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Title	Volga Pedestrian Bridge and Trail Engineering & Design
Completed By	Ryan McDonough, Nathan Gjersvik, Ryan Kowalsky, Spencer McDermott
Date Completed	May 2020
UI Department	Department of Civil & Environmental Engineering
Course Name	CEE:4850:0001 Project Design & Management
Instructor	Richard Fosse
Community Partners	City of Volga

This project was supported by the Iowa Initiative for Sustainable Communities (IISC), a community engagement program at the University of Iowa. IISC partners with rural and urban communities across the state to develop projects that university students and IISC pursues a dual mission of enhancing quality of life in Iowa while transforming teaching and learning at the University of Iowa.

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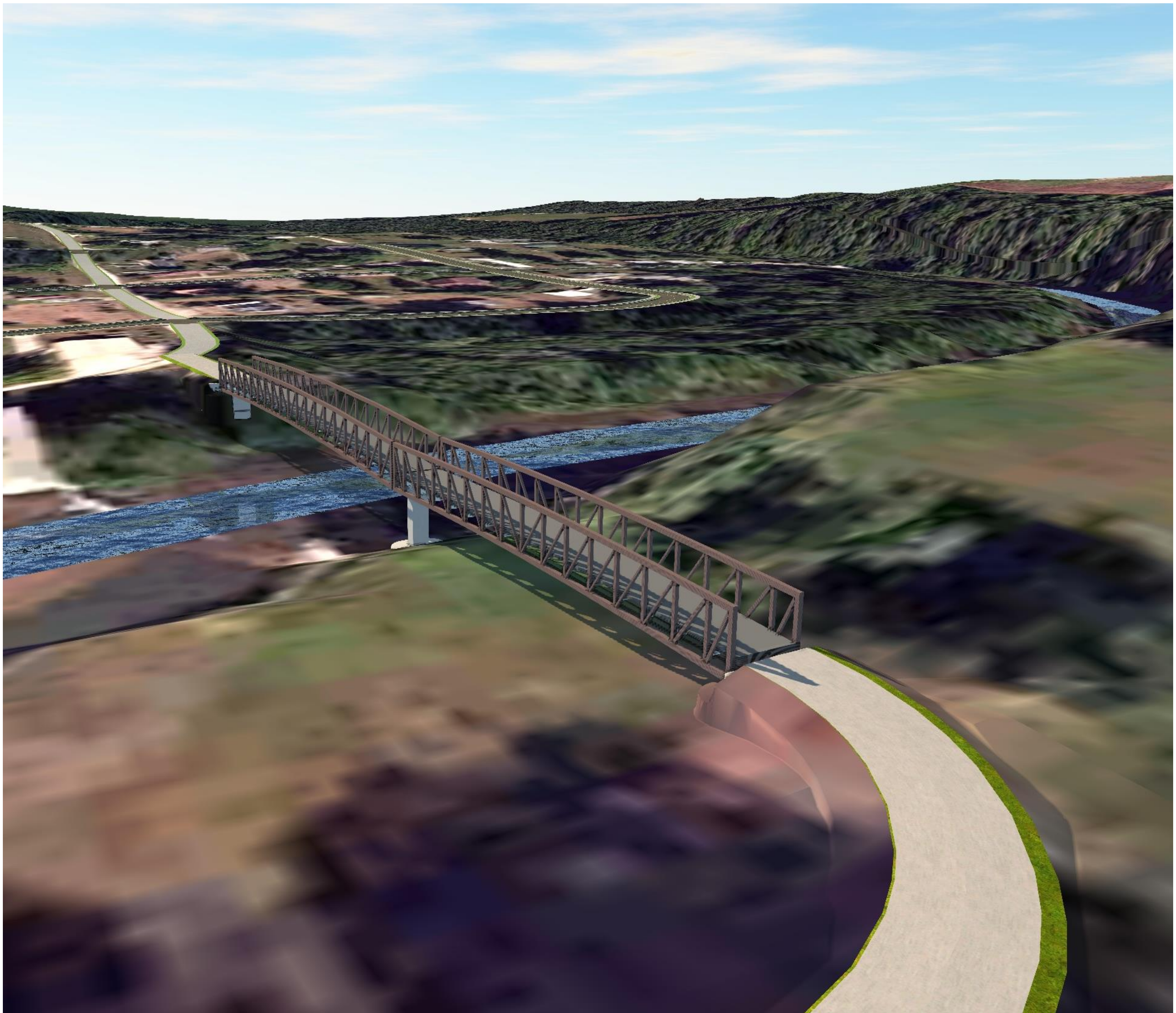

Pedestrian Bridge and Trail

Volga, IA

KGMM Engineering May 8, 2020 University of Iowa

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Section I Executive Summary

The City of Volga, Iowa requested the design of a new pedestrian bridge to connect the east and west ends of town which are separated by the Volga River. The existing bridge that would allow pedestrians to cross the Volga River was partially washed away by a flood and is unserviceable. According to the City, many pedestrians use the existing trail system, however, it is a challenge to travel from the campground to the Reflection Park area because the only available crossing is the county bridge on highway C2W. This vehicular bridge is an unsafe pedestrian crossing, which is a primary driver for the design of a new pedestrian bridge. Another issue the City noted about the remains of the existing bridge is the buildup of debris on the piers when the river is at a high stage. The purpose of this project is to connect the two sides of Volga with a pedestrian bridge along with connecting trails and demolition of the remains of the old bridge. The connection will provide excellent pedestrian mobility within the city, promote safer pedestrian walkways, and furnish better access to city services such as the Volga U Campground and the new Reflection Park.

KGMM Engineering has designed a new pedestrian bridge to cross the Volga River at Volga Street. The new bridge has been designed with a 12-foot width to accommodate pedestrians, cyclists, commercial lawnmowers, and utility terrain vehicles (UTVs). The design width is adequate for multiple simultaneous users and is sufficient to satisfy all AASHTO design standards for pedestrian bridges in the event federal money becomes available for the project. The new bridge will provide pedestrians safe access to key city services and save city staff time when crossing the Volga River. The proposed bridge superstructure is a prefabricated Pratt truss design from Contech Engineered Solutions which is approximately 12-feet wide and 9-feet-10-inches tall, or equal provided by another manufacturer. The truss is made of Corten steel that will achieve a weathered look to provide a natural aesthetic. The bridge will have two spans, one which is 150-feet long and another that is 117-feet long. The bridge superstructure is presented in Section 6.2 of this report. Figure 1.1 represents a 3D rendering of the full project site, including the pedestrian bridge and trail.

Hydraulic and hydrologic design considerations ensure that the bridge and bridge connections can withstand applicable loading scenarios beyond vertical loads due to pedestrians and vehicles, including wind, ice, debris, and water forces. The design freeboard for the proposed bridge is 3.9 feet above the 50-year flood, which is more than the required 3 feet above the 50-year flood and will allow ample space for floating debris to pass under the bridge during a flood event. Also, the backwater 400 feet upstream due to the proposed bridge is 1.32 inches, while the maximum value is 1.5 feet at a location 1.5 times the length of the bridge upstream. See sections 6.3 and 6.4 of this report for the hydrologic and hydraulic designs, respectively.

Reinforced concrete abutments support the bridge on each side of the river. They are situated such that they are out of the 100-year floodplain and will not negatively impact the flow of the river. The bridge spans are attached to the abutments by anchor bolts designed by Contech Engineered Solutions. The total height of each abutment is 11-feet-6-inches, including a 3-foot-10-inch beam seat, a 2-foot stem, and a 1-foot-3-inch approach slab seat. The abutment footing is 10 feet wide by 15 feet long. The abutments are presented in Section 6.5 of this report.

Figure 1.1. 3D rendering of the proposed pedestrian bridge and trail.

The bridge design includes one reinforced concrete pier which is situated out of the main river channel to reduce the amount of debris buildup and loading associated with high flows. The pier is also situated such that any debris that may build up can safely be removed by city staff. A T-shaped pier was chosen for design as it can withstand large hydraulic forces and reduces the column size in the river. The pier is a total of 23-feet tall, including the pier cap, column, and footing. The pier cap width is 15 feet, the column width is 5 feet, and the footing width is 13 feet. The pier footing is founded by deep-seated piles to ensure minimal settlement occurs. The bridge pier is presented in section 6.6 of this report. A protective riprap layer was designed around the pier and both abutments to minimize potential scour and destabilization of the bridge. The proposed riprap design is presented in Section 6.9 of this report.

Connecting to the bridge is a 10-foot wide shared-use path. The path is constructed of a 6-inch thick layer of Portland cement concrete (PCC) with a design cross slope of 1.5% to account for drainage while meeting Americans with Disabilities Act (ADA) regulations. The trail was designed to meet all ADA regulations and followed the IADOT Design Manual standards for pedestrian trail design. The trail will serve pedestrians, cyclists, commercial lawn mowers, and UTVs, and may serve equestrians in the future. The total length of the shared-use trail is approximately 640 feet. The pedestrian trail design is presented in Section 6.8 of this report.

The engineer's project cost estimate has been produced for the pedestrian bridge and trail which includes the cost of materials, labor, equipment, overhead, profit, contingency, possible easements or property acquisition, final design, and administration. The total project cost is estimated to be $1,494,000. Unit costs for each major bid item were determined from RSMeans Cost Handbooks and the IADOT bid tabulations. The full cost estimate is presented in Section VII of this report.

Section II Organization Qualifications and Experience

1. Name of Organization

KGMM Engineering

2. Organization Location and Contact Information

Ryan McDonough – Project Manager
Email: ryan-p-mcdonough@uiowa.edu

3. Organization and Design Team Description

KGMM is a team of senior civil engineering students at the University of Iowa in the capstone design class. The team is comprised of four members: Ryan McDonough, Nathan Gjersvik, Ryan Kowalsky, and Spencer McDermott. Ryan McDonough is specializing in structural design and business management, Nathan Gjersvik is specializing in transportation design with a focus area of civil engineering practice, Ryan Kowalsky is specializing in civil engineering practice, and Spencer McDermott is specializing in structural design and business management. Each individual performed a team role and a substantive task leader role.

Section III Design Services

1. Project Scope

The goal of this project was to reconnect the city of Volga with a pedestrian bridge over the Volga River that divides it, including designing a trail to connect the bridge with the existing network of trails, and estimating the cost for the removal of the existing bridge remains. The designed trail creates a continuous path from the campground on the east side of the river to the kayak/boat launch area of the Reflection Park on the west side. The need for a bridge has been long overdue since the existing bridge partially washed away during a flood in 1999, leaving the bridge unserviceable. KGMM included a cost estimate for the removal of the existing bridge that can be performed along with the construction of the new bridge or divided into a separate phase.

2. Work Plan

Throughout the project, KGMM Engineering followed the Gantt Chart timeline laid out in Figure 3.2.1. KGMM informed the client weekly of goals, completed tasks, and problems that were faced with the project.

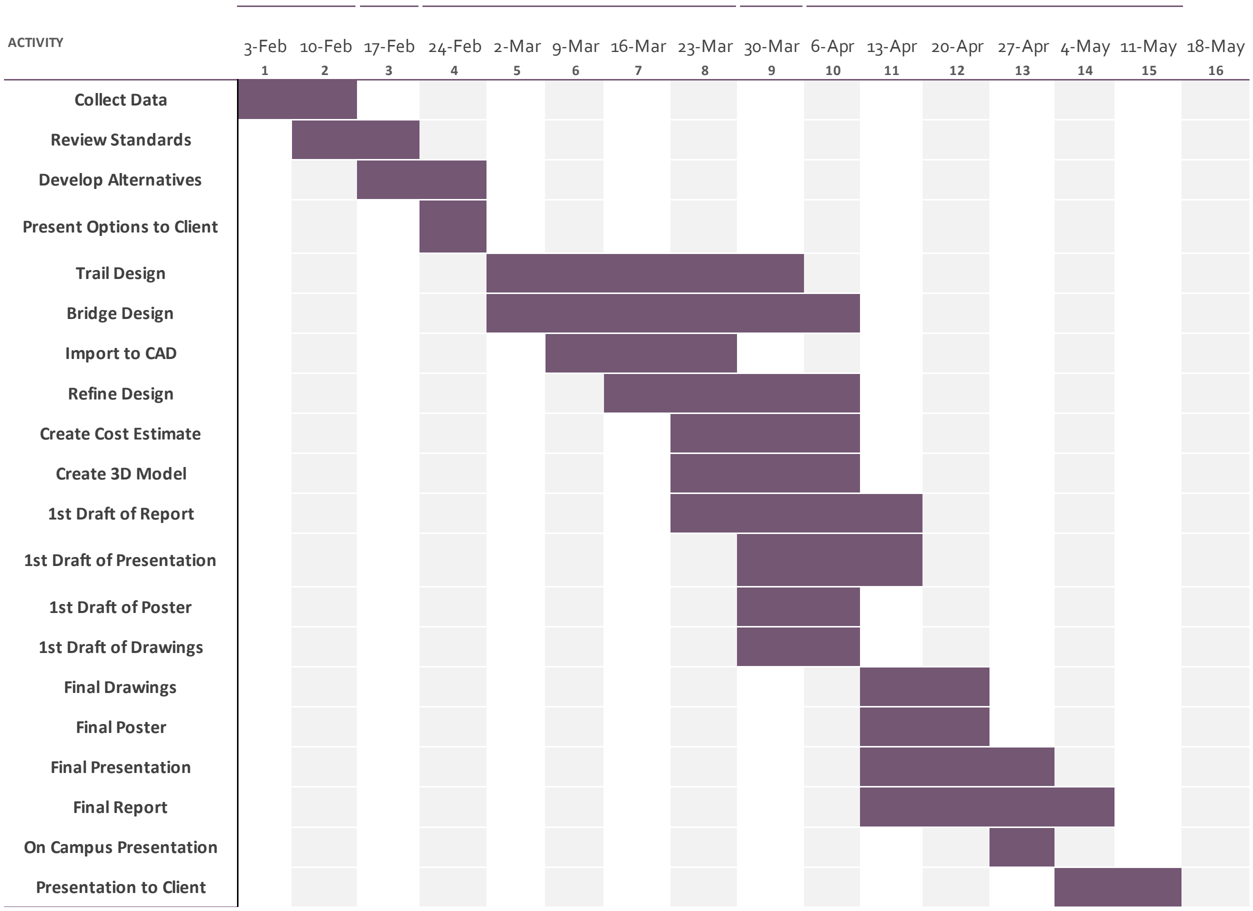


Figure 3.2.1. Project Gantt Chart.

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

1. Constraints

In brainstorming and developing the options for constructing a bridge to cross the Volga River, the design team came across various project constraints. The primary constraint is that the bridge cannot, in any way, increase the flood risk for the community. The team considered multiple bridge locations to determine the best site to meet all of the City's requests without causing an increased flood risk. Another major constraint was the available budget. The City of Volga may have to receive grant money to pay for the design and construction of the bridge. Budget limited KGMM's design and kept the design team budget conscious.

2. Challenges

A challenge the team noticed immediately is the amount of debris buildup on the piers of the washed-out bridge. This posed as a hurdle the team, which the resolved by constructing a 150-foot span to keep the center pier out of the normal flow of the river. Another challenge was the unprecedented Coronavirus (COVID-19). The University of Iowa transitioned to online courses mid-semester, which made the flow of the project much more difficult as each of the team members worked remotely and could not meet face to face for collaboration. COVID-19 also limited our team to only one site visit, so the team had to rely heavily on aerial data and available maps of Volga.

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

The addition of a pedestrian bridge in Volga will bring many positive outcomes to the community. With the town currently having limited access to cross to the other side of town without driving all the way around, it leaves a divided community. This pedestrian bridge, KGMM feels, will tie the community and the walking trail systems back together.

A positive impact this bridge and trail system will have on the community is connecting the campground and the old middle school on the east side of the river to the west side, which is home to the kayak entrance as well as multiple other attractions soon to come in the Reflection Park. This bridge will allow for additional tourism and revenue as campers will have direct access to the gorgeous Volga Reflections Park as well as the kayak boat ramp.

Section V Alternative Solutions that were Considered

The layout and terrain in the city of Volga presented many unique options for designing a bridge to cross the Volga River. KGMM Engineering collaborated with the City to produce three potential locations for bridge crossings that would each satisfy the City's needs. Alternative 1 was a bridge crossing the Volga River between Volga Street and Chase Street. Alternative 2 included the removal and replacement of the existing Cass Street bridge which is partially washed out due to flooding. Alternative 3 included a pedestrian connection to the existing C2W bridge south of town. Figure 5.1 graphically shows the locations of each alternative considered.

The advantages and disadvantages of each alternative were critically analyzed to choose the best project site for the City. Alternative 1 presented many advantages, including proximity to the Reflection Park and existing trails, the highest dike elevation on the west side, the possibility for another senior design group to provide a culvert for street drainage, only one pier in the river, and proximity to the largest population of children in the city. However, Alternative 1 would possibly require a temporary construction easement for private properties located near the dike. Alternative 2 was advantageous because it would remove the old bridge which is considered an "eyesore," it is located in the center of town, it can easily connect to the school and gymnasium on the east side of the river, and it could help another senior design group alleviate some flow at the flood gates. However, Alternative 2 would require the most initial funding, it would have at least two piers in the river, and it is at an elevation that has previously been flooded. The only advantage of Alternative 3 is the possibility of simply connecting to the existing C2W bridge, eliminating the need for a completely new bridge. However, Alternative 3 is located far from the center of town and the trail connection would traverse through an area that is frequently flooded.

Upon seeking input from the City, KGMM Engineering chose Alternative 1 for design. A new bridge across Volga Street would meet all of the City's requirements, including connecting the east and west ends of town, connection to existing trails, fewer piers in the river, and low cost.

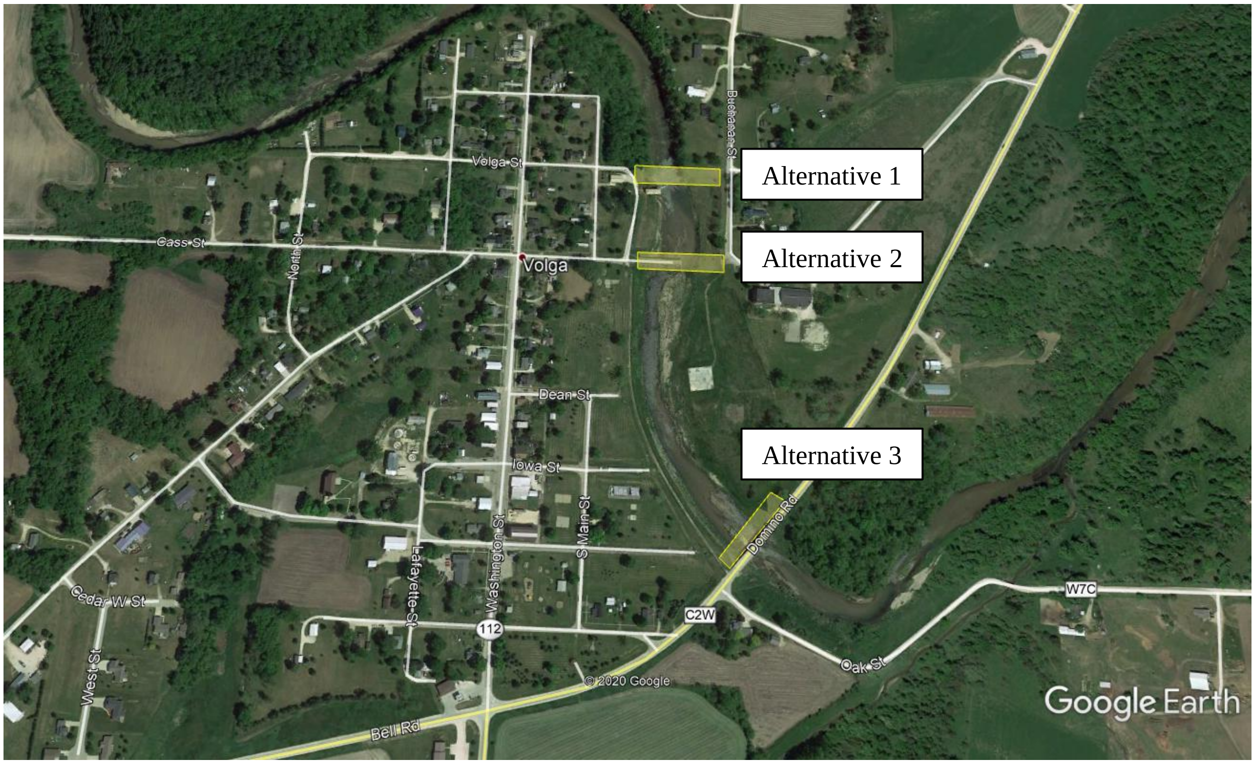


Figure 5.1. Volga pedestrian bridge alternative locations considered.

<u>Section VI</u> Final Design Details

The goal of this project was to produce a pedestrian bridge that would suit the needs of the City of Volga for many years to come. The delivered design can accommodate pedestrians, cyclists, UTVs, and commercial lawnmowers, and connects the two seemingly disconnected sides of Volga. The following sections describe each design element and the description includes the methods used to size elements, select materials, and estimate quantities. See Appendices F and G for design drawings and design renderings/models, respectively.

1. Bridge Deck

Our team began designing the bridge deck by reading through the Iowa DOT Design manual for pedestrian bridges to determine the loading scenarios we would need to consider. The next step was to calculate loads to use for the bridge deck following AASHTO LRFD standards. The load combination used was a 90 psf live load on the full deck or one 20,000 lb vehicle load to represent a maintenance vehicle, a 35 psf wind load on the full height of the bridge as if it was enclosed, and a 20 psf upward wind force applied at 3’ from the edge of the deck in the transverse direction per AASHTO 3.8.2. From these two resources the team determined that the bridge deck would be a 12’ wide, 6” deep concrete slab that will be doubly reinforced with #6 deformed reinforcing steel (rebar). Figure 6.1.1 shows an image of the cross-section of the bridge. The rebars will be spaced 6” on-center (OC) in the east-west direction and 1’ OC in the north-south direction. The concrete will be poured over a trapezoidal metal deck form designed by the prefab company that will be attached to steel W14x43 floor beams. The bridge deck will share the camber of the truss and will drain at the supports and laterally due to the pavement crown applied when paving. The hand-rail system will be attached directly to the bridge truss. The hand and toe rub rail will be C4x1x10 GA steel members and the safety rail will be steel HSS 1x1x1/8.

