 595.43\ kip \cdot ft$$

$$M1_{neg} := 1190.76\ kip \cdot ft$$

$$R1 := 156.94\ kip$$

Adjacent Loading, span (1&2), due to symetry (3&4)



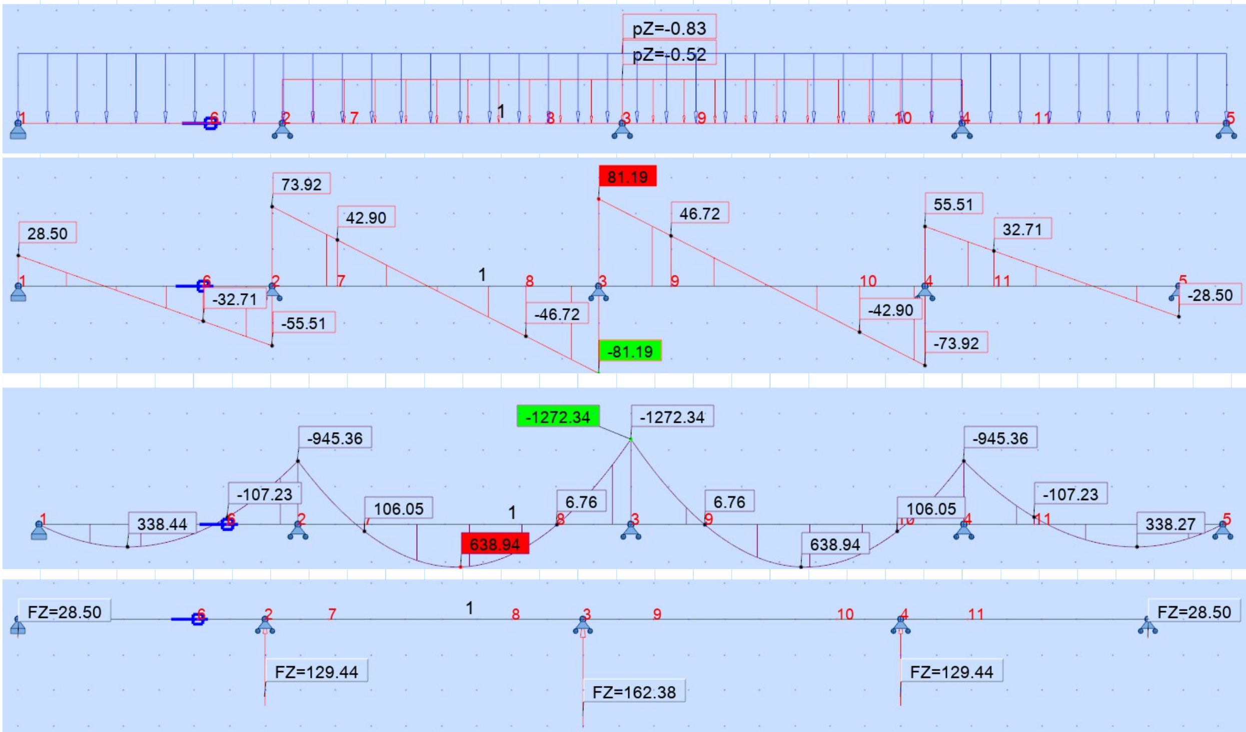

$V2 := 79.80\ \boldsymbol{kip}$

$M2_{pos} := 676.45\ \boldsymbol{kip} \cdot \boldsymbol{ft}$

$M2_{neg} := 1170.81\ \boldsymbol{kip} \cdot \boldsymbol{ft}$

$R2 := 159.84\ \boldsymbol{kip}$

Adjacent Loading, span (2&3)



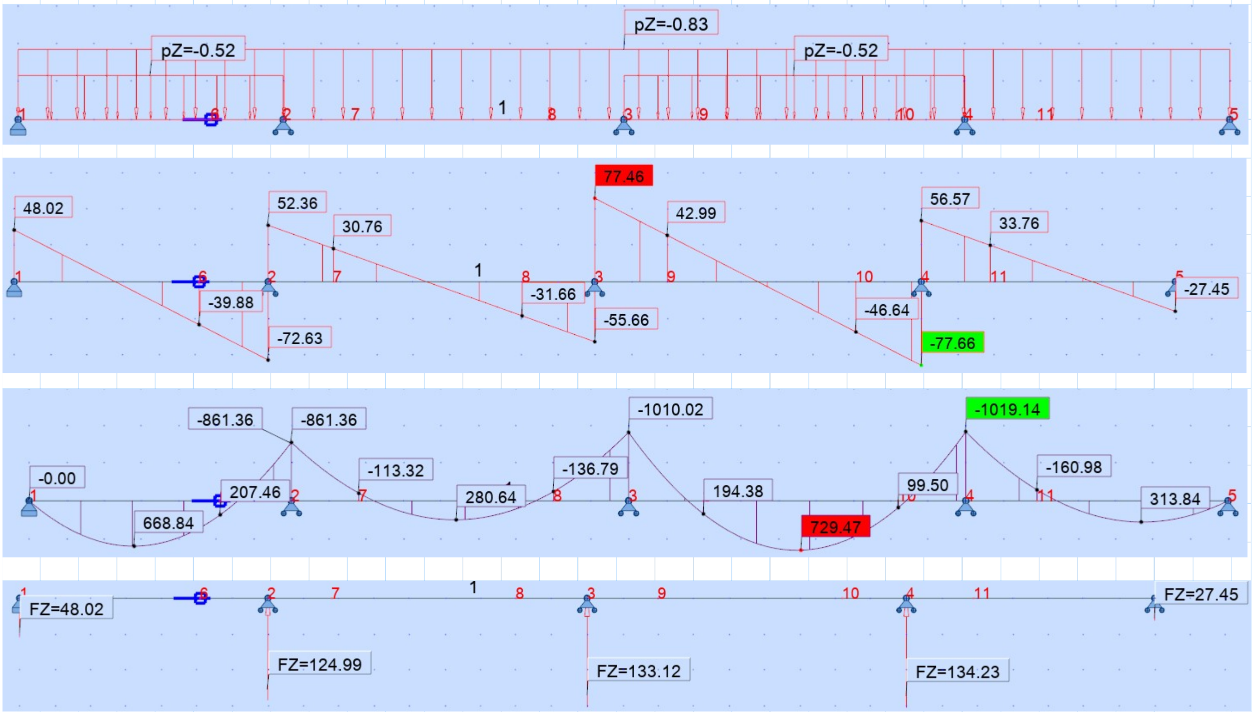

$V3 := 81.19\ kip$

$M3_{pos} := 638.94\ kip \cdot ft$

$M3_{neg} := 1272.34\ kip \cdot ft$

$R3 := 162.38\ kip$

Alternate Loading, span (1&3), due to symetry (2&4)



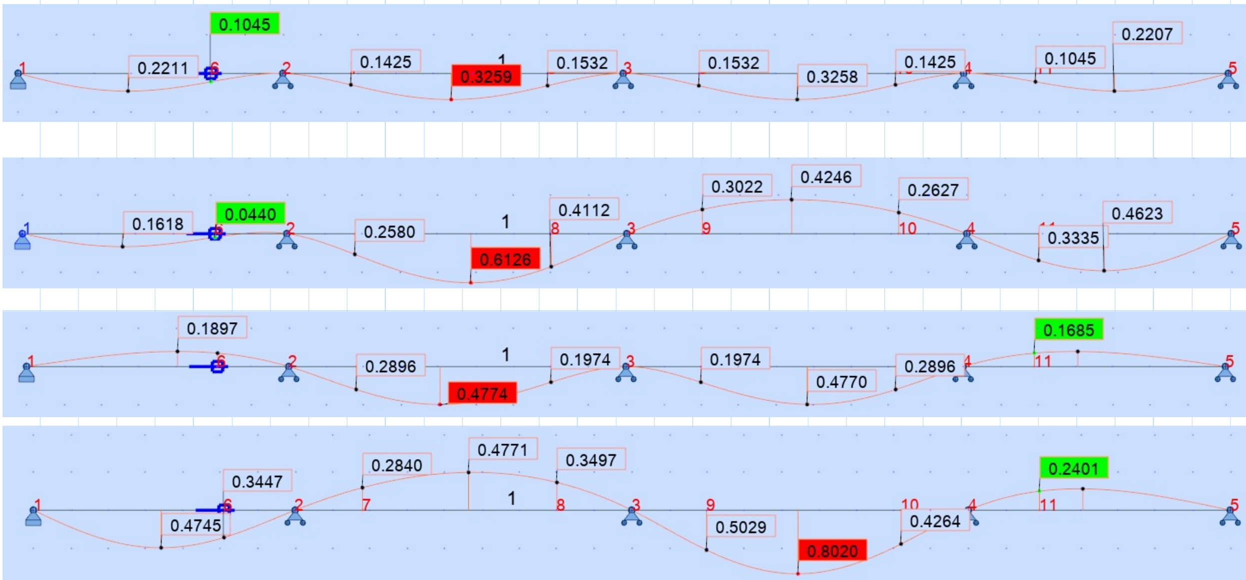

$V4 := 77.66\ \mathbf{kip}$

$M4_{pos} := 729.47\ \mathbf{kip} \cdot \mathbf{ft}$

$M4_{neg} := 1019.14\ \mathbf{kip} \cdot \mathbf{ft}$

$R4 := 134.23\ \mathbf{kip}$

## Live Load Calculation




$\delta_1 := 0.3259\ in$

$\delta_2 := 0.6126\ in$

$\delta_3 := 0.4774\ in$

$\delta_4 := 0.802\ in$

The Deflection calculations were analyzed by only using the unfactored live loads. this is what AISC code requires

Design values that need to be met

$V_{max} := \max(V1, V2, V3, V4) = 81.19\ kip$

$M_{maxpos} := \max(M1_{pos}, M2_{pos}, M3_{pos}, M4_{pos}) = 729.47\ ft \cdot kip$

$M_{maxneg} := \max(M1_{neg}, M2_{neg}, M3_{neg}, M4_{neg}) = 1272.34\ ft \cdot kip$

$\delta_{max} := \max(\delta_1, \delta_2, \delta_3, \delta_4) = 0.802\ in$

$R_{max} := \max(R1, R2, R3, R4) = 162.38\ kip$ $\qquad \dfrac{R_{max}}{2} = 81.19\ kip$

Select initial Size, W24X306

the deflection limit is shown below, our beam passes the deflection check

## BEAM CHECK

$\delta_{maxbeam} := \dfrac{90\ ft}{800} = 1.35\ in$

$DCR_{\delta} := \dfrac{\delta_{max}}{\delta_{maxbeam}} = 0.594$ this is less than 1 so our design passes deflection

Section Properties W24X306

| | | | | |
|---|---|---|---|---|
| $d := 27.1\ in$ | $t_w := 1.26\ in$ | $b_f := 13.4\ in$ | $t_f := 2.28\ in$ | $r := 0.95\ in$ |
| $A := 89.7\ in^2$ | $I_x := 10700\ in^4$ | $I_y := 919\ in^4$ | $Z_x := 922\ in^3$ | $S_x := 789\ in^3$ |
| $Z_y := 214\ in^3$ | $J := 17\ in^4$ | $C_w := 142000\ in^6$ | | $S_y := 137\ in^3$ |
| $E := 29000\ ksi$ | $F_y := 50\ ksi$ | $F_u := 65\ ksi$ | $r_{ts} := 3.81\ in$ | $h_0 := 24.8\ in$ |
| $S2 := 90\ ft$ | $S1 := 70\ ft$ | $r_y := 3.2\ in$ | $r_x := 10.9\ in$ | |

## 70 Foot span, we will check for two spans because it is a symetric bridge

Check for flexural strength major axis, Minor axis calc not needed

Limiting width- thickness ratio for flange (AISC B4.table B4.1b)

$\lambda_f := \dfrac{1}{2} \dfrac{b_f}{t_f} = 2.939$

$\lambda_{pf} := 0.38 \sqrt{\dfrac{E}{F_y}} = 9.152$

$$\lambda_{rf} := 1 \cdot \sqrt{\frac{E}{F_y}} = 24.083$$

$$\begin{array}{l} \text{if } \lambda_f < \lambda_{pf} \\ \quad \| \text{“Compact”} \\ \text{else} \\ \quad \| \text{“Non compact”} \end{array} = \text{“Compact”}$$

