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FINAL DELIVERABLE

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Completed By	Chuanjing Hu, Dylan Bolton, James Scher
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Instructor	Paul Hanley
Community Partners	Boone County Conservation

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Provost's Office of Outreach and Engagement
The University of Iowa
111 Jessup Hall
Iowa City, IA, 52241
Phone: 319.335.0684
Email: outreach-engagement@uiowa.edu
Website: http://outreach.uiowa.edu/

DON WILLIAMS RECREATIONAL AREA PEDESTRIAN BRIDGE

Ogden, IA

Proposal prepared by:

DCJ Bridge Consultants

University of Iowa

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Section I Executive summary

We are a group of three civil engineering students at the University of Iowa. As part of our senior curriculum we are taking a capstone design course in which we complete a preliminary design for a real world engineering project. The work contained in this report, as well as the design drawings, is for academic purposes only.

We have designed a brand new pedestrian bridge to replace the existing one that connects campgrounds A and B at the Don Williams Recreational Area in Ogden, IA. The new bridge, shown in figure 1, will be constructed in the same location as the existing bridge. The new bridge has been designed with a ten foot width to allow not only for pedestrians, but also bicycles and commercial lawn mowers to use it as well. The width is sufficient for foot traffic, bikes, UTV's, and commercial lawn mowers. Pedestrians will continue to save time when traveling between campgrounds A and B via the new bridge. Additionally, lawn mowers, landscaping crews, maintenance workers, or park staff using UTV's, will save time when crossing the new bridge. The bridge has been raised to a height that is approximately the same at the trail leading to it, and about 16 feet over the water. The bridge superstructure is a Howe truss, 10.25 feet wide, and 4 feet deep, made of steel that will over time provide a natural rustic look shown in figure 1. The bridge also has two fishing areas, one on either side of the bridge, similar to the existing bridge. The fishing outcrops are 8 feet wide by 8 feet long, with a slightly lower handrail to allow for ease of fishing over the side. The bridge deck is made of steel grating.

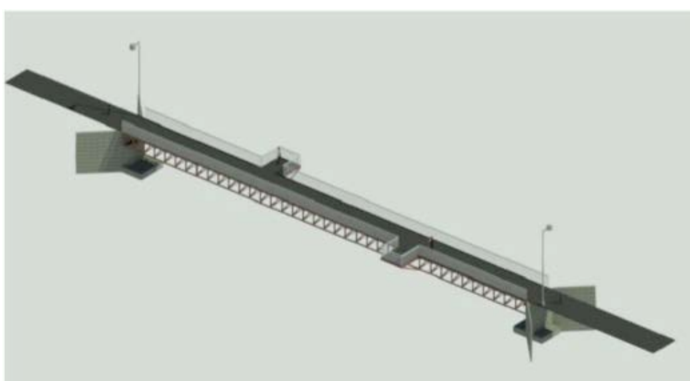


Figure 1. Final bridge design.

The bride will be supported by reinforced concrete abutments on each side of the lakeshore. Each abutment also has a wingwall to support the soil around the approach. The bridge will sit on pin and roller attachments on the abutment. The abutment is shown in figure 5.

To protect the lakeshore around the abutments we included the design of a riprap layer to prevent erosion in this critical area. The riprap was designed using NRCS design guides.

Connecting to the bridge will be a 10 foot wide asphalt trail. The trail has been designed according to Iowa DOT standards for Shared Use Path Design. The trail will be ADA compliant and have a longitudinal slope of less than 2%. The trail will serve for pedestrian use as well as bicycles, UTV's, and commercial lawn mowers. We attempted to keep the trail costs low be reducing the amount of clearing, grading, cut, and fill that would be required.

Several challenges and constraints were considered during the design of the new pedestrian bridge. One challenge was ensuring that unauthorized motor vehicles will not be able to cross the bridge. Another challenge was to design a bridge that would be meet ADA standards, making it accessible. A constraint was to ensure the the new bridge would house the water main connecting campgrounds A and B. The new bridge is designed to support that water main, and keep the existing route with minimal changes.

Along with our own bridge design we also reached out to an engineering firm, Bridge Brothers, for an alternative design. They provided us with the design and cost of their pedestrian bridge. Their design was similar to ours, in that it was a deck truss, 152 foot span, with 2 fishing outcrops. The main difference between their design and ours was simply cost.

The total cost of the project, which includes all design elements, as well as materials and labor is $248,672. Compared to the cost of the prefabricated bridge option this is the best design choice.

Section II Organization Qualification and Experience

Name of Organization

DCJ Bridge Consultants

Organization Location and Contact Information

DCJ Bridge Consultants is a group of three Civil Engineering students attending the University of Iowa and can be reached through the Project Manager's (Dylan Bolton's) email at dylan-bolton@uiowa.edu or evening time phone number at 217-430-0400.

Organizational Design Team Description

We are a team of students at the University of Iowa in senior year capstone design class. Dylan Bolton focuses on structures and specializes in structural analysis and bridge design. Chuanjing Hu focuses on structures and specializes in foundations & abutments as well as trail design. James Scher focuses on environmental and hydraulics issues and specializes in riprap design and as well as trail design.

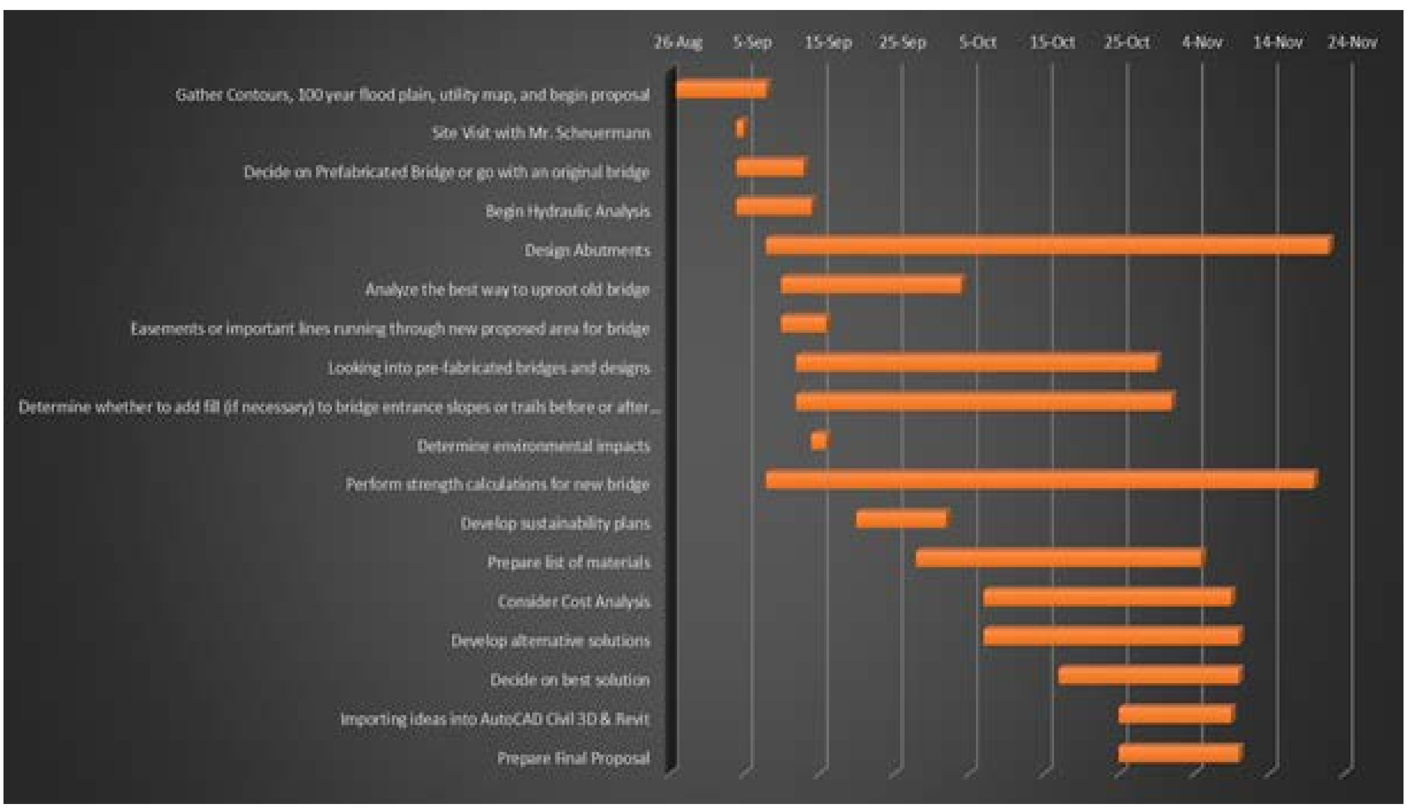
Section III Design Services

Project Scope

The project goal was the design of a single span pedestrian bridge that could accommodate foot traffic, bicycles, UTVs, and commercial lawn mowers. Additionally, outcrops that would allow people to fish off the side without obstructing pedestrians and bicyclists was a design objective. The bridge would be supported by abutments on either side, strong enough to carry all loads. The shoreline around the abutments would be protected from erosion by a layer of riprap. Another project goal was the design of a trail that would connect campgrounds A and B with the bridge. The last project goal was the additional features, such as removable steel bollards, safety signs, the water main, and lighting. We will discuss each of the design elements in the final design section of the report.

The deliverables to our client include a report, a drawing set, a display poster, a presentation, and 3D renderings of our project design. The deadlines for the project submission is December 7th, 2018.

Work Plan



Task	Start Date	End Date	Duration (Days)
Gather Contours, 100 year flood plain, utility map, and begin proposal	27-Aug	7-Sep	12
Site Visit with Mr. Scheuermann	4-Sep	4-Sep	1
Decide on Prefabricated Bridge or go with an original bridge	4-Sep	12-Sep	9
Begin Hydraulic Analysis	4-Sep	13-Sep	10
Design Abutments	8-Sep	2-Nov	75
Analyze the best way to uproot old bridge	10-Sep	3-Oct	24
Easements or important lines running through new proposed area for bridge	10-Sep	15-Sep	6
Looking into pre-fabricated bridges and designs	12-Sep	29-Oct	48
Determine whether to add fill (if necessary) to bridge entrance slopes or trails before or after the new bridge is to be added	12-Sep	31-Oct	50
Determine environmental impacts	14-Sep	15-Sep	2
Perform strength calculations for new bridge	8-Sep	31-Oct	73
Develop sustainability plans	20-Sep	1-Oct	12
Prepare list of materials	28-Sep	4-Nov	38
Consider Cost Analysis	7-Oct	6-Nov	33
Develop alternative solutions	7-Oct	7-Nov	34
Decide on best solution	17-Oct	7-Nov	24
Importing ideas into AutoCAD Civil 3D & Revit	25-Oct	8-Nov	15
Prepare Final Proposal	25-Oct	9-Nov	16

Section IV Constraints, Challenges and Impacts

Constraints

The project design was constrained by several things. The first constraint was that the bridge must be a single span. The client did not want any supporting columns in the water. The next constraint was the material choice for the bridge. The client wanted an aesthetic bridge, with a rusted metal look. The next constraint was to include a minimum of two fishing outcrops, one on each side of the bridge. We noted that fishing from the bridge was actually a main attraction (aside from quicker access between campgrounds), and was a required feature of the new bridge. The next constraint for the new bridge was a deck wide enough to allow to bicycles to cross. The existing bridge and trail design cannot accommodate bicycles or UTV's. The last constraint was to keep the existing water main on the bridge. Currently the water main runs down the hillslope, crosses the bridge, and runs back up the opposite hillslope. The water main supplies potable water to the shower facility at campground B. This water main would need to be included in the design of the new bridge. We will discuss each of these constraints in the final design section.

Challenges

The project involved several challenges that we considered during the design process. The first challenge was relocating the water main. Since the existing bridge supports the water main, and will have to be demolished before construction of the new bridge begins, that means the water must temporarily be shut off. We noted that this will likely only affect water service to the shower facility at campground B. The interruption in water service will only be for the duration of the bridge superstructure construction, and will resume normal function as soon as this construction phase is complete. The next challenge was figuring out a way to decrease the trail approach angles to the bridge. The existing bridge has a staircase leading down to it on either side of the span. This presents a problem for bicycles, lawn mowers, and ADA compliance. The next challenge was to find a way to prevent unauthorized motor vehicles from using the bridge. The next challenge was to discourage park visitors from jumping off the bridge. It was determined that jumping off the bridge into the water below is unsafe.