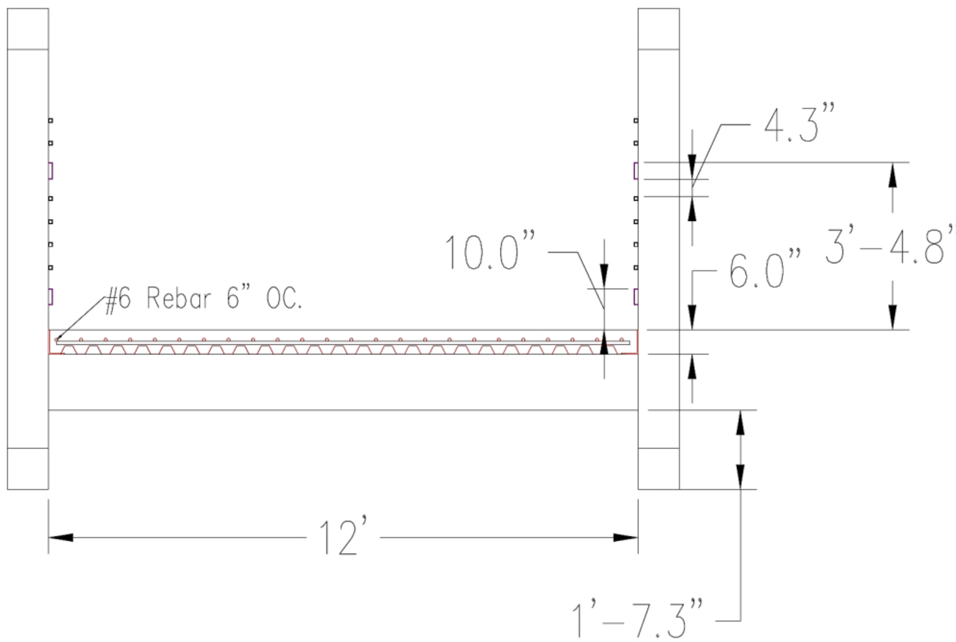


Figure 6.1.1 Bridge cross-section

2. **Bridge Truss**

The bridge truss KGMM used was a prefabricated steel truss designed by Contech Engineered Solutions from Alexandria, Minnesota. This is one of many prefab companies in the area, and others include Bridge Brothers and Pioneer Bridges. We decided to use a prefab company to design the steel truss because we determined it would be cheaper to design and construct. We recommend receiving bids from each of these companies to find the most economical choice. The bridge design we chose was a two-span bridge with spans of 150 feet and 117 feet. By using these spans, the 267-foot total span can be cleared with one pier outside of the main river channel. This was a concern expressed by our client as they wanted to minimize the obstruction of the river flow and prevent debris buildup. The truss is designed using HSS members (F_y =50 kip). Figure 6.2.1 shows a member schedule for the truss. The individual bridge spans are shown below in Figures 6.2.2 and 6.2.3 as well as the entire bridge in Figure 6.2.4. The loading on the bridge was based on LRFD standards and the Iowa DOT Design Manual. The weathered steel finish we chose was the most economical solution and will have a rustic look and does not require repainting. They also offer a painted finish for an additional cost. Figure 6.2.5 shows examples of different finishes.

SCHEDULE OF MEMBERS	
TOP CHORD	HSS 10 x 10 x 3/8
BOTTOM CHORD	HSS 10 x 10 x 3/8
VERTICAL	HSS 8 x 8 x 3/8
END VERTICAL	HSS 10 x 10 x 3/8
DIAGONAL	HSS 6 x 4 x 1/4 ①
BRACE DIAGONAL	HSS 4 x 4 x 1/4
FLOOR BEAM	W 14 x 43
END FLOOR BEAM	HSS 10 x 10 x 3/8 (STACKED) ②
TOE RAIL	C4 x 1 x 10 GA (R.F.)
RUB RAIL	C4 x 1 x 10 CA (R.F.)
SIDE DAM	∠ 6 x 4 x 5/16
SAFETY RAIL	HSS 1 x 1 x 1/8

① USE HSS 8 x 6 x 3/8 END TWO BAYS, HSS 6 x 6 x 1/4 3RD & 4TH BAYS. TYP EACH END. DOUBLE MITER ALL DIAGONALS.

② USE 2 HSS 10 x 10 x 3/8 STACKED FOR END FLOOR BEAMS. TYP EACH END.

Figure 6.2.1. Schedule of members.

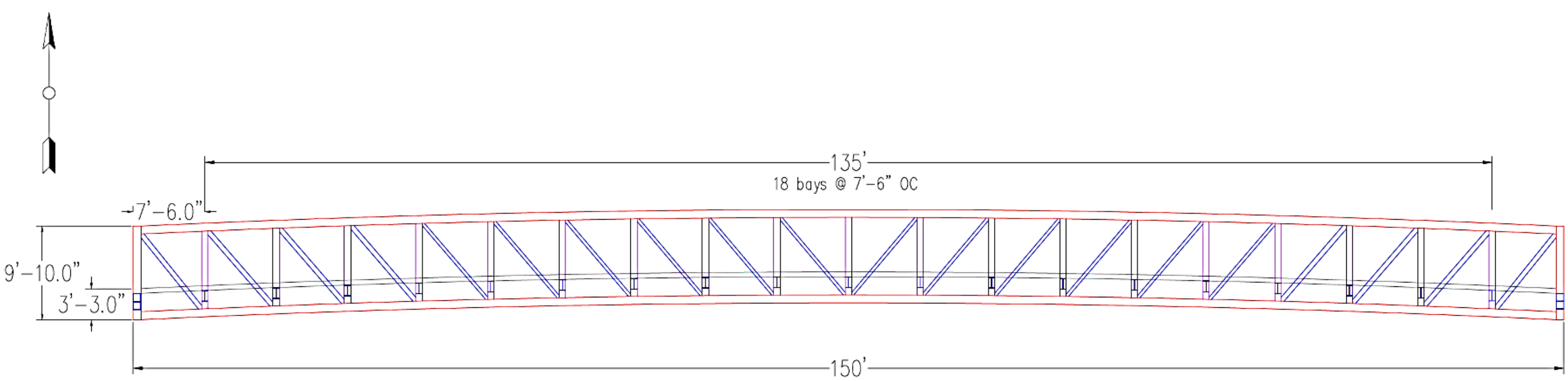


Figure 6.2.2. 150' truss

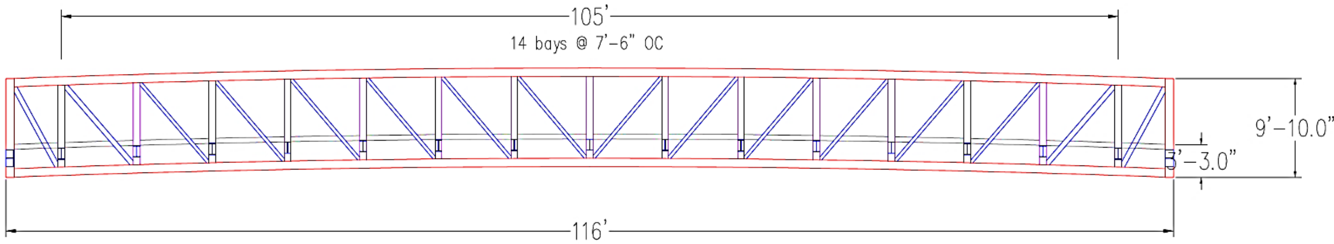


Figure 6.2.3. 117' truss.

Figure 6.2.4. Full bridge span from 3D model.

Figure 6.2.5. Weathered steel vs. painted finish.

3. Hydrologic Information

The Volga River hydrologic information was determined using the online program StreamStats from the United States Geological Survey (USGS). The analysis point selected indicated an upstream drainage area of 262 square miles. The exceedance probability discharges for the 2, 5, 10, 25, 50, 100, 200, and 500-year flood were determined using StreamStats and are as follows:

- Q_{2} = 5,150 cfs
- Q_{5} = 9,230 cfs
- Q_{10} = 12,200 cfs
- Q_{25} = 16,100 cfs
- Q_{50} = 19,300 cfs
- Q_{100} = 22,300 cfs
- Q_{200} = 25,500 cfs
- Q_{500} = 29,700 cfs

According to the Iowa DOT LRFD Bridge Design Manual Section 3.2.2, the design exceedance probability discharge for a bridge is the 50-year flood, so a river flow rate of 19,300 cfs was used for bridge design. See Appendix A for the full StreamStats output. The base flood elevation (BFE) was determined using ESRI ArcMap and was determined to be 794.5 feet upon linear interpolation of the water surface elevation (WSEL) data provided by the Iowa Department of Natural Resources (IDNR). See Appendix B for the full BFE calculation process.

4. Hydraulic Design

The Hydraulic analysis was completed using the Army Corps of Engineers program, HEC-RAS. The Iowa DNR states that for any bridge or culvert structure there must be a minimum freeboard of 3.0 feet during a 50-year flood event and a maximum backwater of fewer than 1 foot during a 100-year flood event. Upon completion of HEC-RAS analysis it was determined that during a 50-year flood event, the freeboard was roughly 3.9 feet. Figure 6.4.1 depicts the water surface level for the 50-year event. The backwater analysis determined that the backwater created by the construction of the bridge was 1.32" at a distance of roughly 450 feet upstream. Figure 6.4.2 displays the water surface profile before construction, and Figure 6.4.3 displays the water surface profile after construction. Once the bridge at Cass St. is removed, the backwater will drop below the 1.32" and should see a net positive decrease in water level due to the old bridge having two piers in the river.

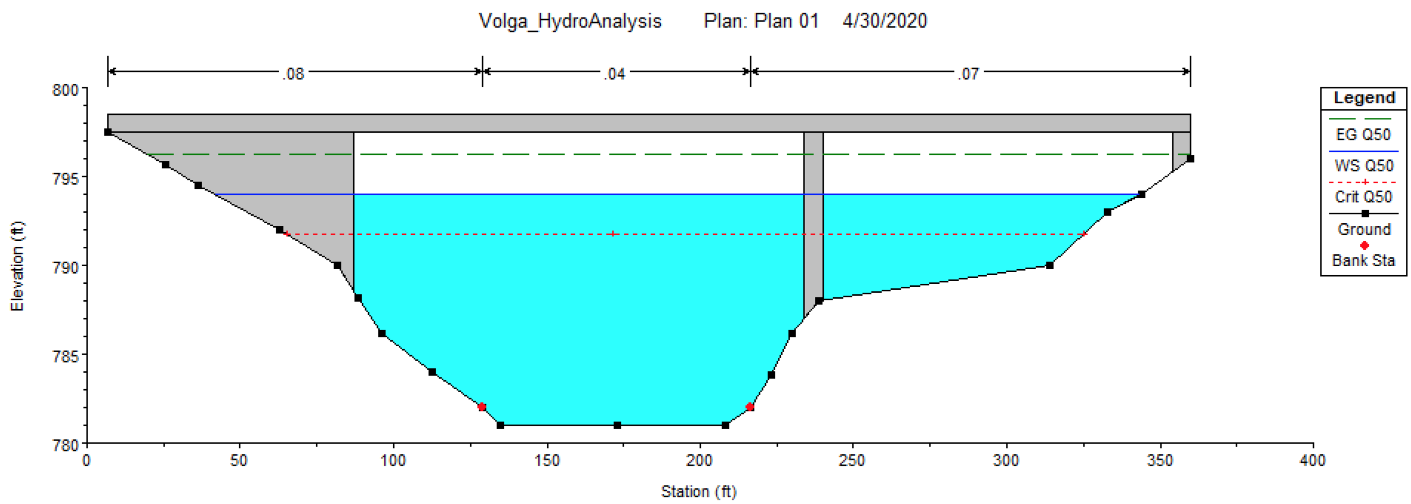


Figure 6.4.1. Water surface elevation during the 50-year flood event.

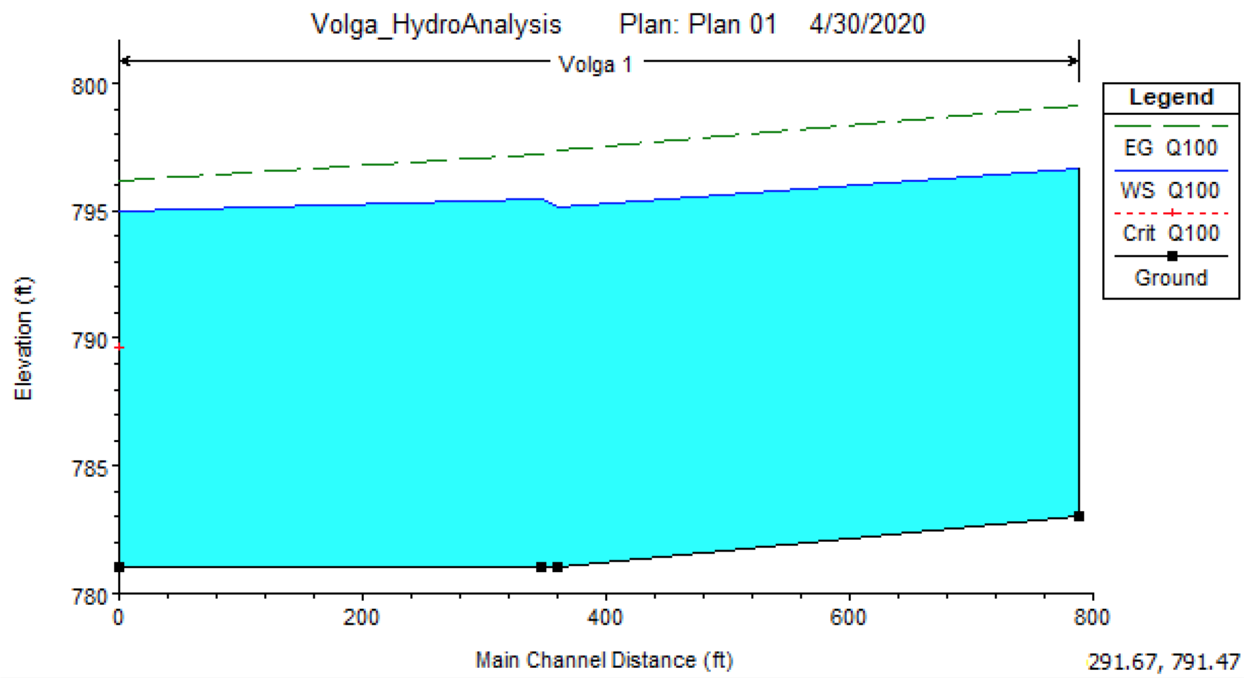


Figure 6.4.2. Water surface profile before bridge construction.

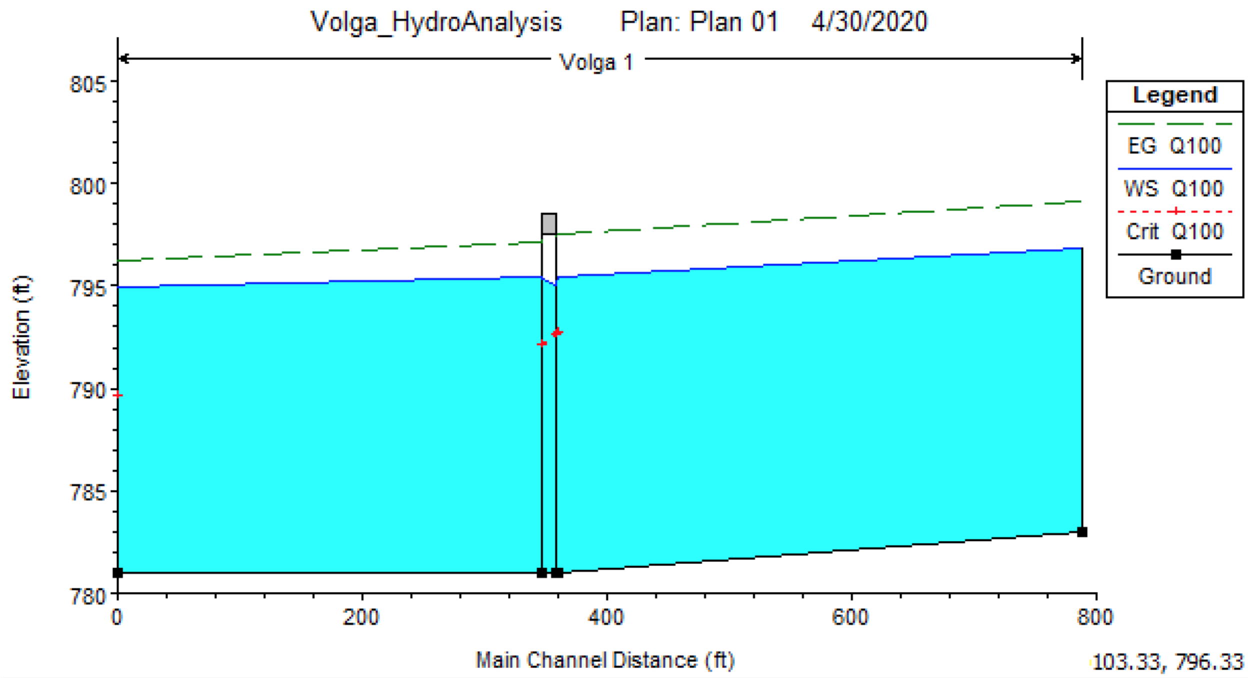
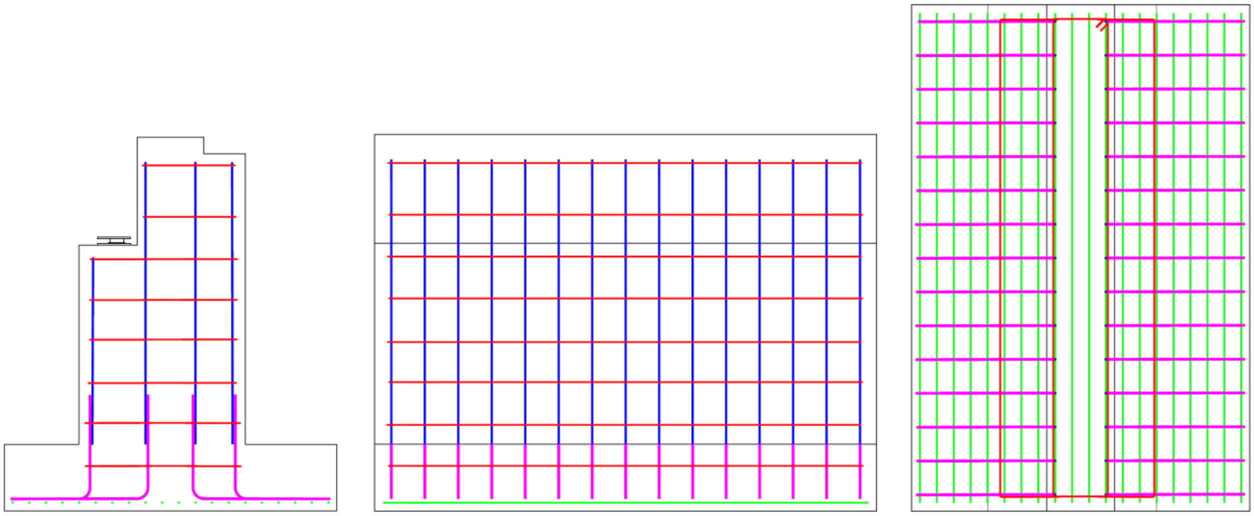


Figure 6.4.3. Water surface profile after bridge construction.

5. Abutment Design

The bridge abutments were designed using AASHTO LRFD methods presented in the IADOT Bridge Design Manual (BDM). The bearing capacity calculations were based on the methods presented in *Foundation Analysis and Design* by Joseph E. Bowles. As the designed bridge is intended to be prefabricated, it is not feasible to use integrated abutments for the bridge. Therefore, KGMM chose stub abutments with spread footings as foundations. The abutment width (dimension parallel to the bridge direction) was 10.0 feet and the length (dimension perpendicular to the bridge direction) was 15.0 feet. The abutment stem was designed to be a total of 5.0 feet wide, with a 1-foot-inch beam seat and a 1-foot-3-inch approach slab seat. The steel reinforcement at the abutment bottom was chosen to be 19 #5 rebars at 6-inch OC spacing. The dowel bars are designed to be #8 rebars and connect the abutment footing and stem. The stem reinforcement is made up of #6 rebars. A splice length of 1-foot-6-inches is used for the stem reinforcement and the dowel bars. The height of the stem is 9-feet-10-inches, with a 3-foot-10-inch beam seat and a 6-inch approach slab seat. The total height of the abutment and footing is 11-feet-10-inches. See Figure 6.5.1 for the final abutment design and Appendix C for abutment design calculations. Consult the design drawings for element dimensions and details as well as a reinforcement key.



(a) (b) (c)

Figure 6.5.1. (a) Abutment cross section; (b) profile view; (c) plan view.

A three-dimensional (3D) rendering of the abutment was developed by KGMM is shown in Figure 6.5.2 such that the client can easily visualize the finished product.

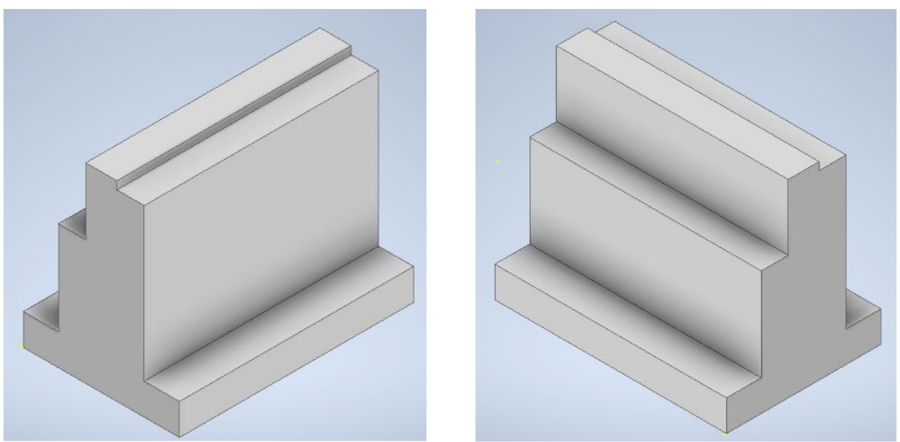


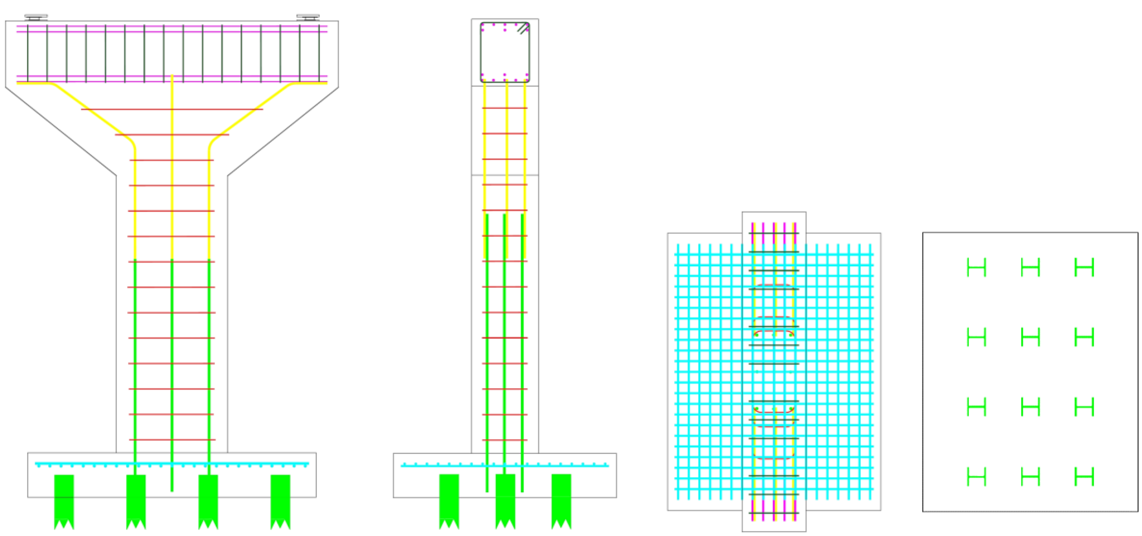
Figure 6.5.1. 3D rendering of the abutments from two different viewpoints.

One of the major design considerations for this bridge was the elevation at which the low steel would be set at. According to the IDNR, for new bridges and roadway embankments, the freeboard must be 3 feet or more between the BFE and the low superstructure horizontal bridge member unless a licensed engineer provides certification that the bridge is designed to withstand the applicable effects of ice and the horizontal stream loads and uplift forces associated with the 100-year flood. Thus, the bridge was designed with the low steel 3.9 feet above the 50-year flood. The low steel elevation is 795.5 feet (NAD83-11 datum) and is the same elevation as the beam seat elevation and the top of pier elevation. The low steel elevation choice also affected the location of the pier and trail, so careful consideration was taken when determining final elevations.