Limiting width- thickness ratio for web

$$\lambda_w := \frac{\left(d - t_f - t_f - 2 \cdot r\right)}{t_w} = 16.381$$

$$\lambda_{pw} := 3.76 \cdot \sqrt{\frac{E}{F_y}} = 90.553$$

$$\lambda_{rw} := 5.7 \cdot \sqrt{\frac{E}{F_y}} = 137.274$$

$$\begin{array}{l} \text{if } \lambda_w < \lambda_{pw} \\ \quad \| \text{“Compact”} \\ \text{else} \\ \quad \| \text{“Non compact”} \end{array} = \text{“Compact”}$$

$$C_b := 2.150$$

Yielding

$$M_{n1} := F_y \cdot Z_x = 3841.667 \; kip \cdot ft$$

LTB

$$L_p := \left(1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}}\right) = 135.637 \; in$$

$c := 1$ for doubly symetric I shape

flange centroid

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_0} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_0}\right)^2 + 6.76 \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 395.358\ in$$

$$L_b := S1 = 840\ in$$

$L_b > L_r = 1$ true, provide bracing at each pier

$$F_{cr} := \frac{C_b \cdot \pi^2\ E}{\left(\frac{L_b}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J \cdot c}{S_x \cdot h_0} \left(\frac{L_b}{r_{ts}}\right)^2} = 26.234\ ksi$$

$$M_{n2} := F_{cr} \cdot S_x = 1724.862\ kip \cdot ft$$

$$M_{nnom} := min\left(M_{n1}, M_{n2}\right) = 1724.862\ kip \cdot ft$$

$$\phi M_n := 0.9\ M_{nnom} = 1552.376\ kip \cdot ft$$

Check Flexural strength

$$DCR := \frac{M_{maxneg}}{\phi M_n} = 0.82$$ Chapter 5

if $1 \ge DCR$
  "We are good"
else
  "resize section"
= "We are good"

Check for Shear Strength Y axis

$$A_w := \left(d - 2\ t_f\right)\ \left(t_w\right) = 28.4\ in^2 \qquad C_v := 1$$

$$V_n := 0.6 \cdot F_y \cdot A_w \cdot C_v = 852.012\ kip$$

$$\phi V_n := 1 \cdot V_n = 852.012\ kip$$

$$DCR := \frac{V_{max}}{\phi V_n} = 0.095$$

$$\text{if } 1 \ge DCR \quad \| \text{“We are good”} \quad \text{else} \quad \| \text{“resize section”} \quad = \text{“We are good”}$$

## 90 Foot span

Check for flexural strength major axis, Minor axis calc not needed

Limiting width- thickness ratio for flange (AISC B4.table B4.1b)

$$\lambda_f := \frac{1}{2} \frac{b_f}{t_f} = 2.939$$

$$\lambda_{pf} := 0.38 \sqrt{\frac{E}{F_y}} = 9.152$$

$$\lambda_{rf} := 1 \cdot \sqrt{\frac{E}{F_y}} = 24.083$$

$$\text{if } \lambda_f < \lambda_{pf} \quad \| \text{“Compact”} \quad \text{else} \quad \| \text{“Non compact”} \quad = \text{“Compact”}$$

Limiting width- thickness ratio for web

$$\lambda_w := \frac{\left(d - t_f - t_f - 2 \cdot r\right)}{t_w} = 16.381$$

$$\lambda_{pw} := 3.76 \cdot \sqrt{\frac{E}{F_y}} = 90.553$$

$$\lambda_{rw} := 5.7 \cdot \sqrt{\frac{E}{F_y}} = 137.274$$

$$\text{if } \lambda_w < \lambda_{pw} \quad \| \text{“Compact”} \quad \text{else} \quad \| \text{“Non compact”} \quad = \text{“Compact”}$$

$C_b := 2.360$ modification factor for non uniform bending

Yielding

$$M_{n1} := F_y \cdot Z_x = 3841.667\ kip \cdot ft$$

LTB

$$L_p := \left(1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}}\right) = 135.637\ in$$

$c := 1$ for doubly symetric I shape

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_0} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_0}\right)^2 + 6.76 \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 395.358\ in$$

$$L_b := S2 = 1080\ in$$

$L_b > L_r = 1$ true, provide lateral bracing at each pier

$$F_{cr} := \frac{C_b \cdot \pi^2\ E}{\left(\frac{L_b}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J \cdot c}{S_x \cdot h_0} \left(\frac{L_b}{r_{ts}}\right)^2} = 21.342\ ksi$$

$$M_{n2} := F_{cr} \cdot S_x = 1403.219\ kip \cdot ft$$

$$M_{nnom} := min\left(M_{n1}, M_{n2}\right) = 1403.219\ kip \cdot ft$$

$$\phi M_n := 0.9\ M_{nnom} = 1262.897\ kip \cdot ft$$

Check Flexural strength

$$DCR := \frac{M_{maxneg}}{\phi M_n} = 1.007$$

```
if 1.05 ≥ DCR              = "We are good"
  ‖ "We are good"
else
  ‖ "resize section"
```

Check for Shear Strength Y axis

$$A_w := \left(d - 2\ t_f\right)\ \left(t_w\right) = 28.4\ in^2 \qquad C_v := 1$$

$$V_n := 0.6 \cdot F_y \cdot A_w \cdot C_v = 852.012\ kip$$

$$\phi V_n := 1 \cdot V_n = 852.012\ kip$$

$$DCR := \frac{V_{max}}{\phi V_n} = 0.095$$

```
if 1 ≥ DCR                    = "We are good"
  ‖ "We are good"
else
  ‖ "resize section"
```

Our section passed Shear, Flexure, LTB, and deflection.

using robot we can confirm that our section passes, and my calculations are right. some dicreptancy though. since robot is more accurate with dimensions. image of the check is shown bellow

CODE: *ANSI/AISC 360-10 An American National Standard, June 22, 2010*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 1 beam POINT: 1 COORDINATE: x = 0.50 L = 160.00 ft

LOADS:
*Governing Load Case:* 9 Adjacent load 2 (1+2)*1.20+5*1.60

MATERIAL:
STEEL Fy = 36.00 ksi Fu = 58.00 ksi E = 29000.01 ksi

SECTION PARAMETERS: W 24x306

| | | | |
|---|---|---|---|
| d=27.10 in | Ay=61.104 in2 | Az=34.146 in2 | Ax=89.700 in2 |
| bf=13.40 in | Iy=10700.000 in4 | Iz=919.000 in4 | J=117.000 in4 |
| tw=1.26 in | Sy=789.668 in3 | Sz=137.164 in3 | |
| tf=2.28 in | Zy=922.000 in3 | Zz=214.000 in3 | |

MEMBER PARAMETERS:

| | | |
|---|---|---|
| | | Cb 1.0 |
| Ly = 320.00 ft | Lz = 320.00 ft | |
| Ky = 1.00 | Kz = 1.00 | Lb = 90.00 ft |
| KLy/ry = 351.59 | KLz/rz = 1199.69 | Cb = 1.00 |

INTERNAL FORCES: DESIGN STRENGTHS
Mry = -1272.34 kip*ft Fib*Mny = 1304.12 kip*ft
Vrz = 81.19 kip Fiv*Vnz = 737.55 kip

SAFETY FACTORS
Fib = 0.90 Fiv = 1.00

SECTION ELEMENTS:
Flange = Compact Web = Compact

VERIFICATION FORMULAS:
Mry/(Fib*Mny) = 0.98 < 1.00 LRFD (H1-1b) Verified
Vrz/(Fiv*Vnz) = 0.11 < 1.00 LRFD (G2-1) Verified

LIMIT DISPLACEMENTS

***Deflections (LOCAL SYSTEM):***
uyt = 0.0000 in < uyt max = L/1000.00 = 3.8400 in Verified
***Governing Load Case:*** 1 DL1
uzt = 0.8017 in < uzt max = L/1000.00 = 3.8400 in Verified
***Governing Load Case:*** 6 Alternating

***Displacements (GLOBAL SYSTEM):*** *Not analyzed*

***Section OK !!!***

## Beam Splice Connection Design

Determine splicing locations for the bridge. I will be applying a full dead load onto the new deck. then I will find the point of inflections and that will indicat the location of splices

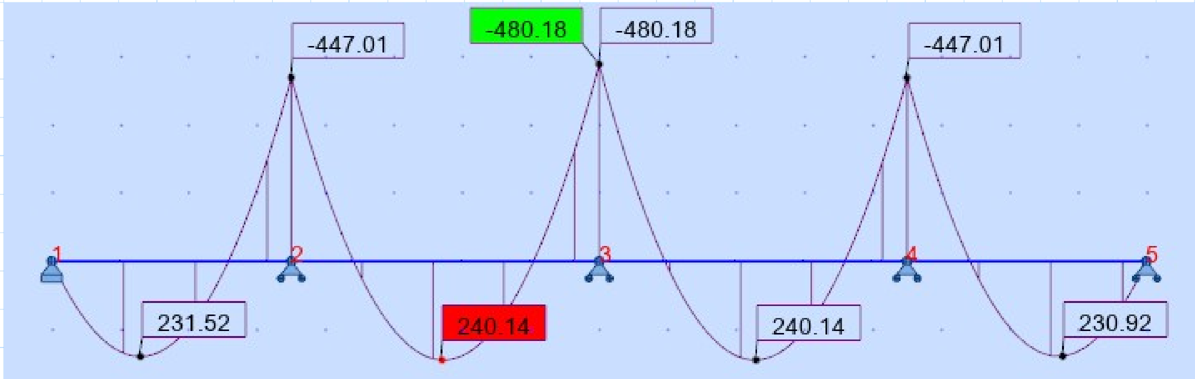



figure above is the moment only due to dead load

Location of points of inflections

$PI_1 := 51\ ft$ $\qquad$ $PI_6 := 320\ ft - PI_1 = 269\ ft$

$PI_2 := 88\ ft$ $\qquad$ $PI_5 := 320\ ft - PI_2 = 232\ ft$

$PI_3 := 140\ ft$ $\qquad$ $PI_4 := 320\ ft - PI_3 = 180\ ft$

We found the points of inflection now we will specify the Lengths of each beam and design the moment connection so the beam will act as a continoious section

after doing some research I found that I beams can be hot rolled up to 150ft in length, even more in some instances. The issue with this is transportation of such beams. In our project our span is 320ft.

The IOWA DOT requires a permit for trcuks with indivisable loads, and a maximum length for an oversized truck is 120ft

Segment lengths

$PI_1 = 51\ ft$

$PI_2 - PI_1 = 37\ ft$

$PI_3 - PI_2 = 52\ ft$

$PI_4 - PI_3 = 40\ ft$

we will have 7 segemnts
3 sets of 2 and 1 unique
length

$PI_5 - PI_4 = 52\ ft$

$PI_6 - PI_5 = 37\ ft$

$PI_1 = 51\ ft$

Moment Connector

Design for the worst senario and use it for all splices, the bolts are A490-X steel.
Plate is A913-70 Steel W section is A992-50

Steel

$F_y := 50\ ksi$ $F_u := 65\ ksi$ $F_{yplate} := 70\ ksi$ $F_{uplate} := 90\ ksi$

$F_{ubolt} := 150\ ksi$ $F_{nv} := .563 \cdot F_{ubolt} = 84.45\ ksi$

Bolt 1in d

$db := \frac{7}{8}\ in$ $A_b := \frac{\pi}{4}\ db^2 = 0.601\ in^2$

Plate

$\phi M_n := 0.9 \cdot F_y \cdot Z_x = 3457.5\ kip \cdot ft$ Full moment capacity of beam section

$\phi V_u := 0.6 \cdot F_y \cdot d \cdot t_w = 1024.38\ kip$

Design momnet and shear(Mu=50%*phiMn, and Vu=50%*phiVu)

$M_u := .5 \cdot \phi M_n = 1728.75\ kip \cdot ft$

$V_u := .5 \cdot \phi V_u = 512.19\ kip$

$P_u := \frac{M_u}{d} = 765.498\ kip$ flange force

**WEB**

Bolts needed web, and length for web plate

# of Web bolts

$$\phi R_n := 0.75 \cdot F_{nv} \cdot 4 \cdot A_b = 152.345 \ kip$$

$$\#bolts_{web} := \text{Ceil}\left(\frac{V_u}{\phi R_n}, 1\right) + 1 = 5$$

5 Bolts on each line of each member, detail will be included at the end, for added safety

Web Plate

$$A_{pw} := \frac{V_u}{0.6 \cdot F_{yplate}} = 12.195 \ in^2$$

area of plate, use shear Vu since it will be primarily resisting shear

since we are not accounting for welding we will use the whole flange width for the plate and the plate will be 3/8in min practical plate thickness

$$b_{wp} := d - 2\ t_f - 2\ r - 1\ in = 19.64\ in$$

an extra 1in for workability

$$t_{wptest} := \frac{A_{pw}}{2\left(b_{wp} - \#bolts_{web} \cdot \left(\frac{13}{16}\ in\right)\right)} = 0.391\ in$$

we will use a 7/16 thick plate

$$t_{wp} := \frac{4}{5}\ in$$

$$s := 3 \cdot db = 2.625\ in$$

min spacing

$$Sv_{wp} := \frac{b_{wp} - 3\ in}{\#bolts_{web}} = 3.328\ in$$

3in for end of plate, 1.5 in each side

$$L_w := (s \cdot 3) + 6\ in = 13.875\ in$$

use 2PLs-7/16 x 31.1 x 15 with 3 in spacing between bolts in the vertical direction