Societal Impact within the Community and/or State of Iowa:

The purpose of this study is to identify the population, economic, and social aspects of the Don Williams Recreation bridge project within the City of Ogden also Boone County.

Population Characteristics:

The Don Williams Recreation Area, is Boone County's largest conservation park. Boone County's estimated population is 26484 people, according to the 2017 United States census from the US Census Bureau. The median age of Boone's people is approximately 41, with residents identified as 96.6% white, 1.2% Black or African American, 0.5% native American and 0.5% of Asian descent. Between April, 2010 and July, 2017 the population of Boone County grew from

26306 to 26484 with a 0.7% increase. Ogden, where the project is located, has a population of 2022 people. The overall median age is 46. According to the American Community survey, there were 992 households in the city and have a median house value of $97400.

Labor Force:

According to the 2007-2011 American Community Survey, the labor force for the City of Ogden, Iowa is made up of 64.3% of the total population being at ages 16 years and older.

Industry Distribution in Boone County:

Based on data from 2017 Iowa's Workforce and the Economy, Boone serves as the home to several industries which include: Glycerin Group, LLC. "The company was awarded tax benefits from High Quality Jobs (HQJ) for this $27 million capital investment that is set to create 41 jobs at a qualifying wage of $21.58 per hour."

Social Impact:

Based on the local survey, the Don Williams Recreation Area has been a popular area for locals to hike, fish, and camp. The project is to replace the current wooden bridge located between Campgrounds A and B, and allow small UTVs to safely cross the bridge. This would provide convenience for visitors and park staff to travel between campgrounds while not bothering fishers on the bridge. The biggest social concern for the project will be the impact on the guests visiting during the bridge construction phase, which will be a consideration in the design.

Environmental Impact:

The development of a new bridge at Don Williams Recreation Area has some natural environmental impact on Ogden. The construction of a leading path to the bridge might cause minimal effects on trees and other shrub if the path is decided to be relocated. Wildlife impact is minimal due to the construction area's small size. The major concern when developing the abutment for the bridge will cause some negative impact on the surface water. Runoff from the site could possibly travel through the Don Williams Lake and Bluff Creek and cause negative health effects on visitors hiking and camping in the Recreation Area, as well as the fish barrier located just downstream of this pedestrian bridge. Other than some trees, the project will not have drastic impacts on the landscape in the area. During construction, the area of disturbed soil will be less one acre, therefore a Storm Water Pollution Prevention Program (SWPPP) will not be required. It is worth noting, however that if multiple construction projects occur at the same time, in the same general area, that together would have one acre or more of disturbed soil, a SWPPP might be required.

Sustainable Practices:

To make the bridge and trail design as sustainable as possible we have chosen materials and designs that will last as long as possible. The trail has been designed with a gravel base and concrete (PCC) paved layer. Adding the gravel base will cost more initially, but it will increase the lifespan of the concrete by reducing cracking over time. The bridge was designed with a steel superstructure as well as a steel deck. The superstructure does not require paint, and will naturally weather over time. This eliminates the need for repainting. The lifespan of the steel will depend on environmental and site conditions, as well as maintenance. A typical steel bridge is designed to last 75 years.

Section V Alternative Solutions that were Considered

During the design phase we considered multiple design options for the project. The design options needed to take into account the projects constraints, challenges, and the client's preferences. Our goal was to deliver to our client, the best possible design solution, for the lowest cost.

Our first design alternative was a steel girder bridge with a 152 foot span. A girder bridge uses girders (steel in this case), to support the deck and loads on the bridge. Steel girders can be manufactured to various lengths, and joined together to span the design length of 152 feet. An advantage of a girder bridge is the simplicity of the design and ease of construction. A downside of this type of bridge is the high cost of long steel girders sections. To be able to support the loads on the bridge, the web of the girder must be 44 inches in our calculations. Long span, 40 inch deep sections of steel beam are extremely expensive. For this reason we decided to look for other options for the superstructure of the bridge.

Our next design alternative was a deck truss bridge. In this design the superstructure of the bridge was a Howe style truss located under the deck of the bridge. Truss bridges allow for longer span lengths without the need for support columns. Truss bridges are strong, and can be constructed with smaller sections of steel than a girder bridge, which means less heavy equipment such as cranes are needed to construct the bridge. The overall cost of a steel truss bridge was significantly less in our design. For this main reason we selected a steel truss bridge as our final design.

Another alternative was the height of the bridge; keeping it at its present height or raising it by 10 feet. An advantage of the existing bridge height would be a slightly shorter span length. Keeping the existing bridge height would require the abutments to be placed at the edge of the water lakeshore, and would give us a span length of 125 feet. Another advantage is the project would require smaller abutments (height only). A major disadvantage of this design is the amount of work that would have to go into the trail construction. To build a trail connecting to the existing bridge height would require significantly more cut and fill, as well as a switchback trail, to avoid a steep down sloping trail. The slope of the switchback would allow for ADA compliant accessibility, however it was not feasible to build a switchback trail that would easily accommodate UTV's and commercial lawn mowers. The cut and fill combined with the switch back trail would also increase the price of the project.

An alternative was to raise the height of the bridge to match the existing ground elevations at the top of the hillslope on either side of the bridge. This places the bridge deck approximately 16 feet over the water surface. There were no real disadvantages to this design alternative. An advantage is that the trail work would require less cut and fill, and no switchback. The trail would also be ADA compliant, and would accommodate bicycles, and UTV's. Another advantage is the amazing view that park visitors will enjoy while using the bridge. The bridge will provide unobstructed views of the lake and surrounding park area.

Another option for the bridge, is to buy a prefabricated bridge. Rather than hiring an engineering firm to design the entire project including the bridge, the bridge can be purchased separately. The engineering firm will however need to design the other elements of the project including the abutments. Prefabricated bridges can be purchased to fit the needs of this project, but will vary in design and cost.

The last design alternative we considered was the material for the trail. The trail options were gravel, gravel with concrete pavement, gravel with asphalt pavement. We compared the initial costs, the aesthetics, the lifespan, and ADA standards to determine the best option. Using only a layer of gravel is the cheapest option. However it would not meet ADA standards. A disadvantage is the potential for lawsuits, since the trail is open to the public, and would not be ADA compliant. Another disadvantage is that the park would not qualify for Federal grant money for an ADA approved trail.

Section VI Final Design Details

The project goal was the design of a single span pedestrian bridge that could accommodate foot traffic, bicycles, UTVs, and commercial lawn mowers. Additionally, outcrops that would allow people to fish off the side without obstructing pedestrians and bicyclists was a design objective. The bridge would be supported by abutments on either side, strong enough to carry all loads. The shoreline around the abutments would be protected from erosion by a layer of riprap. Another project goal was the design of a trail that would connect campgrounds A and B with the bridge. The last project goal was the additional features, such as removable steel bollards, safety signs, the water main, and lighting. We will discuss each of the design elements of the project as well as the decision making process for our choices.

Bridge Design

The bridge we have designed is a 152 foot, single span steel bridge, that has a 10 foot wide deck. The width of the bridge deck is wide enough to allow for pedestrians, bicycles, UTV's, or commercial lawn mowers to safely pass. This will save time for park staff, maintenance workers, or landscapers who need to get UTV's back and forth from campgrounds A and B.

The bridge has been raised to a height that is approximately the same at the trail leading to it, and about 16 feet over the water. The selected elevation of the bridge has several advantages. During a 100 year flood event, the water level will rise about 5.5 feet. Even under these conditions, the lowest point of the bridge truss will still be about 7 feet above the water. Another advantage is the amazing view that park visitors will enjoy while using the bridge. The bridge will provide unobstructed views of the lake and surrounding park area. The last advantage is that by placing the bridge at almost the same elevation as the top of the hillslopes on each side of the lake, the trail can remain nearly flat. This means no need for stairs, ramps, or excessive grading to connect the bridge and trail.

The bridge superstructure is a Howe truss, 10.25 feet wide, and 4 feet deep, made of steel that will over time provide a natural rustic look. A 3D rendering of the bridge is shown in figure 3. Steel was best material for the bridge superstructure for a number of reasons. Steel is strong, lightweight, cost effective, and can be aesthetically pleasing. Our design has taken into account the client's preference to have a naturally weathered look for the bridge superstructure. Over time the steel will weather and achieve this look.

The steel truss is located under the bridge deck, rather than above, for several reasons. One reason is that having the truss under the deck allowed for us to design the fishing outcrops. When the truss is above the bridge deck, it makes it challenging to create a design with overhanging areas on the side that are still accessible. Having the truss above the bridge deck would also make it challenging to cast a fishing line over the side without interference from the steel beams. The next reason is improved aesthetics. The bridge looks cleaner and more open when the deck is on top of the truss. Pedestrians passing over the bridge will have an unobstructed view of the surrounding water and park area.

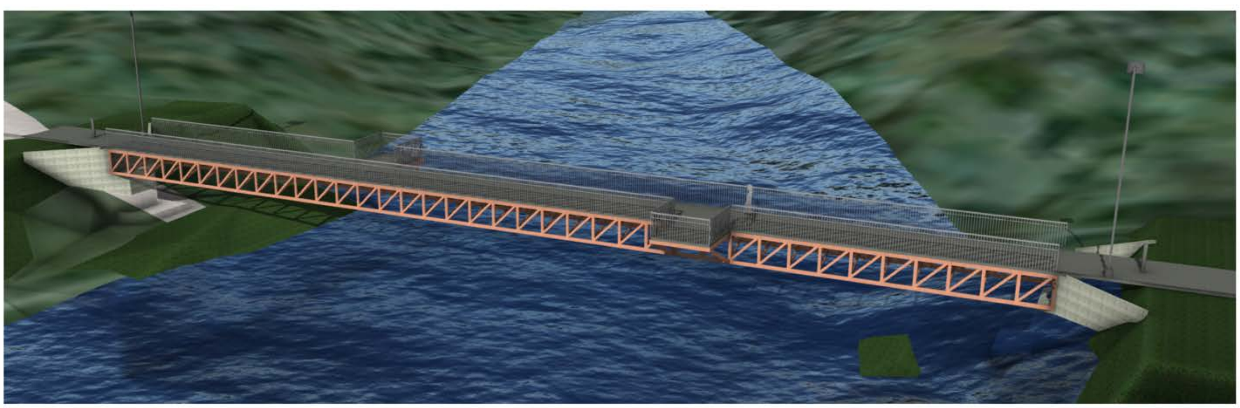


Figure 3. InfraWorks 3D rendering of bridge design.

Fishing has always been a major attraction of the existing bridge, so the new design also has two fishing areas, one on either side of the bridge. Design drawings of the fishing outcrops are shown in figure 4. The fishing outcrops are 8 feet wide by 8 feet long, with a slightly lower handrail to allow for ease of fishing over the side. This will provide enough from for multiple fishers, while still allowing other traffic to safely use the bridge. Spans are at every 4 feet center-to-center where each floor beam will be located, which will be placed under the steel grated deck. The steel deck will be a Stainless Steel, Type 304, 4.50 # grating(standard) with a 58% open area. The deck dimensions will be 48” by 120” panels (120” spanned laterally to completely cover the 10’ deck) and is 0.625” thick. Calculations for the bridge design are in appendix B.

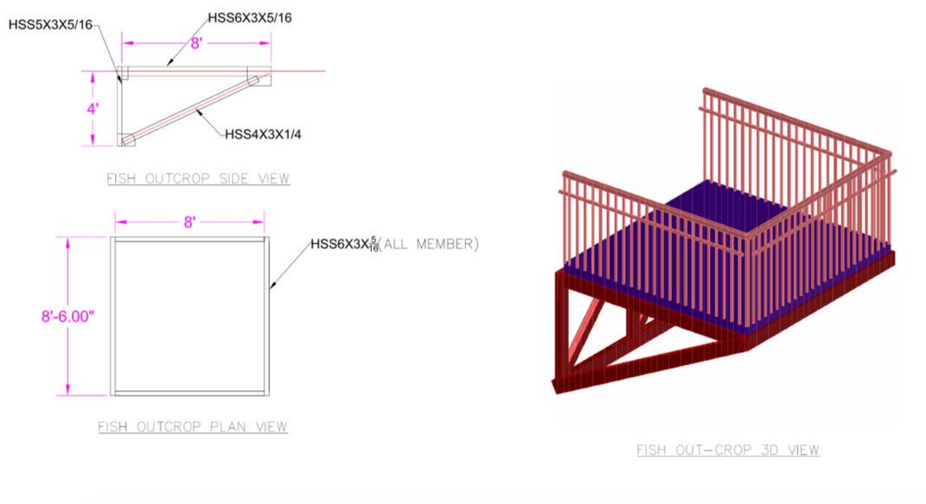


Figure 4. Fishing outcrop drawings.