6. Pier Design

The bridge T-pier was designed based on AASTHO LRFD Section 3.6.5 presented in the IADOT BDM Section 6.6 along with Excel calculation files produced and published by IADOT. Hydraulic loading was computed using AASHTO LRFD 3.7, and the design was checked to ensure the pier has adequate strength to resist wind, ice, and water loading. The pier cap and pier cap overhang were designed using an IADOT Excel file called LRFD_Cap_Design_General.xlsb which is published on the Final Design Software section of the BDM. Using this software, the pier cap was checked for flexural strength and shear strength for the applied loading. Outputs include rebar dimensions and details as well as shrinkage and temperature reinforcement. The pier piles were designed using the IADOT Excel file Pile_Length_LRFD_WEAP.xlsb which determines the number of piles and pile length for the given loading, soil information, and pile type. The pier pile footing was designed using the IADOT Excel file LRFD_Footing_Design_General.xlsb and uses the pile information along with the soil information to determine the dimensions and required reinforcement for pier footings. See Figure 6.6.1 for general schematics of the pier. Upon completion of the design, the pier dimensions are as follows:

- Pier cap height = 3.0 feet
- Pier cap depth = 3.0 feet
- Pier cap overhang = 5.0 feet
- Pier column width = 5.0 feet
- Pier column depth = 3.0 feet
- Pier column height (un-tapered) = 12.5 feet
- Pier footing width = 10.0 feet
- Pier footing length = 13.0 feet
- Pier footing thickness = 4.0 feet
- Pier piles: 12 HP10x42 piles with a contract length of 65.0 feet, spaced at 3.0 feet OC, embedded 1.0 feet, with an edge spacing of 1.0 feet.



(a) (b) (c) (d)

Figure 6.6.1. (a) Pier cross section; (b) profile view; (c) plan view; (d) pile cap plan view.

A 3D rendering was also developed for the pier and pile cap, as shown in Figure 6.6.2.

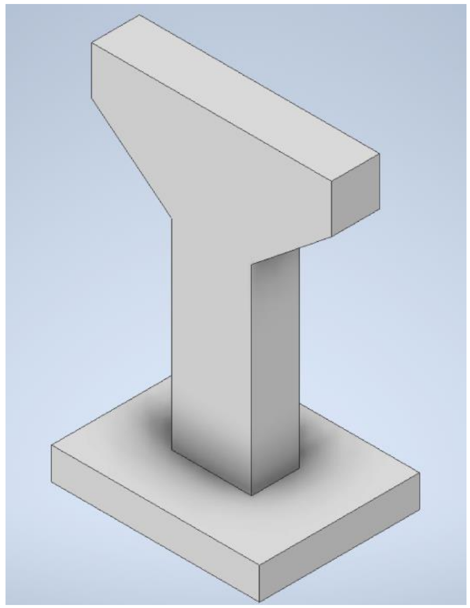


Figure 6.6.2. 3D rendering of the pier and pile cap.

7. Pedestrian Trail

Trail design followed the Iowa DOT Design Manual sections 12A-2 Standards for Accessibility and 12B-2 Shared Use Path Design. The trail was determined to be Type 2 based on the criteria of a path serving as a transportation route to facilities that fulfill a basic life need, provide access to services, or provide a safe route for non-drivers. The recommended shared-use trail was designed as a 10-foot wide by 6-inch Portland Cement Concrete (PCC) shared path trail, with a 2-foot graded earth shoulder on both sides of the trail. The trail should be machine placed and broom finished, or burlap drag surfaced to provide texture. The path was designed with a maximum cross slope of 2.0% with a construction target value of 1.5%. Based on a design speed of 18 mph for bicyclists, the minimum radius for any curve on the path is 60 feet, with grades equal to or less than 5.0% to meet ADA regulations. All portions of the trail were designed to comply with ADA regulations. See Figure 6.7.1 for the trail typical section.

The earthwork and grading for the trail follow the Iowa DOT Design Manual sections 10B-1 Seeding, Fertilizing, and Mulching and 12B-2 Shared Use Path Design. Volumetric analysis was completed in Autodesk Civil3D and the Cut/Fill quantity amounted to 669.19 cubic yards of fill. The target cross slope of 1.5% will be more than adequate to ensure that water drains off of the sidewalk and down into the river valley below. There will be 4 feet on both sides of the trail corridor that will also need to be seeded and fertilized as they will be cleared during construction. The total area needed to be seeded and fertilized is 0.156 acres. See Appendix D for the earthwork report provided by Civil3D.

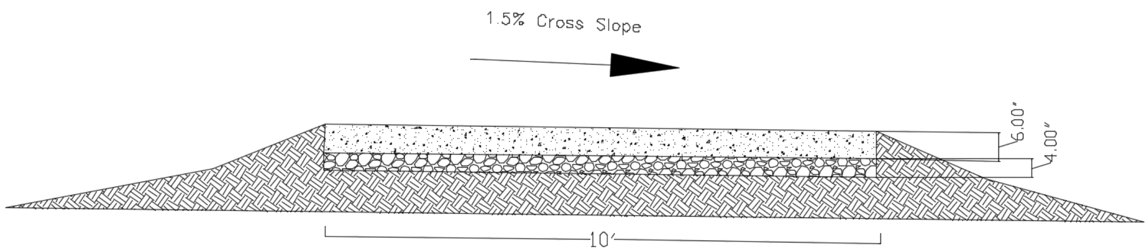


Figure 6.7.1. Typical trail cross-section.

8. Riprap Design

Scour protection for the bridge pier and abutments is critical for the long-term serviceability of the bridge. To minimize scour potential at the bridge pier and abutments, a protective riprap layer was designed. The riprap layer design was based on recommendations from the IADOT BDM section 3.7.3.5 and section C3.2.2.7, and the FHWA publication *Evaluating Scour at Bridges*, *Fifth Edition*, commonly known as HEC-18. According to the IADOT BDM 3.7.3.5, slope protection for bridges typically is specified to a minimum of the 50-year flood elevation. Using hydraulic and hydrologic data for the Volga River, the references recommended a 2- to 3-foot thick layer of Class E revetment stone which extends 10 feet upstream and downstream from the abutments and is 10 feet wide in both directions at the pier. A layer of engineering fabric is recommended under the layer of riprap, according to IADOT BDM guidelines and standard drawings. See Appendix E for the riprap design calculations. Figure 6.8.1 depicts the final riprap design configuration and its relation to the substructure elements.

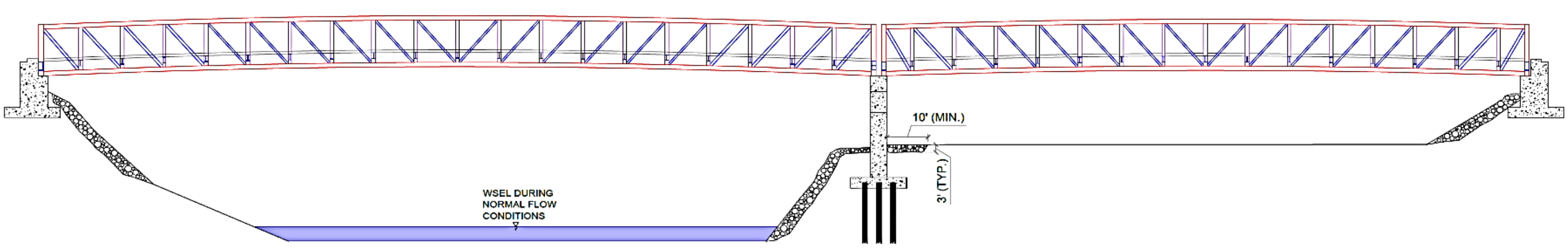


Figure 6.8.1. Riprap design configuration.

<u>Section VII</u> Engineer's Cost Estimate

The primary source used to estimate the cost of the Volga Pedestrian Bridge and Trail was the Iowa Department of Transportation's Bid Tabulation. The costs associated with the steel trusses for the prefabricated bridge were estimated by Contech and the remaining bid item costs were determined by KGMM. Table 7.1 displays the material unit legend to clarify the units used for cost estimation.

Table 7.1. Material unit legend.

Unit Name	Unit Description
CY	Cubic Yard
EA	Each
STA	Per Station
ACRE	Acre
SY	Square Yard
SF	Square Foot
HR	Hour
LF	Linear Feet
TON	Ton (2000 lbs)

Table 7.2 displays the final cost estimate for the Volga Pedestrian Bridge and Trail. KGMM recommends performing the entire project in a single phase which includes the construction of the pedestrian bridge and trail as well as the demolition of the remains of the Cass Street Bridge. Performing the project in a single phase would decrease the overall project cost as equipment would only be transported once and would be in the area for the Cass Street bridge demolition. The crane and crane operator could quickly and easily demolish the Cass Street bridge after installing the bridge superstructure, which would reduce the costs associated with equipment mobilization and rental. However, the client may phase the project in two steps if necessary. KGMM recommends that Phase 1 consist of the construction of the pedestrian trail and the bridge substructures as well as the installation of the prefabricated bridge superstructure. It is recommended that Phase 2 consist of the demolition of the Cass Street bridge. The total project cost estimate including contingency, engineering fees, and administration fees is $1,494,000.00.

Table 7.2. Final project cost estimate.

Volga Pedestrian Trail and Bridge

Bridge (Superstructure and Substructure)				
Item	**Unit**	**Quantity**	**$ Price/Unit**	**Total**
Steel Truss	EA	4	$ 219,000.00	$ 876,000.00
Deck(267x12x.5)	CY	60	$ 200.00	$ 12,000.00
Subgrade Stabilization Material	SY	260	$ 3.00	$ 780.00
Scour Protection	CY	173	$ 40.00	$ 6,925.00
Cast-In-Place Portland Cement Concrete	CY	90.5	$ 243.00	$ 22,000.00
#4 Reinforcement Bar	TON	0.25	$ 695.00	$ 175.00
#9 Reinforcement Bar	TON	3.1	$ 800.00	$ 2,500.00
#11 Reinforcement Bar	TON	0.49	$ 835.00	$ 410.00
Concrete Pump Rental	HR	20	$ 175.00	$ 3,500.00
H-Pile installation	LF	780	$ 71.00	$ 55,500.00
			Total Bridge Cost	**$ 979,790.00**

Trail				
Item	**Unit**	**Quantity**	**$ Price/Unit**	**Total**
Recreational Trail, PCC 6"	SY	710	$ 34.98	$ 24,900.00
Special Backfill	CY	562.79	$ 21.13	$ 11,900.00
Corrugated Metal Culvert (12")	LF	14	$ 21.50	$ 301.00
Seeding and Fertilizing	ACRE	0.1304	$ 2,027.37	$ 265.00
Clearing and Grubbing	ACRE	0.3260	$ 1,757.56	$ 575.00
Granular Subbase	SY	710	$ 8.20	$ 5,825.00
Special Compaction of Subgrade for Recreational Trail	STA	6.39	$ 365.65	$ 2,350.00
Erosion Control	EA	2	$ 500.00	$ 1,000.00
Aluminum, Reflective Trail Signs	EA	5	$ 25.00	$ 125.00
Wood Posts for Trail Signs (4x4)	EA	5	$ 16.98	$ 90.00
			Total Trail Cost	**$ 47,340.00**

Bridge Removal				
Bridge Removal (350x68)	SF	23800	$ 7.00	$ 167,000.00
			Total Removal Cost	**$ 167,000.00**

Phase 1 Construction Cost	$ 1,027,200.00
Phase 2 Construction Cost	$ 167,000.00
Total Construction Cost	$ 1,194,200.00
Easements	$ -
10% Contingencies	$ 120,000.00
15% Engineering and Administration	$ 179,130.00
Total Project Cost	**$ 1,494,000.00**

Appendices

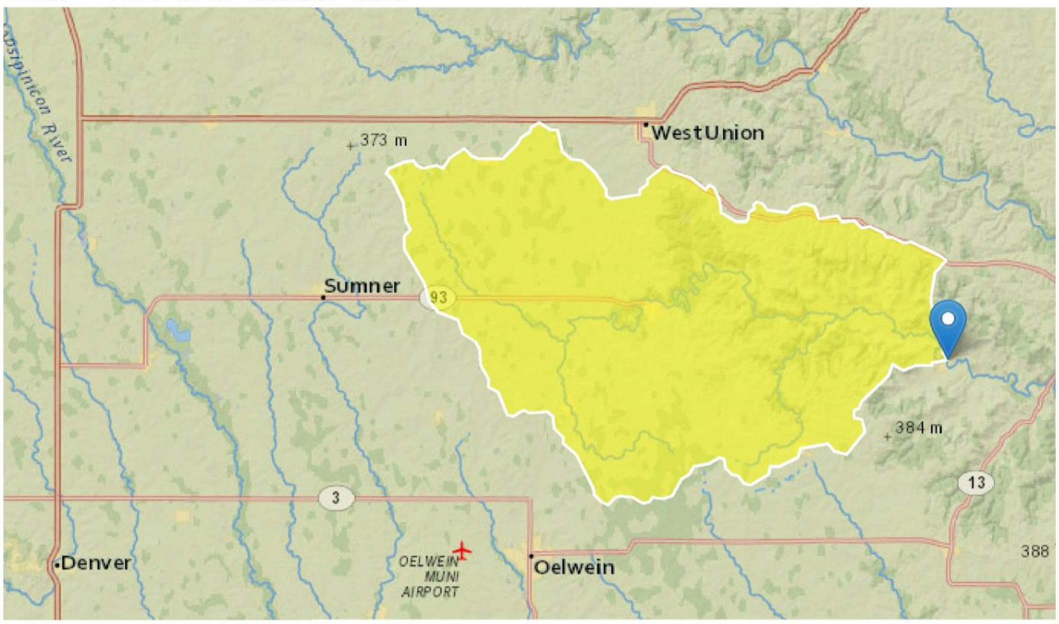
The following appendices contain the outputs, design calculations, assumptions, and standards referenced for each design element.

Appendix A – StreamStats Output

The USGS StreamStats program was implemented to determine design flood flowrates for the Volga River at the bridge location. See below the StreamStats output file.

StreamStats Report Volga IA 31 Mar 2020

Region ID: IA
Workspace ID: IA20200331210840015000
Clicked Point (Latitude, Longitude): 42.80587, -91.53860
Time: 2020-03-31 16:08:57 -0500



Location of proposed Pedestrian Bridge.

Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
DRNAREA	Area that drains to a point on a stream	262	square miles
STREAM_VARG	Streamflow variability index as defined in WRIR 02-4068, computed from regional grid	0.405	dimensionless
DRNFREQ	Number of first order streams per square mile of drainage area	1.24	1st-order streams per square mile

Parameter Code	Parameter Description	Value	Unit
SSURGOC	Percentage of area of Hydrologic Soil Type C from SSURGO	5.4	percent
PRECIP	Mean Annual Precipitation	35.4	inches
RSD	Relative stream density first defined in SIR 2012_5171	0.31	dimensionless
HYSEP	Median percentage of baseflow to annual streamflow	57.37	percent
SSURGOB	Percentage of area of Hydrologic Soil Type B from SSURGO	92.9	percent
SSURGOD	Percentage of area of Hydrologic Soil Type D from SSURGO	0.31	percent
DESMOIN	Area underlain by Des Moines Lobe	0	percent
BSHAPE	Basin Shape Factor for Area	2.88	dimensionless
SSURGOKSAT	Saturated hydraulic conductivity in micrometers per second from NRCS SSURGO database	19.55	micrometers per second
I24H10Y	Maximum 24-hour precipitation that occurs on average once in 10 years	4.37	inches
SSURGOA	Percentage of area of Hydrologic Soil Type A from SSURGO	1.44	percent

General Flow Statistics Parameters[Low Flow Northeast annual 2012 5171]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	262	square miles	1.4	6506
STREAM_VARG	Streamflow Variability Index from Grid	0.405	dimensionless	0.206	0.61
DRNFREQ	Drainage Frequency	1.24	1st-order streams per square mile	0.295	2.78

General Flow Statistics Flow Report[Low Flow Northeast annual 2012 5171]

PIl: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	PIl	PIu

Statistic	Value	Unit	PII	Plu
Harmonic Mean Streamflow	59.1	ft^3/s	22	159

General Flow Statistics Citations

Eash, D.A., and Barnes, K.K.,2012, Methods for estimating selected low-flow frequency statistics and harmonic mean flows for streams in Iowa: U.S. Geological Survey Scientific Investigations Report 2012-5171, 99 p. (http://pubs.usgs.gov/sir/2012/5171/)

Flow-Duration Statistics Parameters[Statewide Flow Duration 2012 5232]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	262	square miles	15.5	7782
SSURGOC	SSURGO Percent Hydrologic Soil Type C	5.4	percent	0.09	83.5
PRECIP	Mean Annual Precipitation	35.4	inches	27.7	38
RSD	Relative Stream Density	0.31	dimensionless	0.22	0.49
HYSEP	Hydrograph separation percent	57.37	percent	20.3	78
STREAM_VARG	Streamflow Variability Index from Grid	0.405	dimensionless	0.21	0.76
SSURGOB	SSURGO Percent Hydrologic Soil Type B	92.9	percent	5.7	99.4
SSURGOD	SSURGO Percent Hydrologic Soil Type D	0.31	percent	0	57

Flow-Duration Statistics Flow Report[Statewide Flow Duration 2012 5232]

PII: Prediction Interval-Lower, Plu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
1 Percent Duration	1770	ft^3/s	23.5
5 Percent Duration	638	ft^3/s	23.6
10 Percent Duration	339	ft^3/s	24.2
15 Percent Duration	356	ft^3/s	24.6
20 Percent Duration	279	ft^3/s	22.1
30 Percent Duration	200	ft^3/s	17.1

Statistic	Value	Unit	SEp
40 Percent Duration	144	ft^3/s	14.9
50 Percent Duration	111	ft^3/s	16.4
60 Percent Duration	83.1	ft^3/s	22.1
70 Percent Duration	54.8	ft^3/s	32.4
80 Percent Duration	49.6	ft^3/s	40.1
85 Percent Duration	42.7	ft^3/s	42.5
90 Percent Duration	38.1	ft^3/s	51
95 Percent Duration	26.3	ft^3/s	74.9
99 Percent Duration	18.7	ft^3/s	

Flow-Duration Statistics Citations

Linhart, S.M., Nania, J.F., Sanders, C.L., Jr., and Archfield, S.A.,2012, Computing daily mean streamflow at ungaged locations in Iowa by using the Flow Anywhere and Flow Duration Curve Transfer statistical methods: U.S. Geological Survey Scientific Investigations Report 2012–5232, 50 p. (http://pubs.usgs.gov/sir/2012/5232/)

Peak-Flow Statistics Parameters[46 Percent (121 square miles) Peak Region 2 2013 5086]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	262	square miles	0.08	7783
DESMOIN	Des Moines Lobe	0	percent	0	100
BSHAPE	Basin Shape Factor	2.88	dimensionless	0.806	13.94

Peak-Flow Statistics Parameters[54 Percent (141 square miles) Peak Region 3 2013 5086]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	262	square miles	0.05	2809
BSHAPE	Basin Shape Factor	2.88	dimensionless	0.339	13.523
SSURGOKSAT	SSURGO Saturated Hydraulic Conductivity	19.55	micrometers per second	1.883	33.572

Peak-Flow Statistics Parameters[46 Percent (121 square miles) Peak Region 2 DA only 2015 5055]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	262	square miles	0.08	7783

Peak-Flow Statistics Parameters[54 Percent (141 square miles) Peak Region 3 DA only 2015 5055]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	262	square miles	0.05	2809

Peak-Flow Statistics Flow Report[46 Percent (121 square miles) Peak Region 2 2013 5086]

PII: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	PII	PIu	SEp
2 Year Peak Flood	4420	ft^3/s	2130	9190	46.8
5 Year Peak Flood	8180	ft^3/s	5410	12400	25.7
10 Year Peak Flood	11600	ft^3/s	8290	16200	20.8
25 Year Peak Flood	16700	ft^3/s	12200	22800	19.4
50 Year Peak Flood	20000	ft^3/s	14400	27700	20.4
100 Year Peak Flood	23100	ft^3/s	16200	33000	22.3
200 Year Peak Flood	29300	ft^3/s	19700	43600	24.9
500 Year Peak Flood	31500	ft^3/s	20100	49400	28.2

Peak-Flow Statistics Flow Report[54 Percent (141 square miles) Peak Region 3 2013 5086]

PII: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	PII	PIu	SEp
2 Year Peak Flood	4530	ft^3/s	2270	9060	43.1
5 Year Peak Flood	8970	ft^3/s	5430	14800	30.4
10 Year Peak Flood	11900	ft^3/s	7590	18700	27
25 Year Peak Flood	16400	ft^3/s	10500	25500	26.5
50 Year Peak Flood	19000	ft^3/s	11900	30300	27.8
100 Year Peak Flood	22400	ft^3/s	13800	36400	29.1
200 Year Peak Flood	25900	ft^3/s	15600	43000	30.5
500 Year Peak Flood	30000	ft^3/s	17100	52500	33.7

Peak-Flow Statistics Flow Report[46 Percent (121 square miles) Peak Region 2 DA only 2015 5055]

PII: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	PII	PIu	SEp
2 Year Peak Flood	3590	ft^3/s	1720	7530	47.4
5 Year Peak Flood	7000	ft^3/s	4450	11000	28.2
10 Year Peak Flood	9630	ft^3/s	6590	14100	23.6
25 Year Peak Flood	13200	ft^3/s	8990	19300	24
50 Year Peak Flood	15900	ft^3/s	10600	23900	25.4
100 Year Peak Flood	18800	ft^3/s	12200	28900	26.9
200 Year Peak Flood	21700	ft^3/s	13600	34500	29.1
500 Year Peak Flood	25500	ft^3/s	15200	42700	32.6

Peak-Flow Statistics Flow Report[54 Percent (141 square miles) Peak Region 3 DA only 2015 5055]

PII: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	PII	PIu	SEp
2 Year Peak Flood	5150	ft^3/s	2580	10300	44
5 Year Peak Flood	9230	ft^3/s	5330	16000	34.4
10 Year Peak Flood	12200	ft^3/s	7180	20700	33.2
25 Year Peak Flood	16100	ft^3/s	9440	27600	33.6
50 Year Peak Flood	19300	ft^3/s	10900	34000	35.6
100 Year Peak Flood	22300	ft^3/s	12300	40500	37.6
200 Year Peak Flood	25500	ft^3/s	13700	47800	39.7
500 Year Peak Flood	29700	ft^3/s	15100	58500	43.2

Peak-Flow Statistics Citations

Eash, D.A., Barnes, K.K., and Veilleux, A.G.,2013, Methods for estimating annual exceedance-probability discharges for streams in Iowa, based on data through water year 2010: U.S. Geological Survey Scientific Investigations Report 2013-5086, 63 p. with a (http://pubs.usgs.gov/sir/2013/5086/)

Eash, D.A.,2015, Comparisons of estimates of annual exceedance-probability discharges for small drainage basins in Iowa, based on data through water year 2013: U.S. Geological Survey Scientific Investigations Report 2015–5055, 37 p. (http://dx.doi.org/10.3133/sir20155055.)

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Application Version: 4.3.11

Appendix B – Base Flood Elevation Calculations

The base flood elevation (BFE) for the 100-year flood was determined using data obtained from IDNR, a preliminary flood insurance rate map (FIRM) from FEMA, and a rating curve for the Volga river produced using the USGS WaterWatch program.

ESRI ArcMap shapefiles containing water surface elevations (WSELs) of the 100-year flood were provided from IDNR. The WSEL downstream from the bridge is 793.4639 feet and the WSEL upstream from the bridge is 795.4095 feet. See Figure B.1 and Figure B.2 for Civil3D screenshots of the provided section locations and WSELs.