## Flange Outside

splices are designed as tension members, the plate width is same as the flange width

$b_{fo} := b_f = 13.4\ in$ $\qquad et_{fo} := 1\ in$ $\qquad$ estimated thickness

fracture and yielding limit state must be investigated

$$T_u := \frac{M_u}{d + et_{fo}} = 738.256\ kip$$

$$A_g := \frac{T_u}{0.9 \cdot \left(F_{yplate}\right)} = 11.718\ in^2$$

$$A_n := \frac{T_u}{0.85 \cdot F_{uplate}} = 9.65\ in^2$$

$$\begin{array}{l} \text{if } A_n \le 0.85 \cdot A_g \\ \quad \| \text{“we do not need to consider block shear”} \\ \text{else} \\ \quad \| \text{“consider block shear”} \end{array} = \text{“we do not need to consider block shear”}$$

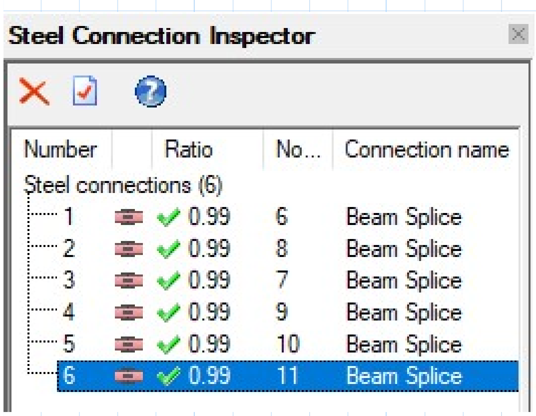
$$R_{tfo} := \frac{A_n}{b_{fo} - 2\left(\frac{3}{4}\ in + \frac{1}{8}\ in\right)} = 0.828\ in$$

Bolts needed flange outside, and length for web plate

$$\#boltf := \mathrm{Ceil}\left(\frac{T_u}{45.1\ kip}, 1\right) + 1 = 18$$

So 8 bolts in two rows

$L_f := (22 \cdot 3\ in) + 2\ in = 68\ in$ $\qquad$ Use 1" X 16.8" X 68" plate



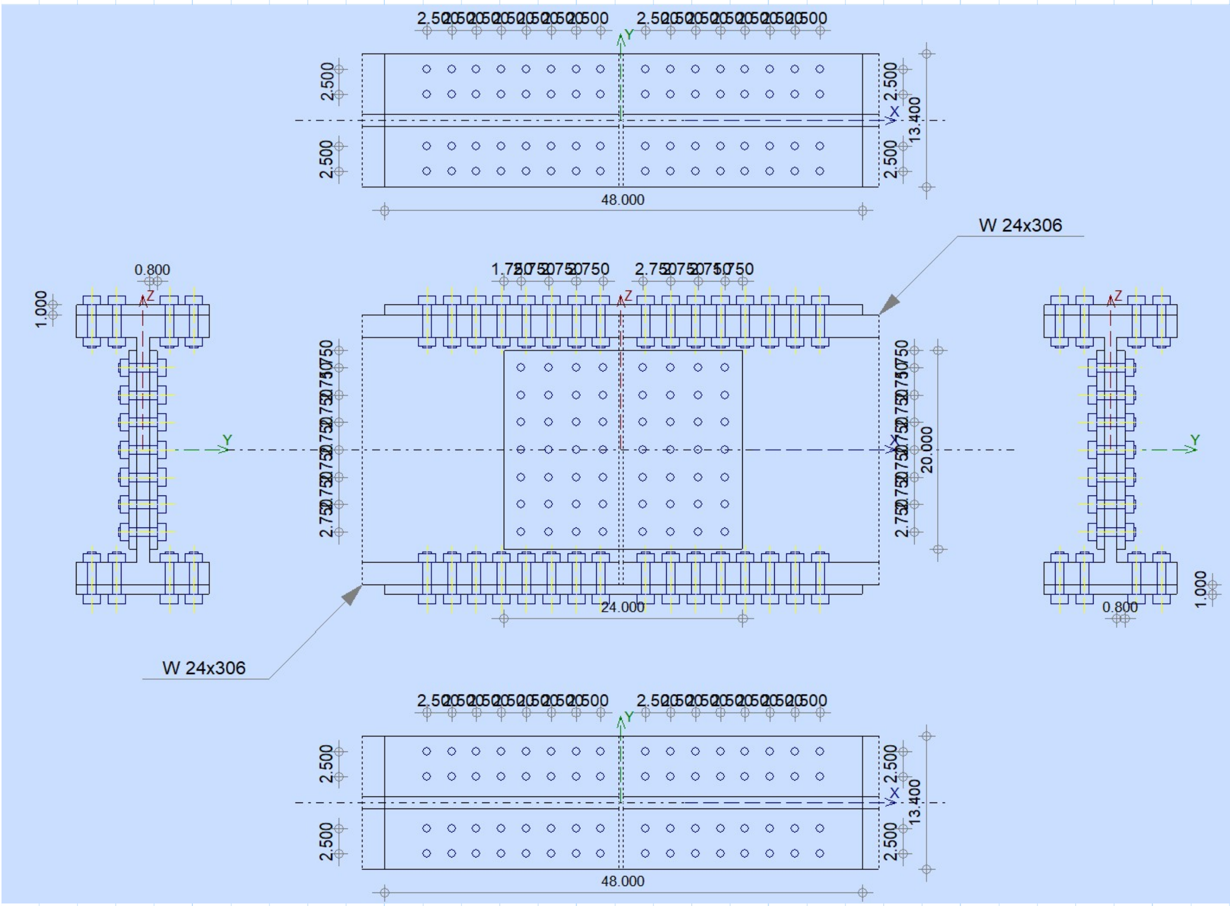
Steel Connection Inspector

| Number | Ratio | No... | Connection name |
|---|---|---|---|
| Steel connections (6) | | | |
| 1 | 0.99 | 6 | Beam Splice |
| 2 | 0.99 | 8 | Beam Splice |
| 3 | 0.99 | 7 | Beam Splice |
| 4 | 0.99 | 9 | Beam Splice |
| 5 | 0.99 | 10 | Beam Splice |
| 6 | 0.99 | 11 | Beam Splice |

over designed in robot to get a better factor of safety, I used 50% of the moment capacity of the beam, instead of the maximum moment at the splice connection generated by the loads

| Number | Ratio | No... | Connection name |
|---|---|---|---|
| Steel connections (6) | | | |
| 1 | 0.12 | 6 | Beam Splice |
| 2 | 0.11 | 8 | Beam Splice |
| 3 | 0.09 | 7 | Beam Splice |
| 4 | 0.11 | 9 | Beam Splice |
| 5 | 0.08 | 10 | Beam Splice |
| 6 | 0.11 | 11 | Beam Splice |

Under maximum moment under a servise load the capacity does not exceed 12%. so Our section is way over designed. but this is the weakest point in the beam, and typically engineers design for 50% of the design moment and shear, that is why my connection is so big.

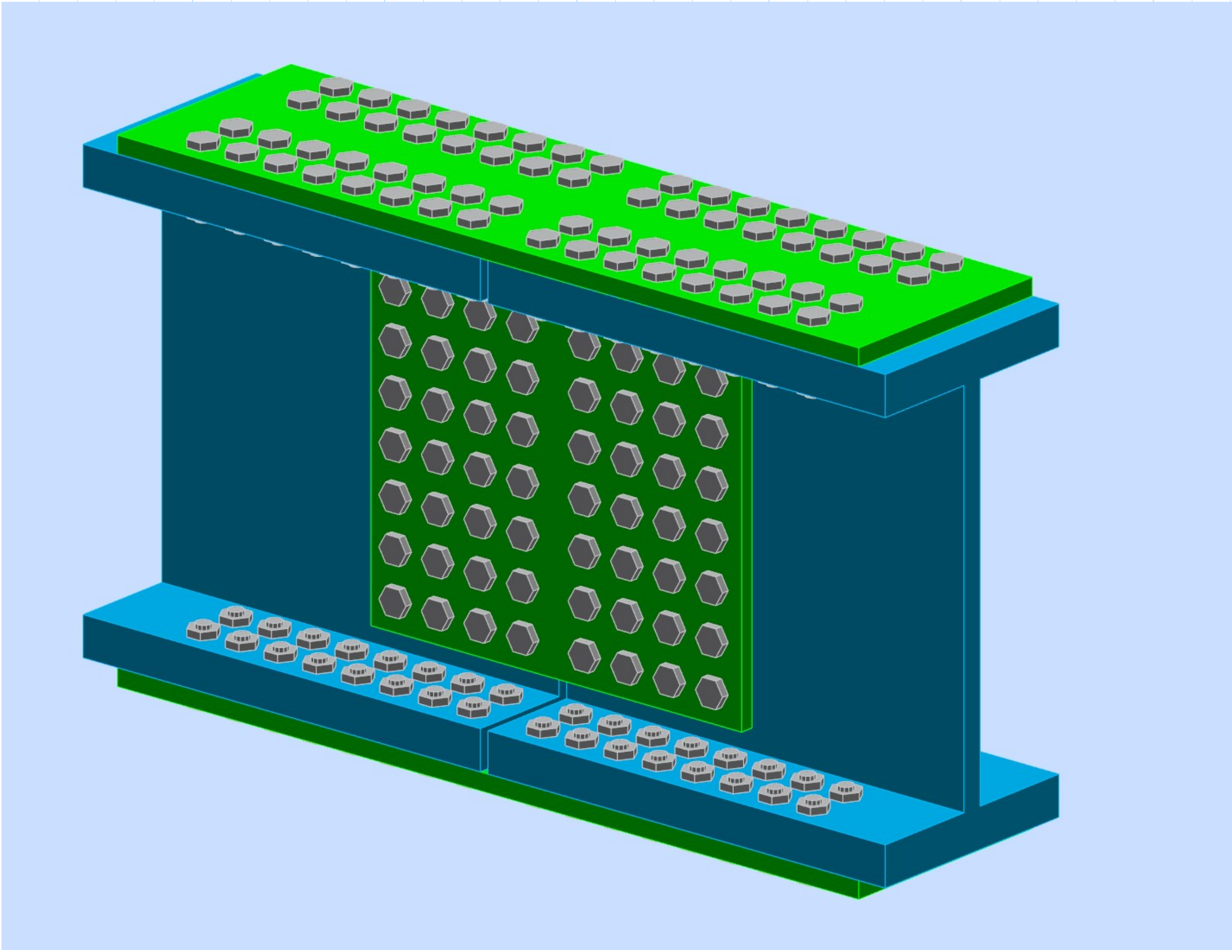



Actual plate sizes based on Robot analysis is:

Web: 4/5" X 24" X 20" using A913-70 steel 7 bolts ber row, 4 rows. spaced at 2.750in

Flanges: 1" X 48" X 13.4 " using A913-70 steel 8 bolts ber row, 4 rows per flange. spaced at 2.50in

Bolts: 7/8" X bolts



## Shear Stud Design

Assume studs Fu=65ksi use 7/8" x 4" Nelson Stud $fc' := 4\ ksi$

$w_c := 150\ pcf$ $fc1' := 4000$ $R_g := 1$ $R_p := 0.75$ $F_{ustud} := 65\ ksi$

$D_s := \frac{7}{8}\ in$ $h_s := 8\ in$ $A_{sa} := \frac{\pi}{4} D_s^{\ 2} = 0.601\ in^2$ $F_{ystud} := 50\ ksi$

$Y_{conc} := 1\ ft + 8\ in$

| | | | |
|---|---|---|---|
| 101-098-032 | 22 | 350 | 1138 |

$$E_c := \left(57000 \cdot \sqrt{fc1'}\right)\ psi = 3604996.533\ psi$$

$$Q_n := min\left(\frac{1}{2} A_{sa} \cdot \sqrt{fc' \cdot E_c}, R_g \cdot R_p \cdot A_{sa} \cdot F_u\right) = 29.314\ kip$$

stud diameter needs to be between 3/4 in and 2.5tf

$\frac{3}{4}\ in < D_s < 2.5 \cdot t_f = 1$ true

Stud Spacing needs to be greater than 6Ds and less than 8*Ycon

$s := 9\ in$

$6\ D_s < s < 8\ Y_{conc} = 1$

Stud height must be greater than 4Ds

$h_s > 4\ D_s = 1$

Concrete cover must be over 1/2 in

$Y_{conc} - h_s = 12\ in$

$L := 90\ ft$

$$N := \left(1 \cdot \frac{\frac{L}{2}}{s}\right) = 60$$

$\Sigma Q_n := N \cdot Q_n = 1758.862\ kip$ $\square > \square$ $M_{maxneg} = 1272.34\ ft \cdot kip$