Abutment Design

We designed the abutment as a non-Integral abutment without piles, the total width of the abutment was 11 feet, and the length was 10 feet. The stem width was designed as 3 feet, and the connection spacing between the truss and the abutment was 2 feet. The bottom footing reinforce for the abutment was # 7 bar with 12 inches spacing, #6 “O” bar with 9 inches spacing was designed for connecting footing and stem. The main stem reinforce was designed to use #7 bars and #5 “Lw” bars both with 12 inches spacing, and top stem was using #5 bar with 10 inches spacing. The material of the abutment will be normal concrete, and the reinforcing steel will be use A 572 Gr. 60. Wing walls were designed for each abutment with 45 degree along the bridge direction. The abutment was designed to be backfilled with gravel with D50 of 2.2 inches for drainage purpose also with 6 inches underdrain wrapped pipe at the bottom. The abutment was designed and checked based on AASHTO LRFD Bridge Design Specification Section 3- 6 and Iowa DOT LRFD Bridge Design Manual Section 5 to 6 including bearing capacity, sliding, overturning and settlement. Calculations for the abutment design are in appendix A.

One of the challenges with this project was to keep the bridge span length reasonably short to help keep the costs low. The placement of the abutments is what impacted this challenge the most. We decided to place the abutments close to the lakeshore, which lessens the distance between them, thus decreasing the span length. The abutments are actually placed on the hillslope of the lakeshore so they are above the water surface elevation.

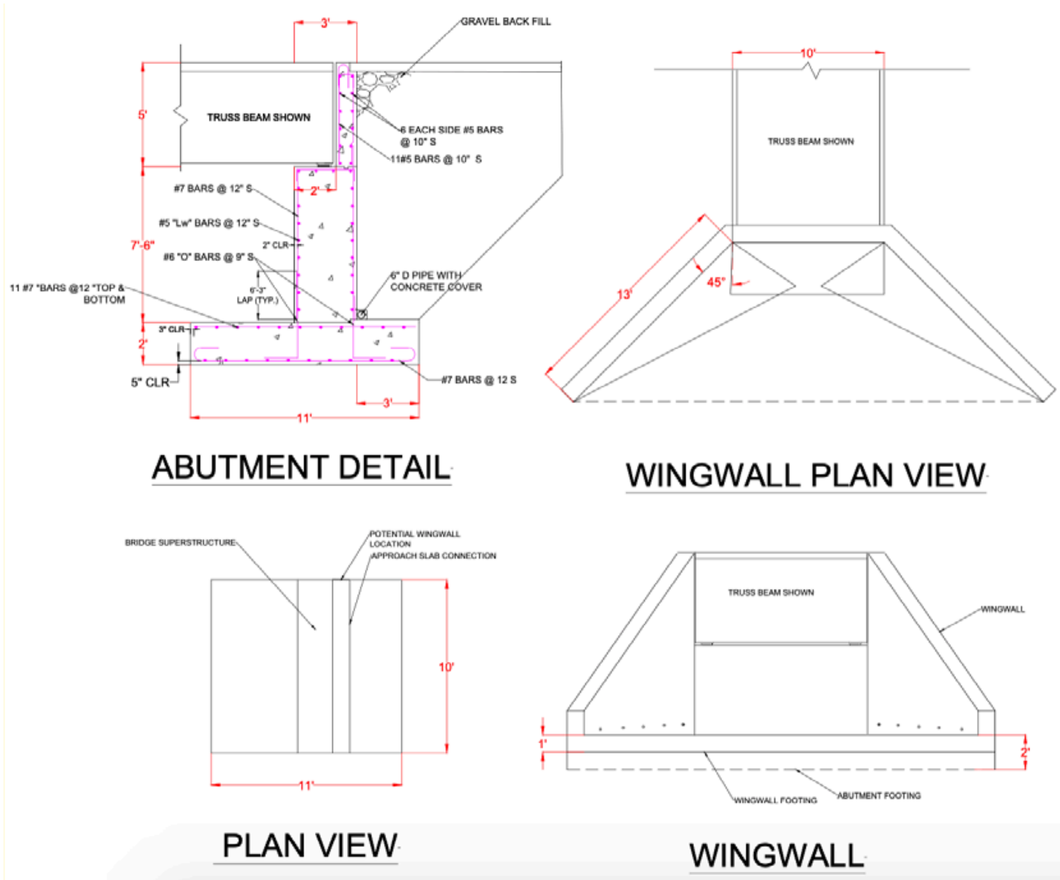


Figure 5. Abutment design drawings.

Riprap Design

To protect the lakeshore we completed a riprap design, using the Natural Resource Conservation Service (NRCS) document for slope protection for dams and lakeshores, from the Minnesota technical note 2. Our area of interest for this project was the lakeshore around the bridge abutments. We designed riprap to protect this area from erosion from wind generated waves hitting the lakeshore. Since the lake has a water velocity of nearly zero at the bridge site, the only bank erosion would be from waves and potentially from ice. Using local wind data and a series of other design factors, we determined the median stone size of 8 inches, the upper and lower protection boundaries, type A cross section, geotextile lining, and a thickness of 12 inches or riprap. The combination of the geotextile and the 8 inch stone layer will ensure that the waves will not erode the soil around the abutment. Figure 6 shows the design drawing for the riprap. The calculations for the design are in appendix C.

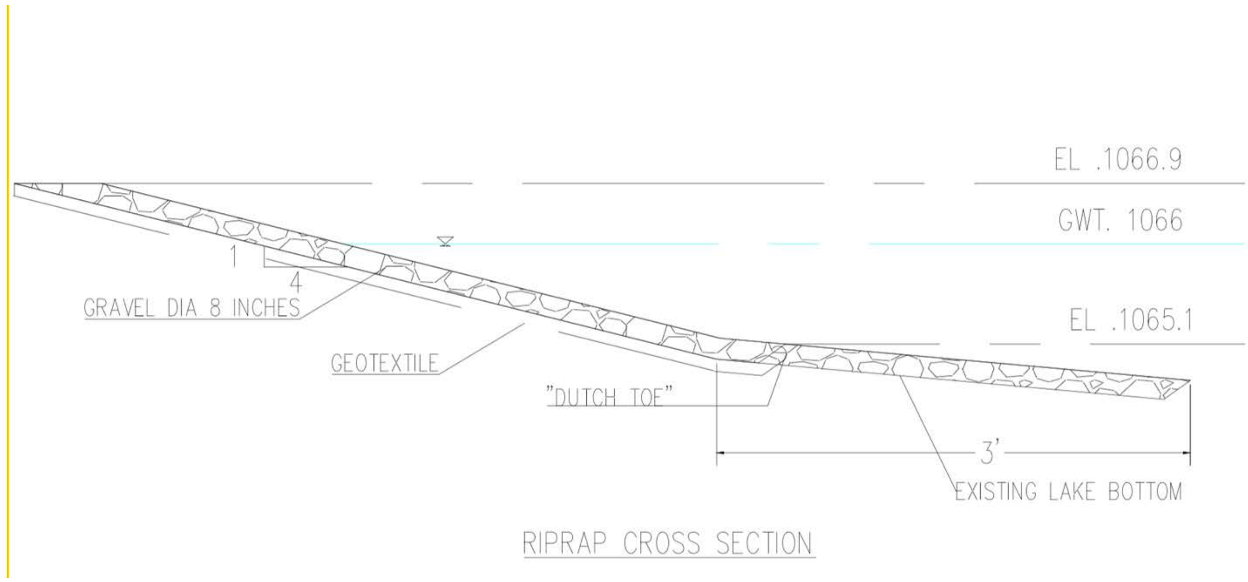


Figure 6. Riprap design drawing.

Trail Design

The new trail was designed using civil 3d software, contour data from the IDNR, and aerial images from google maps and IDNR. The standards used were from the Iowa DOT Design Manual for Shared Use Path Design. The goal was to design the new trail to be in the same location as the existing gravel trail to minimize cost, and reduce the amount of vegetation and trees that would need to be cleared. The trail location is shown in figure 7.

Figure 7. Overview of trail design.

The trail was designed as a 10 foot wide HMA pavement with a 2 foot wide graded shoulder on either side. The pavement thickens is the recommended is 5 inches. The gravel base is designed as 4 inches in depth. The cross slope is 1.5% for drainage. The longitudinal slope of our trail does not exceed 2%. Cross section drawings are shown in figure 8. The benefit of a paved trail design that it is ADA compliant which means it is accessible to all visitors to the park, and you decrease the likelihood of lawsuits. Since the bike trail meets a local road, a clear separation of the paths should be marked with signs, to alert drivers that this path is not for cars.

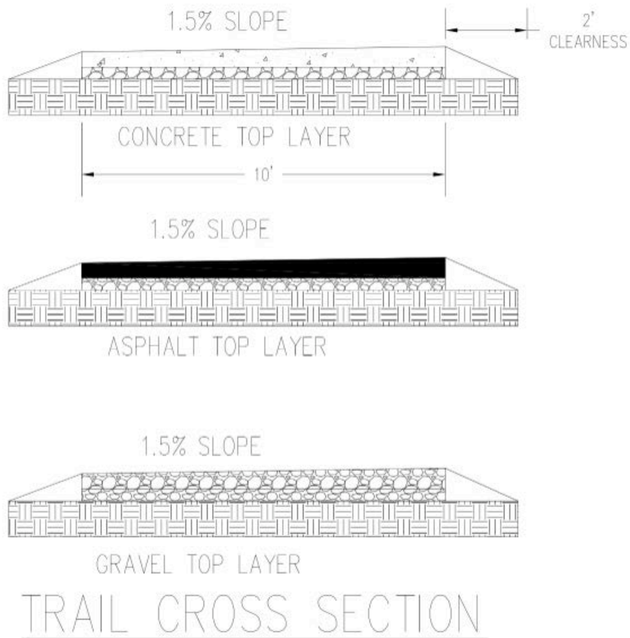


Figure 8. Trail cross section design drawings.

Additional Features

We included two removable steel bollards in our design, one on each side of the bridge. This is a commonly used safety feature that presents a physical barrier to stop unauthorized vehicle from entering a specific area. The removable steel bollards are placed at either end of the bridge, and are secured to the ground with either a lock or a bolt. They can simply be removed and placed to the side to allow commercial lawn mowers to use the bridge.

Another challenge we addressed in the project was to discourage park visitors from jumping off the bridge into the water below. We determined that the best course of action to discourage this behavior was to install warning placards along the bridge. The message written on the placard should warn visitors to the danger of jumping. This could include information about shallow water depth, hidden objects below the surface of the water, and dangers of old fishing lines and hooks potentially in the water below.

The project also needed to address the constraint of the existing water main on the bridge. Currently a water main runs along the bridge connecting the water supply to campground B. Our final design includes the addition of a new section of water main to be installed under the deck of the new bridge. Since the new bridge is in the same location as the existing bridge, there will be minimal rerouting of the water main.

Lastly, we decided to install a total of 2 light poles, one on either side of the bridge, that would provide enough lighting to illuminate the bridge approach path, as well as the full span or the bridge. A single light pole on both sides of the bridge will accomplish this, adding to the safety of the bridge, while keeping costs low.

Section VII Engineer’s Cost Estimate

The primary source used to estimate the cost of the Don Williams Recreation Area Pedestrian Bridge proposal was RSMeans. The primary book used to calculate estimated values was from the 2019 Heavy Constructions Cost book, however, values were also pulled from the 2019 Site Work & Landscape Cost book, 2019 Assemblies Cost book, and 2019 Concrete Masonry Costs book.