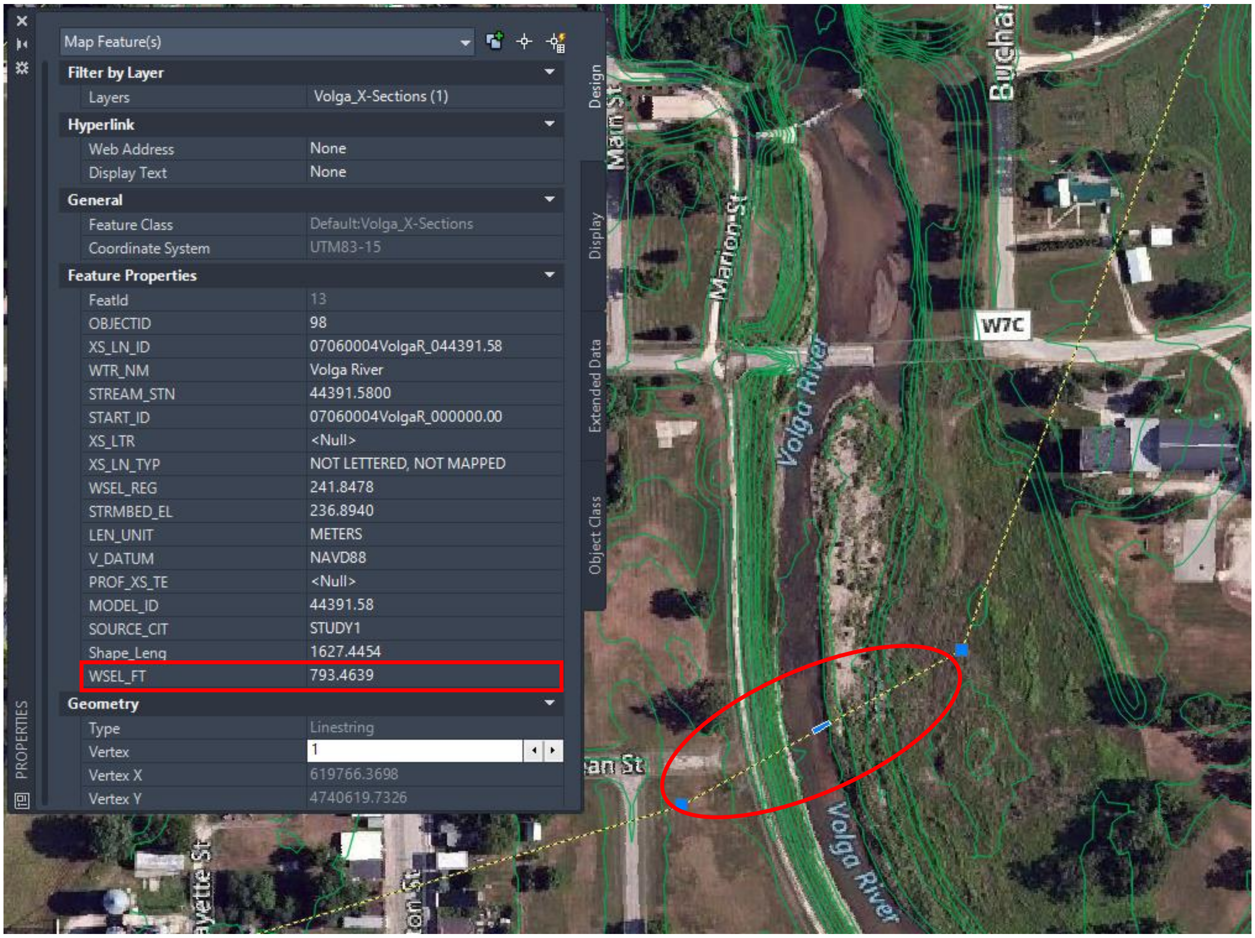


Figure B.1. Provided WSEL downstream from the bridge.

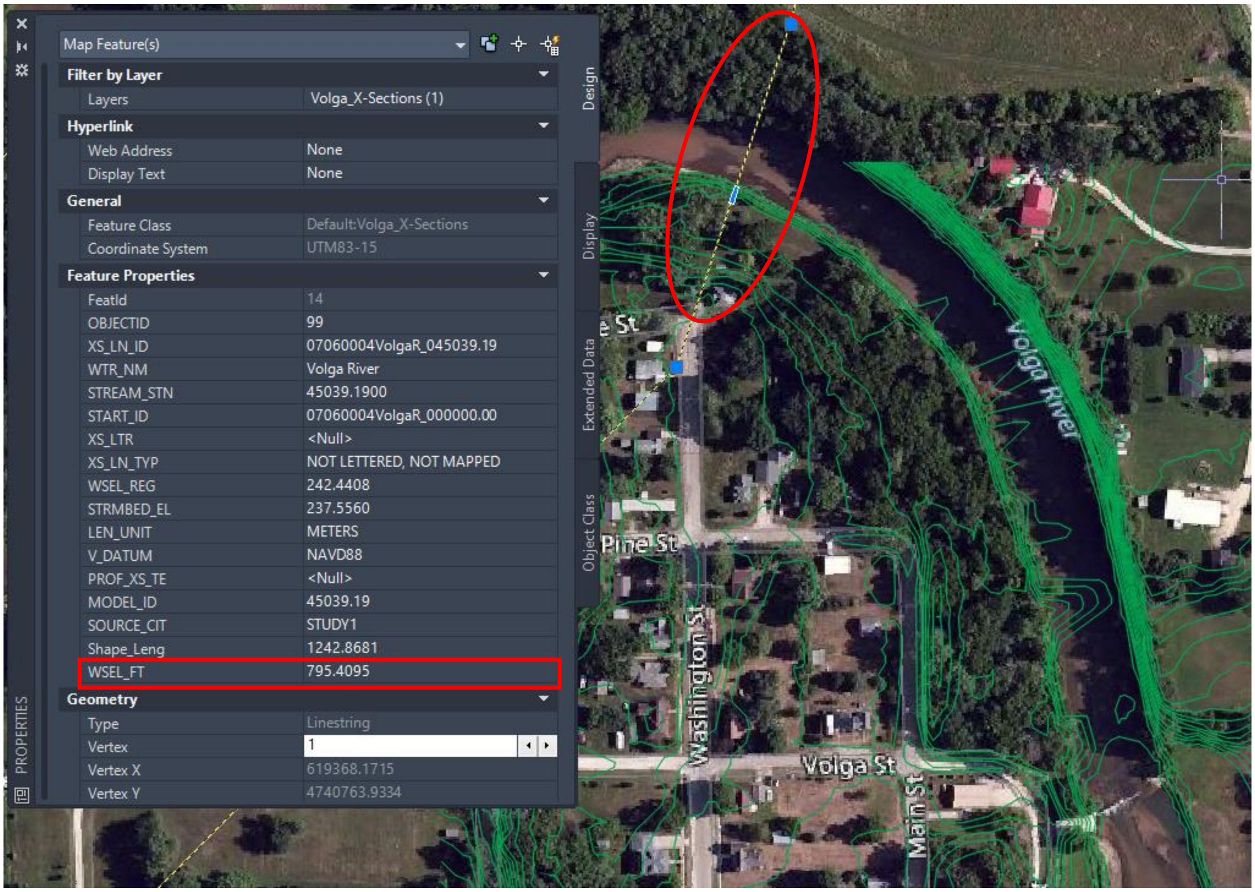


Figure B.2. Provided WSEL upstream from the bridge.

To estimate the WSEL at the proposed bridge location, the bridge was estimated to be halfway between the two sections provided. See Figure B.3 which depicts the relative locations of the sections and the bridge. The yellow lines represent the locations of the sections and the red line indicates the location of the proposed bridge.

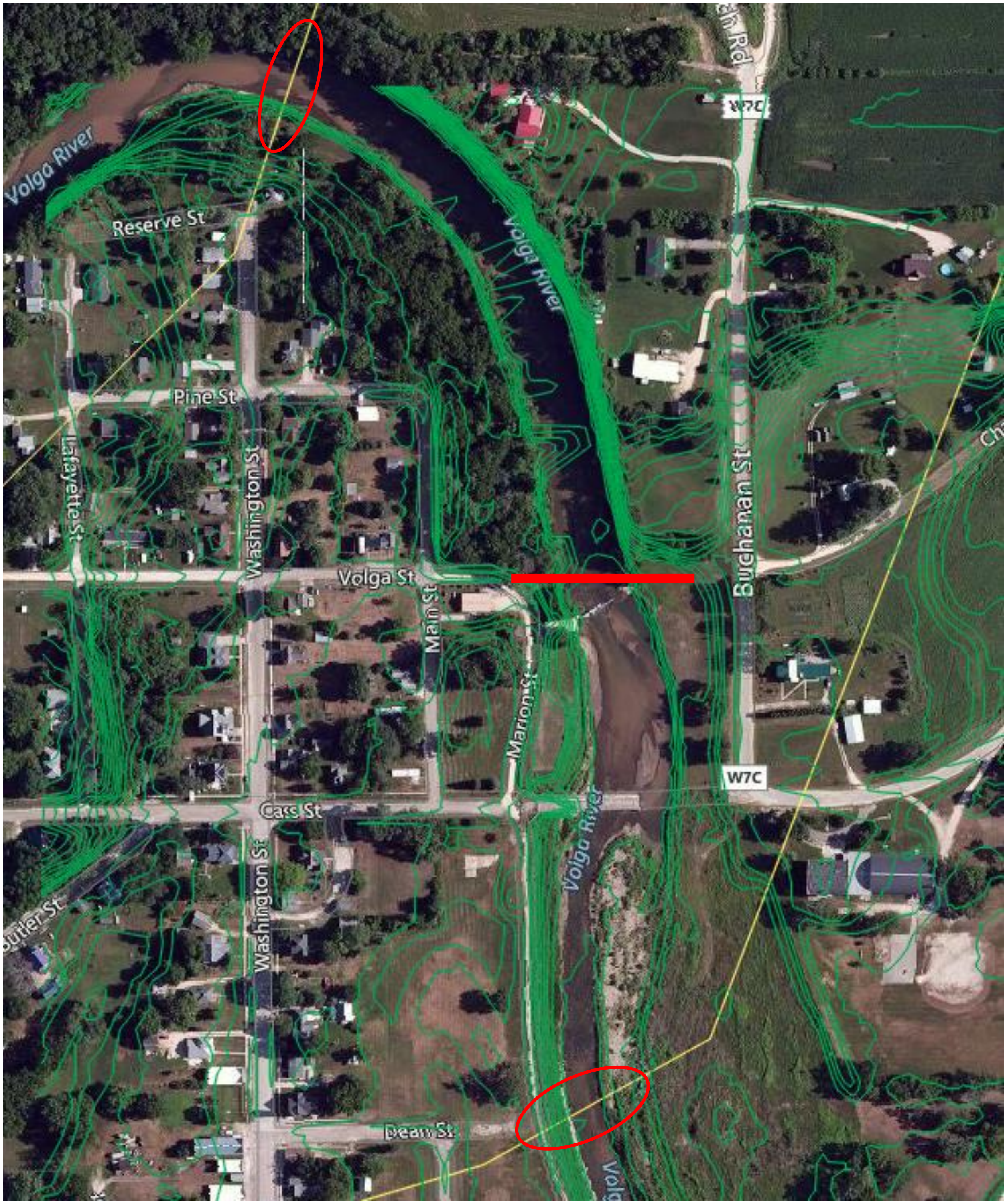


Figure B.3. Relative locations of the proposed bridge and the provided sections.

The calculation of the WSEL at the bridge is as follows:
WSEL = 0.5 x (793.4639 ft + 795.4095 ft) = 794.4367 ft ≅ 794.5 ft. This is the estimated BFE for the bridge location. To check that the estimated BFE is accurate, the 794.0 feet contour line

from Civil3D was compared to the preliminary FIRM from FEMA. See the comparison in Figure B.4.

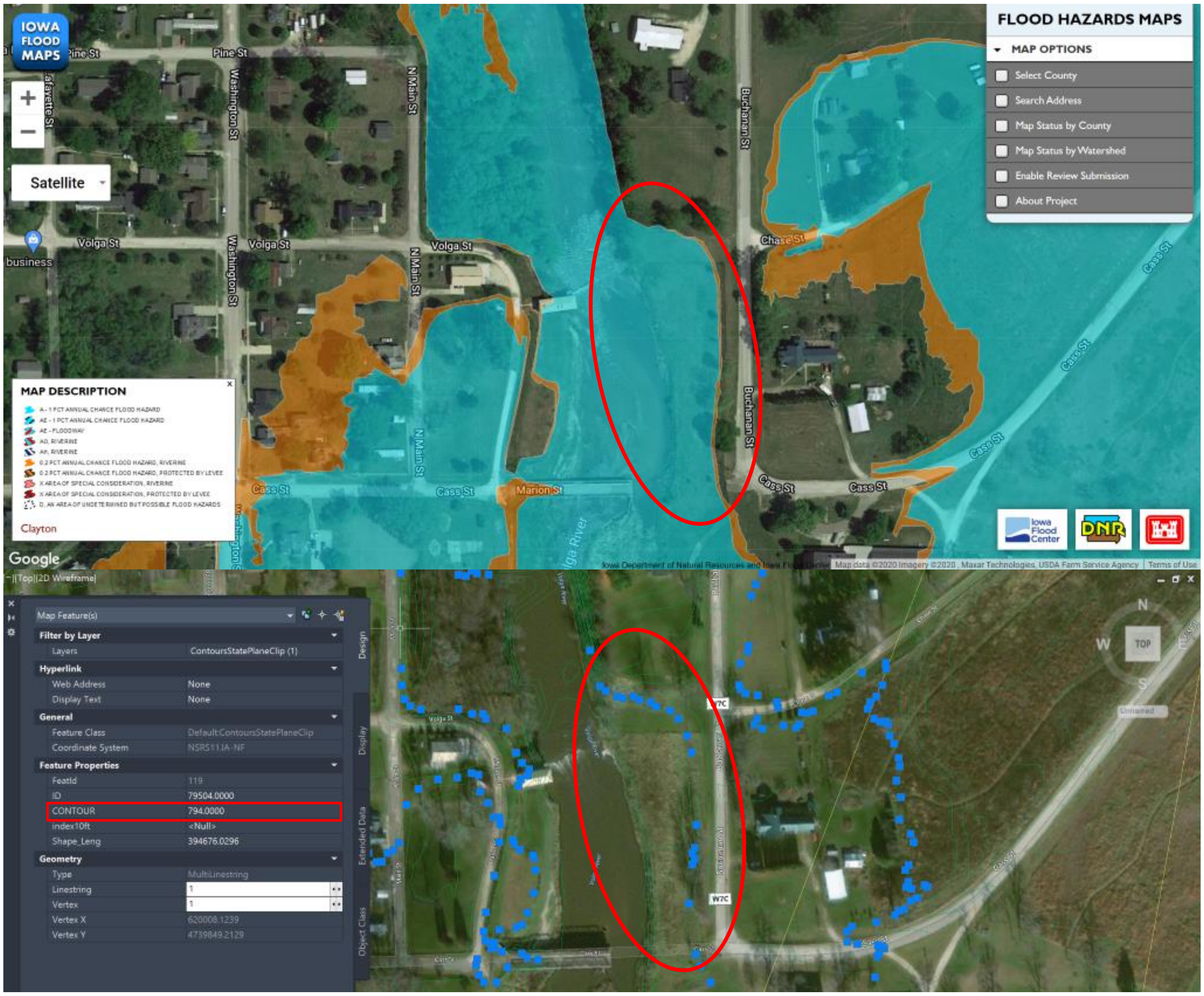


Figure B.4. Contour line comparison to the preliminary FEMA FIRM.

As shown in Figure B.4, the 794.0 contour line matches well with the blue 100-year flood elevation provided by FEMA. One last check of the BFE comes from USGS WaterWatch. A rating curve for the Volga River at Littleport, Iowa was created using USGS WaterWatch. Rating curves are used to estimate the WSEL for different discharge values. The discharge for a 100-year flood was determined to be 22,300 cfs, so the gage height at Littleport is estimated to be 21 feet. See the rating curve in Figure B.5. The datum for the gage height at Littleport is 677.0 feet above NGVD29, so a total height of 698.0 feet was determined for the BFE at Littleport. Next, the BFE was be converted to NAD83-11 so that a proper comparison could be made. The conversion from NGVD29 to NAD83-11 is shown in Figure B.6. From the conversion, the BFE for Littleport is 800.17 feet. A 5-foot increase in the BFE from Volga to Littleport is reasonable, therefore the BFE at Volga is reasonably accurate.

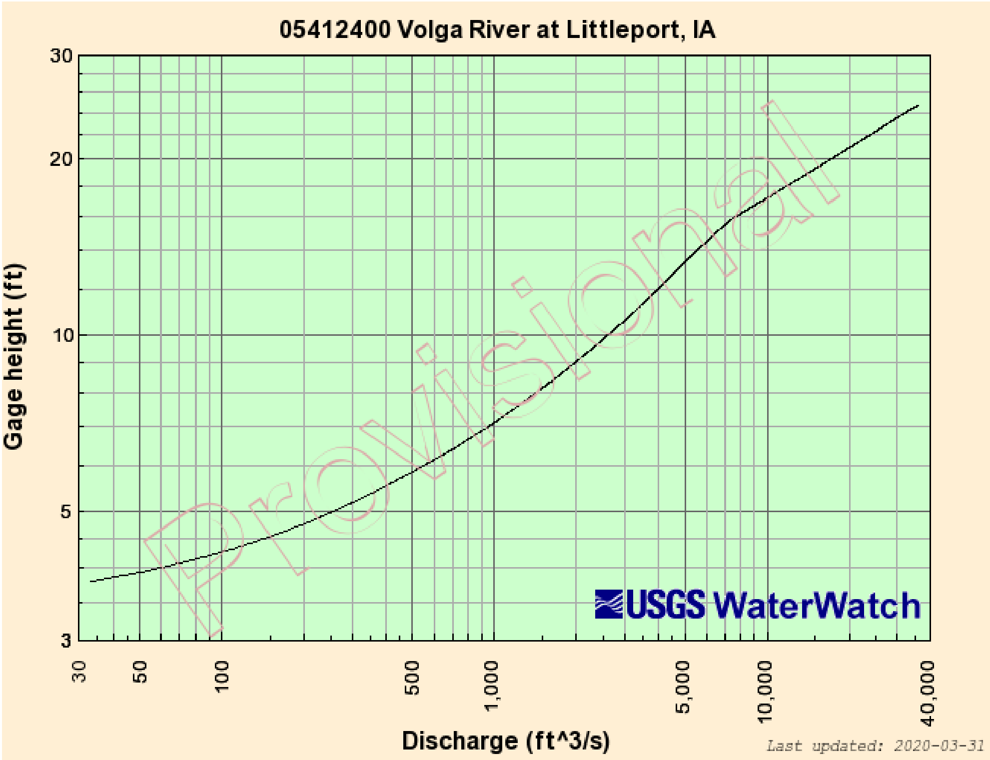


Figure B.5. Rating curve for the Volga River at Littleport, Iowa. From USGS WaterWatch.

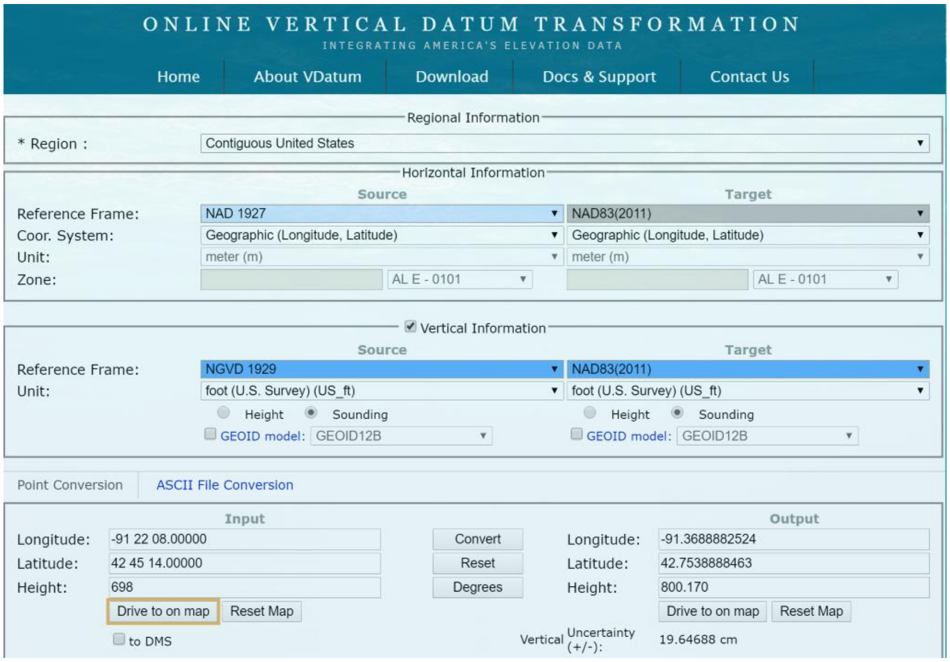


Figure B.6. Vertical datum transformation.

Appendix C – Substructure Design Calculations

The substructure designed includes abutments on the east and west ends of the pedestrian bridge as well as one pier to connect the two bridge spans. Below are the abutment design calculations.

Computation of Bearing Capacity of Spread Footing Stub Abutment:

Material properties:

Concrete density: $Wc := 0.150 \frac{kip}{ft^3}$

Concrete 28-day compressive strength: $f_c' := 4.0\ ksi$

Reinforcement strength: $fy := 60\ ksi$

Soil properties: (assumed)

Backfill:

- Unit weight: $\gamma b := 120\ pcf$
- Active pressure coefficient: $Kab := 0.33$
- Passive pressure coefficient: $Kpb := 3$
- Internal friction angle: $\phi' := 29\ deg$

In-situ soil:

- Unit weight: $\gamma := 120\ pcf$
- Saturated unit weight: $\gamma sat := 135\ pcf$
- Water unit weight: $\gamma w := 62.4\ pcf$
- Effective unit weight: $\gamma' := \gamma sat - \gamma w = 72.6\ pcf$
- Active pressure coefficient: $Ka := 0.4$
- Internal friction angle: $\phi := 32\ deg$

Cohesion: $c' := 0\ psf$

	$+\Delta H/H$	$-\Delta H/H$	K_o	K_a	K_p
Granular	0.0005–0.002	0.005–0.01	0.5	0.33	3.0
Cohesive	0.01–0.02	0.02–0.04	0.6	0.4	2.4

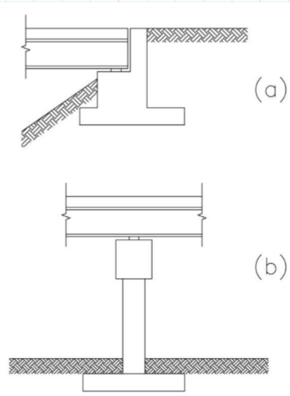
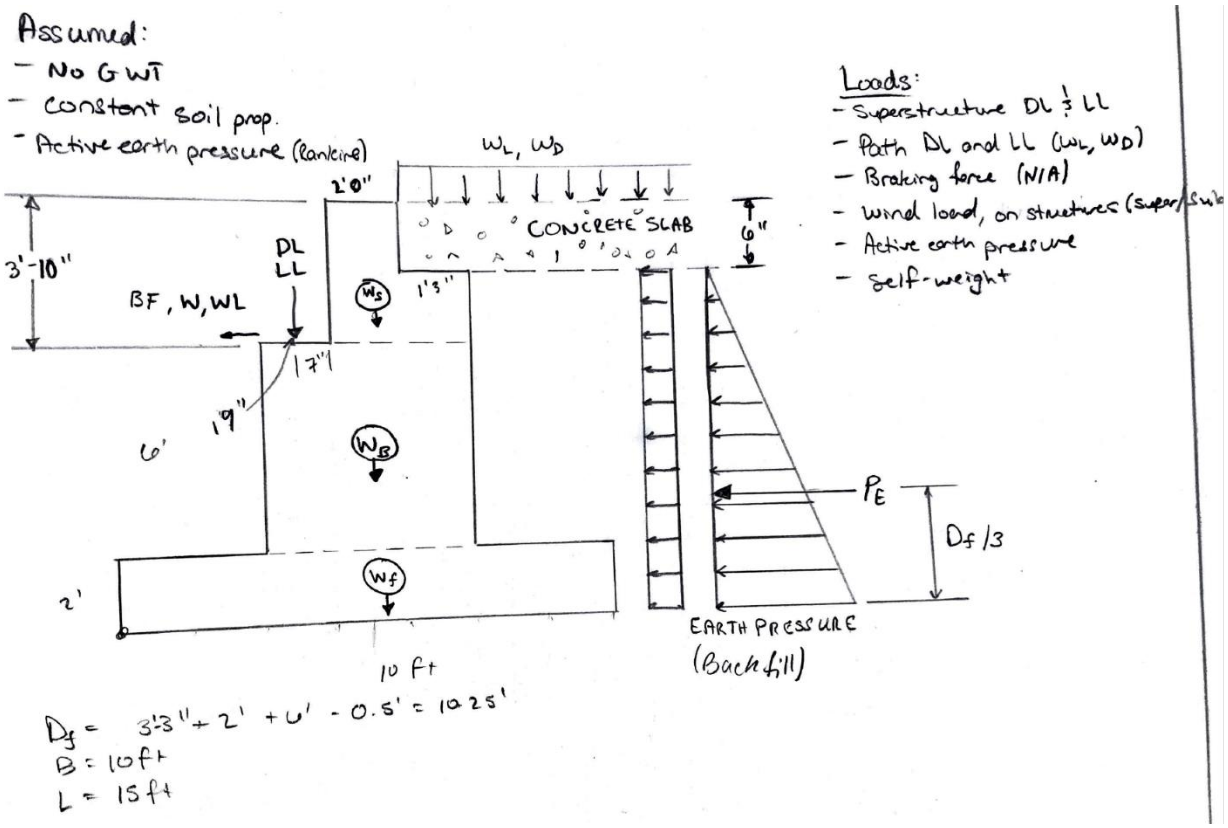


Figure 1 Spread footing applications: a) spread footing for a stub abutment; b) spread footing for a pier.