Adequte design

$L := 70\ ft$ $\qquad s := 7\ in$

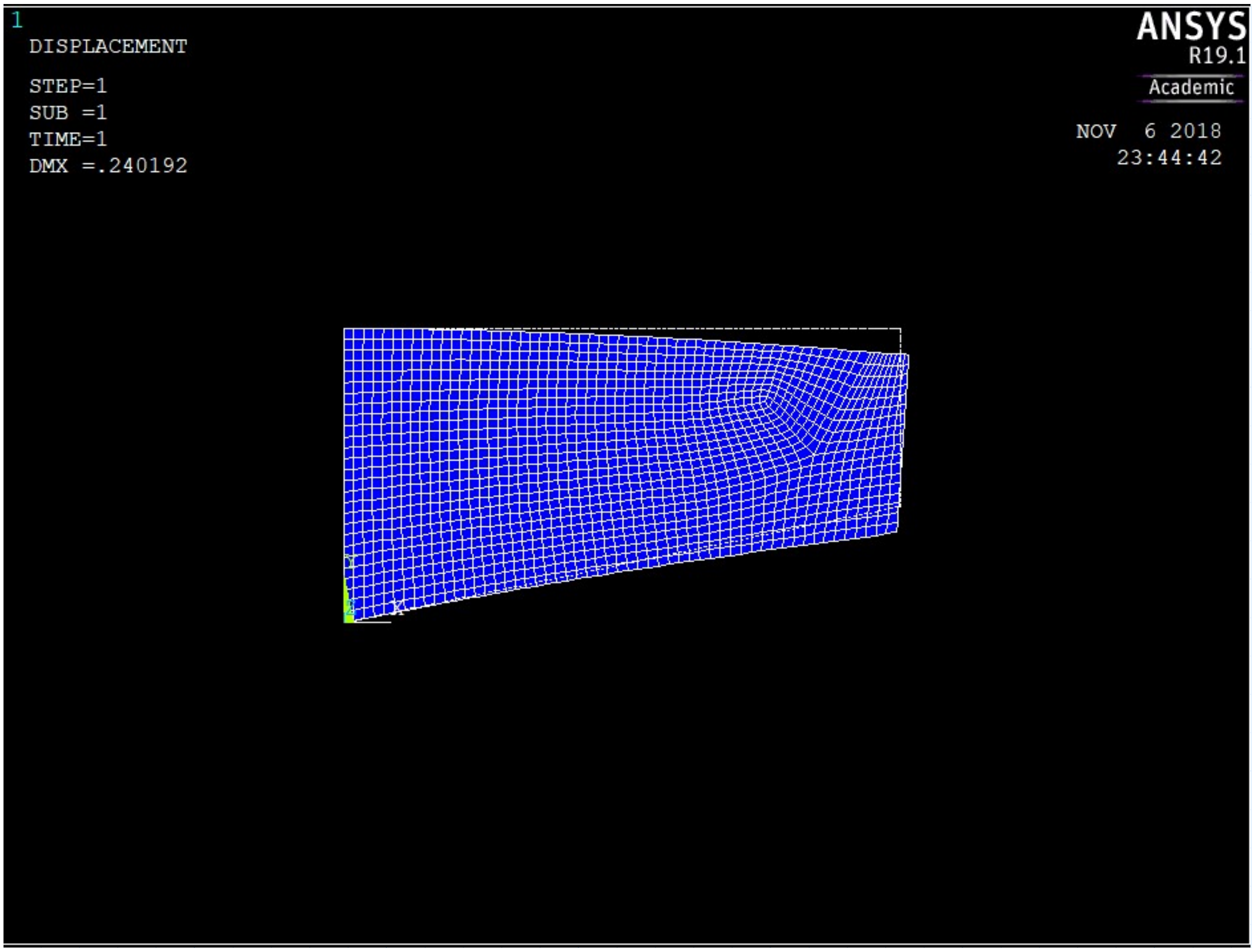
$$N := \left(1 + \frac{\frac{L}{2}}{s}\right) = 61$$

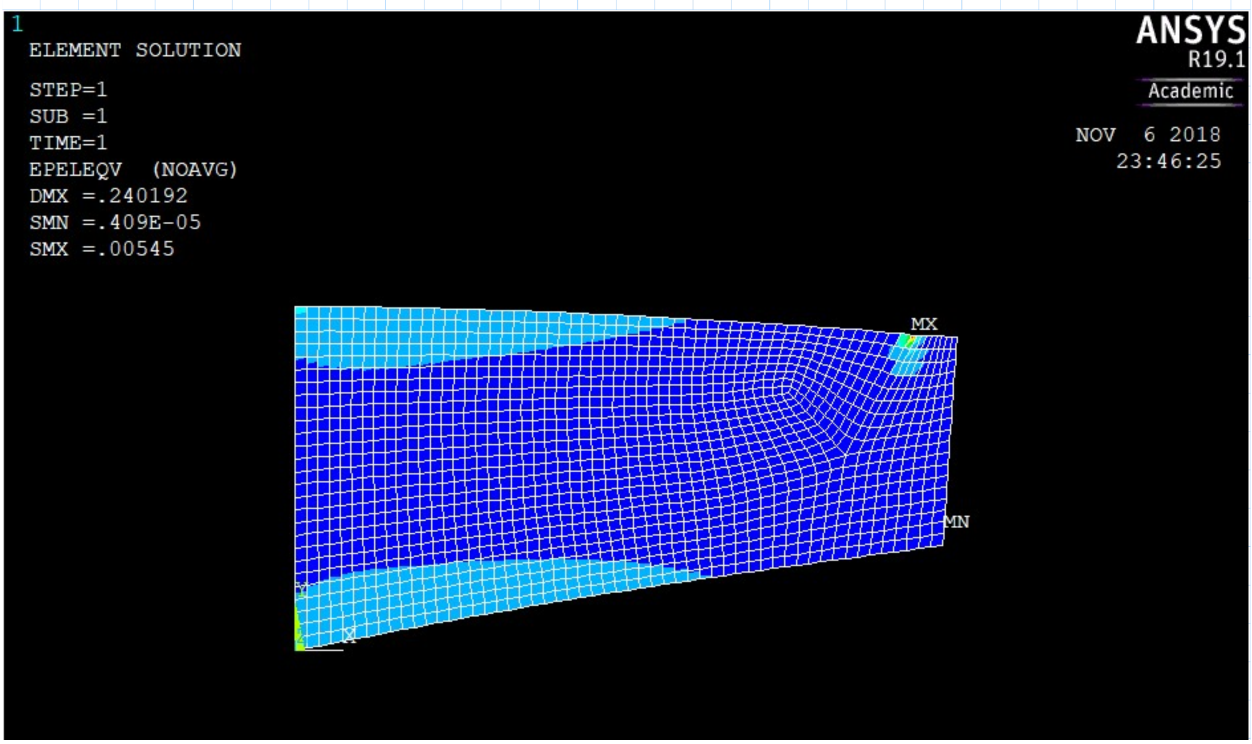
$\Sigma Q_n := N \cdot Q_n = 1788.177\ kip \qquad > \qquad M_{maxneg} = 1272.34\ ft \cdot kip$

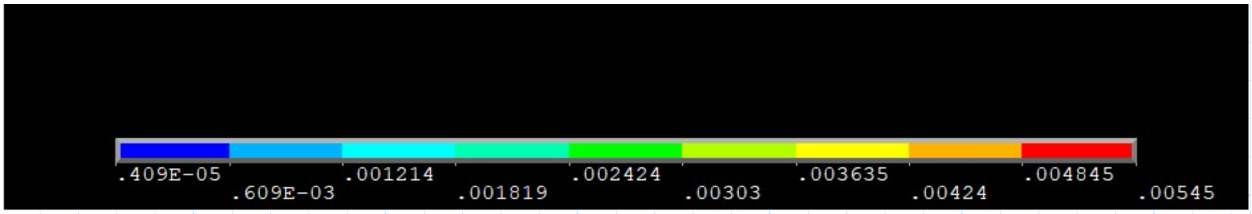
Adequte design

For the 70 foot span we will need 122 studs
and for 90 foot span we will need 120 studs
we have 2 spans for each so our total studs will be:

$\#ofstuds := 122 \cdot 2 + 120 \cdot 2 = 484$


1
DISPLACEMENT
STEP=1
SUB =1
TIME=1
DMX =.240192
ANSYS
R19.1
Academic
NOV 6 2018
23:44:42
Y
X


1
ELEMENT SOLUTION
STEP=1
SUB =1
TIME=1
EPELEQV (NOAVG)
DMX =.240192
SMN =.409E-05
SMX =.00545
ANSYS
R19.1
Academic
NOV 6 2018
23:46:25
MX
MN
Y
X


.409E-05
.609E-03
.001214
.001819
.002424
.00303
.003635
.00424
.004845
.00545

## ABUTMENT DESIGN FOR RED BRIDGE

### *Abutment Wall Design*

### *Variable Definition:*

$H_1 := 5\ in$ Trail Pavement Thickness

$H_2 := 6\ in$ Trail Subbase Thickness

$E_g := 796.2\ ft$ Approach Elevation

$E_b := 779.0\ ft$ Elevatin of Bottom of Foundation

$H_4 := 3.5\ ft$ Foundation Depth (Frost Depth - Iowa)

$H_w := E_g - E_b = 17.2\ ft$ Height of Wall

$x_4 := 2.0\ ft$ Width of Toe

$H_5 := 0.1 \cdot H_w = 1.72\ ft$ Hight of Footing

$H_3 := H_w - (H_1 + H_2 + H_5) = 14.563\ ft$

$x_3 := 1.5\ ft$ Width Needed for Roler/Pin Support

$t_{stem} := 0.1 \cdot H_w = 1.72\ ft$ Total Width of Stem

$x_2 := t_{stem} - x_3 = 0.22\ ft$ Width of Heel Side of Stem

$x_1 := 2.0\ ft$ Width of Heel

$B := x_1 + x_2 + x_3 + x_4 = 5.72\ ft$

$\gamma_c := 145\ pcf$ Unit Weight of Concrete

$\gamma_g := 115\ pcf$ Unit Weight of Engineered Subbase

$\gamma_1 := 110\ pcf$ Unit Weight of Engineered Backfill

$\phi'_1 := 45\ °$ Angle of Friction of Engineered Backfill

$\gamma_2 := 130\ pcf$ Saturated Unit Weight of Engineered Backfill $\beta := 0\ °$

$\phi'_2 := 45\ °$ Angle of Friction of Engineered Backfill $c'_2 := 0\ psf$

$w_1 := x_1 \cdot H_1 \cdot \gamma_c = 0.121\ klf$

$w_2 := x_1 \cdot H_2 \cdot \gamma_g = 0.115\ klf$

$w_3 := x_1 \cdot H_3 \cdot \gamma_1 = 3.204\ klf$

$w_4 := x_2 \cdot \left(H_1 + H_2 + H_3\right) \cdot \gamma_c = 0.494\ klf$

$w_5 := x_3 \cdot \left(H_2 + H_3\right) \cdot \gamma_c = 3.276\ klf$

$w_6 := x_4 \cdot \left(H_4 - H_5\right) \cdot \gamma_2 = 0.463\ klf$

$w_7 := B \cdot H_5 \cdot \gamma_c = 1.427\ klf$

$W_{Bridge} := 47823\ lbf = 47.823\ kip$ Wight of Superstructure

$L_{pedestrian} := 85\ \frac{lbf}{ft^2} \cdot \left(16\ ft \cdot 120\ ft\right) = 163.2\ kip$ Weight of Pedestrians

$L_a := 16.1\ ft + 4\ ft = 20.1\ ft$ Length of Abutment

$$P := \frac{\frac{W_{Bridge}}{2} + \frac{L_{pedestrian}}{2}}{L_a} = 5.249\ klf$$