Prices From	To	Rounded to nearest
$0.01	$5.00	$0.01
5.01	20.00	0.05
20.01	100.00	1.00
100.01	1,000.00	5.00
1,000.01	10,000.00	25.00
10,000.01	10,000.00	100.00
50,000.01	Up	500.00

Figure 9. RSMeans rounding standards

Table 3. Material Quantities

Estimated Bridge Quantities			
Item Number	Item	Unit	Total
1	Concrete Abutments	CY	37
2	#5 Steel Rebar for Concrete Abutments	FT	427
3	#6 Steel Rebar for Concrete Abutments	FT	716
4	#7 Steel Rebar for Concrete Abutments	FT	1148
5	Asphalt	CY	43
6	HSS6x3x5/16	EA	82
7	HSS5x3x5/16	EA	78
8	HSS4x3x1/4	EA	196
9	W12x16	EA	38
10	Steel Grated Deck	PP	37
11	Steel Bollard	EA	2
12	Concrete Pad for Steel Bollard	EA	2
13	Light Pole	EA	2
14	Handrail	LF	304
15	Water main	EA	2
16	Placard	EA	2
17	Bronze Drainage Pipe	EA	1
18	Gravel for Abutment	CY	37
19	Riprap	LCY	31.8

A legend has been provided to clarify quantity units used to calculate costs.

Table 4. Legend of Material Quantities

Legend	
Unit Nomenclature	Unit Name
CY	Cubic Yard
LCY	Linear Cubic Yard
FT	Feet
EA	Each
PP	Per Panel

Once quantities were pulled together and in appropriate units, the final cost table was pulled together and thrown into Microsoft Excel based on RSMeans values. This is shown in table 5.

Table 5. Preliminary Cost of Pedestrian Bridge

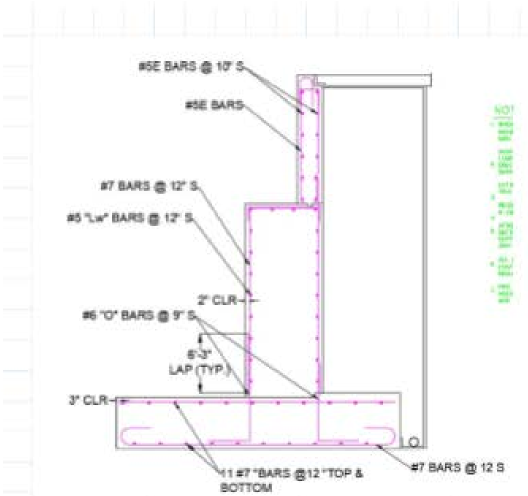
Total Project Cost

Project Item	Material	Labor	Equipment	Total	Total Including Overhead & Profit	**Notes:**		
Demolition with clean up	$ -	$ 18,738.00	$ 15,375.00	$ 34,113.00	$ 37,524.30	Average Cost for Demo in RS Means is $56.23 so we split this between labor and equipment since no material is needed to be bought		
Concrete Abutments with Steel Rebar	$ 3,777.65	$ 12,000.00	$ 19.07	$ 15,796.72	$ 17,376.39	Average Cost for Concrete for Shallow Foundations is $95/CY and $7/square foot for forms (from the side view)	$200 per hour for Excavation & 280 per hour for Steel Reinforcement	$0.52/cubic yard for equipment with direct chute
Asphalt Trails	$ 3,495.00	$ -	$ -	$ 3,495.00	$ 3,844.50	Assumed 4" depth of asphalt. The price includes labor and equipment	Pulled from Iowa DOT trails 2000	
Price of roller to compact concrete	$ -	$ 245.16	$ 48.92	$ 294.09	$ 323.50	6" Lift measured by the cubic yard	1467.8 cubic feet needed for North & South trail == 54.36 cubic yards of concrete needed	
Curing for Concrete	$ 42.54	$ 35.70	$ -	$ 78.24	$ 86.06	With burlap, 4 uses assumed, 7.5 oz.	54.36 CY = 54.36/((6/12)/3) == 326.15 SF == 3.26 CSF	
Members of Steel Truss Bridge	$ 43,827.51	$ -	$ -	$ 43,827.51	$ 48,210.26	Pricing Pulled from MidwestSteelSupply.com	Shipping cost is included in material cost	
Steel Erecting for Light Steel Tubing	$ -	$ 8,721.18	$ 4,360.59	$ 13,081.77	$ 14,389.95	Pricing found in RS Means from Structural Steel Framing, Structural Tubing A500GrB, 4 to 6" light section	990 Cubic feet needed for both abutments == 36.67 cubic yards	
Steel Erecting for W12x16 Floor Beams	$ -	$ 1,250.20	$ 653.60	$ 1,903.80	$ 2,094.18	Pricing found in RS Means from Structural Steel Framing, Structural Steel Members, W12x16 Section		
Grated Deck	$ 67,861.92	$ -	$ -	$ 67,861.92	$ 74,648.11	Quoted from McNichols.com @ 1785.84/panel (with shipping) and in need of 38 panels across bridge		
Steel Bollards	$ 180.00	$ -	$ 36.00	$ 216.00	$ 237.60	$90 per steel bollard when buying 2 to prevent vehicular traffic from uline.com model H-4970 Heavy Duty	$18 per 4 anchors for each bollard installation kits for concrete pads when buying 2	
Concrete Pad for Steel Bollards	36.9	11.2	48.1	$ 96.20	$ 105.82	Assumed 1 cubic yard of concrete needed per pad and same price of elevated slabs, less than 6" thick, pumped		
Light Poles	$ 2,600.00	$ 940.00	$ 120.00	$ 3,660.00	$ 4,026.00	Conservatively used Galvanized Steel, Exterior Lighting, 30' high Lighting Poles from RS Means		
Handrails	$ 10,336.00	$ 3,237.60	$ 276.64	$ 13,850.24	$ 15,235.26	Metal Railings (Galvanized Steel), Pipe and Tube Railings, 1-1/4" Railings and cost is based on Linear Foot from RS Means		
Water main	$ -	$ 564.00	$ -	$ 564.00	$ 620.40	1" to 2" Service		
2 Placards	$ 53.60	$ -	$ -	$ 53.60	$ 58.96	Placards priced from https://www.jjkeller.com/shop/Product/Custom-OSHA-ANSI-Sign-and-Label		
Bronze Drainage Pipe	$ 4,100.00	$ 130.00	$ -	$ 4,230.00	$ 4,653.00	3 & 4" Pipe size from Facility Storm Drainage section		
Gravel for Abutments	$ 1,283.45	$ 916.75	$ 1,466.80	$ 3,667.00	$ 4,033.70	990 Cubic feet needed for both abutments == 36.67 cubic yards	Average price of pea gravel per cubic yard is 35 dollars conservatively	
Pricing of Riprap	$ 1,908.00	$ 731.40	$ 11.40	$ 2,650.80	$ 2,915.88	Need from James	Machine placed for slope protection measured in linear cubic yards	
Geotextile for Riprap	$ 680.00	$ -	$ -	$ 680.00	$ 748.00	Price per SY that comes in a standard 15'x300' roll		
Freight Cost	$ -	$ 15,946.00	$ -	$ 15,946.00	$ 17,540.60			
		Total Cost Estimate for Pedestrian Bridge =			$ 248,672.47			

Table 6. A simpler breakdown brings us to the same result in a more general fashion

Total Project Cost					
Project Item	Material	Labor	Equipment	Total	Total Including Overhead & Profit
Demoltion with Clean Up	$ -	$ 18,738.00	$ 15,375.00	$ 34,113.00	$ 37,524.30
Bridge Material	$ 111,731.97	$ 10,252.24	$ 5,063.11	$ 127,047.33	$ 139,752.06
Asphalt Trails	$ 3,495.00	$ -	$ -	$ 3,495.00	$ 3,844.50
Concrete Abutments with Rebar	$ 3,777.65	$ 12,000.00	$ 19.07	$ 15,796.72	$ 17,376.39
Riprap	$ 2,588.00	$ 731.40	$ 11.40	$ 3,330.80	$ 3,663.88
Additional Utilities	$ 18,589.95	$ 5,799.55	$ 1,947.54	$ 26,337.04	$ 28,970.74
Shipping Costs	$ -	$ 15,946.00	$ -	$ 15,946.00	$ 17,540.60
Total					$ 248,672.47

Appendix A Abutment Calculations



Design Property:

$Wc := 0.15\ \frac{kip}{ft^3}$

$Ws := 0.49\ \frac{kip}{ft^3}$

Backfill soil:

(i) . Backfill soil:

$\gamma b := 120\ pcf$

$Kab := 0.33$

$Kpb := 3$

$\phi' := 29\ °$

(2) In situ soil:

$\gamma := 120\ pcf$ $\quad\quad$ $\gamma w := 62.4\ pcf$

$\gamma sat := 135\ pcf$

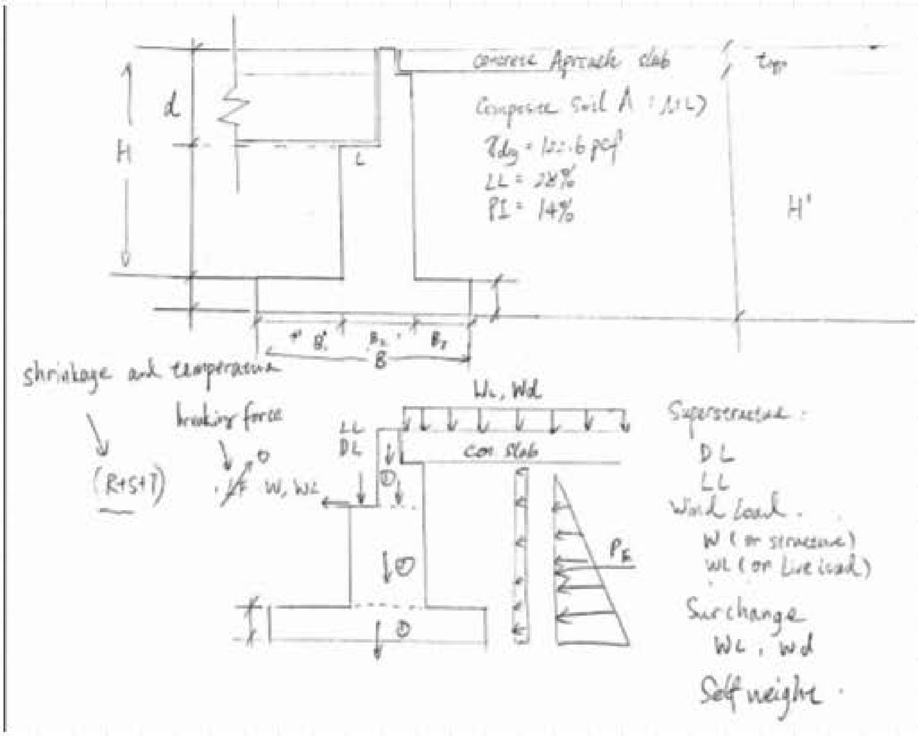
$\gamma' := \gamma sat - \gamma w = 72.6\ pcf$

$Ka := 0.42$

Table 2. Consistency of granular soil.