The abutment free-body diagram for analysis:



Abutment Loading Scenario:

Superstructure:
- Dead load of 47,875 lb (not including slab weight)
- Live load of 40,500 lb

Bridge reactions:
- Vertical dead reaction = 47,875 lb (4 per bridge)
- Vertical live reaction = 40,500 lb (4 per bridge)
- Vehicle reaction = 10,000 lb
- 20 psf wind uplift = -15,375 lb windward side and -5,125 lb leeward side
- Vertical wind reaction = ± 9,895 lb
- Horizontal wind reaction = 25,815 lb
- Longitudinal thermal reaction = 7,185 lb (at each of 4 base plates)

- Bridge total weight = 191,500 lb (not including slab weight)
- Bridge lifting weight = 81,700 lb (including slab weight)

$BridgeWeight := 191.5\ kip$

$VehicleLoad := 10\ kip$

$DL := 0.5 \cdot BridgeWeight + VehicleLoad = 105.75\ kip$

$BridgeLiveLoad := 4 \cdot 40.5\ kip = 162\ kip$

$LL := 0.5 \cdot BridgeLiveLoad = 81\ kip$

$deg := \frac{\pi}{180}$

Assumed Footing Dimensions:

Footing width: $B := 10\ ft$

Footing length: $L := 15\ ft$

Footing thickness: $t_f := 2\ ft$

Footing Depths:

Slab thickness: $t_{slab} := 6\ in = 0.5\ ft$

Backwall required height: $h_{bw} := 39\ in = 3.25\ ft$

Footing depth: $D_f := h_{bw} + t_f - t_{slab} + 6\ ft = 10.75\ ft$

Bearing Capacity Evaluation using Vesic's Bearing Capacity Equation:

Factor of Safety: $FS_q := 3$

Vesic's bearing capacity equation:

$$q_n = c'N_c\left(s_c d_c i_c b_c g_c\right) + q_s N_q\left(s_q d_q i_q b_q g_q\right) + \frac{1}{2}\gamma B N_\gamma\left(s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma\right)$$

$N_q := \exp(\pi \cdot \tan(\phi)) \cdot (\tan(45 \cdot deg + 0.5 \cdot \phi))^2 = 23.177$

$N_c := \dfrac{N_q - 1}{\tan(\phi)} = 35.49$

$N_\gamma := 2\left(N_q + 1\right)\tan(\phi) = 30.215$

Shape factors:

$s_c := 1 + \left(\dfrac{B}{L}\right)\left(\dfrac{N_q}{N_c}\right) = 1.435$

$s_q := 1 + \left(\dfrac{B}{L}\right)\tan(\phi) = 1.417$

$s_\gamma := 1 - 0.4\left(\dfrac{B}{L}\right) = 0.733$

Depth factors:

$\dfrac{D_f}{B} = 1.075 \quad \therefore \quad k := \operatorname{atan}\left(\dfrac{D_f}{B}\right) = 0.822\ rad$

$d_c := 1 + 0.4\ k = 1.329$

$d_q := 1 + 2\ k \cdot \tan(\phi)\left(1 - \sin(\phi)\right)^2 = 1.227$

$d_\gamma := 1$

Load inclination factors:

Since no Service I limit state required, the horizontal loads are neglected.

$i_c := 1 \quad i_q := 1 \quad i_\gamma := 1$

Base inclination factors:

Level base, so all are equal to 1.

$b_c := 1 \quad b_q := 1 \quad b_\gamma := 1$

Ground inclination factors:

Footing near slope, so need these factors.

$\beta := 15$ (assumed river bank slope, degrees)

$g_c := 1 - \dfrac{\beta}{147} = 0.898 \qquad g_q := (1 - \tan(\beta \cdot deg))^2 = 0.536 \qquad g_\gamma := g_q = 0.536$

Groundwater effects:
Assumed no groundwater table, so no groundwater effects on soil unit weight.

Soil surcharge load:

$q_s := D_f \cdot \gamma b = 1290 \ psf$

Allowable bearing pressure:

$q_n := c' \cdot N_c \cdot (s_c \cdot d_c \cdot i_c \cdot b_c \cdot g_c) + q_s \cdot N_q \cdot (s_q \cdot d_q \cdot i_q \cdot b_q \cdot g_q) + 0.5 \cdot \gamma b \cdot B \cdot N_\gamma \cdot (s_\gamma \cdot d_\gamma \cdot i_\gamma \cdot b_\gamma \cdot g_\gamma)$

$q_n = 34.971 \ ksf$

Dead load eccentricity:

$e_{DL} := 19 \ in = 1.583 \ ft$

Reduction factor due to eccentricity:

$R_e := 1 - \left(\frac{e_{DL}}{B}\right)^{0.5} = 0.602$

Reduced allowable bearing pressure:

$q_n' := q_n \cdot (1 - R_e) = 13.915 \ ksf$

Applied Bearing Pressure:

Total vertical load (dead + live), no load combinations:

$P_{tot} := DL + LL = 186.75 \ kip$

$A_f := B \cdot L = 150 \ ft^2$

$q_{applied} := \frac{P_{tot}}{A_f} = 1.245 \ ksf$

Bearing Capacity Safety Factor Check:
Check if the allowable bearing pressure is greater than the applied pressure multiplied by a factor of safety

$FS_q \cdot q_{applied} = 3.735 \ ksf$ < $q_n' = 13.915 \ ksf$ OK $FS_{true} := \frac{q_n'}{q_{applied}} = 11.177$

Overturning Stability Analysis:

Active earth pressure:

$P_a := 0.5 \cdot \gamma b \cdot Kab \cdot D_f^{\ 2} = 2.288 \ \frac{kip}{ft}$ $\qquad h_a := \frac{2\ D_f}{3} = 7.167\ ft$

Overturning moments: moments about footing heel
Active earth pressure

$M_o := P_a \cdot (D_f - h_a) \cdot L = 122.987\ ft \cdot kip$

Resisting moments: moments about footing heel
Live load, dead load, concrete weight, and trail slab weight

$M_L := LL \cdot (5\ ft - (1\ ft + 7\ in)) = 276.75\ ft \cdot kip$

$M_D := DL \cdot (5\ ft - (1\ ft + 7\ in)) = 361.313\ ft \cdot kip$

$M_R := M_L + M_D = 638.063\ ft \cdot kip$

Overturning factor of safety:

$FS_o := \frac{M_R}{M_o} = 5.188$ > 3 OK

Sliding Stability Analysis:

Total vertical loading:

$P_{tot} = 186.75\ kip$

Frictional resistance:

$F_{max} := P_{tot} \cdot \tan(\phi) + B \cdot L \cdot c' = 116.694\ kip$

Factor of safety againat sliding:

$FS_v := \frac{F_{max}}{P_a \cdot L} = 3.4$ > 3 OK

Settlement Analysis using Bowles' Method:

Assumed in-situ elastic soil properties: (from Bowles text)

Modulus of elasticity: $E := 3000\ psi$

Poisson's ratio: $\mu := 0.3$

Elastic settlement calculation at center of footing:

$H_{eq} := 5 \cdot B = 50\ ft$

$q_{net} := q_{applied} = 8.646\ psi$

$\alpha := 4 \qquad B' := \frac{B}{2} = 5\ ft \qquad L' := \frac{L}{2} = 7.5\ ft$

$M := \frac{L'}{B'} = 1.5 \qquad N := \frac{H_{eq}}{B'} = 10$

$DepthRatio := \frac{D_f}{B} = 1.075 \qquad LengthRatio := \frac{L}{B} = 1.5$

$I_f := 0.7$ From D/B plot in Bowles text

$$I_1 := \frac{1}{\pi} \cdot \left(M \cdot \ln\left(\frac{\left(1+\sqrt{M^2+1}\right) \cdot \sqrt{M^2+N^2}}{M \cdot \left(1+\sqrt{M^2+N^2+1}\right)} \right) + \ln\left(\frac{\left(M+\sqrt{M^2+1}\right) \cdot \sqrt{1+N^2}}{M+\sqrt{M^2+N^2+1}} \right) \right) = 0.584$$

$$I_2 := \frac{N}{2 \cdot \pi} \cdot \operatorname{atan}\left(\frac{M}{N \cdot \sqrt{M^2+N^2+1}} \right) = 0.023$$

$$I_s := I_1 + \left(\frac{1-2\ \mu}{1-\mu} \right) \cdot I_2 = 0.598$$

$$\delta_{ERigidCenter} := 0.93 \cdot \left(\alpha \cdot I_s \cdot I_f \cdot \left(\frac{q_{net} \cdot \left(1-\mu^2\right)}{E} \right) B' \right) = 0.245\ in$$

Checking settlement at corners:

$\alpha := 1 \qquad B' := B \qquad L' := L \qquad M := \frac{L'}{B'} = 1.5 \qquad N := \frac{H_{eq}}{B'} = 5$

$$I_1 := \frac{1}{\pi} \cdot \left(M \cdot \ln\left(\frac{\left(1+\sqrt{M^2+1}\right) \cdot \sqrt{M^2+N^2}}{M \cdot \left(1+\sqrt{M^2+N^2+1}\right)} \right) + \ln\left(\frac{\left(M+\sqrt{M^2+1}\right) \cdot \sqrt{1+N^2}}{M+\sqrt{M^2+N^2+1}} \right) \right) = 0.496$$

$$I_2 := \frac{N}{2 \cdot \pi} \cdot \operatorname{atan}\left(\frac{M}{N \cdot \sqrt{M^2+N^2+1}} \right) = 0.045 \qquad I_s := I_1 + \left(\frac{1-2\ \mu}{1-\mu} \right) \cdot I_2 = 0.521$$

$$\delta_{ERigidCorner} := 0.93 \cdot \left(\alpha \cdot I_s \cdot I_f \cdot \left(\frac{q_{net} \cdot (1-\mu^2)}{E} \right) B' \right) = 0.107 \; in$$

Since both the corner settlement and the center settlement values for the abutment foundation are very small, the design is OK. Settling limits are approximately 0.5" inches for footings.

Abutment Structural Analysis Using ACI 318-11:

Abutment details and dimensions:

Thickness of stem	$t_{stem} := 2 \; ft$
Length of toe	$l_{toe} := 2.75 \; ft$
Length of heel	$l_{heel} := 2.25 \; ft$
Thickness of footing	$t_{foot} := 2 \; ft$
Concrete cover	$d_c := 3 \; in$
Stem Steel	#5 @ 12 in
Heel Steel	#5 @ 12 in
Transverse reinforcing	#5 @ 12 in
Yield strength of steel	$f_y := 60 \; ksi$
Resistance Factor for Tension Controlled Concrete	$\phi_f := 0.90$
Resistance Factor for Shear	$\phi_v := 0.90$

Loading and soil data:

Allowable bearing capacity w/ factor of safety $q_b := \frac{q_n'}{FS_q} = 4638.411\ psf$

Self pressure $q_{self} := t_{foot} \cdot Wc + \gamma b \cdot (D_f - t_{foot}) = 1350\ psf$

Allowable soil pressure $q_{allow} := q_b - q_{self} = 3288.411\ psf$

Area of footing needed $A_{needed} := \frac{P_{tot}}{q_{allow}} = 56.79\ ft^2$

Footing weight $W_{foot} := Wc \cdot B \cdot L \cdot t_{foot} = 45000\ lbf$

Overburden weight $W_{OB} := 0.5\ \gamma b \cdot B \cdot L \cdot (D_f - t_{foot}) = 78750\ lbf$

Soil pressure $q_{soil} := (P_{tot} + W_{foot} + W_{OB}) \div (L \cdot B) = 2070\ psf$
OK since $q_{allow} = 3288\ psf$

Upward pressure on the footing for use in shear and flexural design:

Factored vertical load:

$P_u := \max(1.4 \cdot DL, 1.2 \cdot DL + 1.6 \cdot LL) = 256.5\ kip$

Upward pressure on the footing from the soil: $q_u := \frac{P_u}{A_f} = 1.71\ ksf$

Steel cover and diameters:

$d_b := 0.625\ in$ $c_c := 3\ in$ $c_1 := c_c + 0.5\ d_b = 3.313\ in$ $c_2 := c_c + d_b + 0.5\ d_b = 3.938\ in$

$c_{avg} := 0.5\ (c_1 + c_2) = 3.625\ in$

Effective depth of footing: $d := t_{foot} - c_{avg} = 20.375\ in$

Check for one-way shear at critical section, distance d from face of stem:

$V_{uOneWay} := q_u \cdot B \cdot \left(\frac{L - c_1}{2} - d\right) = 96.855\ kip$

$\lambda := 1$ normal weight concrete

$\phi V_c := 0.75 \cdot 2 \cdot 1 \cdot \sqrt{4000} \cdot 10 \cdot 12 \cdot \frac{20.375}{1000} = 231.953$ $\therefore$ $\phi V_c := 231.953\ kip$

Since $V_u < \phi V_c$, the footing thickness is adequate for one-way shear.

Check for two-way (punching shear:

$$V_{uPunchOut} := q_u \cdot \left(A_f - \left(c_1 + d\right)\left(c_2 + d\right)\right) = 249.661 \ kip$$

$$\beta := \frac{c_1}{c_2} = 0.841 \qquad b_o := 2 \cdot \left(c_1 + d\right) + 2 \cdot \left(c_2 + d\right) = 96 \ in \qquad \alpha_s := 40$$

$$\phi V_{cPunchOut} := 0.75 \cdot min\left(4, \left(2 + \frac{4}{\beta}\right), \left(2 + \alpha_s \cdot \frac{d}{b_o}\right)\right) \cdot 1 \cdot \sqrt{4000} \cdot 96 \cdot 20.375 = 371124.906$$

$$\phi V_{cPunchOut} := 371.125 \ kip$$

Since $V_{uPunchOut} < \phi V_{cPunchOut}$, the footing is adequate for two-way shear.

Flexural reinforcement:

Width of rectangular section: $b := B = 120 \ in$

Bending moment (cantilever beam: $M_u := q_u \cdot B \cdot \left(\frac{L - c_1}{2}\right)\left(\frac{L - c_2}{4}\right) = 461.76 \ ft \cdot kip$

Preliminary area of steel using rule of thumb: $A_s = M_u / (4 \ d)$

$$A_{sPrelim} := \frac{461.76}{4 \cdot 20.375} = 5.666 \qquad A_b := 0.310 \ in^2 \qquad N_{bars} := \frac{5.666}{0.310} = 18.277$$

Use 19#5 bars. Minimum steel area based on shrinkage and temperature for slabs:

$\rho_{min} := 0.0018$ for $f_y = 60000 \ psi$ $\qquad A_s := 19 \cdot A_b = 5.89 \ in^2$

$A_{sMin} := \rho_{min} \cdot b \cdot t_{foot} = 5.184 \ in^2$ Trial area more than this so OK.

Max bar spacing: $s_{max} := min\left(3 \cdot t_{foot}, 18 \ in\right) = 18 \ in$

Bar spacing provided: $s_{prov} := \frac{b}{19} = 6.316 \ in$ Less than max so OK.

Check flexural strength of rectangular section using standard procedure:

$$\beta_1 := \max\left(0.65, 0.85 - 0.05 \cdot \frac{f_c' - 4000 \ psi}{1000 \ psi}\right) = 0.85 \qquad E_s := 29000 \ ksi \qquad \varepsilon_{ty} := \frac{f_y}{E_s} = 0.002$$

$A_{sTensionControlled} := \frac{0.85 \cdot f_c' \cdot b \cdot \beta_1}{f_y} \cdot \left(\frac{3 \cdot d}{8}\right) = 44.163 \ in^2$ We provide less than this so OK

$$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c' \cdot b} = 0.866 \ in \qquad M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 587.289 \ ft \cdot kip \qquad \phi_F := 0.9$$

$$M_r := \phi_F \cdot M_n = 528.56 \ ft \cdot kip \qquad M_u = 461.76 \ ft \cdot kip$$

Created with PTC Mathcad Express. See www.mathcad.com for more information.

Since flexural capacity is greater than flexural strength required, the design is adequate.

Check rebar development length:

$\psi_t := 1 \quad \psi_e := 1 \quad \psi_s := 0.8 \quad \lambda := 1 \quad c_s := 3\ in \quad c_t := 3\ in \quad s_b := s_{prov} = 6.316\ in$

$\psi_g := 1 \quad R_s := M_u \div (\phi_F \cdot M_n) = 0.874 \quad c_b := min(c_s, c_t, s_b \cdot 0.5) = 3\ in$

$n := 2 \quad A_{st} := 0.2\ in^2 \quad A_{tr} := 2 \cdot A_{st} = 0.4\ in^2 \quad s_{tr} := 12\ in$

$K_{tr} := \frac{40 \cdot A_{tr}}{s_{tr} \cdot n} = 0.667\ in \quad R_t := min\left(2.5\ in, \frac{c_b + K_{tr}}{0.65}\right) = 2.5\ in$

$$l_d := \max\left(12\ in, \frac{3}{40} \cdot R_s \cdot \frac{60000}{\lambda \cdot min(100, \sqrt{4000})} \cdot \frac{\psi_s \cdot \psi_g \cdot min(\psi_t \cdot \psi_e, 1.7)}{2.5} \cdot d_b\right) = 12.432\ in$$

Footing size is enough to accomodate the full bar development. OK.

Bearing capacity of column at base:

$columnWidth := 2\ ft = 24\ in \quad A_1 := columnWidth \cdot columnWidth = 576\ in^2$

$l := min(L, 2 \cdot t_{foot} + columnWidth + 2 \cdot t_{foot}) = 120\ in$

$A_2 := l^2 = 14400\ in^2 \quad N_1 := 0.65 \cdot (0.85 \cdot f_c' \cdot A_1) = 1272.96\ kip$

$N_2 := 0.65 \cdot min\left(0.85 \cdot f_c' \cdot A_1 \cdot \sqrt{\frac{A_2}{A_1}}, 2 \cdot 0.85 \cdot f_c' \cdot A_1\right) = 2545.92\ kip$

$\phi P_{nb} := min(N_1, N_2) = 1272.96\ kip$ > factored load from column, P_u. OK.

Dowel bar connection to stem: since concrete bearing strength at column base is adequate, we just need the minimum area for dowels.

$A_{DMin} := 0.005 \cdot A_1 = 2.88\ in^2$ provide 4#8 bars. $A_{sDowel} := 4 \cdot 0.79\ in^2 = 3.16\ in^2$

Development for dowel bars in compression:

$$l_{dc} := \max\left(8\ in, \frac{0.015 \cdot 60000 \cdot 1\ in}{\lambda \cdot min(100, \sqrt{4000})}, 0.0003 \cdot 60000 \cdot 1\ in\right) = 18\ in$$

Checking footing thickess to accomodate dowel bars:

radius of dowel bar end: $r := 0.5 \cdot 1\ in = 0.5\ in$ $d_{bDowel} := 1.0\ in$

$h_{req} := l_{dc} + r + d_{bDowel} + d_b + 3\ in = 23.125\ in$ 24 in provided so OK.

Dowel bar splice into stem:

$$l_{dcCol} := \max\left(8\ in, \frac{0.015 \cdot 60000 \cdot d_{bDowel}}{\lambda \cdot min\left(100, \sqrt{4000}\right)}, 0.0003 \cdot 60000 \cdot d_{bDowel}\right) = 18\ in$$

$$l_{splice} := \max\left(12\ in, l_{dcCol}, 0.0005 \cdot 60 \cdot 1\ in \cdot 40\right) = 18\ in$$

Extend dowel bars 1.5 ft into the stem.

See the separate appendix for design drawings and final abutment details. Below are the design calculations for the pier column, footing, and pile foundation.

Volga Pier Calculations Using AASHTO LRFD and Iowa DOT BDM:

Hydraulic data:

Average flow velocity: $V_{avg} := 5.5\ \frac{ft}{s}$

Low flow elevation above top of footing: $H_w := 4.0\ ft$

Water force (WA) calculations:

Drag coefficients:

Longitudinal $C_D := 1.40$ (assume debris is present)

Lateral $C_L := 0.00$ (no skew between pier and stream flow)

Average hydraulic pressures:

Longitudinal pressure: $p_{Davg} := \frac{C_D \cdot 5.5^2}{1000} = 0.042$ $p_{Davg} := 0.042\ ksf$

Lateral pressure: $p_{Lavg} := \frac{C_L \cdot 5.5^2}{1000} = 0$ $p_{Lavg} := 0\ ksf$

Maximum pressures:

Longitudinal: $p_{Dmax} := 2 \cdot p_{Davg} = 0.084\ ksf$

Lateral: $p_{Lmax} := 2 \cdot p_{Lavg} = 0\ ksf$

Column depth (width with respect to the stream channel): $D_c := 3.0\ ft$

Stream force on column:

Longitudinal: $P_D := p_{Dmax} \cdot D_c = 0.252\ \frac{kip}{ft}$

Lateral: $P_L := p_{Lmax} \cdot D_c = 0\ \frac{kip}{ft}$

Total column buoyancy force: $F_{BC} := H_w \cdot 5\ ft \cdot D_c \cdot 0.0624\ \frac{kip}{ft^3} = 3.744\ kip$ up

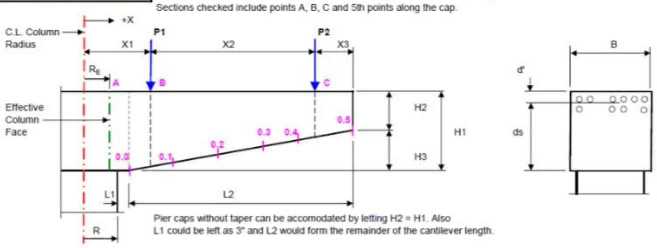
Column buoyancy force per foot: $w_{BC} := 5\ ft \cdot D_c \cdot 0.0624\ \frac{kip}{ft^3} = 0.936\ \frac{kip}{ft}$ up

Footing buoyancy force: $F_{BF} := 2\ ft \cdot 8\ ft \cdot 6\ ft \cdot 0.0624\ \frac{kip}{ft^3} = 5.99\ kip$ up

IADOT Pier Calculations:

Pier cap overhang design:

PIER CAP OVERHANG DESIGN Includes T-Piers or Frame Piers with either one or two beams on the overhang. Sections checked include points A, B, C and 5th points along the cap.