$x'_1 := \frac{x_1}{2} + x_2 + x_3 + x_4 = 4.72\ ft$

$x'_2 := x'_1 = 4.72\ ft$

$x'_3 := x'_1 = 4.72\ ft$

$x'_4 := \frac{x_2}{2} + x_3 + x_4 = 3.61\ ft$

$x'_5 := \frac{x_3}{2} + x_4 = 2.75\ ft$

$x'_6 := \frac{x_4}{2} = 1\ ft$

$x'_7 := \frac{B}{2} = 2.86\ ft$

$x'_P := x'_5 = 2.75\ ft$

$$K_a := \cos(\beta) \cdot \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi'_1)^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi'_1)^2}} = 0.172$$

$$P_a := \frac{1}{2} \cdot \gamma_1 \cdot H_w^{\ 2} \cdot K_a = 2.792 \ \frac{kip}{ft}$$

$$z_{bara} := \frac{H_w}{3} = 5.733 \ ft$$

$$P_H := P_a \cdot \cos(\beta) = 2.792 \ \frac{kip}{ft}$$

$$P_v := P_a \cdot \sin(\beta) = 0 \ \frac{kip}{ft}$$

$$\Sigma M_o := P_H \cdot z_{bara} = 16.006 \ kip$$

$$\Sigma M_R := w_1 \cdot x'_1 + w_2 \cdot x'_2 + w_3 \cdot x'_3 + w_4 \cdot x'_4 + w_5 \cdot x'_5 + w_6 \cdot x'_6 + w_7 \cdot x'_7 + P \cdot x'_P = 46.007 \ kip$$

$FS_o := \frac{\Sigma M_R}{\Sigma M_o} = 2.874$ $\quad FS_o \geq 2-3$ Yes, OK! Overturning Factor of Safety

$$F_{max} := (w_1 + w_2 + w_3 + w_4 + w_5 + w_6 + w_7 + P) \cdot \tan(\phi'_2) + B \cdot \frac{1}{2} \cdot c'_2 = 14.349 \ klf$$

$FS_V := \frac{F_{max}}{P_H} = 5.14$ $\quad FS_V \geq 1.5$ Yes, OK! Sliding Factor of Safety

$$X_{barR} := \frac{\Sigma M_R - \Sigma M_o}{w_1 + w_2 + w_3 + w_4 + w_5 + w_6 + w_7 + P} = 2.091 \ ft$$

$e := \frac{B}{2} - X_{barR} = 0.769 \ ft$ $\quad e < var$ Yes, OK! Check for Uplfit

$$var := \frac{B}{6} = 0.953 \ ft$$

$s_c := 1$
$s_q := 1$
$s_\gamma := 1$

$d_c := 1.00$
$d_q := 1.00$
$d_\gamma := 1.00$

$$N_q := e^{\pi \cdot \tan(\phi'_2)} \cdot \tan\left(45\ ° + \frac{\phi'_2}{2}\right)^2 = 134.874$$

$$N_c := \frac{N_q - 1}{\tan(\phi'_2)} = 133.874$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_2) = 271.748$$

$$m := \frac{2 + \frac{B}{L_a}}{1 + \frac{B}{L_a}} = 1.778$$

$$H_i := P_H = 2.792\ klf$$

$$den := \frac{F_{max}}{\tan(\phi'_2)} = 14.349\ klf$$

$$i_q := \left(1 - \frac{H_i}{den}\right)^m = 0.681$$

$$i_\gamma := \left(1 - \frac{H_i}{den}\right)^{m+1} = 0.548$$

$$i_c := i_q - \left(\frac{1.0 - i_q}{N_q - 1}\right) = 0.678$$

$$B' := B - 2 \cdot e = 4.182\ ft$$

$$q'_N := c'_2 \cdot N_c \cdot d_c \cdot i_c + \gamma_1 \cdot H_4 \cdot N_q \cdot d_q \cdot i_q + \frac{1}{2} \cdot \gamma_2 \cdot B' \cdot N_\gamma \cdot d_\gamma \cdot i_\gamma = 526.598\ psi$$

$$q' := \frac{(w_1 + w_2 + w_3 + w_4 + w_5 + w_6 + w_7 + P) + \gamma_1 \cdot x_3 \cdot (H_4 - H_5)}{B'} = 24.316\ psi$$

$FS_q := \frac{q'_N}{q'} = 21.657$ $\quad FS_q \ge 3$ $\quad$ Yes, OK! $\quad$ Bearing Capacity Factor of Safety

***Wing Wall Design (Symmetric About Centerline Abutment Wall)***

***Variable Definition:***

$E_g := 796.2\ ft$ Ground Elevation

$E_f := 779.0\ ft$ Foundation Elevation (Bottom)

$H_1 := 5\ in$ Trail Pavement Thickness

$H_2 := 6\ in$ Subbase Thickness

$H_5 := 3.5\ ft$ Foundation Depth (Frost Depth - Iowa)

$H_w := E_g - E_f = 17.2\ ft$ Height of Wall

$x_1 := 4.0\ ft$ Width of Foundation

$x_3 := 1.4142\ ft$ Width of Toe

$H_4 := 0.1 \cdot H_w = 1.72\ ft$ Hight of Footing

$H_3 := H_w - \left(H_1 + H_2 + H_4\right) = 14.563\ ft$

$t_{stem} := 1.2162\ ft$ Total Width of Stem

$B := x_1 + t_{stem} + x_3 = 6.63\ ft$

$\gamma_c := 145\ pcf$ Unit Weight of Concrete

$\gamma_1 := 110\ pcf$ Unit Weight of Engineered Backfill

$\phi'_1 := 45\ °$ Angle of Friction of Engineered Backfill

$\gamma_2 := 130\ pcf$ Saturated Unit Weight of Engineered Backfill $\beta := 0\ °$

$\phi'_2 := 45\ °$ Angle of Friction of Engineered Backfill $c'_2 := 0\ psf$

$w_1 := x_1 \cdot H_3 \cdot \gamma_1 = 6.408\ klf$

$w_2 := t_{stem} \cdot \left(H_w - H_4\right) \cdot \gamma_c = 2.73\ klf$

$w_3 := x_3 \cdot \left(H_5 - H_4\right) \cdot \gamma_2 = 0.327\ klf$

$w_4 := B \cdot H_4 \cdot \gamma_c = 1.654\ klf$

$x'_1 := \dfrac{x_1}{2} + t_{stem} + x_3 = 4.63\ ft$

$x'_2 := \dfrac{t_{stem}}{2} + x_3 = 2.022\ ft$

$x'_3 := \dfrac{x_3}{2} = 0.707\ ft$

$x'_4 := \dfrac{B}{2} = 3.315\ ft$

$$K_a := \cos(\beta) \cdot \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi'_1)^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi'_1)^2}} = 0.172$$

$P_a := \dfrac{1}{2} \cdot \gamma_1 \cdot {H_w}^2 \cdot K_a = 2.792\ \dfrac{kip}{ft}$

$z_{bara} := \dfrac{H_w}{3} = 5.733\ ft$

$P_H := P_a \cdot \cos(\beta) = 2.792\ \dfrac{kip}{ft}$

$P_v := P_a \cdot \sin(\beta) = 0\ \dfrac{kip}{ft}$

$\Sigma M_o := P_H \cdot z_{bara} = 16.006\ kip$

$\Sigma M_R := w_1 \cdot x'_1 + w_2 \cdot x'_2 + w_3 \cdot x'_3 + w_4 \cdot x'_4 = 40.905\ kip$

$FS_o := \dfrac{\Sigma M_R}{\Sigma M_o} = 2.556$ $FS_o \geq 2-3$ Yes, OK! Overturning Factor of Safety

$F_{max} := \left(w_1 + w_2 + w_3 + w_4\right) \cdot \tan(\phi'_2) + B \cdot \dfrac{1}{2} \cdot c'_2 = 11.119\ klf$

$FS_V := \dfrac{F_{max}}{P_H} = 3.983$ $FS_V \geq 1.5$ Yes, OK! Sliding Factor of Safety

$$X_{barR} := \frac{\Sigma M_R - \Sigma M_o}{w_1 + w_2 + w_3 + w_4} = 2.239\ ft$$

$$e := \frac{B}{2} - X_{barR} = 1.076\ ft$$ $e < var$ Yes, OK! Check for Uplfit

$$var := \frac{B}{6} = 1.105\ ft$$

$s_c := 1$
$s_q := 1$
$s_\gamma := 1$

$d_c := 1.00$
$d_q := 1.00$
$d_\gamma := 1.00$

$$N_q := e^{\pi \cdot \tan(\phi'_2)} \cdot \tan\left(45\ ° + \frac{\phi'_2}{2}\right)^2 = 134.874$$

$$N_c := \frac{N_q - 1}{\tan(\phi'_2)} = 133.874$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_2) = 271.748$$

$L_w := 10\ ft$

$$m := \frac{2 + \frac{B}{L_w}}{1 + \frac{B}{L_w}} = 1.601$$

$H_i := P_H = 2.792\ klf$

$$den := \frac{F_{max}}{\tan(\phi'_2)} = 11.119\ klf$$

$$i_q := \left(1 - \frac{H_i}{den}\right)^m = 0.629$$

$$i_\gamma := \left(1 - \frac{H_i}{den}\right)^{m+1} = 0.471$$

$$i_c := i_q - \left(\frac{1.0 - i_q}{N_q - 1}\right) = 0.627$$

$B' := B - 2 \cdot e = 4.479\ ft$

$$q'_N := c'_2 \cdot N_c \cdot d_c \cdot i_c + \gamma_1 \cdot H_4 \cdot N_q \cdot d_q \cdot i_q + \frac{1}{2} \cdot \gamma_2 \cdot B' \cdot N_\gamma \cdot d_\gamma \cdot i_\gamma = 370.505 \ psi$$

$$q' := \frac{(w_1 + w_2 + w_3 + w_4) + \gamma_1 \cdot x_3 \cdot (H_4 - H_5)}{B'} = 16.81 \ psi$$

$FS_q := \frac{q'_N}{q'} = 22.041$ $FS_q \geq 3$ Yes, OK! Bearing Capacity Factor of Safety

***Required Concrete Volume for the abutment***

Abutment Wall Footing:

$B_{wallfooting} := 6.25\ ft$ Base of abutment wall footing

$H_{wallfooting} := 1.75\ ft$ Height of abutment wall footing

$L_{wallfooting} := 20.1667\ ft$ Length of abutment wall footing

$V_{wallfooting} := B_{wallfooting} \cdot H_{wallfooting} \cdot L_{wallfooting} = 8.169\ yd^3$ Volume of abutment wall footing

Abutment Wall Stem (Part 1 - Skinny Portion):

$B_{wallheel} := 0.75\ ft$ Base of abutment wall stem (part 1)

$H_{wallheel} := 15.5\ ft$ Height of abutment wall stem (part 1)

$L_{wallheel} := 20.1667\ ft$ Length of abutment wall stem (part 1)

$V_{wallheel} := B_{wallheel} \cdot H_{wallheel} \cdot L_{wallheel} = 8.683\ yd^3$ Volume of abutment wall stem (part 1)

Abutment Wall Stem (Part 2 - Thick Portion):

$B_{walltoe} := 1.5\ ft$ Base of abutment wall stem (part 2)

$H_{walltoe} := 14.0\ ft$ Height of abutment wall stem (part 2)

$L_{walltoe} := 20.1667\ ft$ Length of abutment wall stem (part 2)

$V_{walltoe} := B_{walltoe} \cdot H_{walltoe} \cdot L_{walltoe} = 15.685\ yd^3$ Volume of abutment wall stem (part 1)

Wing Wall Base (Part 1 - Thick Portion):

$B_{wingheel} := 4.0\ ft$ Base of wing wall footing (Part 1)

$H_{wingheel} := 1.75\ ft$ Height of wing wall footing (Part 1)

$L_{wingheel} := 11.1667\ ft$ Length of wing wall footing (Part 1)

$V_{wingheel} := B_{wingheel} \cdot H_{wingheel} \cdot L_{wingheel} = 2.895\ yd^3$ Volume of wing wall footing (Part 1)

$N := 2$ Number of Part 1 Bases

$V_{wingheel} := N \cdot V_{wingheel} = 5.79\ yd^3$ Total volume of Part 1 Base

Wing Wall Base (Part 2 - Stem Portion):

$B_{wingstem} := 1.583\ ft$ Base of wing wall footing (Part 2)

$H_{wingstem} := 1.75\ ft$ Height of wing wall footing (Part 2)

$L_{wingstem} := 11.583\ ft$ Length of wing wall footing (Part 2)

$V_{wingstembase} := B_{wingstem} \cdot H_{wingstem} \cdot L_{wingstem} = 1.188\ yd^3$ Volume of wing wall footing (Part 2)

$N := 2$ Number of Part 2 Bases

$V_{wingstembase} := N \cdot V_{wingstembase} = 2.377\ yd^3$ Total volume of Part 2 Base

Wing Wall Base (Part 3 - Skinny Portion):