Consistency	D_r (%)	N_{60}	N'_{60}	ϕ' (°)
Very loose	0-15	< 4	< 3	< 28
Loose	15-35	4-10	3-8	28-30
Medium dense	35-65	10-30	8-25	30-36
Dense	65-85	30-50	25-42	36-41
Very dense	85-100	> 50	> 42	> 41

Dimesion of the abutment:



Dimesion:

$H' := 11\ ft$

$t_{approachSlab} := 4\ in$

$d := 3\ ft$

$d' := H' - d = 8\ ft$

$tf := 2\ ft$

$H := H' - tf = 9\ ft$

$B1 := 5\ ft$

$B2 := 3\ ft$

$B3 := 3\ ft$

$B := B1 + B2 + B3 = 11\ ft$

$L := 1.5\ ft$

$L' := B2 - L = 1.5\ ft$

$h := H - t_{approachSlab} = 8.667\ ft$

$Df := h = 8.667\ ft$

$W_{abutment} := 10\ ft$

$\phi' := 29°$

$Ka := \tan(45° - \phi')^2 = 0.082$

Loads act on the abutment:

$DL := 11.913\ \frac{kip}{ft}$ Dead load from superstructure

$LL := 0.9\ \frac{kip}{ft}$ Live load from superstructure

$WS := 0.25\ \frac{kip}{ft}$ Wind load on superstructure

$WL := 0.05\ \frac{kip}{ft}$ Wind load on live load

$RST := 0 = 0\ \frac{s^2}{kg} \cdot \frac{kip}{ft}$ shrinkage and temperature force (assume 0)

Surcharge LL:

$$wl := 2 \cdot Wc \cdot t_{approachSlab} = 0.1\ \frac{kip}{ft^2}$$

$$wd := Wc \cdot t_{approachSlab} = 0.05\ \frac{kip}{ft^2}$$

$$vl := wl \cdot B3 = 0.3\ \frac{kip}{ft}$$

$$vd := wd \cdot B3 = 0.15\ \frac{kip}{ft}$$

$$Pe := \frac{1}{2}\ \gamma b \cdot Kab \cdot h^2 = 1.487\ \frac{kip}{ft}$$

$$Hl := Kab \cdot wl \cdot h = 0.286\ \frac{kip}{ft}$$

$$Hd := Kab \cdot wd \cdot h = 0.143\ \frac{kip}{ft}$$

Vertical Load Table:

Service Vertical Loads

Item	selfweight 1	selfweight 2	selfweight 3	backfill soil 4	DL	LL	VL	VD	Sum
I	3.3	3.375	0.75	5.1	0.17	0.425	0.425	0.5	14.045
II	3.3	3.375	0.75	5.1	0.17	0.5525	0.5525	0.5	14.3
III	3.3	3.375	0.75	5.1	0.17	0.34	0.34	0.5	13.875
IV	3.3	3.375	0.75	5.1	0.17	0	0	0.5	13.195
V	0	0	0	0	0	0.31875	0.31875	0	0.6375

Service Horizontal Loads

Item	Pe	Hd	Hl	WS	WL	LF	RST	Sum
I	2.93095	0.20075	0.4015	0	9.5	0	0	13.0332
II	0.20075	0.20075	0.4015	0	0	0	0	0.803
III	0.4015	0.20075	0.4015	0	0	0	0	1.00375
IV	0.169	0.20075	0.4015	0	0	0	0	0.77125
V	0	0	0	0	0	0	0	0

Service Vertical moments

Item	selfweight 1	selfweight 2	selfweight 3	backfill soil 4	DL	LL	VL	VD	Sum

M	18.15	21.9375	5.625	48.45	1.02	2.55	4.0375	4.75	
I	18.15	21.9375	5.625	48.45	1.02	2.55	4.0375	4.75	106.52
II	18.15	28.51875	5.625	48.45	1.02	3.315	5.24875	4.75	115.0775
III	18.15	17.55	5.625	48.45	1.02	2.04	3.23	4.75	100.815
IV	18.15	0	5.625	48.45	1.02	0	0	4.75	77.995
V	0	16.453125	0	0	0	1.9125	3.028125	0	21.39375

Service Horizontal moments

Item	Pe	Hd	Hl	WS	WL	LF	RTS	Sum
M	11.88663056	1.221229167	2.442458333	1.6055	0.475	0	0	
I	11.88663056	1.221229167	2.442458333	0.48165	0.475	0	0	16.50697
II	11.88663056	1.221229167	2.442458333	0	0	0	0	15.55032
III	11.88663056	1.221229167	2.442458333	0	0	0	0	15.55032
IV	11.88663056	1.221229167	2.442458333	1.12385	0	0	0	16.67417
V	0	0	0	0	0	0	0	0

Eccentricity Check

Service	V	H	Mv	Mh	xo	e	emax	qt	q (Ksf)	Check
I	14.045	13.0332	106.52	16.50696806	6.408902	0.908902	1.833333	1.90982	1.095741	Pass
II	14.3	0.803	115.0775	15.55031806	6.959943	1.459943	1.833333	2.335232	1.027307	Pass
III	13.875	1.00375	100.815	15.55031806	6.145202	0.645202	1.833333	1.705273	1.128929	Pass
IV	13.195	0.77125	77.995	16.67416806	4.647278	0.852722	1.833333	1.757479	1.419648	Pass
V	0.6375	0	21.39375	0	33.55882	28.05882	1.833333	0.944938	0.009498	Not Pass

Bearing Capacity Assessment

Service	V	H	q	qn	FS	Fsreq	Check
I	14.045	13.0332	0.127681818	1	7.831969	3	Pass
II	14.3	0.803	0.13	1	7.692308	3	Pass
III	13.875	1.00375	0.126136364	1	7.927928	3	Pass
IV	13.195	0.77125	0.119954545	1	8.336491	3	Pass
V	0.6375	0	0.005795455	1	172.549	3	Pass

Assume no ground water table

Vdead	13.195
Cu	2.3751
Ca	1.18755

Sliding check

Service	V	H	Vn	FS	Fsreq	Check
I	14.045	13.0332	140.6305	10.79017432	1.5	Pass
II	14.3	0.803	140.6305	175.1313823	1.5	Pass
III	13.875	1.00375	140.6305	140.1051059	1.5	Pass
IV	13.195	0.77125	140.6305	182.3410049	1.5	Pass
V	0.6375	0	140.6305	#DIV/0!	1.5	#DIV/0!

Vn=Af*Ca+0.5tf*L
Af=B*Wabut
ca= 0.5Cu
Cu = 0.18 Vdead (Assume basi

Factored Service Vertical

Item	selfweight 1	selfweight 2	selfweight 3	backfill soil 4	DL	LL	VL	VD	Sum
I	4.125	4.21875	0.9375	6.63	0.2125	0.74375	0.74375	0.625	18.23625
II	4.125	4.21875	0.9375	6.63	0.2125	0.57375	0.57375	0.625	17.89625
III	4.125	4.21875	0.9375	6.63	0.2125	0	0	0.625	16.74875
IV	4.95	5.0625	1.125	6.63	0.255	0	0	0.75	18.7725
V	4.125	4.21875	0.9375	6.63	0.2125	0.57375	0.57375	0.625	17.89625

Factored Service Horizontal

Item	Pe	Hd	Hl	WS	WL	LF	RST	Sum
I	4.396425	0.301125	0.60225	0	0	0	0	0.501875
II	4.396425	0.301125	0.60225	0	0	0	0	0.21125
III	4.396425	0.301125	0.60225	0.2366	0	0	0	0.0625

IV	4.396425	0.301125	0.60225	0	0	0	0	0
V	4.396425	0.301125	0.60225	0.0676	0.05	0	0	0

Strength Vertical moments

Item	selfweight 1	selfweight 2	selfweight 3	backfill soil 4	DL	LL	VL	VD	Sum
M	18.15	21.9375	5.625	48.45	1.02	2.55	4.0375	4.75	
I	22.6875	27.421875	7.03125	62.985	1.275	4.4625	7.065625	5.9375	138.8663
II	22.6875	27.421875	7.03125	62.985	1.275	3.4425	5.450625	5.9375	136.2313
III	22.6875	27.421875	7.03125	62.985	1.275	0	0	5.9375	127.3381
IV	27.225	32.90625	8.4375	62.985	`	0	0	7.125	138.6788
V	22.6875	27.421875	7.03125	62.985	1.275	3.4425	5.450625	5.9375	136.2313

Strength Horizontal moments

Item	Pe	Hd	Hl	WS	WL	LF	RTS	Sum
M	11.88663056	1.221229167	2.442458333	1.6055	0.475	0	0	
I	17.82994583	1.83184375	3.6636875	0	0	0	0	23.32548
II	17.82994583	1.83184375	3.6636875	0	0	0	0	23.32548
III	17.82994583	1.83184375	3.6636875	2.2477	0	0	0	25.57318
IV	17.82994583	1.83184375	3.6636875	0	0	0	0	23.32548
V	17.82994583	1.83184375	3.6636875	0.6422	0.475	0	0	24.44268

Eccentricity Check

Service	V	H	Mv	Mh	xo	e	emax	Check	qt	q (Ksf
I	18.23625	0.501875	138.86625	23.32547708	6.335775	0.835775	1.833333	Pass	0.066424	0.03
II	17.89625	0.21125	136.23125	23.32547708	6.308907	0.808907	1.833333	Pass	0.027678	0.01
III	16.74875	0.0625	127.338125	25.57317708	6.075973	0.575973	1.833333	Pass	0.007467	0.00
IV	18.7725	0	138.67875	23.32547708	6.144801	0.644801	1.833333	Pass	0	
V	17.89625	0	136.23125	24.44267708	6.24648	0.74648	1.833333	Pass	0	

Bearing Capacity Assessment

Service	V	H	q	qn	FS	Fsreq	Check
I	18.23625	0.501875	0.165784091	1	6.031942	3	Pass
II	17.89625	0.21125	0.162693182	1	6.146539	3	Pass
III	16.74875	0.0625	0.152261364	1	6.567654	3	Pass
IV	18.7725	0	0.170659091	1	5.859635	3	Pass
V	17.89625	0	0.162693182	1	6.146539	3	Pass

Assume no ground water table?

Vdead	16.74875
Cu	3.014775
Ca	1.5073875

Sliding check

Service	V	H	Vn	FS	Fsreq	Check
I	18.23625	0.501875	140.6305	280.2102117	1.5	Pass
II	17.89625	0.21125	140.6305	665.7065089	1.5	Pass
III	16.74875	0.0625	140.6305	2250.088	1.5	Pass
IV	18.7725	0	140.6305	#DIV/0!	1.5	#DIV/0!
V	17.89625	0	140.6305	#DIV/0!	1.5	#DIV/0!

Vn=Af*Ca+0.5tf*L
Af=B*Wabut
ca= 0.5Cu
Cu = 0.18 Vdead (Assume bas

Settlement

B' = B/2	5.5	μs	0.35
α	4	Es	290.076
N = 5*B/B'	10	2~20	
M = L/B	0.909090909		
I1	0.5455		

I2	0.0195	
Is= I1+((1-2mu)/(1-mu))*I2		0.5545
B/L	1.1	
Df/B	1.106060606	
If	0.694	
q = max	0.170659091	
Se	0.004370674 ft	
	0.052448093 in	

Abutment Rebar detail:

$f'c := 3000\ psi$ $\quad \gamma := 120\ pcf$ $\quad b := B2 = 36\ in$

$fy := 6000\ psi$ $\quad \phi' := 29°$ $\quad \mu s := 0.35$

$Astem := B2 = 36\ in$

$Try\#7bar$ $\quad db := 0.875\ in$ $\quad @12in$ $\quad \beta 1 := 0.85$

$cover := 2\ in$ $\quad Asb := 0.6\ in^2$

reinforcement in Stem bars:

$$MU := Pe \cdot \frac{h}{3} + Hl \cdot h \cdot 0.5 + Hd \cdot h \cdot 0.5 = 6.155\ \frac{kip \cdot ft}{ft}$$

$$Mu := MU \cdot ft - 0.8075\ kip \cdot ft = 5.348\ kip \cdot ft$$

$$ds := Astem - cover - \frac{db}{2} = 33.563\ in$$

$$c := \frac{Asb \cdot fy}{0.85\ f'c \cdot \beta 1 \cdot b} = 0.046\ in$$

$$\frac{c}{ds} = 0.001$$

$$a := \beta 1 \cdot c = 0.039\ in$$

$$\varepsilon t := 0.003 \cdot \left(\frac{ds}{c} - 1\right) = 2.179 \qquad tension$$

$$\phi Mn := 0.9 \cdot Asb \cdot fy \cdot \left(ds - \frac{a}{2}\right) = 9.057\ kip \cdot ft \quad > \quad Mu = 5.348\ kip \cdot ft \qquad OK$$

reinforcement in Stem "O" bars:

$d := 0.5 \cdot B1 = 30\ in$ $\quad t := 0.5 \cdot B2 = 18\ in$

$b := B1 = 60\ in$

$$Vu := 1.7 \cdot Pe = 2.528\ \frac{kip}{ft}$$

$$Mu := \left(Vu \cdot \frac{h}{\ }\right) \cdot ft = 7.304\ ft \cdot kip$$

$ds = 33.563\ in$

$C1 := 1.7 \cdot f'c \cdot b \cdot \frac{d}{2\ fy} = 765\ in^2$

$C2 := 6.8 \cdot f'c \cdot b \cdot \frac{Mu}{4 \cdot 0.9\ fy^2} = 827.765\ in^4$

$As := C1 - \left(C1^2 - C2\right)^{0.5} = 0.541\ in^2$

"O" bars # 6 @ 9"

$0.44 \cdot \frac{12}{9} = 0.587$ $>$ $As = 0.541\ in^2$ *OK*

$\rho bal := \frac{0.85\ \beta1 \cdot 3}{60} \cdot \frac{87}{87+60} = 0.021$

$\rho max := 0.75\ \rho bal = 0.016$

$\rho act := \frac{As}{b \cdot ds} = 2.688 \cdot 10^{-4}$ $<$ $\rho max = 0.016$ *OK*

Shear check:

$P := Pe \cdot \frac{(h-ds)^2}{h^2} = 682.198\ \frac{lbf}{ft}$

$Vu := 1.7 \cdot P \cdot ft = \left(1.16 \cdot 10^3\right)\ lbf$

$\phi V := 0.85 \cdot 2 \cdot (3000)^{0.5} \cdot 12 \cdot 21.563 = 2.409 \cdot 10^4$ $>$ $Vu = \left(1.16 \cdot 10^3\right)\ lbf$ *OK*

reinforcement in "Lw" bars:

$Asmin := 0.002 \cdot 12\ in \cdot \frac{B2}{2} = 0.432\ in^2$

For #5bar

$As\#5 := 0.31\ in^2$ $>$ $Asmin = 0.432\ in^2$ *OK*

reinforcement in "Lb" bars:

$Asmin := 0.0018 \cdot 12\ in \cdot B = 2.851\ in^2$

For #5bar

$As\#5 \cdot 11 = 3.41\ in^2$ $>$ $Asmin = 2.851\ in^2$ *OK*

Appendix B Bridge Calculations

AASHTO LRFD Guide Specification Pedestrian Bridge Design Example

Half-Through Truss Bridge with Tubular Members

ILLUSTRATIVE EXAMPLE OF KEY PROVISIONS OF GUIDE SPECIFICATIONS

Load and Resistance Factor Design

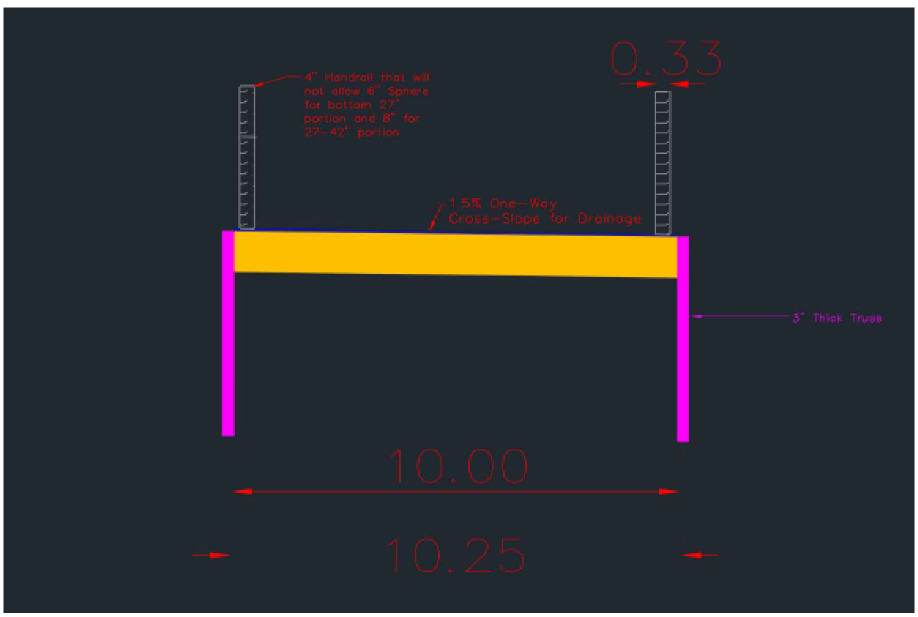
GENERAL INFORMATION

Specifications Used:

- *AASHTO LRFD Bridge Design Specifications, 2008 (AASHTO LRFD)*
- *AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals, 2008 (AASHTO Signs)*
- *LRFD Guide Specifications for Pedestrian Bridges (Specification)*

Geometry:

Span =	152	ft.	
Deck width, w_{deck} =	10	ft.	
CL-CL trusses =	10.25	ft.	
A500, Gr. B, F_y =	46	ksi	(Tubing)
A992, Gr. 50, Fy =	50	ksi	(W-Shape Floor Beams)



TRUSS MEMBERS: Structural Tubing & Floor Beams

Top and Bottom Chords:

Section: 6 x 3 x 5/16" structural tubing

A =	4.68	in^2	L =	8.00 ft
w =	16.96	plf		

Vertical Posts

Section: 5 x 3 x 5/16" structural tubing					
$A =$	4.1	in^2	$L =$	4.00 ft	
$w =$	14.83	plf			
$I_x = I_c =$	12.6	in^4			

Diagonals:

Section: 4 x 3 x 1/4" structural tubing					
$A =$	3.09	in^2	$L =$	5.66 ft	
$w =$	10.51	plf			
				8.94 ft	(Out-Crops)

Sway Bracing

Section: 4 x 3 x 1/4" structural tubing				
$A =$	3.09	in^2	$L =$	10.77 ft
$w =$	10.51	plf		

FLOORBEAMS:

Section:	W12x16			
$I_x = I_b =$	103	in^4	$L =$	10.00 ft
$S_x =$	17.1	in^3		

Spacing = 4 ft. at each panel point

DEAD LOAD:

Weight of each truss =	127	plf per truss	(Done separately based on 152' Truss)
Deck loading =	4.5	psf	(From McNichols Type 304, 58% Open Area)

Weight of deck = 5 psf x 5.00
25 plf

Total dead load per truss = 127 plf + 25 plf
= 152 plf Use 160 plf

PEDESTRIAN LIVE LOAD: (Specification, Article 3.1)

MAIN MEMBERS: Trusses

- The deck area may be used to compute design pedestrian live load for all main member components (truss members). The deck area is the non-zero influence surface for all such components.

- Use 85 psf without impact.

Live load per truss = pedestrian loading x deck width / 2
= 85 psf x 5.0
= 425 plf

SECONDARY MEMBERS: Deck, Stringers, Floorbeams
- Use 85 psf without impact.

VEHICLE LOAD: (Specification, Article 3.2)

- Vehicular access is prevented by steel bollards locked in the fixed position above concrete pad at the edge of the steel bridge, therefore, the pedestrian bridge should not be designed for an occassional single maintenance vehicle load.

- Use Table 3.2-1 for Minimum Axle Loads and Spacings if needed.

Moving Load was neglected. Specifics were discussed with client and H-5 was not needed for the design of the Pedestrian Bridge

WIND LOAD: (Specification, Article 3.4)
- Assume 100 mph design wind.

- Use wind load as specified in the *AASHTO Signs* , Articles 3.8 and 3.9.

- The design life shall be taken as 50 years for the purpose of calculating the wind loading.

Horizontal Wind Loading
- Apply the design horizontal wind pressure on the truss components.

P_z = design wind pressure on superstructure using AASHTO Signs, Eq. 3-1 or Table 3-7, psf
= $0.00256K_zGV^2I_rC_d$ (AASHTO Signs, Eq. 3-1)

where:

K_z = height and exposure factor from AASHTO Signs, Eq. C3-1 or Table 3-5
= 1.00 (conservatively taken from Table 3-5 for a height of 32.8 ft.)

G = gust effect factor
= 1.14 (minimum)

V = basic wind velocity
= 100 mph

I_r = wind importance factor from Specification, Article 3.4
= 1.15

C_d = wind drag coefficient from AASHTO Signs, Table 3-6
= 2.00

P_z = 67.1 psf (Alternatively, AASHTO Signs, Table 3-7 may be used with a C_d value of 2.0 applied)

Projected vertical area per linear foot:

Chords:	2 @ 3 in./ 12 x 8 ft. long / 4 ft.	1.00	SF/ft.
Verticals:	3 in./ 12 x 4 ft. long / 4 ft.	0.25	SF/ft.
Diagonals:	3 in./12 x 5.66 ft. long / 4 ft.	0.35	SF/ft.
Total per Truss:		1.60	SF/ft.
Deck + Floor Members:	0.625"/12 + 12" / 12	1.05	SF/ft.

WS_H = total horizontal wind on superstructure, plf
= (2 trusses x 1.60 SF/ft. + 1.05 SF/ft.) x 67.1 psf
= 286 plf

Note: The full lateral wind loads must be resisted by the entire superstructure. Appropriate portions of the design wind loads must also be distributed to the truss top chord for design lateral forces on the truss verticals.

Vertical Wind Loading

- Apply a vertical pressure of 0.020 ksf over the full deck width concurrently with the horizontal loading. This loading shall be applied at the windward quarter point of the deck width.

WS_V = vertical wind load on the full projected area of the superstructure applied at the windward quarter point, plf

$= P_v * w_{deck}$

where:

P_v = vertical wind loading on superstructure, ksf

= 0.020 ksf

w_{deck} = total deck width, ft.

= 10.0 ft.

Therefore,

WS_V = 0.020 ksf x 1000 x 10.00 ft.

= 200 plf

Vertical load on leeward truss = 200 plf x (7.5 ft. + (0.5 in. + 2.5 in.) / 12) / 10.25 ft.

= 147.6 plf

Vertical load on windward truss = 200 plf x (2.5 ft. + (0.5 in. + 2.5 in.) / 12) / 10.25 ft.

= 52.4 plf (uplift)

TOTAL VERTICAL LOADS PER TRUSS: (Specification, Article 3.7)

DEAD LOAD (DC1+DC2):	160	plf
LIVE LOAD (Pedestrian, PL):	425	plf
WIND (Overturning, WS):	148	plf

Load Factors (AASHTO LRFD Table 3.4.1-1)

Limit State	DC1 & DC2	PL	WS
Str I	1.25	1.75	0
Str III	1.25	0	1.40
Ser I	1.00	1.00	0.30

STRENGTH I LIMIT STATE ($\gamma_{DC1+DC2}$*(DC1+DC2) + γ_{PL}*PL)

= 944 plf

STRENGTH III LIMIT STATE ($\gamma_{DC1+DC2}$*(DC1+DC2) + γ_{WS}*WS_V)

= 407 plf

SERVICE I LIMIT STATE ($\gamma_{DC1+DC2}$*(DC1+DC2) + γ_{PL}*PL + γ_{WS}*WS_V)

= 629 plf

TRUSS MEMBER DESIGN LOADS:

Panel point load from controlling load comb. = 0.944 klf x 4.0 ft. panel = 3.78 k/panel

Maximum Truss Member Axial Loads (from separate truss analysis on Autodesk Robot & Hand Calculations to verify):

Diagonal Bar 7	3.56	k	(compression)
Vertical Bar 42	3.85	k	(compression)
Bottom Chord Bar 455	2.61	k	(tension)
Top Chord Bar	1.27	k	(tension)

TRUSS TOP CHORD LATERAL SUPPORT: (Specification, Article 7.1)

- Assume the truss verticals are adequate to resist the lateral force per Specification, Article 7.1.1 (Must verify assumption; see section titled "LATERAL FORCE TO BE RESISTED BY VERTICALS")
- Lateral support is provided by a transverse U-frame consisting of the floorbeam and verticals.

Determine the design effective length factor, K, for the individual top chord members supported between the truss verticals using Specification, Table 7.1.2-1.

Compute CL/P_c for use in the Table.

where:

$C = P/\Delta$
$= 2.917$ k/in. (from a separate 2D analysis)

L = unbraced length of the chord in compression (i.e. length of two panel points points), in.
$= 96$ in.

P_c = desired critical buckling load (i.e. factored compressive force) multiplied by 1.33, k (Specification, Article 7.1.2)
$= 5.1205$ k

$CL/P_c = 54.69$

n = number of panels
$= 38$

Therefore,

$1/K = 0.825$ (Specification, by conservative interpolation of Table 7.1.2-1)
$K = 1.21$

TOP CHORD COMPRESSIVE RESISTANCE: (AASHTO LRFD, Article 6.9.2)

Check the slenderness ratio against the limiting value.

For main members: $KL/r \leq 120$
For bracing members: $KL/r \leq 140$

Section: 6 x 3 x 5/16" Structural Tube

$A =$ 4.1 in^2

r_x = radius of gyration about the x-axis, in.
= 2.07 in.

r_y = radius of gyration about the y-axis, in.
= 1.19 in.

K = 1.21

L = 96 in.