Pier caps without taper can be accomodated by letting H2 = H1. Also L1 could be left as 3" and L2 would form the remainder of the cantilever length.

General Input Note: X1, H3, and R_E are calculated based on other dimensions

All dimensions are along skew of pier cap

Is this a T-Pier? Y or N	Y		This entry only affects the calculation of R_E
Edge Dist. bt. Column and Cap Taper, L1 (Typ. 3")	3.000	in	
Taper Length, L2	5.000	ft	
Column Radius, R	2.500	ft	
Cap Height, H1	7.000	ft	Note: Input items P1 and X2 shall be set to 0 if there is only one beam on the overhang. [P1 shall only be considered to be on the overhang if it falls to the right of the effective column face.]
Cap Height at End, H2	3.000	ft	
Beam Spa, X2 -- Enter 0 if only one beam on overhang	0.000	ft	
Distance bt. Exterior Beam and Cap End, X3	1.000	ft	
Pier Cap Depth, B	3.000	ft	
Estimated Dist. from Cap Top to C.G. of Top Reinf., d'	5.000	in	

Concentrated Beam Loads	Unfactored Loads		Hover for extended comment	Load Factors	Factored Loads		
Enter 0 kips for all Int. Beam Loads (P1 Loads) if only one beam is on the overhang.	Interior Beam, P1	Exterior Beam, P2			Interior Beam, P1	Exterior Beam, P2	
DC Load (DC1 and DC2)	0.000	47.875	k	1.250	0.000	59.844	k
DW Load	0.000	0.000	k	1.500	0.000	0.000	k
LL+IM Load (Truck with Impact and Lane)	0.000	40.500	k	1.750	0.000	70.875	k
Total Load	0.000	88.375	k		0.000	130.719	k

	Unfactored Load		Load Factors	Factored Load	
Distributed Pier Diaphragm Weight	0.001	k/ft	1.250	0.001	k/ft

Bar Size for the Side Reinforcing Bars	5	Currently not used

Concrete Strength, f'c	4.000	ksi	Aashto Lrfd 5.4.2.1, typically 4 ksi
Flexural Reinforcement Yield Strength, fy	60.000	ksi	Aashto Lrfd 5.4.3.1, typically 60 ksi
Shear Reinforcement Yield Strength, fy	60.000	ksi	Aashto Lrfd 5.4.3.1, typically 60 ksi
Reinforcement Modulus of Elasticity, Es	29000.000	ksi	Aashto Lrfd 5.4.3.2, typically 29,000 ksi
Flexure Resistance Factor, ϕ_f	0.900		Aashto Lrfd 5.5.4.2, begin by assuming a tension-controlled section, $\phi = 0.90$
Shear Resistance Factor, ϕ_v	0.900		Aashto Lrfd 5.5.4.2, typically $\phi = 0.90$
Exposure Factor, γ_e (Typically 1.00)	1.000		Aashto Lrfd 5.6.7, Class 1 and 2 exposure factors are 1.00 and 0.75 respectively
Concrete Unit Weight, γ_c	0.150	kcf	

Intermediate Calculations Aashto Lrfd 3.5.1 and 5.4.2.4, typically 0.150 kcf

Whitney Stress Block Factor, α_1	0.850		Aashto Lrfd 5.6.2.2
Whitney Stress Block Factor, β_1	0.850		Aashto Lrfd 5.6.2.2
Compression-controlled Strain Limit, ε_{cl}	0.002		Aashto Lrfd 5.6.2.1
Tension-controlled Strain Limit, ε_{tl}	0.005		Aashto Lrfd 5.6.2.1
Concrete Modulus of Elasticity, Ec	4266.223	ksi	Aashto Lrfd 5.4.2.4, aggregate correction factor is set to 1.0
Modular Ratio, n	7.000		Aashto Lrfd 5.6.1, rounded to nearest integer
Height of Tapered Section, H3	4.000	ft	
Distance from C.L. Column to Interior Beam, X1	6.750	ft	If only one beam is on the overhang then this is the distance to the exterior beam
Dist. from C.L. Column to Effective Column Face, R_E	1.309	ft	Calculation is slightly different depending on pier type

	Critical Points			Fifth Points Along Taper						
	A	B	C	0	1	2	3	4	5	
Dist. from C.L. Column to Point of Interest, X	1.309	6.750	6.750	2.750	3.750	4.750	5.750	6.750	7.750	ft
Dist. from Cap End to Point of Interest, X1+X2+X3-X	6.441	1.000	1.000	5.000	4.000	3.000	2.000	1.000	0.000	ft
Section Height, Hx	7.000	3.800	3.800	7.000	6.200	5.400	4.600	3.800	3.000	ft
Estimat'd Dist. from Cap Bot. to C.G. of Bar Group, d_s	6.583	3.383	3.383	6.583	5.783	4.983	4.183	3.383	2.583	ft
Factored Shear, Vu, due to P1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	k
Factored Shear, Vu, due to P2	130.72	130.72	130.72	130.72	130.72	130.72	130.72	130.72	0.00	k
Factored Shear, Vu, due to Diaphragm	0.01	0.00	0.00	0.01	0.01	0.00	0.00	0.00	0.00	k
Factored Shear, Vu, due to Pier Cap	19.74	1.91	1.91	14.06	10.35	7.09	4.28	1.91	0.00	k
Total Factored Shear, Vu	150.46	132.63	132.63	144.79	141.07	137.81	135.00	132.63	0.00	k
Factored Moment, Mu due to P1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	k*ft
Factored Moment, Mu due to P2	711.24	0.00	0.00	522.88	392.16	261.44	130.72	0.00	0.00	k*ft
Factored Moment, Mu due to Diaphragm	0.03	0.00	0.00	0.02	0.01	0.01	0.00	0.00	0.00	k*ft
Factored Moment, Mu due to Pier Cap	54.82	0.92	0.92	30.47	18.30	9.62	3.98	0.92	0.00	k*ft
Total Factored Moment, Mu	766.09	0.92	0.92	553.36	410.47	271.06	134.70	0.92	0.00	k*ft

Estimate Flexural Reinforcement Required

	Critical Points			Fifth Points Along Taper						
	A	B	C	0	1	2	3	4	5	
Rough Estimate of A_s required at each section	2.170	0.005	0.005	1.564	1.320	1.011	0.598	0.005	0.000	in^2

Rough Estimate of Maximum A_s required		2.170	in^2
Estimate of the number of bars required for bar sizes:	#7	4	
	#8	3	
	#9	3	
	#10	2	
	#11	2	

Note: The flexural reinforcement information on the left is an estimate of what is required for the overhang. In the next section of the spreadsheet the user can enter the actual reinforcement to be used in the design checks.

Enter Flexural and Shear Reinf. for Design Checks

Stirrup Bar Size (i.e. #4, #5, etc.)	4	
Number of Stirrup Legs (Typ. 4 legs for double hoops)	2	
Total Area of Shear Stirrups	0.400	in^2

Stirrup spacing is entered later.

Bar Size for Flexural Reinforcement (i.e. #9, #10, etc.)	9

Layer	d' (in) by Layer	No. of Bars per Layer
1	3.1875	8
2	7.1875	
3	11.1875	
4	15.1875	

Total Bar Area Input, A_s provided	8.000	in^2
Distance from Top of Cap to C.G. of Bar Group, d'	3.188	in

Total bar area is lumped at its center of gravity.

Enter Flexural Reinf. Development Lengths Aashto Lrfd 5.10.8.2

Available Cap Length for #9 Development Length	75.292	in	Measured from critical sxn and assumes 2" end clearance.

Straight Bar Development Length (Non-Epoxy Coated Reinforcement)			Aashto Lrfd 5.10.8.2.1a-c
Basic Development Length for a # 9 Bar	81.216	in	
Reinforcement Location Factor, λ_{rl}	1.300	Top bar factor	
Distance from Bar Center to Nearest Concrete Surface	3.188	in	Used in determination of c_b. Set to cell D113 by default, user can overwrite.
One-half Center to Center Bar Spacing	4.267	in	Used in determination of c_b. User can overwrite.
Reinforcement Confinement Factor, λ_{rc}	0.400		A_{tr} and therefore k_{tr} assumed to be 0.
Excess Reinforcement Factor, λ_{er}	0.271	By default set equal to g96/g118. User can overwrite.	
Development Length for a # 9 Bar	12.000	in	
Enter Develop Length Used for a # 9 Bar	12.000	in	
Is Cap Long Enough for Develop Length?	YES		

Standard Hook Development Length (Non-Epoxy Coated Reinforcement)			Aashto Lrfd 5.10.8.2.4
Basic Development Length for a # 9 Bar	21.432	in	
Reinforcement Confinement Factor, λ_{rc}	0.800	User can overwrite.	
Excess Reinforcement Factor, λ_{er}	0.271	By default set equal to g96/g118. User can overwrite.	
Development Length for a # 9 Bar	9.024	in	
Enter Develop Length Used for a # 9 Bar	10.000	in	
Is Cap Long Enough for Develop Length?	YES		

Enter Bar End Type for Development (S = straight, H = hook)	S		
Development Length Used for a # 9 Bar	12.000	in	This is used to determine effective area of flexural reinforcement.

Check Flexural Reinforcement

Flexural Capacity Check Aashto Lrfd 5.6.3.2

	Critical Points			Fifth Points Along Taper						
	A	B	C	0	1	2	3	4	5	
Bar Area Provided, A_s	8.000	8.000	8.000	8.000	8.000	8.000	8.000	8.000	0.000	in^2
Fraction of Bar Area that is Effective	1.000	0.833	0.833	1.000	1.000	1.000	1.000	0.833	0.000	
Effective Bar Area Provided, A_{se}	8.000	6.667	6.667	8.000	8.000	8.000	8.000	6.667	0.000	in^2
Factored Applied Moment, Mu_z	766.088	0.919	0.919	553.359	410.466	271.062	134.696	0.919	0.000	k*ft
Depth of Equivalent Stress Block, a	3.922	3.268	3.268	3.922	3.922	3.922	3.922	3.268	0.000	in
Distance from Cap Bottom to C.G. of Bar Group, d_s	80.813	42.413	42.413	80.813	71.213	61.613	52.013	42.413	32.813	in
Factored Flexural Resistance, $Mr_z = \phi Mn_z$	2838.662	1223.355	1223.355	2838.662	2493.062	2147.462	1801.862	1223.355	0.000	k*ft
Is Mr_z >= Mu_z ?	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	

Minimum Reinforcement Check Aashto Lrfd 5.4.2.6 and 5.6.3.3

Yield to Ult. Tensile Strength Ratio, γ_3	0.750	Using 0.75 rather than 0.67 is conservative for Grade 60 reinforcement.	
Flexural Cracking Variability Factor, γ_1	1.600		
Modulus of Rupture, $f_r = 0.37 \cdot f_c^{0.5}$	0.740	ksi	Using a coefficient of 0.37 rather than 0.24 from Aashto Lrfd C5.4.2.6 is conservative.

	Critical Points			Fifth Points Along Taper						
	A	B	C	0	1	2	3	4	5	
Factored Flexural Resistance, $Mr_z = \phi Mn_z$	2838.662	1223.355	1223.355	2838.662	2493.062	2147.462	1801.862	1223.355	0.000	k*ft
Section Modulus of Cap, S_z	24.500	7.220	7.220	24.500	19.220	14.580	10.580	7.220	4.500	ft^3
Cracking Moment, $Mcr_z = \gamma_3 \cdot \gamma_1 \cdot f_r \cdot S_z$	3132.864	923.236	923.236	3132.864	2457.700	1864.374	1352.886	923.236	575.424	k*ft
Is Mr_z >= Mrc_z ?	No	Yes	Yes	No	Yes	Yes	Yes	Yes	No	

---- OR ----

	Critical Points			Fifth Points Along Taper						
	A	B	C	0	1	2	3	4	5	
Effective A_{se} provided	8.000	6.667	6.667	8.000	8.000	8.000	8.000	6.667	0.000	in^2
$1.33 \cdot Mu_z$	1018.90	1.22	1.22	735.97	545.92	360.51	179.15	1.22	0.00	k*ft
A_s required based on $1.33 \cdot Mu_z$	2.826	0.006	0.006	2.036	1.714	1.307	0.768	0.006	0.000	in^2
Is Eff. A_{se} prov'd >= A_s req'd based on $1.33 \cdot Mu_z$?	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	

Flexure Resistance Factor and Steel Yield Checks Aashto Lrfd 5.5.4.2 and 5.6.2.1
If section is in Transition, then the user may adjust the Flexural Phi Factor, ϕ, in cell G54. If the section is compression-controlled then do not use this sheet.
If rebar does not yield ($f_s < f_y$) then do not use this sheet.

	Critical Points			Fifth Points Along Taper						
	A	B	C	0	1	2	3	4	5	
Location of Neutral Axis, c	4.614	3.845	3.845	4.614	4.614	4.614	4.614	3.845	0.000	in
Net Tensile Strain in the Extreme Tension Steel, ε_t	0.050	0.030	0.030	0.050	0.043	0.037	0.031	0.030	0.000	in/in
Tension Controlled? $\varepsilon_t >= \varepsilon_{tl}$ = 0.005	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	n/a	
Compression Controlled? $\varepsilon_t <= \varepsilon_{cl}$ = 0.002	No	No	No	No	No	No	No	No	n/a	
Transition? $\varepsilon_{tl} > \varepsilon_t > \varepsilon_{cl}$	No	No	No	No	No	No	No	No	n/a	
Flexure Phi Factor, ϕ, for Design	0.900	0.900	0.900	0.900	0.900	0.900	0.900	0.900	0.750	
c/d_s	0.057	0.091	0.091	0.057	0.065	0.075	0.089	0.091	0.000	
$0.003/(0.003 + \varepsilon_{cl})$ where ε_{cl} = 0.002	0.600	0.600	0.600	0.600	0.600	0.600	0.600	0.600	0.600	
Is $c/d_s <= 0.003/(0.003 + \varepsilon_{cl})$ such that $f_s = f_y$?	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	

	Critical Points			Fifth Points Along Taper					
	A	B	C	0	1	2	3	4	5
Is Flexural Reinf. Adequate?	YES	YES	YES	YES	YES	YES	YES	YES	YES

Check Shear Reinforcement

To increase size of shear reinforcement bars go to cell G106.
Maximum stirrup spacing is 12" based on the Bridge Design Manual (BDM 6.6.4.1.1.1).

Simplified Shear Design Aashto Lrfd 5.7 and specifically 5.7.3.4.1, β = 2.0 and θ = 45 degrees

	Critical Points			Fifth Points Along Taper						
	A	B	C	0	1	2	3	4	5	
Total Factored Shear, Vu	150.463	132.633	132.633	144.788	141.074	137.810	134.996	132.633	0.000	k
Effective Shear Depth, dv	78.852	40.779	40.779	78.852	69.252	59.652	50.052	40.779	32.813	in
Max. Permissible Factored Shear Resistance, Vr_{max}	2554.796	1321.224	1321.224	2554.796	2243.756	1932.716	1621.676	1321.224	1063.125	k
Factored Concrete Shear Resistance, ϕVc	322.926	167.003	167.003	322.926	283.611	244.295	204.980	167.003	134.379	k
Req'd Factored Shear Reinforcement Resistance, ϕVs	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	k
Stirrup Spacing, s, Aashto Lrfd Eq. 5.7.3.3-4	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	in
Stirrup Spacing, s, Aashto Lrfd Eq. 5.7.2.5-1	10.549	10.549	10.549	10.549	10.549	10.549	10.549	10.549	10.549	in
Stirrup Spacing, s, Aashto Lrfd Eq. 5.7.2.6-1 & 2	24.000	24.000	24.000	24.000	24.000	24.000	24.000	24.000	24.000	in
Final Stirrup Spacing, s	10.500	10.500	10.500	10.500	10.500	10.500	10.500	10.500	10.500	in

General Shear Design Aashto Lrfd 5.7 and specifically 5.7.3.4.2, variable β and θ
In Aashto Lrfd 5.7.3.4.2 there are a number of bulleted items that should be considerd when using this method for shear design. This spreadsheet does reduce A_s for the section under consideration based on development length. This spreadsheet does not base ε_s on the calculated value at d_v when the section under consideration is closer than d_v to the face of the support. Axial forces in the cap are not considered.

	Critical Points			Fifth Points Along Taper						
	A	B	C	0	1	2	3	4	5	
Total Factored Shear, Vu	150.463	132.633	132.633	144.788	141.074	137.810	134.996	132.633	0.000	k
Total Factored Moment, Mu	766.088	0.919	0.919	553.359	410.466	271.062	134.696	0.919	0.000	k*ft
Effective Shear Depth, dv	78.852	40.779	40.779	78.852	69.252	59.652	50.052	40.779	32.813	in
Max. Permissible Factored Shear Resistance, Vr_{max}	2554.796	1321.224	1321.224	2554.796	2243.756	1932.716	1621.676	1321.224	1063.125	k
Flexural Reinf. Strain, ε_s, Aashto Lrfd Eq. 5.7.3.4.2-4	0.00130	0.00137	0.00137	0.00125	0.00122	0.00119	0.00116	0.00137	0.00000	in/in
$\beta = 4.8/(1 + 750^*\varepsilon_s)$, Aashto Lrfd Eq. 5.7.3.4.2-1	2.433	2.366	2.366	2.479	2.510	2.538	2.563	2.366	4.800	
$\theta = 29 + 3500^*\varepsilon_s$, Aashto Lrfd Eq. 5.7.3.4.2-3	33.540	33.802	33.802	33.369	33.257	33.158	33.073	33.802	29.000	deg
Factored Concrete Shear Resistance, ϕVc	392.850	197.535	197.535	400.296	355.975	310.050	262.679	197.535	322.510	k
Req'd Factored Shear Reinforcement Resistance, ϕVs	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	k
Stirrup Spacing, s, Aashto Lrfd Eq. 5.7.3.3-4	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	in
Stirrup Spacing, s, Aashto Lrfd Eq. 5.7.2.5-1	10.549	10.549	10.549	10.549	10.549	10.549	10.549	10.549	10.549	in
Stirrup Spacing, s, Aashto Lrfd Eq. 5.7.2.6-1 & 2	24.000	24.000	24.000	24.000	24.000	24.000	24.000	24.000	24.000	in
Final Stirrup Spacing, s	10.500	10.500	10.500	10.500	10.500	10.500	10.500	10.500	10.500	in

Additional Longitudinal Reinforcement

Aashto Lrfd 5.7.3.5

See Aashto Lrfd 5.7.3.5 and BDM 6.6.4.1.1.1 for additional information regarding the applicability of this provision.

Base θ off of Shear Method 1 or 2	1	Method 1 = Simplified Shear, 2 = General Shear Design

The user has the opportunity to enter a stirrup spacing. The stirrup size and number of legs remain the same.

	Critical Points			Fifth Points Along Taper						
	A	B	C	0	1	2	3	4	5	
Stirrup Spacing Used	10.500	10.500	10.500	10.500	10.500	10.500	10.500	10.500	10.500	in
θ based on Shear Method 1	45.000	45.000	45.000	45.000	45.000	45.000	45.000	45.000	45.000	deg
Mu/(dv*φf)	129.541	0.301	0.301	93.570	79.029	60.588	35.882	0.301	0.000	k
Vu	150.463	132.633	132.633	144.788	141.074	137.810	134.996	132.633	0.000	k
Vu/φv	167.181	147.369	147.369	160.875	156.749	153.122	149.996	147.369	0.000	k
0.5*Vs	83.591	46.604	46.604	80.438	78.374	68.173	57.202	46.604	0.000	k
As*fy = Mu/(dv*φf) + (Vu/φv - 0.5*Vs)*cotθ	213.131	101.066	101.066	174.007	157.403	145.537	128.676	101.066	0.000	k
Total As Needed	3.552	1.684	1.684	2.900	2.623	2.426	2.145	1.684	0.000	in²
Additional Longitudinal Reinf. Needed, Alx	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	in²

Crack Control and S&T Reinforcement

Crack Control: Flexure Reinforcement Aashto Lrfd 5.6.7. Spacing should also comply with Aashto Lrfd 5.10.3.1 and 5.10.3.2.

See cell G56 to change the Exposure Factor, γ_e, which is typically set to 1.00 (Class 1) for pier caps.

If fss >= 0.60*fy = 36 ksi, then the user needs to redesign the section.

	Critical Points			Fifth Points Along Taper						
	A	B	C	0	1	2	3	4	5	
Concrete Cover Thickness to Reinf. Center, d_c	3.188	3.188	3.188	3.188	3.188	3.188	3.188	3.188	3.188	in
βs	1.056	1.107	1.107	1.056	1.064	1.074	1.088	1.107	1.139	
Service Moment, Ms due to P1	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	k*ft
Service Moment, Ms due to P2	480.849	0.000	0.000	353.500	265.125	176.750	88.375	0.000	0.000	k*ft
Service Moment, Ms due to Diaphragm	0.021	0.001	0.001	0.013	0.008	0.005	0.002	0.001	0.000	k*ft
Service Moment, Ms due to Pier Cap	43.857	0.735	0.735	24.375	14.640	7.695	3.180	0.735	0.000	k*ft
Total Service Moment, Ms	524.726	0.736	0.736	377.888	279.773	184.450	91.557	0.736	0.000	k*ft
Reinforcment Ratio, ρ	0.00275	0.00524	0.00524	0.00275	0.00312	0.00361	0.00427	0.00524	0.00677	
Factor for Distance to Neutral Axis, k	0.178	0.237	0.237	0.178	0.188	0.201	0.216	0.237	0.264	
Reinforcement Stress at Service Level, f_{ss}	10.354	0.028	0.028	7.456	6.288	4.813	2.846	0.028	0.000	ksi
Max. Spa. of Top Layer of Neg. Flexural Reinf., s	57.6	22378.0	22378.0	82.5	98.3	129.1	219.8	22378.0	n/a	in

Crack Control: Skin Reinforcement Aashto Lrfd 5.6.7. Spacing should also comply with Aashto Lrfd 5.10.3.1 and 5.10.3.2.

	Critical Points			Fifth Points Along Taper						
	A	B	C	0	1	2	3	4	5	
Is Skin Reinf. Required ? (Is $d_{ext\ tens\ bar}$ > 3.00'?)	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	No	
Area of Skin Reinf. Required per Face, A_{sk}	0.610	0.149	0.149	0.610	0.495	0.379	0.264	0.149	0.000	in²/ft
Max Spacing of Skin Reinf. Required	12.000	7.069	7.069	12.000	11.869	10.269	8.669	7.069	5.469	in

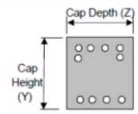
Shrinkage and Temp. Reinforcement Aashto Lrfd 5.10.6. Spacing should also comply with Aashto Lrfd 5.10.3.1 and 5.10.3.2.