$B_{wingtoe} := 1.417\ ft$ Base of wing wall footing (Part 3)

$H_{wingtoe} := 1.75\ ft$ Height of wing wall footing (Part 3)

$L_{wingtoe} := 13.0\ ft$ Length of wing wall footing (Part 3)

$V_{wingtoe} := B_{wingtoe} \cdot H_{wingtoe} \cdot L_{wingtoe} = 1.194\ yd^3$ Volume of wing wall footing (Part 3)

$N := 2$ Number of Part 3 Bases

$V_{wingtoe} := N \cdot V_{wingtoe} = 2.388\ yd^3$ Total volume of Part 3 Base

Wing Wall Stem:

$B_{wingstem} := 2.25\ ft$

$H_{wingstem} := 15.5\ ft$

$L_{wingstem} := 10.0\ ft$

$V_{wingstem} := B_{wingstem} \cdot H_{wingstem} \cdot L_{wingstem} = 12.917\ yd^3$

$N := 2$

$V_{wingstem} := N \cdot V_{wingstem} = 25.833\ yd^3$

## Weep Holes:

$r_{weep} := \frac{0.25}{2}\ ft$ Radius of weep holes

$A_{weep} := \pi \cdot r_{weep}^{\ 2} = 0.049\ ft^2$ Area of weep holes

$L_{weep} := 2.25\ ft$ Length of weep holes

$V_{weep} := A_{weep} \cdot L_{weep} = 0.004\ yd^3$ Volume of weep holes

$N_{weep} := 8$ Number of weep holes

$V_{weep} := V_{weep} \cdot N = 0.008\ yd^3$ Total volume of weep holes

## Total Volume of Concrete in Abutment Wall and Its Wing Walls:

$V_T := V_{wallfooting} + V_{wallheel} + V_{walltoe} + V_{wingheel} + V_{wingstembase} + V_{wingtoe} + V_{wingstem} - V_{weep}$

$N := 2$ Number of abutments with wing walls

$V_T := N \cdot V_T = 137.835\ yd^3$

## Abutment Reinforcemnt Design:

WALL

DESIGN ABUTMENT STEM

$f'_c := 4\ ksi \qquad f_y := 60\ ksi$

Assume #7 Rebar

$D_{bar} := 0.875\ in \qquad A_{bar} := 0.6\ in^2$

$b := 1\ ft \qquad h := 1.72\ ft \qquad t_{STEM} := 1.72\ ft \qquad cover := 2.5\ in$

$\phi := .9 \qquad \beta_1 := .85$

$w_1 := 0.121\ kip \qquad x'_1 := 4.72\ ft$

$w_2 := 0.115\ kip \qquad x'_2 := 4.72\ ft$

$w_3 := 3.204\ kip \qquad x'_3 := 4.72\ ft$

$w_4 := 0.494\ kip \qquad x'_4 := 3.61\ ft$

$w_5 := 3.276\ kip \qquad x'_5 := 2.75\ ft$

$w_6 := 0.463\ kip \qquad x'_6 := 1\ ft$

$w_7 := 1.427\ kip \qquad x'_7 := 2.86\ ft$

$w_{bridge} := 5.249\ kip \qquad x'_{bridge} := 2.75\ ft$

$M_{stem} := w_1 \cdot x'_1 + w_2 \cdot x'_2 + w_3 \cdot x'_3 + w_4 \cdot x'_4 + w_5 \cdot x'_5 + w_{bridge} \cdot x'_{bridge} = 41.464\ ft \cdot kip$

<u>Design For Flexure</u>

$I_g := \frac{1}{12} (b) (h)^3 = 8792.838\ in^4$

$f_r := .24 \cdot \left(\sqrt{4}\right)\ ksi = 0.48\ ksi$

$y_t := \frac{h}{2} = 10.32\ in$

$M_{cr} := \frac{f_r \cdot I_g}{y_t} = 34.081\ kip \cdot ft \qquad 1.2 \cdot M_{cr} = 40.897\ kip \cdot ft$

$$M_{STEMdes} := \min\left(1.33 \cdot M_{stem}, 1.2 \cdot M_{cr}\right) = 40.8969 \; kip \cdot ft$$

Effective Depth

$$d_e := t_{STEM} - cover - \frac{D_{bar}}{2} = 17.703 \; in$$

Solve required amount of reinforcing

$$R_n := \frac{M_{STEMdes}}{\phi \cdot b \cdot {d_e}^2} = 0.145 \; ksi$$

$$\rho := .85 \left(\frac{f'_c}{f_y}\right) \left(1 - \sqrt{1 - \frac{2 \cdot R_n}{.85 \cdot f'_c}}\right) = 0.002$$

$$A_s := \rho \cdot b \cdot d_e = 0.525 \; in^2$$

Required Bar Spacing

$$Req_{spacing} := \frac{A_{bar}}{\frac{A_s}{12 \; in}} = 13.719 \; in$$

Use #7 @ 10" $\quad bar_{space} := 10 \; in$

$$A_s := A_{bar} \cdot \frac{12 \; in}{bar_{space}} = 0.72 \; in^2$$

Check max reinforcement limit

$$T := A_s \cdot f_y = 43.2 \; kip$$

$$a := \frac{T}{.85 \cdot f'_c \cdot b} = 1.059 \; in$$

$$c := \frac{a}{\beta_1} = 1.246 \; in$$

$$\frac{c}{d_e} \le 0.42 = 1$$ OK

Check Crack Control

$Z := 130\ \frac{kip}{in}$ (assume abutment will be exposed to deicing salts)

$d_c := 2\ in + \frac{D_{bar}}{2} = 2.438\ in$

$A_c := 2 \cdot (d_c) \cdot bar_{space} = 48.75\ in^2$

$f_{sa} := \frac{Z}{(d_c \cdot A_c)^{\frac{1}{3}}} = 26.443\ ksi$

$f_{sa} \le .6 \cdot f_y = 1$

$E_c := 1820 \cdot (\sqrt{4})\ ksi = 3640\ ksi$ $E_s := 29000\ ksi$

$n := \frac{E_s}{E_c} = 7.967$

$n := 8$

Design for Shear

$V_{stem} := w_1 + w_2 + w_3 + w_4 + w_5 + w_{bridge} = 12.459\ kip$

$\beta := 2.0$ $b_v := 12\ in$ $\phi := .90$

Calculate nominal shear resistance

$d_v := \max\left(d_e - \frac{a}{2}, .9 \cdot d_e, .72 \cdot h\right) = 17.173\ in$

$v_{n1} := .0316 \cdot \beta \cdot (\sqrt{4})\ ksi \cdot b_v \cdot d_v = 26.048\ kip$

$v_{n2} := .25 \cdot f'_c \cdot b_v \cdot d_v = 206.077\ kip$

$V_n := \min(v_{n1}, v_{n2}) = 26.048\ kip$

$V_r := \phi \cdot V_n = 23.443\ kip$

$V_r \geq V_{stem} = 1$ OK

Shrinkage and Temperature Reinforcment

$A_g := b \cdot h = 247.68\ in^2$

$.0015 \cdot A_g = 0.372\ in^2$

$A_s := 2 \cdot A_{bar} \cdot \left(\frac{12\ in}{6\ in}\right) = 2.4\ in^2$

$A_s \geq .0015 \cdot A_g = 1$ OK

#7 @ 10" will be used for the back face flexure reinforcement. The same bar size and spacing will be used for the front face vertical reinforcement to reduce design steps. The horizontal temperature and shrinkage reinforcement will consist of #7 @ 10" for the front and back faces.

DESIGN ABUTMENT FOOTING

$f'_c := 4\ ksi$ $f_y := 60\ ksi$

Assume #7 Rebar

$D_{bar} := 0.875\ in$ $A_{bar} := 0.6\ in^2$

$b := 1\ ft$ $h := 1.72 \cdot ft$ $t_{footing} := 2 \cdot ft$ $cover := 2.5\ in$

$\phi := .9$ $\beta_1 := .85$

$$M_{footing} := w_1 \cdot x'_1 + w_2 \cdot x'_2 + w_3 \cdot x'_3 + w_4 \cdot x'_4 + w_5 \cdot x'_5 + w_6 \cdot x'_6 + w_7 \cdot x'_7 + w_{bridge} \cdot x'_{bridge} = 46.008\ ft \cdot kip$$

Design For Flexure

$I_g := \frac{1}{12}(b)(h)^3 = 8792.838\ in^4$

$f_r := .24 \cdot \left(\sqrt{4}\right)\ ksi = 0.48\ ksi$

$y_t := \frac{h}{2} = 10.32\ in$

$$M_{cr} := \frac{f_r \cdot I_g}{y_t} = 34.081\ kip \cdot ft \qquad 1.2 \cdot M_{cr} = 40.897\ kip \cdot ft$$

$$M_{Fdes} := \min\left(1.33 \cdot M_{footing}, 1.2 \cdot M_{cr}\right) = 40.8969\ kip \cdot ft$$

Effective Depth

$$d_e := t_{footing} - cover - \frac{D_{bar}}{2} = 21.063\ in$$

Solve required amount of reinforcing

$$R_n := \frac{M_{Fdes}}{\phi \cdot b \cdot {d_e}^2} = 0.102\ ksi$$

$$\rho := .85 \left(\frac{f'_c}{f_y}\right)\left(1 - \sqrt{1 - \frac{2 \cdot R_n}{.85 \cdot f'_c}}\right) = 0.002$$

$$A_s := \rho \cdot b \cdot d_e = 0.438\ in^2$$

Required Bar Spacing

$$Req_{spacing} := \frac{A_{bar}}{\dfrac{A_s}{12\ in}} = 16.431\ in$$

Use #7 @ 10" $\qquad bar_{space} := 10\ in$

$$A_s := A_{bar} \cdot \frac{12\ in}{bar_{space}} = 0.72\ in^2$$

Check max reinforcement limit

$$T := A_s \cdot f_y = 43.2\ kip$$

$$a := \frac{T}{.85 \cdot f'_c \cdot b} = 1.059\ in$$

$$c := \frac{a}{\beta_1} = 1.246\ in$$

$$\frac{c}{d_e} \le 0.42 = 1 \qquad \text{OK}$$

Check Crack Control

$$Z := 130\ \frac{kip}{in} \qquad \text{(assume abutment will be exposed to deicing salts)}$$

$$d_c := 2\ in + \frac{D_{bar}}{2} = 2.438\ in$$

$$A_c := 2 \cdot (d_c) \cdot bar_{space} = 48.75\ in^2$$

$$f_{sa} := \frac{Z}{(d_c \cdot A_c)^{\frac{1}{3}}} = 26.443\ ksi$$

$$f_{sa} \le .6 \cdot f_y = 1$$

$$E_c := 1820 \cdot \left(\sqrt{4}\right)\ ksi = 3640\ ksi \qquad E_s := 29000\ ksi$$

$$n := \frac{E_s}{E_c} = 7.967$$

$$n := 8$$

Design for Shear

$$V_{footing} := w_1 + w_2 + w_3 + w_4 + w_5 + w_6 + w_7 + w_{bridge} = 14.349\ kip$$

$$\beta := 2.0 \qquad b_v := 12\ in \qquad \phi := .90$$

Calculate nominal shear resistance

$$d_v := \max\left(d_e - \frac{a}{2}, .9 \cdot d_e, .72 \cdot h\right) = 20.533\ in$$

$$v_{n1} := .0316 \cdot \beta \cdot \left(\sqrt{4}\right)\ ksi \cdot b_v \cdot d_v = 31.145\ kip$$

$v_{n2} := .25 \cdot f'_c \cdot b_v \cdot d_v = 246.397\ kip$

$V_n := \min\left(v_{n1}, v_{n2}\right) = 31.145\ kip$

$V_r := \phi \cdot V_n = 28.03\ kip$

$V_r \geq V_{footing} = 1$ OK

Shrinkage and Temperature Reinforcment

$A_g := b \cdot h = 247.68\ in^2$

$.0015 \cdot A_g = 0.372\ in^2$

$A_s := 2 \cdot A_{bar} \cdot \left(\frac{12\ in}{6\ in}\right) = 2.4\ in^2$

$A_s \geq .0015 \cdot A_g = 1$ OK

#7 @ 10" will be used for the back face flexure reinforcement. The same bar size and spacing will be used for the front face vertical reinforcement to reduce design steps. The horizontal temperature and shrinkage reinforcement will consist of #5 @ 6" for the front and back faces.