KL/r_x = (1.21 x 96 in.) / 2.07 in.
= 56.2 < 120 OK

KL/r_y = (1.00 x 96 in.) / 1.19 in.
= 80.7 < 120 OK

P_r = factored resistance of components in compression, k
$= \phi_c P_n$ (AASHTO LRFD, Eq. 6.9.2.1-1)

where:

ϕ_c = resistance factor for compressive per AASHTO LRFD, Article 6.5.4.2
= 0.9

P_n = nominal compressive resistance per AASHTO LRFD, Article 6.9.4, k

Determine the nominal compressive resistance, P_n

If $\lambda \leq 2.25$, then:

$$P_n = 0.66^{\lambda} F_y A_s$$ (AASHTO LRFD, Eq. 6.9.4.1-1)

If $\lambda > 2.25$, then:

$$P_n = \frac{0.88 F_y A_s}{\lambda}$$ (AASHTO LRFD, Eq. 6.9.4.1-2)

$$\lambda = \left(\frac{KL}{r_s \pi}\right)^2 \frac{F_y}{E}$$ (AASHTO LRFD, Eq. 6.9.4.1-3)

$= 1.05$

where:

A_s = gross cross-sectional area, in^2
= 4.1 in^2

F_y = specified minimum yield strength, ksi
= 46 ksi

E = modulus of elasticity, ksi
= 29,000 ksi

KL/r_s = Maximum of KL/r_x, KL/r_y
= 81

Therefore, the top chord factored resistance is:

$P_n = 0.66^{0.1.05}$ x 46 ksi x 4.10 in^2
= 122 k

$\phi_c P_n$ = 109 k > P_{chord} = 3.85 k OK

LATERAL FORCE TO BE RESISTED BY VERTICALS: (Specification, Article 7.1.1)

H_f = minimum lateral force, k
= $0.01/K^*P_{avg}$

where:
K = 1.21

P_{avg} = average design compressive force in adjacent chord members, k
= 3.85 k

Verify limit 0.01 / 1.21 = 0.008 > 0.003 OK

Therefore,
H_f = 0.01 / 1.21 x 3.85 k
= 0.03 k

Apply H_f as the lateral force at the top of the Truss Verticals. Apply H_f concurrently with other primary forces in the Verticals (combined compression plus bending analysis). Include lateral wind forces for AASHTO LRFD Load Combination Strength III.

Length of vertical = 48.0 in.

Lateral Moment in Vertical due to C = 0.03 k x 48.0 in. = 1.52 k-in.

END POSTS: (Specification, Article 7.1.1)

- Apply the lateral force, C, at the top end of post and design as a cantilever combined with axial load. The lateral force, C, is taken as 1.0% of the end post axial load.

#REF! #REF!

Note: All other truss members are analyzed using conventional methods per AASHTO LRFD.

DEFLECTION: (Specification, Article 5)

Maximum pedestrian LL Deflection = 1/500 of the span length = 152.00 ft. x 12 / 500 = 3.65 in.

From Truss Analysis, LL Deflection (w_{LL} = 0.956 k/ft) = 0.03 in. < L/500 OK

VIBRATIONS: (Specification, Article 6)

Vertical Direction

- Estimate the fundamental frequency in the vertical direction, f, by approximating the truss as a simply supported uniform beam:
- The fundamental frequency in a *vertical* mode without consideration of live load should be greater than 3.0 Hz to avoid the first harmonic.

$$f = 0.18*SQRT(g / \Delta_{DL})$$

where:

g = acceleration due to gravity, ft/s^2
= 32.2 ft/s^2

Δ_{DL} = maximum vertical deflection of the truss due to the dead load, ft.
= 0.0009 ft. (from a separate analysis with w = 0.20 klf per truss)

f = 0.18*SQRT(32.2 / 0.0009) = 34.05 Hz > 3.0 Hz minimum desirable, OK

For illustration purposes, assume higher harmonics (second, third, etc.) are a concern. The bridge should be proportioned such that the following criteria is satisfied:

$$f \geq 2.86 \ln (180 / W)$$

where:

W = full weight of the supported structure including dead load and an allowance for actual pedestrian live load, k

= 2 trusses x 0.16 klf x 152.00 ft.
= 48.64 k (Dead Load Only)

2.86 ln (180 / 48.64) = 3.74 Hz

f = 34.05 Hz is greater than 3.74 Hz, no need to include the pedestrian live load contribution.

Assume some pedestrian live load contribution and re-evaluate the expression:

$W = DL + 10\%LL = 48.6 + 0.10 \times 2 \times (0.425 \text{ klf} \times 152.00 \text{ ft.}) =$ 61.56 k

$2.86 \ln(180 / 61.56) =$ 3.07 Hz < f = 34.05 Hz OK

Lateral Direction

- Estimate the fundamental frequency in the lateral direction, f_{lat}, by approximating the truss as a simply supported uniform beam rotated 90 degrees:
- The fundamental frequency in a *lateral* mode without consideration of live load should be greater than 1.3 Hz to avoid the first harmonic.

Assume the lateral wind bracing is 3 x 3 x 1/4" structural tubing.

$f = 0.18 \cdot \mathrm{SQRT}(g / \Delta_{DL\_Lat})$

where:

g = acceleration due to gravity, ft/s^2
= 32.2 ft/s^2

Δ_{DL_Lat} = maximum lateral deflection of the truss due to the dead load, ft.
= 0.0844 ft. (from a separate analysis)

$f = 0.18 \cdot \mathrm{SQRT}(32.2 / 0.0844) =$ 3.52 Hz > 1.3 Hz minimum desirable, OK

FATIGUE: (Specification, Article 3.5)

Use AASHTO Signs, Article 11.7.3
AASHTO Signs, Article 11.7.4 - Not used as it is assumed that the Pedestrian Bridge is not over a highway

$P_{NW} = 5.2\, C_d\, I_F$

C_d = wind drag coefficient per AASHTO Signs, Table 3-6
= 2.00

I_r = wind importance factor per AASHTO Signs, Table 3-2
= 1.15

P_{NW} = 11.96 psf

WS_H = total horizontal wind on superstructure, plf
= (2 trusses x 1.60 SF/ft. + 1.05 SF/ft.) x 12.0 psf
= 51 plf

FATIGUE Cont'd:

Maximum Member Force:
Vertical Bar 42 — 3.85 kips (from a separate Analysis)

Δf = Stress Range
= (3.85 kips / 4.68 in^2)
= 0.82 in^2

$\gamma(\Delta f) \leq (\Delta F)_n$ (AASHTO LRFD Eq. 6.6.1.2.2-1)

where:

γ = 1.0 (Specification, Article 3.7)

Δf = 0.82 ksi

$(\Delta F)_n = (\Delta F)_{TH}$ (Specification, Article 4.1)

where

$(\Delta F)_n$ = 16 ksi (Category B -base metal) (AASHTO Signs, Table 11-3)

(1.0)(1.12) $\leq$ 16

0.82 < 16 OK

Welded Member connections and Fracture Toughness Requirements are outside the limits of this Pedestrian Bridge design example. They will be the responsibility of the designer.

Appendix C Riprap Calculations

Riprap Calculations - MN DNR

step 1	Hazard	high		
step 2	Effective fetch calculations			
	Fe=	887 feet	=	0.16 mile(s)

Note: if effective Fetch Fe<.5 miles then use 0.5 for Fe — use Fe= 0.5 miles

step 3	describe fluctuation of lake level	minimal		
	Still water elevation(s)			1066 ft
step 4	Wind Direction along critical fetch			NW compass point
step 5	first order weather station			Ogden
	wind stress factor (Ua)			16 mph
step 6	wave period (T)	$T=0.599[Ua \times Fe]^{1/3}$	=	1.20 ft
	wave length (L)	$L=5.12T^2$	=	7.35
step 7	Signifigant wave height (Hs)	$Hs=0.0301*Ua*Fe^{0.5}$	=	0.34 ft
step 8	Design factor (DF) (table 2-2)			1.67

High Hazard: Failure of the measure would threaten existence of a valuable structure or property; distance from shore to anything of value is less than 20 feet.

Table 2-2. Design Factor Selection

Hazard	Riprap Runup & WPH ⋆	Riprap Rock Size	Gabions & C. Block ♠
Low	1.27	1.0	1.27
Moderate	1.37	1.27	1.37
High	1.67	1.27	1.67

♠ C. Block is precast concrete block, any style
⋆ WPH is wave protection height

Design wave height (Ho) =Hs*DF		=	0.57	ft		
step 9	slope ratio	3:1	(such as 3:1, 4:1)	Ho/L=	0.08	
	R/Ho (figure 2-2)	1.47				
	Runup (R) = Ho*R/Ho	=	0.835998	ft		
		use	0.8	ft		
	Setup (S) = 0.1*Ho	=	0.056871	ft	(not more than .5 feet)	
		use	0.1	ft		
step 10	lower limit =1.5*Ho	=	0.9	ft		
	upper limit (WHPH) = R + S	=	0.9	ft		
step 11	upper elevation of protection: (upper) SWL + upper limit	=	1066.9	ft		
	lower elevation of protection: (lower) SWL - lower limit	=	1065.1			

RIPRAP DESIGN

step 12	slope ratio	4 : 1	Design factor (DF) (rock size only) (Table 2-2)	1.27	
	Ho=Hs*DF =	0.4	ft		
	(Hs is the same as determined in step 7)				
step 13	Determine W_{50} (use eqn 2-4 and/or 2-5 or select from the chart in appendix C)				
	Determine or estimate the density, W_r	156	lbs/ft^3		

or specific gravity Gs 2.5 of the rock

Describe rock expected: 50 % rounded 50 % angular

$$W_{50} = \frac{w_r \ Ho^3}{K_{rr}\,(S_r - 1)^3 \cot \gamma}$$ (Eq'n 2-5) = 0.55 lbs

γ = angle of structure slope measured from horizontal in degrees

W50= 0.60 lbs D 50= 2.2 inches

(use table C-4 or C-5 to convert to equivilent size, or use eqn 2-6)

$d = 1.15\,(W/w_r)^{1/3}$ (Eq'n 2-6)

d= 0.18018 ft 2.162161 in

Use D50 = 2.2 in

step 14 Gradation calculated for this location

D100	2.0*D50=	4.4 in	2.5*D50=	5.5 in
D85	1.6*D50=	3.5 in	2.1*D50=	4.6 in
D50	1.0*D50=	2.2 in	1.5*D50=	3.3 in
D15	0.3*D50=	0.7 in	0.5*D50=	1.1 in

For rock that is partially angular and partially rounded, a combination of K factors may be used. For example, with 2 layers of rock under breaking wave conditions, rock that is considered to be 30% angular and 70% rounded has a K of $0.3(K_{rr}) + 0.7(K_D) = 0.3(2.2)+0.7(1.2) = 1.5$.
The tables in Appendix C or equation 2-6 should be used to convert W_{50} to d_{50}, being certain to use the correct specific gravity for the rock that will be installed. Over much of Minnesota, a specific gravity of 2.50 is reasonable; in northeastern Minnesota, often rock is used with a specific gravity of 2.65.

$d = 1.15\,(W/w_r)^{1/3}$ (Eq'n 2-6)

where, d = equivalent stone dimension in feet and the other parameters are the same as defined step 15 Thickness of riprap = 1.25*maximum D100

D100= 6.9 in

Use minimum of 12 in

Step 16 Ocertopping protection

step a) Elevation of top of bank (determined in field) 1068 ft
step b) Upper elevation of protection (calculated in step 11) 1066.9 ft

step c) If step b is higher than step a, an overtopping apron is required.
[(step b)-(step a)]*3=width of apron shoreward (must be > 1.5 ft)

not needed

Width of overtoping appron (Wo)= N/A ft not less than 5 ft

Use Wo= N/A ft

Special considerations related to the OHW elevation:

Protection is nearly to OHW; vegitate to top of bank or to 100 yr flood elevation of 1070.5

Step 17 End Protection. Choose method A or B from figure 2-4
Method A X to secure end points
Method B

Step 18 Toe protection: (figures 2-5 and 2-6)
Follow steps a through f for an La or Lc toe; use step g for an Lb toe. Use step h for a type d toe.

step a.	1.25*D50(riprap)=	2.75	in
step b.	Elevation of existing lake bottom near shore =	1065.4	ft
step c.	Lower elevation of protection (computed in step 11) =	1065.1	
step d.	[(step b)-(step a)]*3=	0.8	ft
step e.	Determine whether step a or step d results in a larger value. Write it here	2.75	in

step f. The value in step e must no be less than 3 feet (if it is use 3.0 feet) nor larger than 6 feet (if it is use 6 feet). This value is the length La or Lc as depicted in in Figure 2-5 and 2-6

La or Lc= 3 feet — go to step 19

step g. Lb= 8*D50= N/A in N/A ft

step h. Ld= the shorter value of 1) 6 feet (more at engineers discretion) or 2) the lower elevation of protection calculated in step 11

N/A ft

Notes: Reason to use type A toe

A type a toe (with either a geotextile or a granular bedding/filter) is meant for lakeshores with shallow water and a flatter lakebed slope. A trench is cut in the bottom to install the toe. The type a toe is preferred for sites where ice action is known to have taken place. It encourages ice to ride up and over the riprap, especially if the slope of the riprap is flatter than 5:1. The ice does not have a protruding riprap toe to push as illustrated in Figure 1-10.