	A	B	C	0	1	2	3	4	5	
Area of Skin Reinf. Required per Face, A_s	0.273	0.218	0.218	0.273	0.263	0.251	0.236	0.218	0.195	in²/ft
Max Spacing of Skin Reinf. Required	12.000	12.000	12.000	12.000	12.000	12.000	12.000	12.000	12.000	in

IADOT Pier Cap Design Calculations:

DISCRETE FLEXURE AND SHEAR DESIGN — Aashto Lrfd 5.6 and 5.7

Pier Cap Design at One Discrete Location



Cap Height (Y)	3.000	ft	
Cap Depth (Z)	3.000	ft	
Concrete Strength, f'c	4.000	ksi	Aashto Lrfd 5.4.2.1
Flexural Reinforcement Yield Strength, fy	60.000	ksi	Aashto Lrfd 5.4.3.1
Shear Reinforcement Yield Strength, fy	60.000	ksi	Aashto Lrfd 5.4.3.1
Reinforcement Modulus of Elasticity, Es	29000.000	ksi	Aashto Lrfd 5.4.3.2
Flexure Resistance Phi Factor, ϕ_f	0.900		Begin by assuming a tension-controlled section, ϕ_f= 0.9 -- Aashto Lrfd 5.5.4.2.
Shear Resistance Phi Factor, ϕ_v	0.900		ϕ_v = 0.9 -- Aashto Lrfd 5.5.4.2.
Concrete Unit Weight, γ_c	0.150	kcf	

Intermediate Calculations — Aashto Lrfd 3.5.1 and 5.4.2.4, typically 0.150 kcf

Whitney Stress Block Factor, α_1	0.850		Aashto Lrfd 5.6.2.2
Whitney Stress Block Factor, β_1	0.850		Aashto Lrfd 5.6.2.2
Compression-controlled Strain Limit, ε_{cl}	0.002		Aashto Lrfd 5.6.2.1
Tension-controlled Strain Limit, ε_{tl}	0.005		Aashto Lrfd 5.6.2.1
Concrete Modulus of Elasticity, Ec	4266.223	ksi	Aashto Lrfd 5.4.2.4, aggregate correction factor is set to 1.0
Modular Ratio, n	7.000		Aashto Lrfd 5.6.1, rounded to nearest integer

Design Bottom Cap Reinforcement

Factored Applied Positive Moment, Mu_z	766.090	k*ft

Rough Estimate of A_s required		5.373	in^2
Estimate of the number of bars required for bar sizes:	#7	9	
	#8	7	
	#9	6	
	#10	5	
	#11	4	

Bar Size for Flexural Reinforcement	9

Reinforcement Layer	d' (in) by Layer, Measured from **Bottom** of Cap	No. of Bars per Layer	d_s (in) by Layer
1	3.1875	8	32.8125
2	7.1875		28.8125
3	11.1875		24.8125
4	15.1875		20.8125

Total Bar Area Input, A_s provided	8.000	in^2
Distance from Top of Cap to C.G. of Bar Group, d_s	32.813	in

Note: Total bar area is lumped at its center of gravity.

Design Top Cap Reinforcement

Factored Applied Negative Moment, Mu_z	766.090	k*ft

Note: Use positive value for negative moments.

Rough Estimate of A_s required		5.373	in^2
Estimate of the number of bars required for bar sizes:	#7	9	
	#8	7	
	#9	6	
	#10	5	
	#11	4	

Bar Size for Flexural Reinforcement	9

Reinforcement Layer	d' (in) by Layer, Measured from **Top** of Cap	No. of Bars per Layer	d_s (in) by Layer
1	3.1875	8	32.8125
2	7.1875		28.8125
3	11.1875		24.8125
4	15.1875		20.8125

Total Bar Area Input, A_s provided	8.000	in^2
Distance from Bottom of Cap to C.G. of Bar Group, d_s	32.813	in

Note: Total bar area is lumped at its center of gravity.

Flexural Capacity Check Aashto Lrfd 5.6.3.2

Factored Applied Moment, Mu_z	766.090	k*ft
Depth of Equivalent Stress Block, a	3.922	in
Factored Flexural Resistance, $Mr_z = \phi Mn_z$	1110.662	k*ft
Is Mu_z <= Mr_z ?	Yes	

Factored Applied Moment, Mu_z	766.090	k*ft
Depth of Equivalent Stress Block, a	3.922	in
Factored Flexural Resistance, $Mr_z = \phi Mn_z$	1110.662	k*ft
Is Mu_z <= Mr_z ?	Yes	

Minimum Reinforcement Check Aashto Lrfd 5.4.2.6 and 5.6.3.3

For modulus of rupture, using a coefficient of 0.37 rather than 0.24 from Aashto Lrfd C5.4.2.6 is conservative.

Modulus of Rupture, $f_r = 0.37{*}f_c^{0.5}$	0.740	ksi
Section Modulus of Cap, S_z	4.500	ft^3
Yield to Ult. Tensile Strength Ratio, γ_3 (0.67 or 0.75)	0.750	
Flexural Cracking Variability Factor, γ_1 (1.6)	1.600	
Cracking Moment, $Mcr_z = \gamma_3{*}\gamma_1{*}f_r{*}S_z$	575.424	k*ft
Is Mcr_z <= Mr_z ?	Yes	

---- OR ----

A_s required based on 1.33*Mu_z	7.298	in^2
Is A_s prov'd >= A_s req'd based on 1.33*Mu_z?	Yes	

Modulus of Rupture, $f_r = 0.37{*}f_c^{0.5}$	0.740	ksi
Section Modulus of Cap, S_z	4.500	ft^3
Yield to Ult. Tensile Strength Ration, γ_3 (0.67 or 0.75)	0.750	
Flexural Cracking Variability Factor, γ_1 (1.6)	1.600	
Cracking Moment, $Mcr_z = \gamma_3{*}\gamma_1{*}f_r{*}S_z$	575.424	k*ft
Is Mcr_z <= Mr_z ?	Yes	

---- OR ----

A_s required based on 1.33*Mu_z	7.298	in^2
Is A_s prov'd >= A_s req'd based on 1.33*Mu_z?	Yes	

Flexure Resistance Factor and Steel Yield Checks Aashto Lrfd 5.5.4.2 and 5.6.2.1

If section is in Transition, then the user may adjust the Flexural Phi Factor, ϕ_f, in cell G10. If the section is compression-controlled then do not use this sheet.

If rebar does not yield ($f_s < f_y$) then do not use this sheet.

Location of Neutral Axis, c	4.614	in	Location of Neutral Axis, c	4.614	in
Net Tensile Strain in the Extreme Tension Steel, ε_t	0.018	in/in	Net Tensile Strain in the Extreme Tension Steel, ε_s	0.018	in/in
Tension Controlled? $\varepsilon_t >= \varepsilon_{tl}$ = 0.005	Yes		**Tension Controlled? $\varepsilon_t >= \varepsilon_{tl}$ = 0.005**	Yes	
Compression Controlled? $\varepsilon_t <= \varepsilon_{cl}$ = 0.002	No		**Compression Controlled? $\varepsilon_t <= \varepsilon_{cl}$ = 0.002**	No	
Transition? $\varepsilon_{tl} > \varepsilon_t > \varepsilon_{cl}$	No		**Transition? ε_{tl} = 0.005 > $\varepsilon_t > \varepsilon_{cl}$ = 0.002**	No	
Flexure Phi Factor, ϕ_f, for Design	0.900		Flexure Phi Factor, ϕ_f, for Design	0.900	
c/d_s	0.141		c/d_s	0.141	
$0.003/(0.003 + \varepsilon_{cl})$ where ε_{cl} = 0.002	0.600		$0.003/(0.003 + \varepsilon_{cl})$ where ε_{cl} = 0.002	0.600	
Is $c/d_s <= 0.003/(0.003 + \varepsilon_{cl})$ such that $f_s = f_y$?	Yes		**Is $c/d_s <= 0.003/(0.003 + \varepsilon_{cl})$ such that $f_s = f_y$?**	Yes	
Is Flex. Reinf. Adequate?	**YES, Flexural Reinf. is Adequate.**		**Is Flex. Reinf. Adequate?**	**YES, Flexural Reinf. is Adequate.**	

Shear Reinforcement Check Aashto Lrfd 5.7 and specifically 5.7.3.4.1 and 2

Simplified Shear Design Aashto Lrfd 5.7 and specifically 5.7.3.4.1, β = 2.0 and θ = 45 degrees

Use positive value for shears.

Factored Applied Shear, Vu	150.460	k	Factored Applied Shear, Vu	150.460	k
Stirrup Bar Size (i.e. #4, #5, etc.)	4		Stirrup Bar Size (i.e. #4, #5, etc.)	4	
Number of Stirrup Legs (Typ. 4 legs for double hoops)	2		Number of Stirrup Legs (Typ. 4 legs for double hoops)	2	
Total Area of Shear Stirrups	0.400	in^2	Total Area of Shear Stirrups	0.400	in^2
Stirrup Spacing at Location of Interest (i.e. at Mu)	10.500	in	Stirrup Spacing at Location of Interest (i.e. at Mu)	10.500	in
Beta, β	2.000		Beta, β	2.000	
Theta, θ	45.000	deg	Theta, θ	45.000	deg
Effective Shear Depth, dv	30.852	in	Effective Shear Depth, dv	30.852	in
Concrete Shear Resistance, V_c	140.388	k	Concrete Shear Resistance, V_c	140.388	k
Stirrup Shear Resistance, V_s	70.518	k	Stirrup Shear Resistance, V_s	70.518	k
Vn_{cs} = Vc + Vs	210.906	k	Vn_{cs} = Vc + Vs	210.906	k
Vn_{max} = 0.25*f'c*bv*dv	1110.662	k	Vn_{max} = 0.25*f'c*bv*dv	1110.662	k
Vn = minimum of Vn_{cs} and Vn_{max}	210.906	k	Vn = minimum of Vn_{cs} and Vn_{max}	210.906	k
ϕ_v*Vn	189.815	k	ϕ_v*Vn	189.815	k
Factored Applied Shear, Vu	150.460	k	Factored Applied Shear, Vu	150.460	k
Is Shear Design Adequate? ϕ_v*Vn >= Vu	Yes		Is Shear Design Adequate? ϕ_v*Vn >= Vu	Yes	

General Shear Design Aashto Lrfd 5.7 and specifically 5.7.3.4.2, variable β and θ

Use positive value for shears and moments.

Factored Applied Shear, Vu	150.460	k	Factored Applied Shear, Vu	150.460	k
Factored Applied Positive Moment, Mu_z	766.090	k*ft	Factored Applied Negative Moment, Mu_z	766.090	k*ft
Stirrup Bar Size (i.e. #4, #5, etc.)	4		Stirrup Bar Size (i.e. #4, #5, etc.)	4	
Number of Stirrup Legs (Typ. 4 legs for double hoops)	2		Number of Stirrup Legs (Typ. 4 legs for double hoops)	2	
Total Area of Shear Stirrups	0.400	in^2	Total Area of Shear Stirrups	0.400	in^2
Stirrup Spacing at Location of Interest (i.e. at Mu)	6.000	in	Stirrup Spacing at Location of Interest (i.e. at Mu)	6.000	in
Effective Shear Depth, dv	30.852	in	Effective Shear Depth, dv	30.852	in
Flexural Reinf. Strain, ε_s, Aashto Lrfd Eq. 5.7.3.4.2-4	0.00193	in/in	Flexural Reinf. Strain, ε_s, Aashto Lrfd Eq. 5.7.3.4.2-4	0.00193	in/in
$\beta = 4.8/(1 + 750*\varepsilon_s)$, Aashto Lrfd Eq. 5.7.3.4.2-1	1.959		$\beta = 4.8/(1 + 750*\varepsilon_s)$, Aashto Lrfd Eq. 5.7.3.4.2-1	1.959	
$\theta = 29 + 3500*\varepsilon_s$, Aashto Lrfd Eq. 5.7.3.4.2-3	35.765		$\theta = 29 + 3500*\varepsilon_s$, Aashto Lrfd Eq. 5.7.3.4.2-3	35.765	
Concrete Shear Resistance, V_c	137.540	k	Concrete Shear Resistance, V_c	137.540	k
Stirrup Shear Resistance, V_s	171.327	k	Stirrup Shear Resistance, V_s	171.327	k
Vn_{cs} = Vc + Vs	308.867	k	Vn_{cs} = Vc + Vs	308.867	k
Vn_{max} = 0.25*f'c*bv*dv	1110.662	k	Vn_{max} = 0.25*f'c*bv*dv	1110.662	k
Vn = minimum of Vn_{cs} and Vn_{max}	308.867	k	Vn = minimum of Vn_{cs} and Vn_{max}	308.867	k
ϕ_v*Vn	277.981	k	ϕ_v*Vn	277.981	k
Factored Applied Shear, Vu	150.460	k	Factored Applied Shear, Vu	150.460	k
Is Shear Design Adequate? ϕ_v*Vn >= Vu	Yes		Is Shear Design Adequate? ϕ_v*Vn >= Vu	Yes	

Additional Longit. Reinf. Check Aashto Lrfd 5.7.3.5

See Aashto Lrfd 5.7.3.5 and BDM 6.6.4.1.1.1 for additional information regarding the applicability of this provision.

Method 1 = Simplified Shear, 2 = General Shear Design		Method 1 = Simplified Shear, 2 = General Shear Design	
Base Check off of Shear Method 1 or 2	1	Base Check off of Shear Method 1 or 2	1

The user has the opportunity to enter a stirrup spacing. The stirrup size and number of legs remain the same.

Stirrup Spacing Used	10.500	in	Stirrup Spacing Used	10.500	in

Values based on shear method chosen.

Total Area of Shear Stirrups	0.400	in^2	Total Area of Shear Stirrups	0.400	in^2
θ	45.000	deg	θ	45.000	deg
Effective Shear Depth, dv	30.852	in	Effective Shear Depth, dv	30.852	in
Vu	150.460	k	Vu	150.460	k
Mu	766.090	k	Mu	766.090	k

$Mu/(dv*\phi_f)$	331.085	k	$Mu/(dv*\phi_f)$	331.085	k
Vu/ϕ_v	167.178	k	Vu/ϕ_v	167.178	k
0.5*Vs	35.259	k	0.5*Vs	35.259	k
$As*fy = Mu/(dv*\phi_f) + (Vu/\phi_v - 0.5*Vs)*\cot\theta$	463.003	k	$As*fy = Mu/(dv*\phi_f) + (Vu/\phi_v - 0.5*Vs)*\cot\theta$	463.003	k
Total As Needed	7.717	in^2	Total As Needed	7.717	in^2
Additional Longitudinal Reinf. Needed, Alx	0.000	in^2	**Additional Longitudinal Reinf. Needed, Alx**	0.000	in^2

If value for additional longitudinal reinforcement is zero or negative it means that this provision of the code is satisfied.

Crack Control: Flexure Reinf. Aashto Lrfd 5.6.7. Spacing should also comply with Aashto Lrfd 5.10.3.1 and 5.10.3.2.

If fss >= 0.60*fy = 36 ksi, then the user needs to redesign the section.

Exposure Factor, γ_e (Typically 1.00)	1.000		Exposure Factor, γ_e (Typically 1.00)	1.000	

Class 1 and 2 exposure factors are 1.00 and 0.75 respectively. | Class 1 and 2 exposure factors are 1.00 and 0.75 respectively.

Concrete Cover Thickness to Reinf. Center, d_c	3.188	in	Concrete Cover Thickness to Reinf. Center, d_c	3.188	in
βs	1.139		βs	1.139	

User Value for Positive Service M_z	766.090	k*ft	User Value for Negative Service M_z	766.090	k*ft

Note: Use positive value for negative moments.

See Aashto Lrfd 5.4.2.4 and 5.7.1 for E_c and n

Reinforcment Ratio, ρ	0.00677		Reinforcment Ratio, ρ	0.00677	
Concrete Modulus of Elasticity, E_c	4266.223	ksi	Concrete Modulus of Elasticity, E_c	4266.223	ksi
Modular Ratio, n	7.000		Modular Ratio, n	7.000	
Factor for Distance to Neutral Axis, k	0.264		Factor for Distance to Neutral Axis, k	0.264	
Reinforcement Stress at Service Level	38.402	ksi	Reinforcement Stress at Service Level	38.402	ksi

Max. Spa. of Bot Layer of Pos. Flexural Reinf., s	9.632	in	**Max. Spa. of Bot Layer of Pos. Flexural Reinf., s**	9.632	in

Crack Control: Skin Reinf. Aashto Lrfd 5.6.7. Spacing should also comply with Aashto Lrfd 5.10.3.1 and 5.10.3.2.

Is Skin Reinf. Required ? (Is $d_{ext\ tens\ bar}$ > 3.00'?)	No		**Is Skin Reinf. Required ? (Is $d_{ext\ tens\ bar}$ > 3.00'?)**	No	

Area of Skin Reinf. Required per Face, A_{sk}	0.000	in^2 per ft	**Area of Skin Reinf. Required per Face, A_{sk}**	0.000	in^2 per ft
Max Spacing of Skin Reinf. Required	5.469	in	**Max Spacing of Skin Reinf. Required**	5.469	in

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Shrinkage and Temp. Reinf. Aashto Lrfd 5.10.6. Spacing should also comply with Aashto Lrfd 5.10.3.1 and 5.10.3.2.

Area of Skin Reinf. Required per Face, A_{sk}	0.195	in^2 per ft	**Area of Skin Reinf. Required per Face, A_{sk}**	0.195	in^2 per ft
Max Spacing of Skin Reinf. Required	12.000	in	**Max Spacing of Skin Reinf. Required**	12.000	in

Pier Pile Footing Design:

Required footing length from IADOT wave equation Excel file calculations:

Design Pile Length -- WEAP

Bridge Description	Volga Pedestrian Bridge
Abut or Pier Description	Pier

Notes:
Do not move, add, or delete rows and columns.
Spreadsheet is for WEAP only as construction control.

Boring No.	0001
Natural Ground Elev	780.00
Water Table Elev	775.00

Link to BDM LRFD Pile Design Examples: https://iowadot.gov/bridge/policy/06-02-00PileDesignExamplesLRFD.pdf
Link to BDM LRFD Pile Training Videos: https://iowadot.gov/bridge/lrfdworkshop
Link to BDM LRFD Pile Design Articles: https://iowadot.gov/bridge/policy/06-02-00PileLRFD.pdf

Bot of Cap Elev	776.00

Scour (Y/N)?	N	Scour Elev	
Prebore (Y/N)?	Y	Prebore Depth (ft)	20.00

Notes:
Do not include scour and prebore in the same pile design.
Do not include scour and downdrag in the same pile design.

Hide Soil Entry Table

Enter Soil Information

Bottom of Boring Elev	680.00

Notes:
If necessary, for nominal friction resistance entry, the user should start a layer elevation at 30' below Natural Ground Elev at: 750.00

Layer No.	Calculated Layer Thickness (ft)	Calculated Cumulative Depth (ft)	Starting Elevation of Layer	Abbreviated Soil Description	C N or R	Enter D for Down-drag	Avg N for Layer (Blows/ft)	Nominal Friction Resistance (k/ft)	Nominal End Bearing Resistance (kips)
1	25.00	25.00	775.00	Silty Sand	N		12	1.20	
2	50.00	75.00	750.00	Silty Sand	N		12	1.20	
3			700.00	Silty Sand	N		12	1.20	
4									
5									
6									
7									
8									
9									
10									
11									
12									
13									
14									
15									
16									
17									
18									
19									
20									

Enter Pile Information

Pile Type	HP10x42
Pile Area (in^2)	12.40
Pile Cutoff (ft)	1.00
Pile Embed in Cap (ft)	2.00
Pile Length Incr (ft)	5.00

Common Geotech DESIGN Resist Factors	
Generalized Soil Category	Track 1 WEAP only
Cohesive	0.65
Mixed	0.65
Non-Cohesive	0.55

Factored Axial Load per Pile, $\Sigma\eta\gamma Q$ (k)	30.00
Geotech DESIGN Friction Resist Factor	0.65
Geotech DESIGN End Brg Resist Factor	0.70
Load Factor for Downdrag	1.00

Geotech DESIGN End Brg Resist Factor for Bedrock only	0.70

#1. Click the Button to Estimate a Contract Pile length.
Based on the estimate set contract length for #2 to back-calculate factored resistance. ==>>

Estimate Pile Contract Length

Note: Estimated pile contract length will not account for minimum bedrock penetration.

#2. Click the Button to Determine Factored Design Resistance Based on User Supplied Pile Contract Length.

User Supplied Contract Pile Length (ft)	65.00

Determine Factored Pile Design Resistance

Note: The user should include minimum bedrock penetration if appropriate.

Location	Start Elev	End Elev	Soil Description	C, N, R	Down-drag	Avg N (Blows/ft)	Length (ft)	Fact Friction Resistance (kips/ft)	Fact Friction Resistance (kips)	Fact End Brg Resistance (kips)	Cum Fact Resistance (kips)
Bot of Cap Elev	776.00	776.00					0.00	0.00	0.00		0.00
No Soil/Prebore Zone	776.00	756.00					20.00	0.00	0.00		0.00
Layer 1	756.00	750.00	Silty Sand	N		12	6.00	0.78	4.68		4.68
Layer 2	750.00	714.00	Silty Sand	N		12	36.00	0.78	28.08	0.00	32.76

Cumulative Pile Length (ft)	62.00
Factored Downdrag Load (k)	0.00
Factored Axial Load per Pile (k)	30.00
Tot Factored Axial Load per Pile (k)	30.00
Tot Factored Pile Resist (k)	32.76

Warnings
No warnings.

Total C+N Pile Length (ft)	42.00	Percent (%) of C+N
Cohesive, C, Pile Length (ft)	0.00	0.00
Non-cohesive, N, Pile Length (ft)	42.00	100.00
Generalized Soil Category: C, N, or M	N	Review resist factor selection to ensure compatability with generalized soil category.

Pile Contract Length (ft)	65.00
Nom Design Friction Pile Resist (k)	50.40
Nom Design End Brg Pile Resist (k)	0.00
Tot Nom Design Pile Resist (k)	50.40

Estimate Target Nominal Pile Driving Resistance (Construction Stage)

Hover for Notes

Pile Contract Length (ft)	65.00
Factored Downdrag Load, $\gamma_{DD}DD$ (k)	0.00
Factored Axial Load per Pile, $\Sigma\eta\gamma Q$ (k)	30.00
Tot Factored Axial Load per Pile (k)	30.00

Consider Setup in Cohesive Soil? Y/N	N	* See NOTE.
Geotech CC Resist Factor	0.65	CC = Construction Control
[redacted]		

Will Pile Tip Out in Bedrock? Y/N	Y
Geotech CC End Brg Resist Factor	0.70

Common Geotech CONSTRUCTION CONTROL (CC) Resistance Factors			
Generalized Soil Category	Track 1 WEAP Only		
	ϕ	ϕ_{DD}	ϕ_{setup}
Cohesive	-	0.65	0.20
Mixed	0.65	-	-
Non-Cohesive	0.55	-	-

For redundant timber pile groups use ϕ of 0.40 for Track 1.

Geotech CC End Brg Resist Factor for Bedrock only	0.70

* NOTE: The user should NOT consider setup when any of the following are present:
  - Soil type classification is mixed or non-cohesive.
  - Pile type is something other than steel H-piles.
  - Construction control does not include WEAP.
  - Downdrag is present or piles are driven to bedrock.
  - Accelerated bridge construction.
  - Soil type classification is cohesive, but overall average blow count, N, is less than 5.

#3. Click the Button to Determine Target Nominal Pile Driving Resistance

Determine Target Nom Pile Driving Resist

Nom Pile Resist in Scour Soil, R_{SCOUR} (k)	0.00
Pile Length subject to Scour (ft)	0.00

The table below will be used to determine the Generalized Soil Category for the pile, the average SPT N-value for the cohesive soil if it is necessary to account for setup, and for the determination of driving resistance due to scour.