WING

DESIGN WING STEM

$f'_c := 4\ ksi$ $f_y := 60\ ksi$

Assume #7 Rebar

$D_{bar} := 0.875\ in$ $A_{bar} := 0.6\ in^2$

$b := 1\ ft$ $h := 1.2162\ ft$ $t_{STEM} := h$ $cover := 2.5\ in$

$\phi := .9$ $\beta_1 := .85$

$w_1 := 6.408\ kip$ $x'_1 := 4.63\ ft$

$w_2 := 2.73\ kip$ $x'_2 := 2.022\ ft$

$w_3 := 0.327\ kip$ $x'_3 := .707\ ft$

$w_4 := 1.654\ kip$

$$M_{stem} := w_1 \cdot x'_1 + w_2 \cdot x'_2 + w_3 \cdot x'_3 = 35.42\ ft \cdot kip$$

Design For Flexure

$$I_g := \frac{1}{12} (b)\ (h)^3 = 3108.556\ in^4$$

$$f_r := .24 \cdot \left(\sqrt{4}\right)\ ksi = 0.48\ ksi$$

$$y_t := \frac{h}{2} = 7.297\ in$$

$$M_{cr} := \frac{f_r \cdot I_g}{y_t} = 17.04\ kip \cdot ft \qquad 1.2 \cdot M_{cr} = 20.448\ kip \cdot ft$$

$$M_{STEMdes} := \min\left(1.33 \cdot M_{stem}, 1.2 \cdot M_{cr}\right) = 20.4477\ kip \cdot ft$$

Effective Depth

$$d_e := t_{STEM} - cover - \frac{D_{bar}}{2} = 11.657\ in$$

Solve required amount of reinforcing

$$R_n := \frac{M_{STEMdes}}{\phi \cdot b \cdot {d_e}^2} = 0.167\ ksi$$

$$\rho := .85 \left(\frac{f'_c}{f_y}\right) \left(1 - \sqrt{1 - \frac{2 \cdot R_n}{.85 \cdot f'_c}}\right) = 0.003$$

$$A_s := \rho \cdot b \cdot d_e = 0.4\ in^2$$

Required Bar Spacing

$$Req_{spacing} := \frac{\frac{A_{bar}}{A_s}}{12\ in} = 18.005\ \frac{1}{ft^2} \cdot in$$

Use #7 @ 10" $\quad bar_{space} := 10\ in$

$$A_s := A_{bar} \cdot \frac{12\ in}{bar_{space}} = 0.72\ in^2$$

Check max reinforcement limit

$$T := A_s \cdot f_y = 43.2\ kip$$

$$a := \frac{T}{.85 \cdot f'_c \cdot b} = 1.059\ in$$

$$c := \frac{a}{\beta_1} = 1.246\ in$$

$$\frac{c}{d_e} \le 0.42 = 1$$ OK

Check Crack Control

$$Z := 130\ \frac{kip}{in}$$ (assume abutment will be exposed to deicing salts)

$$d_c := 2\ in + \frac{D_{bar}}{2} = 2.438\ in$$

$$A_c := 2 \cdot \left(d_c\right) \cdot bar_{space} = 48.75\ in^2$$

$$f_{sa} := \frac{Z}{\left(d_c \cdot A_c\right)^{\frac{1}{3}}} = 26.443\ ksi$$

$$f_{sa} \le .6 \cdot f_y = 1$$

$$E_c := 1820 \cdot \left(\sqrt{4}\right)\ ksi = 3640\ ksi \qquad E_s := 29000\ ksi$$

$$n := \frac{E_s}{E_c} = 7.967$$

$$n := 8$$

Design for Shear

$V_{stem} := w_1 + w_2 + w_3 = 9.465\ kip$

$\beta := 2.0$ $\quad b_v := 12\ in$ $\quad \phi := .90$

Calculate nominal shear resistance

$$d_v := \max\left(d_e - \frac{a}{2}, .9 \cdot d_e, .72 \cdot h\right) = 11.127\ in$$

$v_{n1} := .0316 \cdot \beta \cdot \left(\sqrt{4}\right)\ ksi \cdot b_v \cdot d_v = 16.878\ kip$

$v_{n2} := .25 \cdot f'_c \cdot b_v \cdot d_v = 133.53\ kip$

$V_n := \min\left(v_{n1}, v_{n2}\right) = 16.878\ kip$

$V_r := \phi \cdot V_n = 15.19\ kip$

$V_r \geq V_{stem} = 1$ OK

Shrinkage and Temperature Reinforcment

$A_g := b \cdot h = 175.133\ in^2$

$.0015 \cdot A_g = 0.263\ in^2$

$$A_s := 2 \cdot A_{bar} \cdot \left(\frac{12\ in}{6\ in}\right) = 2.4\ in^2$$

$A_s \geq .0015 \cdot A_g = 1$ OK

#7 @ 10" will be used for the back face flexure reinforcement. The same bar size and spacing will be used for the front face vertical reinforcement to reduce design steps. The horizontal temperature and shrinkage reinforcement will consist of #5 @ 6" for the front and back faces.

DESIGN ABUTMENT FOOTING

$f'_c := 4\ ksi$ $\quad f_y := 60\ ksi$

Assume #7 Rebar

$D_{bar} := 0.875\ in$ $\quad A_{bar} := 0.6\ in^2$

$b := 1\ ft$ $h := 1.72 \cdot ft$ $t_{footing} := h$ $cover := 2.5\ in$

$\phi := .9$ $\beta_1 := .85$

$M_{footing} := w_1 \cdot x'_1 + w_2 \cdot x'_2 + w_3 \cdot x'_3 + w_4 \cdot x'_4 = 41.391\ ft \cdot kip$

<u>Design For Flexure</u>

$$I_g := \frac{1}{12}(b)(h)^3 = 8792.838\ in^4$$

$$f_r := .24 \cdot \left(\sqrt{4}\right)\ ksi = 0.48\ ksi$$

$$y_t := \frac{h}{2} = 10.32\ in$$

$$M_{cr} := \frac{f_r \cdot I_g}{y_t} = 34.081\ kip \cdot ft \qquad 1.2 \cdot M_{cr} = 40.897\ kip \cdot ft$$

$$M_{Fdes} := \min\left(1.33 \cdot M_{footing}, 1.2 \cdot M_{cr}\right) = 40.8969\ kip \cdot ft$$

Effective Depth

$$d_e := t_{footing} - cover - \frac{D_{bar}}{2} = 17.703\ in$$

Solve required amount of reinforcing

$$R_n := \frac{M_{Fdes}}{\phi \cdot b \cdot {d_e}^2} = 0.145\ ksi$$

$$\rho := .85\left(\frac{f'_c}{f_y}\right)\left(1 - \sqrt{1 - \frac{2 \cdot R_n}{.85 \cdot f'_c}}\right) = 0.002$$

$$A_s := \rho \cdot b \cdot d_e = 0.525\ in^2$$

Required Bar Spacing

$$Req_{spacing} := \frac{A_{bar}}{\dfrac{A_s}{12\ in}} = 13.719\ in$$

Use #7 @ 10" $bar_{space} := 10\ in$

$$A_s := A_{bar} \cdot \frac{12\ in}{bar_{space}} = 0.72\ in^2$$

Check max reinforcement limit

$$T := A_s \cdot f_y = 43.2\ kip$$

$$a := \frac{T}{.85 \cdot f'_c \cdot b} = 1.059\ in$$

$$c := \frac{a}{\beta_1} = 1.246\ in$$

$$\frac{c}{d_e} \le 0.42 = 1$$ OK

Check Crack Control

$$Z := 130\ \frac{kip}{in}$$ (assume abutment will be exposed to deicing salts)

$$d_c := 2\ in + \frac{D_{bar}}{2} = 2.438\ in$$

$$A_c := 2 \cdot (d_c) \cdot bar_{space} = 48.75\ in^2$$

$$f_{sa} := \frac{Z}{(d_c \cdot A_c)^{\frac{1}{3}}} = 26.443\ ksi$$

$$f_{sa} \le .6 \cdot f_y = 1$$

$$E_c := 1820 \cdot \left(\sqrt{4}\right)\ ksi = 3640\ ksi$$ $$E_s := 29000\ ksi$$

$$n := \frac{E_s}{E_c} = 7.967$$

$n := 8$

Design for Shear

$V_{footing} := w_1 + w_2 + w_3 + w_4 = 11.119 \; kip$

$\beta := 2.0 \qquad b_v := 12 \; in \qquad \phi := .90$

Calculate nominal shear resistance

$$d_v := \max\left(d_e - \frac{a}{2}, .9 \cdot d_e, .72 \cdot h\right) = 17.173 \; in$$

$v_{n1} := .0316 \cdot \beta \cdot \left(\sqrt{4}\right) \; ksi \cdot b_v \cdot d_v = 26.048 \; kip$

$v_{n2} := .25 \cdot f'_c \cdot b_v \cdot d_v = 206.077 \; kip$

$V_n := \min\left(v_{n1}, v_{n2}\right) = 26.048 \; kip$

$V_r := \phi \cdot V_n = 23.443 \; kip$

$V_r \geq V_{footing} = 1$ OK

Shrinkage and Temperature Reinforcment

$A_g := b \cdot h = 247.68 \; in^2$

$.0015 \cdot A_g = 0.372 \; in^2$

$$A_s := 2 \cdot A_{bar} \cdot \left(\frac{12 \; in}{6 \; in}\right) = 2.4 \; in^2$$

$A_s \geq .0015 \cdot A_g = 1$ OK

#7 @ 10" will be used for the back face flexure reinforcement. The same bar size and spacing will be used for the front face vertical reinforcement to reduce design steps. The horizontal temperature and shrinkage reinforcement will consist of #5 @ 6" for the front and back faces.

## Abutment and Approach Cut/Fill Calculations:

### South Abutment:

$l_{southabut} := 37.392\ ft$ Length of S. Abut. derived from design dimensions.

$w_{southabut} := 13.279\ ft$ Width of S. Abut. derived from design dimensions.

$A_{pvsouthabut} := 18.204\ ft \cdot 6\ ft = 109.224\ ft^2$ Area of plan view of S. Abut. derived from contours width of S. Abut.

$V_{cutsouthabut} := A_{pvsouthabut} \cdot l_{southabut} = 4084\ ft^3$

$V_{cutsouthabut} := \frac{4084}{27} = 151$

### South Abutment Approach:

**Straight Portion of Approach:**

$l_{southapp} := 87.686\ ft$ Length of S. Approach derived from existing contours.

$w_{southapp} := 10.2\ ft$ Width of S. Approach derived from S. Abut. Design dimensions.

$A_{southapp} := \frac{1}{2} \cdot l_{southapp} \cdot w_{southapp} = 447\ ft^2$ Area of S. Approach derived from existing contours and S. Abut. design dimensions.

$V_{fillsouthapp} := A_{southapp} \cdot 37.392\ ft = 16722\ ft^3$

$V_{fillsouthapp} := \frac{16722}{27} = 619$

**Traingles on Sides of South Approach:**

$A_{nssabutapp} := \frac{1}{2} \cdot 170\ ft \cdot 10.2\ ft = 867\ ft^2$

Area of North end of S. Abut. approach (West Side of trail).

$A_{sssabutapp} := \frac{1}{2} \cdot 20\ ft \cdot 1.2\ ft = 12\ ft^2$

Area of South end of S. Abut. approach (West Side of Trail).

$A_{avgssabut} := \frac{A_{nssabutapp} + A_{sssabutapp}}{2} = 439.5\ ft^2$

Avg area of both ends of West Side of trail.

$V_{fsnorthabut} := 2 \cdot A_{avgssabut} \cdot l_{southapp} = 77076\ ft^3$

Volume of West Side of trail of S. Abut. approach (x2 to account for East side of trail).

$V_{fsnorthabut} := \frac{77076}{27} = 2855$

**<u>North Abutment:</u>**

$A_{pvnorthabut} := A_{pvsouthabut} = 109.224\ ft^2$

Area of plan view of N. Abut. derived from contours width of N. Abut.

$V_{fillnorthabut} := A_{pvnorthabut} \cdot (779\ ft - 765\ ft) = 1529\ ft^3$

$V_{fillnorthabut} := \frac{1529}{27} = 57$

## North Abutment Approach:

### Straight Portion of Approach in Lake:

$l_{northapplake} := 156.05\ ft$ Length of N. Approach derived from existing contours.

$w_{northapp} := 37.392\ ft$ Width of N. Approach derived from N. Abut. Design dimensions.