Based on experience, the critical length, La, for this type of toe should be between 3 and 6 feet. The length needed is based on a comparison between what is needed for the rock size vs. the anticipated scour. For rock size, the toe length is estimated by 15 x d50. For scour protection, the length is calculated by subtracting the lower limit elevation calculated in step 10 from the lake bottom elevation near shore, and multiplying that result by 3. Figures 2-5 and 2-6 illustrate the toe layout.

Step 19 Filter or bedding requirments: select one

Use geotextile	X
Use granular filter or bedding	

Granular Filter Design: 1 inch = 25.4 mm

d15 (bedding) (min.) > d15 (riprap)/40 (max.) > 0.42 mm (No. 40 sieve)

	Minimum	Maximum
d100		
d85		
d50		
d15		

d15 (bedding) (max.) < d15 (riprap)/4 (min.)

d85 (bedding) (min.) > d15 (riprap)/4 (max.)

d50 (bedding) (min.) > d50 (riprap)/40 (max.)

Geotextile:

Woven	
Non-woven	X

Per Iowa DOT standards use Non-woven US 160NW.
Specification 4196.01-3 Embankment Erosion Control.

https://www.usfabricsinc.com/specifications/dot/iowa https://windexchange.energy.gov/maps-data/32

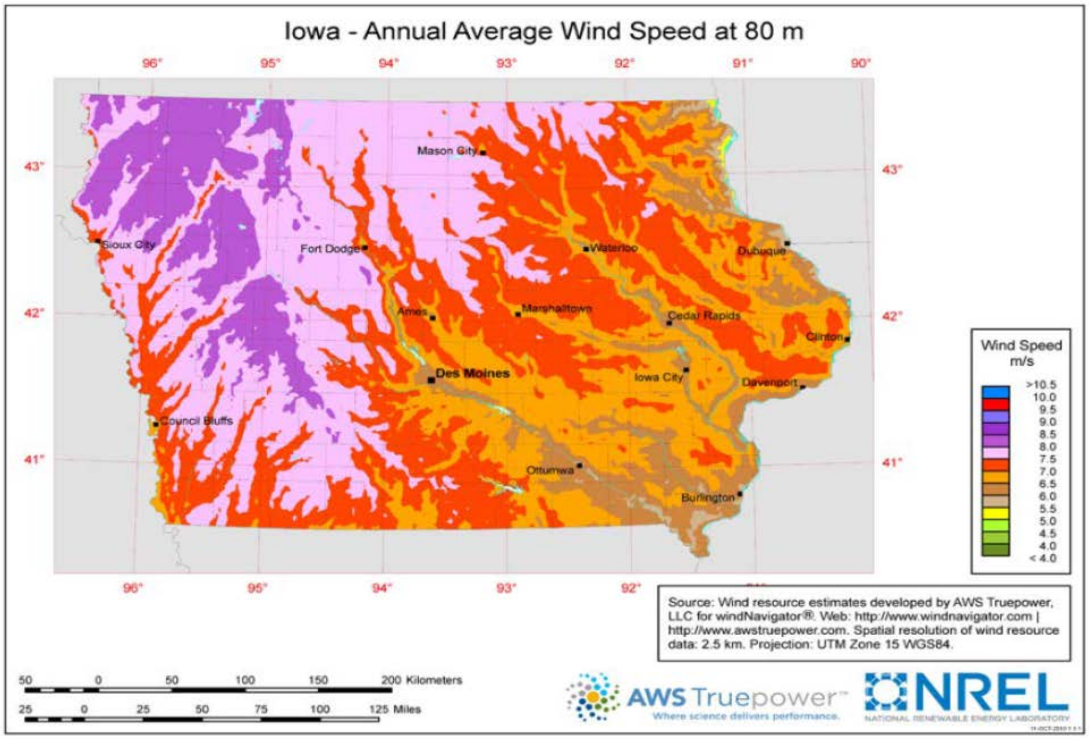


Figure 2-2. Wave Runup Ratio (from Reference #17)

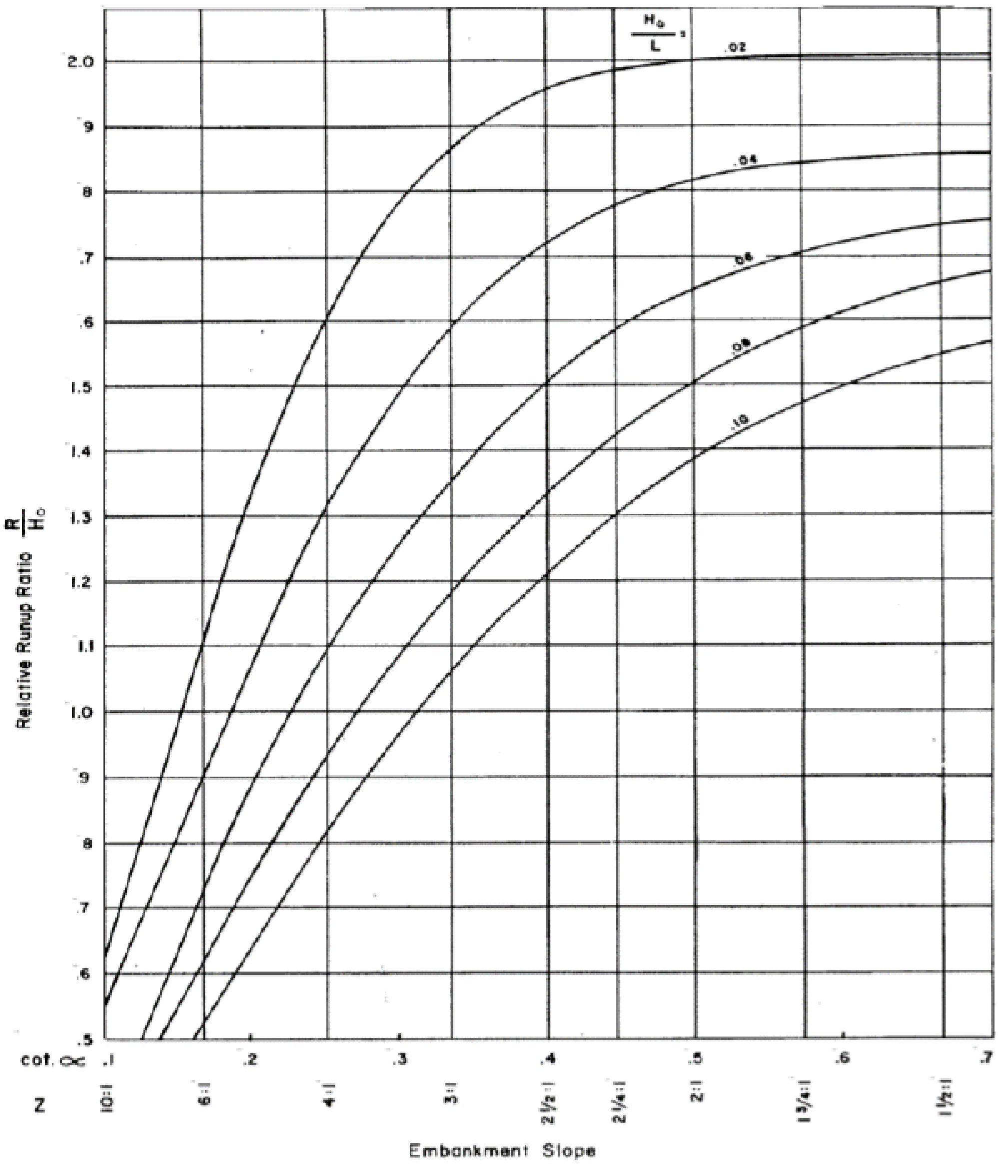


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Table 2-4. Suggested K_D or K_{rr} Values for Use in Determining Armor Unit Weight
Non-Damage Criteria and Minor Overtopping

Armor Units	**Number of Units in Layer**	**Placement**	**K_D or K_{rr} Value Breaking Wave**	**K_D or K_{rr} Value Nonbreaking Wave**
Quarrystone (K_D)				
Smooth, rounded	2	Random	1.2	2.4
Smooth, rounded	>3	Random	1.6	3.2
Rough angular	1	Random	not recommended	2.9
Quarrystone (K_{rr})				
Rough Angular	any	Random	2.2	2.5
(graded)				
Minimal toe**	any	Random	3.5	4.0

Note: The K_D values for smooth, rounded quarrystone for breaking waves are unsupported by test results but were estimated by the authors of the Corps' *Shore Protection Manual*, 1984.
** Meant to be used when a minimal riprap toe is installed in combination with bioengineering techniques.

Table C-4. Equivalent Stone Dimension for a known Stone Weight

Gs = 2.5							
Weight	**Size**	**Weight**	**Size**	**Weight**	**Size**	**Weight**	**Size**
0.5	2.04	**25**	7.50	**90**	11.49	**250**	16.15
1	2.56	**30**	7.97	**95**	11.70	**300**	17.16
2	3.23	**35**	8.39	**100**	11.90	**350**	18.07
3	3.70	**40**	8.77	**110**	12.28	**400**	18.89
4	4.07	**45**	9.12	**120**	12.64	**500**	20.35
5	4.38	**50**	9.44	**130**	12.99	**600**	21.62
6	4.66	**55**	9.75	**140**	13.31	**700**	22.76
7	4.90	**60**	10.04	**150**	13.62	**800**	23.80
8	5.13	**65**	10.31	**160**	13.92	**900**	24.75
9	5.33	**70**	10.57	**170**	14.20	**1000**	25.63
10	5.52	**75**	10.81	**180**	14.47		
15	6.32	**80**	11.05	**190**	14.74		
20	6.96	**85**	11.27	**200**	14.99		

Gs = 2.50
Weight n pounds
Size in inches

from Chap.7 of Corps' Shore Protection Manual

Table C-5. Equivalent Stone Dimension for a known Stone Weight

Gs = 2.65							
Weight	**Size**	**Weight**	**Size**	**Weight**	**Size**	**Weight**	**Size**
0.5	2.00	**25**	7.36	**90**	11.28	**250**	15.85
1	2.52	**30**	7.82	**95**	11.48	**300**	16.84
2	3.17	**35**	8.23	**100**	11.68	**350**	17.73
3	3.63	**40**	8.61	**110**	12.06	**400**	18.54
4	3.99	**45**	8.95	**120**	12.41	**500**	19.97
5	4.30	**50**	9.27	**130**	12.75	**600**	21.22
6	4.57	**55**	9.57	**140**	13.06	**700**	22.34
7	4.81	**60**	9.85	**150**	13.37	**800**	23.36
8	5.03	**65**	10.12	**160**	13.66	**900**	24.29
9	5.23	**70**	10.37	**170**	13.94	**1000**	25.16
10	5.42	**75**	10.61	**180**	14.21		
15	6.21	**80**	10.84	**190**	14.46		
20	6.83	**85**	11.06	**200**	14.71		

Gs = 2.65
Weight in pounds
Size in inches

from Chap.7 of Corps' Shore Protection Manual

Figure 2-5. Three toe types with geotextile

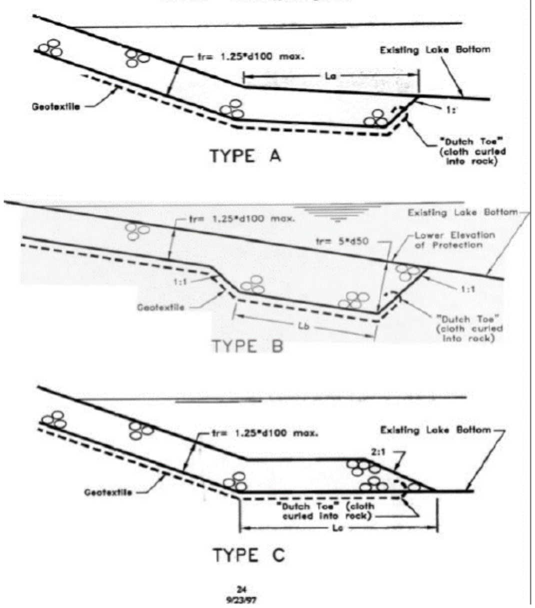

Figure 2-6. Three toe types with granular filter