Location	Start Elev	End Elev	Soil Description	C, N, R	Down-drag	Avg N (Blows/ft)	Length (ft)	Nom Friction Resistance (kips/ft)	Nom Friction Resistance (kips)	Nom End Brg Resistance (kips)	Cum Nom Resistance (kips)
Bot of Cap Elev	776.00	776.00					0.00	0.00	0.00		0.00
No Soil/Prebore Zone	776.00	756.00					20.00	0.00	0.00		0.00
Layer 1	756.00	750.00	Silty Sand	N		12	6.00	1.20	7.20		7.20
Layer 2	750.00	714.00	Silty Sand	N		12	36.00	1.20	43.20	0.00	50.40

Cumulative Pile Length (ft)	62.00	
Total C+N Pile Length (ft)	42.00	Percent (%) of C+N
Cohesive, C, Pile Length (ft)	0.00	0.00
Non-cohesive, N, Pile Length (ft)	42.00	100.00
General Soil Category: C, N, or M	N	Review resist factor selection to ensure compatability with gen soil category.

$\Sigma\eta\gamma Q + \gamma_{DD}DD$ (k)	30.00
Summation of Fact Frict Resist (k)	32.76
Fraction of Friction Resist, F_{fr}	1.00
Fraction of End Brg Resist, F_{eb}	0.00
Targt Nom Pile Driving Resist, R_{ndr-T} (k)	46.15

IADOT Pile Footing Design:

Pile Footing Design Aashto Lrfd 5.13.3

Footing Length (X direction)	13.000	feet
Footing Width (Z direction)	10.000	feet
Footing Depth (Y direction)	4.000	feet

Column Width or Diameter (X direction)	5.000	feet
Column Depth (Z direction)	3.000	feet

Enter column depth of 0 for round columns.

Pile Diameter, dp	10	inches

28 Day Concrete Strength, f'c	3500	psi

Typically 3500 psi for piers.

Only the pile locations in the positive quadrant should be entered since the pile footing is assumed to be symmetrical. The user should include any piles located on the +X and + Z axes and the pile at the center of the footing if present.

Aashto Lrfd 5.13.3.2 makes provision for the tolerance of actual pile location. Office policy is to ignore this provision in footing design.

Pile Number	Positive x-coord (feet)	Positive z-coord (feet)
1	1.5	0
2	4.5	0
3	1.5	3
4	4.5	3
5		
6		
7		
8		
9		
10		
11		
12		
13		
14		
15		
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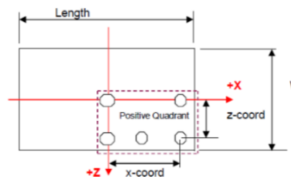

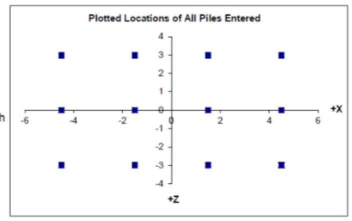

Total Number of Footing Piles	12	
Sx, Pile Section Modulus	24.000	ft
Sz, Pile Section Modulus	30.000	ft

User Loads for Footing Design			Used for:
Factored Pile Resistance, $R_r = \phi R_n$	50.400	kips	One-way (beam) shear
Maximum Factored Pile Load, Pu_m	30.000	kips	Flexure reinforcement
Maximum Factored Average Pile Load, Pu_a	30.000	kips	Two-way (punching) shear

Note: The user can, according to office policy, deduct the factored bouyant weight of the footing from the pile loads for flexure and shear design. Be sure to deduct only the portion of the load going to one pile. Soil load may not be deducted.

Flexural Capacity Check Aashto Lrfd 5.7.3.2 and 5.13.3.4

Design Flexural R/I Parallel to X-axis (Mz moments)			Design Flexural R/I Parallel to Z-axis (Mx moments)		
Clearance to Flexural R/I from Bottom of Footing	14.000	in	Clearance to Flexural R/I from Bottom of Footing	13.000	in
Number of Bars Required for R/I parallel to X-axis	18		Number of Bars Required for R/I parallel to Z-axis	22	
Flexural R/I Bar Size for bars parallel to X-axis	8		Flexural R/I Bar Size for bars parallel to Z-axis	8	
Bar Diameter for # 8	1.000	in	Bar Diameter for # 8	1.000	in
Bar Area for # 8	0.790	in^2	Bar Area for # 8	0.790	in^2
Total Bar Area	14.220	in^2	Total Bar Area	17.380	in^2
Effective Depth for bars parallel to X-axis, de_x or ds_x	33.500	in	Effective Depth for bars parallel to Z-axis, de_z or ds_z	34.500	in

Note: de_x corresponds with design for Mz | Note: de_z corresponds with design for Mx.

Flexure Phi Factor, ϕ	0.900		Flexure Phi Factor, ϕ	0.900	
Factored Applied Moment, Mu_z	180.000	k*ft	Factored Applied Moment, Mu_x	180.000	k*ft
Depth of Equivalent Stress Block, a	2.390	in	Depth of Equivalent Stress Block, a	2.247	in
Factored Flexural Resistance, $Mr_z = \phi Mn_z$	2067.200	k*ft	Factored Flexural Resistance, $Mr_x = \phi Mn_x$	2610.379	k*ft
Is Mu_z <= Mr_z ?	**Yes**		**Is Mu_x <= Mr_x ?**	**Yes**	

Minimum Reinforcement Check Aashto Lrfd 5.7.3.3.2 and 5.4.2.6

Enter: 1 if de + 2" is to be used to calculate Mcr 2 if the footing depth is to be used to calculate Mcr	1		If 1, then de + 2" is used to calculate Mcr; otherwise, the footing depth is used.
Modulus of Rupture, fr	0.692	ksi	

Section Modulus of Concrete Footing, Sz	14.586	ft^3	Section Modulus of Concrete Footing, Sx	20.045	ft^3
120% of the Cracking Moment, 1.2^*Mcr_z	1744.707	k*ft	120% of the Cracking Moment, 1.2^*Mcr_x	2397.700	k*ft
Is 1.2^*Mcr_z <= Mr_z ?	**Yes**		**Is 1.2^*Mcr_x <= Mr_x ?**	**Yes**	

---- OR ---- | ---- OR ----

As required based on 1.33^*Mu_z	1.594	in^2	As required based on 1.33^*Mu_x	1.547	in^2
Is As prov'd >= As req'd based on 1.33^*Mu_z ?	**Yes**		**Is As prov'd >= As req'd based on 1.33^*Mu_x ?**	**Yes**	

Maximum Reinforcement Check Aashto Lrfd 5.5.4.2, 5.7.2.1, and 5.7.3.3

Stress Block Factor, $\beta 1$	0.850		Stress Block Factor, $\beta 1$	0.850	
Location of Neutral Axis, c	2.812	in	Location of Neutral Axis, c	2.643	in
Net Tensile Strain in the Extreme Tension Steel, ε_s	0.033	in/in	Net Tensile Strain in the Extreme Tension Steel, ε_s	0.036	in/in
Is Section Tension Controlled? ε_s >= 0.005	**Yes**		**Is Section Tension Controlled? ε_s >= 0.005**	**Yes**	
Is Section Compression Controlled? ε_s <= 0.002	**No**		**Is Section Compression Controlled? ε_s <= 0.002**	**No**	
Is Section in Transition? 0.005 > ε_s > 0.002	**No**		**Is Section in Transition? 0.005 > ε_s > 0.002**	**No**	
Flexure Phi Factor, ϕ, for Design	0.900		Flexure Phi Factor, ϕ, for Design	0.900	

NOTE: If section is in Transition, then the user must adjust the Flexural Phi Factor, ϕ, in cell G60 or O60.
If section is Compression Controlled, then do not use this spreadsheet, but the user must do a strain compatability analysis.

Is Flexural R/I Adequate?	**YES, Flexural R/I is Adequate.**	**Is Flexural R/I Adequate?**	**YES, Flexural R/I is Adequate.**

Crack Control: Flexure R/I Aashto Lrfd 5.7.3.4.

The requirements of Aashto Lrfd 5.13.3.5 should be included. Spacing should also comply with Aashto Lrfd 5.10.3.1 and 5.10.3.2.
If uplift is present; then, as a minimum, add #5 bars at 12" to the top of the footing in both directions.

Enter: 1 if de + 2" is to be used to calculate cover 2 if the footing dpth is to be used to calculate cover	1	If 1, then de + 2" is to be used to calculate cover; otherwise, the footing depth is used.

Exposure Factor, γ_e	1.000		Exposure Factor, γ_e	1.000	
Concrete Cover Thickness to R/I Center, dc	2.000	in	Concrete Cover Thickness to R/I Center, dc	2.000	in
βs	1.085		βs	1.083	
Maximum Service Pile Load, Ps_m	100.000	kips	Maximum Service Pile Load, Ps_m	100.000	kips
Positive Service Mz	600.000	k*ft	Positive Service Mz	600.000	k*ft

See Aashto Lrfd 5.4.2.4 and 5.7.1 for Ec and n | See Aashto Lrfd 5.4.2.4 and 5.7.1 for Ec and n

Reinforcment Ratio, ρ	0.00354		Reinforcment Ratio, ρ	0.00323	
Concrete Modulus of Elasticity, Ec	3586.616	ksi	Concrete Modulus of Elasticity, Ec	3586.616	ksi
Modular Ratio, n	8.000		Modular Ratio, n	8.000	
Factor for Distance to Neutral Axis, k	0.211		Factor for Distance to Neutral Axis, k	0.203	
Reinforcement Stress at Service Level	16.259	ksi	Reinforcement Stress at Service Level	12.879	ksi
Max. Spacing of Bot Layer of Pos. Flex. R/I, s	**35.669**	in	**Max. Spacing of Bot Layer of Pos. Flex. R/I, s**	**46.195**	in

Crack Control: Skin R/I Aashto Lrfd 5.7.3.4

Is Skin R/I Required ? (Is de = ds > 3.00'?)	**No**		**Is Skin R/I Required ? (Is de = ds > 3.00'?)**	**No**	
Area of Skin R/I Required per Face, Ask	**0.000**	in^2 per ft	**Area of Skin R/I Required per Face, Ask**	**0.000**	in^2 per ft
Max Spacing of Skin R/I Required	**5.583**	in	**Max Spacing of Skin R/I Required**	**5.750**	in

Shrinkage and Temp. R/I and Structural Mass Concrete Aashto Lrfd 5.10.8

Area of Skin R/I Required per Face, Ask	**0.371**	in^2 per ft	**Area of Skin R/I Required per Face, Ask**	**0.398**	in^2 per ft
Max Spacing of Skin R/I Required	**12.000**	in	**Max Spacing of Skin R/I Required**	**12.000**	in

Fatigue in R/I Aashto Lrfd 5.5.3

Office policy is to neglect checking fatigue.

Shear Capacity Check Aashto Lrfd 5.8.1.4, 5.13.3.6 and 5.8.3

Enter 1 to check dv = 0.72*h. Enter 2 to exclude it.	2	
Calculated Effective Shear Depth, dv	32.305	in
User Entry for Effective Shear Depth, dv	32.305	in

See Aashto Lrfd 5.8.2.9.

One Way Shear or Beam Shear Parallel to Z-axis		
Distance from Column Center to Critical Section	5.192	ft
Point of 0 Shear to Equivalent Column Face	2.417	ft
Distance of 3*dv	8.076	ft
Is Point of 0 Shear to Equivalent Column Face < 3*dv?	YES	

If the above is YES then Aashto Lrfd 5.8.3.4.1 may be applied with β = 2.00.

Factor for Tens Trans Diagonally Crack'd Concr, β	2.000	

Aashto Lrfd 5.8.3.3 and 5.8.3.4

Factored Applied Shear, Vu_x	0.000	k
Factored Shear Resistance, $Vr_x = \phi Vn_x = \phi Vc_x$	412.520	k
Is Beam Shear OK? Vu_x <= Vr_x	**YES.**	

One Way Shear or Beam Shear Parallel to X-axis		
Distance from Column Center to Critical Section	4.192	ft
Point of 0 Shear to Equivalent Column Face	1.917	ft
Distance of 3*dv	8.076	ft
Is Point of 0 Shear to Equivalent Column Face < 3*dv?	YES	

If the above is YES then Aashto Lrfd 5.8.3.4.1 may be applied with β = 2.00.

Factor for Tens Trans Diagonally Crack'd Concr, β	2.000	

Aashto Lrfd 5.8.3.3 and 5.8.3.4

Factored Applied Shear, Vu_z	0.000	k
Factored Shear Resistance, $Vr_z = \phi Vn_z = \phi Vc_z$	536.276	k
Is Beam Shear OK? Vu_z <= Vr_z	**YES.**	

Two Way Shear or Punching Shear			
Distance from Column Center to Critical Section	3.846	ft	Parallel to Z-axis
Distance from Column Center to Critical Section	2.846	ft	Parallel to X-axis
Perimeter of the Critical Section, bo	26.768	ft	
Ratio of Long Side to Short Side, βc	1.667		
Factored Applied Shear, Vu_x	262.170	k	

Final Pier and Pier Footing Dimensions & Details:

Pier dimensions, reinforcement, and situation:

Pier cap:	
Cap height:	3.0 ft
Cap depth:	3.0 ft
Top cap reinforcement:	6#9 @ 6" c/c and 3#9 @12" c/c
Bottom cap reinforcement:	6#9 @ 6" c/c and 3#9 @12" c/c
Shear reinforcement:	Single hoop #4 stirrups @ 10.5" c/c
Skin reinforcement:	Not required
S&T reinforcement:	1#4 @ 12" c/c
Pier cap overhang:	
Cap overhang flexural reinforcement:	4#9 @ 10" c/c
Cap overhang shear reinforcement:	Single hoop #4 stirrups @ 10.5" c/c
Skin reinforcement:	4#4, two on each side @ 10.5"c/c
S&T reinforcement:	2#4 @ 10.5" c/c
Pier column:	
Column width:	3.0 ft
Column height (untapered):	12'-6"
Reinforcement:	8#9, square symmetric spacing
Pier footing:	
Footing width:	10'-0"
Footing length:	13'-0"
Footing depth:	4'-0"
Short dimension reinforcement:	18#8 symmetrically placed
Long dimesion reinforcement:	22#8 symmetrically placed
Skin reinforcement:	Not required.
S&T reinforcement:	#4 @ 6" c/c
Pier piles:	
Shape:	HP10x42
Number:	12
Edge spacing:	1'-2"
Spacing:	3'-0"
Embedment:	1'-0"
Contract length:	65 ft
Axial load capacity:	$P_{np} := 179\ \textbf{kip}$ (BDM Table 6.2.6.1-1)
Total factored load:	$P := 1.2 \cdot (4 \cdot 47.875\ \textbf{kip}) + 1.6 \cdot (2 \cdot 40.5\ \textbf{kip}) = 359.4\ \textbf{kip}$

Appendix D – Earthwork Report

See Figure D.1 for the earthwork report provided by Civil3D.

Cut/Fill Report

Generated: 2020-04-27 13:47:03

By user: kowalsky

Drawing: \\iowa.uiowa.edu\shared\Engineering\Home\kowalsky\windowsdata\Desktop\Final Trail Drawings\\\iowa.uiowa.edu\shared\Engineering\Home\kowalsky\windowsdata\Desktop\Final Trail Drawings\Volga_4_15.dwg

Volume Summary

Name	Type	Cut Factor	Fill Factor	2d Area (Sq. Ft.)	Cut (Cu. Yd.)	Fill (Cu. Yd.)	Net (Cu. Yd.)
Cut-FillBeforeBridge1	full	1.000	1.000	2804.25	1.21	274.17	272.95<Fill>
CutFillAfterBridge1	full	1.000	1.000	10135.33	3.56	293.40	289.84<Fill>

Totals

	2d Area (Sq. Ft.)	Cut (Cu. Yd.)	Fill (Cu. Yd.)	Net (Cu. Yd.)
Total	12939.58	4.78	567.57	562.79<Fill>

* Value adjusted by cut or fill factor other than 1.0

Figure D.1. Cut/Fill earthwork report from Civil3D.

Appendix E – Riprap Design Calculations

Below are the equations and methods used to design the riprap layer around the pier and abutments.

Riprap at abutments: IADOT BDM C3.2.2.7 Scour.

- V_{avg} = Q/A = 2.33 ft/s (from HEC-RAS output)
- If V_{avg} < 8 ft/s, use Class E revetment stone
- If V_{avg} >= 8 ft/s, use Class B revetment stone

  $\therefore$ USE CLASS E
- Riprap upstream and downstream from the abutment, about 10'
- Riprap 2' deep into existing soil to prevent floodway constriction

Final design: 2-foot deep riprap layer which extends 10 feet upstream and downstream from the abutment with engineering fabric underlain.

Riprap at piers: HEC-18 section 7.5.1 and IADOT Standard Bridge Sheet 1006C - MACADAM STONE SLOPE PROTECTION - STUB ABUTMENT.

- $D_{50}=\frac{(K\,V)^2}{153.6}=\frac{(1.7*2.33*1.3)^2}{153.6}=0.17$ ft.
- D_{50} = median stone diameter, ft
- K = coefficient of pier shape, 1.5 for round nose and 1.7 for square nose
- V = velocity approaching pier = (Q/A)*C, C = 0.9 for near bank/ straight, 1.7 for middle/ curved. $\therefore$ Use 1.3 for between middle and edge of channel and straight bank.
- Use Class E since D_{50} < 1 ft (Class E D_{50} = 1.0 ft)
- Width of riprap should be 2 x pier column width minimum. IADOT usually uses 25' for county bridges, but no need for this pedestrian bridge.
- Thickness = 3 x D_{50} = 3 x 1.0 ft = 3.0 ft
- Width = 2 x 5.0' = 10.0' (column width = 5.0 ft)

Final design: 3-foot deep riprap layer which extends 10 feet in all directions from the pier with engineering fabric underlain.

Appendix F – Design Drawings

The design drawings are available in a file titled “Volga Pedestrian Bridge.pdf” located in the project submittals folder.

Appendix G – Design Renderings and Models

This section is a collection of images from the 3D model created using Autodesk Infraworks.

Figure G.1. 3D rendering of the entire project, looking north.

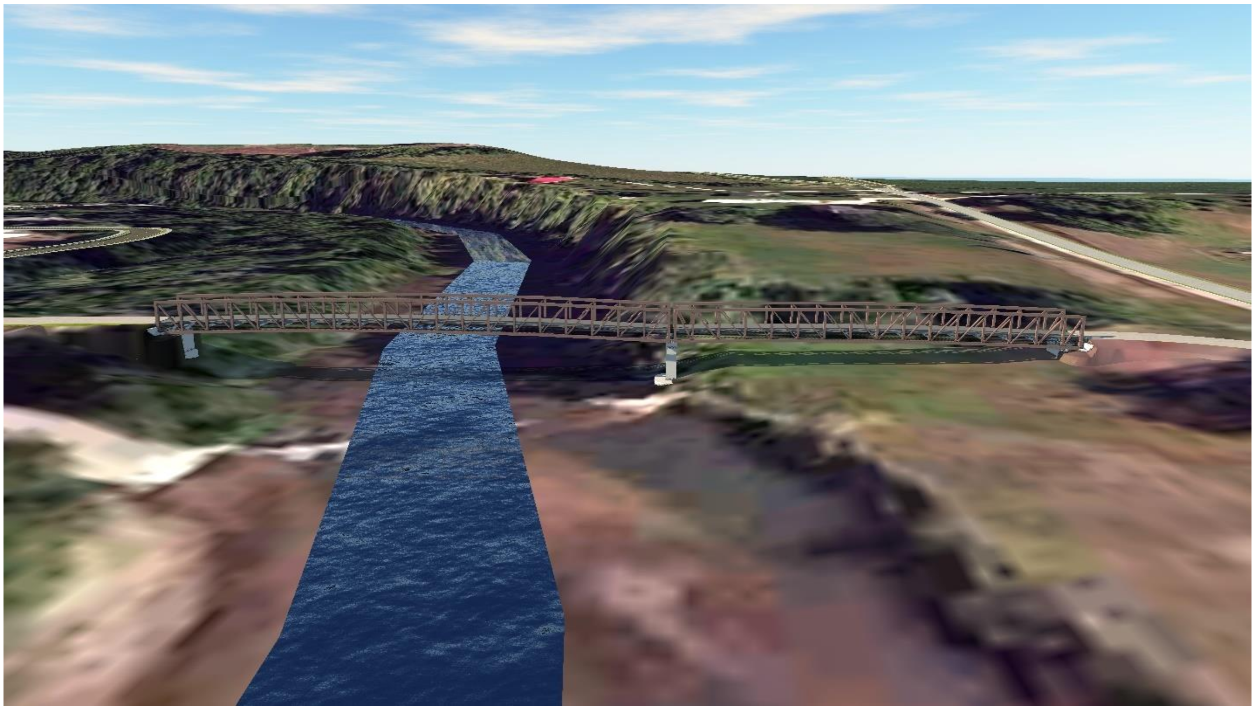

Figure G.2. 3D rendering of the pedestrian bridge, looking north.

Figure G.3. 3D rendering view looking west.

Figure G.4. 3D rendering of the bridge approach, looking west.

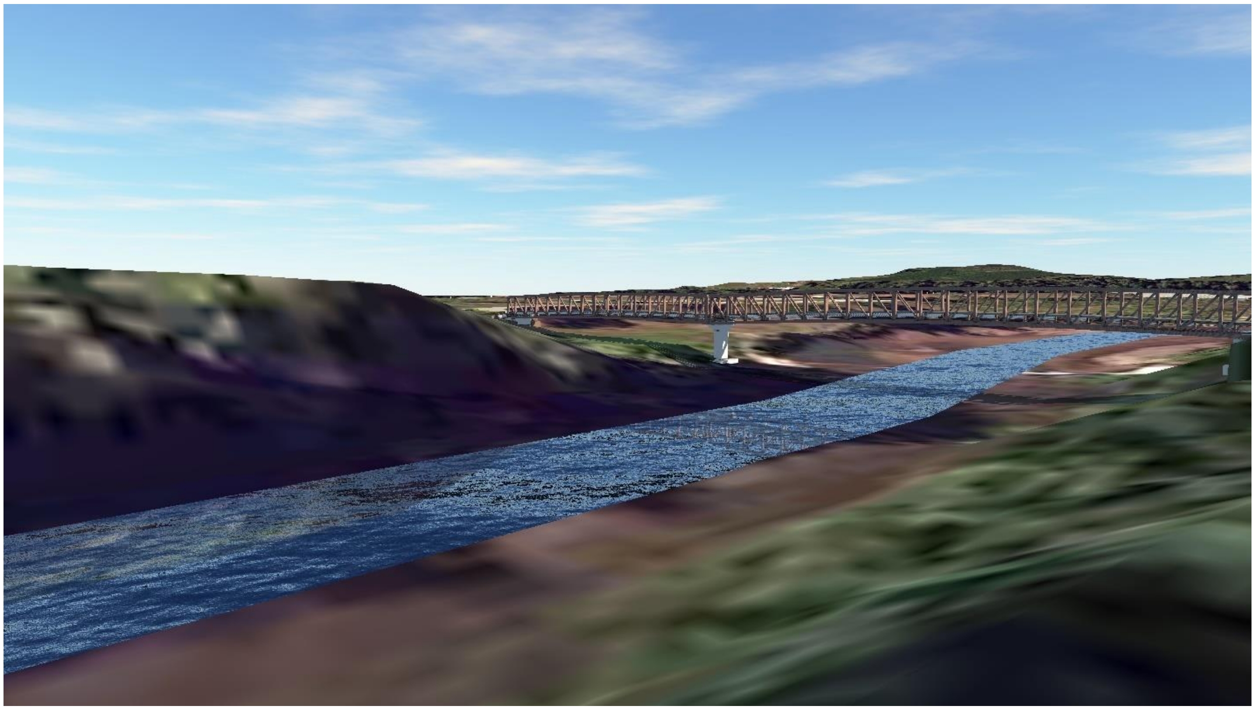

Figure G.5. 3D rendering view looking south from the Reflection Park.

Figure G.6. 3D rendering pedestrian view looking west, showing the expansion joint.

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