$A_{northapplake} := l_{northapplake} \cdot w_{northapp} = 5835\ ft^2$ Area of N. Approach under lake derived from existing contours and N. Abut. design dimensions.

$$V_{fillnorthapplake} := A_{northapplake} \cdot (779\ ft - (780\ ft - 15\ ft)) = 81690\ ft^3$$

$$V_{fillnorthapplake} := \frac{81690}{27} = 3026$$

### Straight Portion of Approach:

$A_{northapp} := 10\ ft \cdot 240\ ft = 2400\ ft^2$ Area of N. Approach derived from existing contours and N. Abut. design dimensions.

$$V_{fillnorthapp} := A_{northapp} \cdot 37.392\ ft = 89741\ ft^3$$

$$V_{fillnorthapp} := \frac{89741}{27} = 3324$$

**Traingles on Sides of North Approach:**

$$A_{nsnabutapp} := \frac{1}{2} \cdot 20\ ft \cdot 1\ ft = 10\ ft^2$$

Area of North end of N. Abut. approach (West Side of trail).

$$A_{ssnabutapp} := \frac{1}{2} \cdot 620\ ft \cdot 31\ ft = 9610\ ft^2$$

Area of South end of N. Abut. approach (West Side of Trail).

$$A_{avgsnabut} := \frac{A_{nsnabutapp} + A_{ssnabutapp}}{2} = 4810\ ft^2$$

Avg area of both ends of West Side of trail.

$$V_{fsnorthabut} := 2 \cdot A_{avgsnabut} \cdot 240\ ft = 2308800\ ft^3$$

Volume of West Side of trail of N. Abut. approach (x2 to account for East side of trail).

$$V_{fsnorthabut} := \frac{2308800}{27} = 85511$$

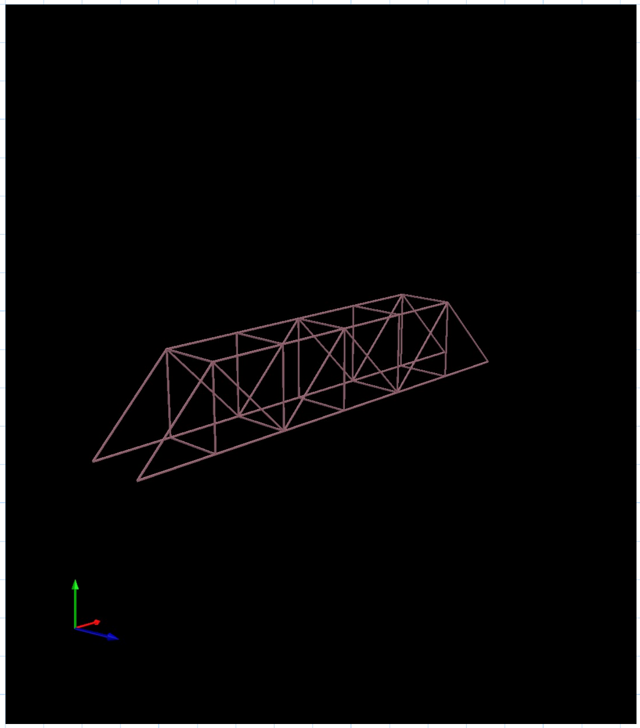
$$F_{net} := 85511 + 3324 + 3026 + 57 + 2855 + 619 - 151$$

$F_{net} = 95241$ cubic yards — Fill required for abutments prior to use of tril cut quantities.

$C_{net} := 73981$ cubic yards — Net cut of trail system.

$F_{net} := F_{net} - C_{net} = 21260$ — Total fill required for Red Bridge abutments.

## Red Bridge Geometry



$Span := 120\ ft \qquad \delta := 45\ °$

$Spantop := \dfrac{Span}{\dfrac{6}{4}} = 80\ ft$

$H := \dfrac{Span}{6} \cdot \tan(\delta) = 20\ ft$

$W := 15\ ft$

Girder estimation

$bf := \dfrac{W}{20} = 9\ in$

Assume W10x49 A36 steel

$A := 14.4\ in^2 \qquad d := 10 \qquad bf := 10 \qquad wt := 49\ \dfrac{lb}{ft}$

$Lg1 := \sqrt{(H^2) + H^2} = 28.284\ ft$

$Topwt := (Lg1 \cdot wt \cdot 2 + H \cdot wt \cdot 4)\ 2 = 13383.717\ lb$

## CLEARING AND GRUBBING AREA ESTIMATES

### *Area of Clearing and Grubbing*

#### For Quarry Springs Park Trail:

$d_1 := 1350\ ft$
$d_2 := 200\ ft$

$A_1 := d_1 \cdot d_2 = 6.198\ acre$ Medium Trees

$d_3 := 800\ ft$
$d_4 := 600\ ft$

$A_2 := d_3 \cdot d_4 = 11.019\ acre$ Small Trees

#### For Abutments:

$d_5 := 300\ ft$
$d_6 := 100\ ft$

$A_3 := d_5 \cdot d_6 = 0.689\ acre$ Medium Trees

***Red Bridge Deck Cost Estimate:***

$w := 7.5\ in$ Width of Single Board

$Cost := 70$ Cost per 4" x 8" x 16'

$Length := 120\ ft$ Total Length of Red Bridge

$s := \frac{1}{8}\ in$ Minimum Spacing Between Each Board

$N_1 := \frac{Length}{w} = 192$ Number of Boards Without Minimum Spacing

$x := 191 \cdot \frac{1}{8}\ in = 2\ ft$ Total Spacing Required Over Total Length of Red Bridge

$Length := Length - x = 118.01\ ft$ Total Distance to be Spanned By Boards

$N_2 := \frac{Length}{w} = 189$ Number of Boards Required with Minimum Spacing

$C_T := Cost \cdot N_2 = 13217$

$C_T = \$13217$ Total Cost of Lumber

***Sandblasting, Paint, and Primer Quantity Estimate:***

$High := 12.63$ High Cost per Square Foot

$Low := 6.25$ Low Cost per Square Foot

$Cost := \frac{High + Low}{2} = 9.44$ Median of High Cost and Low Cost per Square Foot

$W_{bridge} := 47040\ lb = 23.52\ ton$ Weight of the Red Bridge

$A := 155\ \frac{ft^2}{ton}$ Estimated Surface Area per Ton of Steel

$SA := W_{bridge} \cdot A = 3646\ ft^2$ Estimated Surface Area of Red Bridge Based on Weight

$T_c := Cost \cdot SA = 34414\ ft^2$

$T_{cost} = \$34414$ Total Cost to Sandblast, Paint, and Prime the Red Bridge

## APPENDIX . B Cost Estimates

| Highway 117 Sidewalk Expansion Construction Cost Estimate | | | |
|---|---|---|---|
| **Item description** | **Quantity** | **Unit Price** | **Bid Amount** |
| Engineered backfill (CY) | 25 | 44.2 | $1,125 |
| Shoulder Finish (LF) | 8 | 84 | $675 |
| Removal of Fense (LF) | 320 | 3 | $965 |
| Brdige Approach (LF) | 50 | 191.5 | $9,575 |
| Concrete Pavement (SY) | 171.8519 | 60 | $10,500 |
| Structural Steel (lb) | 97920 | 2.1 | $206,000 |
| Performed Neoprene Joint (LF) | 20 | 354 | $7,100 |
| Neoprene Gland Installation (LF) | 20 | 58.85 | $1,175 |
| Railing (LF) | 320 | 124 | $40,000 |
| Ornamental Railing (LF) | 320 | 55 | $17,600 |
| Concrete Grinding (CY) | 53.33333 | 15 | $800 |
| Survey | lump | lump | $5,000 |
| Temp Barrier (LY) | 500 | 6.55 | $3,275 |
| Traffic Control | lump | lump | $9,000 |
| Flaggers (per peson) | 8 | 315 | $2,525 |
| Mobilization | lump | lump | $60,000 |
| Removable Tape Markings (LF) | 500 | 100 | $50,000 |
| Painted Markings (LF) | 500 | 35 | $17,500 |
| Seeding and Fertalizing (acre) | 0.15 | 3500 | $525 |
| Abutment Steel (lb) | 35000 | 2.07 | $72,500 |
| Structural Concrete (CY) | 75 | 974.3 | $73,000 |
| Structural Steel Brackets (lb) | 15000 | 2.1 | $31,500 |
| Structural Steel Solums (lb) | 436.5833 | 2.1 | $915 |
| | | | |
| | | **Total** | $621,255 |

| Downtown Sidewalk Construction Cost Estimate | | | |
|---|---|---|---|
| **Item description** | **Quantity** | **Unit Price** | **Total Amount** |
| Excavation & Fill | 338 (CY) | 7.6 | $2,575 |
| Concrete Pavement | 1490 (SY) | 20.5 | $30,600 |
| Removal of Pavment | 1490 (SY) | 6.5 | $9,700 |
| Traffic Control | lump | lump | $9,000 |
| Mobilization | lump | lump | $60,000 |
| Temp Barrier | 559 (LY) | 6.55 | $3,925 |
| Flaggers | 2 (per peson) | 315 | $630 |
| Removable Tape Markings | 1676 (LF) | 100 | $168,000 |
| Painted Markings | 1676 (LF) | 35 | $58,700 |
| Galvanized Corrugated Metal Culverts | 15 (LF) | 1025 | $15,400 |
| | | | |
| | | **Total** | $358,530 |

| Remaining Quarry Springs Park Trail Construction Cost Estimate | | | |
|---|---|---|---|
| **Item Description** | **Quantity** | **Unit Price** | **Total Amount** |
| Cut & Chip Medium Trees | 17.2 (Acre) | 5250 | $90,500 |
| Grub Stumps & Remove | 17.2 (Acre) | 3225 | $55,500 |
| Excavation & Fill | 278720 (B.C.Y.) | 7.00 | $1,951,000 |
| Plastic Netting | 10000 (S.Y.) | 2.39 | $24,000 |
| Subbase 6" Thick | 53550 (S.Y.) | 6.70 | $359,000 |
| Concrete Paving - 6" Thick | 53550 (S.Y.) | 24.50 | $1,312,000 |
| | | | |
| | | **Total** | $3,792,000 |

| Phase One: Quarry Springs Park Trail Construction Cost Estimate | | | |
|---|---|---|---|
| **Item Description** | **Quantity** | **Unit Price** | **Total Amount** |
| Cut & Chip Medium Trees | 6.89 (Acre) | 5250 | $36,000 |
| Grub Stumps & Remove | 6.89 (Acre) | 3225 | $22,300 |
| Excavation & Fill | 43212 (B.C.Y.) | 7.00 | $302,500 |
| Plastic Netting | 3360 (S.Y.) | 2.39 | $8,050 |
| Subbase 6" Thick | 4762 (S.Y.) | 6.70 | $32,000 |
| Concrete Paving - 6" Thick | 4762 (S.Y.) | 24.50 | $117,000 |
| | | | |
| | | **Total** | $517,850 |

| Red Bridge Abutment Construction Cost Estimate | | | |
|---|---|---|---|
| **Item Description** | **Quantity** | **Unit Price** | **Total Amount** |
| Cut & Chip Medium Trees | 1 (Acre) | 6875 | $4,125 |
| Grub Stumps & Remove | 1 (Acre) | 4300 | $2,600 |
| Plastic Netting, Stapled | 5640 (S.Y.) | 2.39 | $13,500 |
| Excavation & Fill | 42520 (B.C.Y.) | 7.6 | $323,500 |
| Steel Reinforcement | 18 (Ton) | 505 | $9,100 |
| Structural Concrete | 138 (C.Y.) | 515 | $71,100 |
| | | | |
| | | **Total** | $423,925 |

| Red Bridge Refurbishing Cost Estimate | | | |
|---|---|---|---|
| **Item Description** | **Quantity** | **Unit Price** | **Total Amount** |
| Sandblasting & Painting | 3646 (S.F.) | 9.44 | $34,500 |
| Deck Replacement | 189 (Ea.) | 70 | $13,300 |
| Railing System | 120 (L.F.) | 65 | $7,800 |
| Lighting | 4 (Ea.) | 320 | $1,300 |
| Rub Rail | 30 (Ea.) | 5 | $150 |
| Spot Repair Contingency | Lump | Lump | $70,000 |
| | | | |
| | | **Total** | $127,050 |

| Transportation of Red Bridge Cost Estimate | |
|---|---|
| **Item Description** | **Cost** |
| Lift Structure from Current Abutments onto Truck | $82,000 |
| Transportation of Slurcture to New Location | $50,000 |
| Placement of Structure on Bank From Truck | $82,000 |
| Placement of Structure on New Foundaiton | $82,000 |
| | |
| Total | $296,000 |

| Total Construction Cost Estimate w/ Phase 1 Trail System | |
|---|---|
| Element of Project | Total Cost |
| Quarry Springs Park Trail | $517,850 |
| Sidewalk Design | $358,530 |
| Red Bridge Abutments | $423,925 |
| Bridge Expansion | $621,255 |
| Red Bridge Refurbishing | $127,050 |
| Transportation of Red Bridge | $296,000 |
| | |
| **Total** | $2,344,610 |