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Right Hand Fork Pedestrian Bridge Final Report

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RIGHT HAND FORK PEDESTRIAN BRIDGE FINAL REPORT

by

Ren Gibbons

Thesis submitted in partial fulfillment
of the requirements for the degree

of Bachelor's of Science

HONORS IN UNIVERSITY STUDIES
WITH DEPARTMENTAL HONORS

in

Civil Engineering
in the Department of Civil and Environmental Engineering

Approved:

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UTAH STATE UNIVERSITY
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RIGHT HAND FORK PEDESTRIAN BRIDGE FINAL REPORT

Prepared by

Stream Spanners

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For the Logan Ranger District
Uinta-Wasatch-Cache National Forests

A report submitted to fulfill requirements
for the CEE 4880 Design III course.

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Team Member Roles

This table lists each team member in alphabetical order and their responsibility or function on the team. Function and specialties include discipline, CEE specialty, and function such as team leader, scheduler, records keeper, etc. This is used to demonstrate the multidisciplinary nature of the project.

Function or specialty on team	Last name	First name
Environmental Coordinator/Hydrological Expert	Bassett	Stetson
CAD Expert/Hydraulic Expert	Curtis	Kedric
Record Keeper/Hydraulic Expert	Fisher	Dexter
Team Leader/Structural Expert	Gibbons	Ren
Cost Evaluator/Gantt Chart Expert/Structural Evaluator	Lundgreen	Jeff

EXECUTIVE SUMMARY

Right Hand Fork is a tributary canyon whose stream enters the Logan River nine miles east of the mouth of Logan Canyon near Logan, Utah. The access bridge to a popular recreational area, the Hobbit Caves, was damaged in 2011 by flooding. We propose that a new bridge be designed and constructed for the Forest Service to replace the current structure to ensure public safety and minimize environmental impact. We plan to use a longer bridge span that will situate the abutments higher on the stream bank and reduce possible water damage during flood events. Bridge costs will be minimized and will meet required construction and aesthetic standards required by the Forest Service.

The current bridge is a 15-foot timber span placed on existing earth footings. High runoff discharges in 2011 caused the northern earth abutment to shear and settle due to scour. The bridge span now sags at approximately 20°. While still usable, the bridge's structural integrity has been compromised, and the footing will presumably continue to settle until the bridge fails. Figure 1 shows the extent of the damage done to the bridge.

Figure 1. The condition of the existing bridge.

We have completed a topographical survey and hydrological report to determine the best location for the structure to prevent future water damage. We performed structural analysis and design for a safe and cost-effective replacement pedestrian bridge. We independently prepared a steel bridge and a timber bridge design. The preferred alternative for structural materials is a 40-ft timber glulam girder span and Gabion foundations. We propose that construction will begin as early as fall 2015. We also propose biannual inspections beginning once construction is complete to ensure proper maintenance and a 50-year life as required by the Forest Service.

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PROBLEM STATEMENT

Right Hand Fork, a tributary canyon of Logan Canyon in the Bear River Range, is home to a recreational area called the Hobbit Caves. Figure 2 shows the location of Right Hand Fork with respect to Logan, Utah. The Hobbit Caves attract recreationalists for rock climbing, camping, and bon fires. In 2011, high runoff discharges damaged the pedestrian bridge that connects the road to the recreational site. The structural integrity of the bridge has been compromised (twisted and sagging at 20°) caused by differential movement of the abutments due to scour. To protect the aquatic life in the Right Hand Fork stream and provide easy access for the public, a new pedestrian bridge will be proposed for construction beginning in as soon as late summer 2015.

Figure 2. Right Hand Fork Location (courtesy of Google Maps).

Figure 3. Current Bridge location (courtesy of Google Maps).

PROJECT DESCRIPTION

OBJECTIVES

- Complete a topographic survey of Right Hand Fork near the Hobbit Caves recreation area.
- Perform a hydrological study of the Right Hand Fork stream.
- Select a reasonable location for a new pedestrian bridge.
- Determine the most suitable and economic material.
- Perform structural analysis using RISA-2D.
- Complete bridge design using LRFD
    - Steel alternative designed according to the American Institute of Steel Construction Manual
    - Wood alternative designed according to National Design Standard by the American Wood Council
- Produce bridge drawings and specifications using AutoCAD.
- Collaborate with the Forest Service to ensure the bridge will not disrupt the local ecosystem (unable to complete due to Forest Service noncompliance with the Stream Spanners Group).
- Conform to all Forest Service design and construction standards.
- Attend National Environmental Policy Act (NEPA) meetings for project approval (unable to complete).

METHODS

Hydrology and Hydraulics

The first step of the design process was to complete a topographic survey of the existing site. We borrowed a Topcon total station from Utah State University's Department of Civil and Environmental Engineering. We performed two surveys to collect cross section data for the stream and the location of the existing bridge. We collected five soil samples from the surrounding area to get an estimate of the soil type in Right Hand Fork (See Table 7 in Appendix II-A). Using the "Feel Method" (Appendix II-A) we were able to determine the possible soil type for the five samples.

The next step was to conduct a hydrological study of Right Hand Fork to determine the peak runoff for a design storm. In order to determine the peak runoff we needed to gather some watershed characteristic data and determine a design storm. We used the USGS Streamstats website to delineate the watershed for Right Hand Fork and determine the area, average slope, and catchment length (See Table 1). Using ArcGIS we determined the land cover type percentages for the entire watershed (See Table. 5 in Appendix II-A). Using this information and Table 9-2 from Part 630 in the National Engineering Handbook, we made assumptions for the SCS curve numbers for the watershed to describe the precipitation losses during a design storm. We used a weighted curve number to calculate some parameters for a Clark unit hydrograph such as potential maximum retention, lag time, and time of concentration. Table 2 shows the different possible weighted curve numbers and the associated parameters for the Clark unit hydrograph.

Table 1. Basin Characteristics

Basin Characteristics				
Area	25.2	mi^2		
Average Slope	34.2	%		
Catchment Length	8	mi	42240	ft

Table 2. Watershed Variables Used in HEC-HMS

Watershed Variables				
Possible Curve Numbers:	56	75	66	61
Potential Max Retention,S (hr):	7.86	3.33	5.15	6.39
Time Lag tL (hr):	2.08	1.26	1.61	1.83
Time of Concentration tc (hr):	3.47	2.10	2.69	3.05

With all this information, a model was created in HEC-HMS to simulate the peak runoff of the watershed for a design storm. We used precipitation frequency data from the National Weather Service's website to simulate a 100-year 12-hour frequency storm (See Appendix II-A). Assuming a weighted curve number of 66, the resulting peak flow was approximately 710 cfs (See Figure 4).

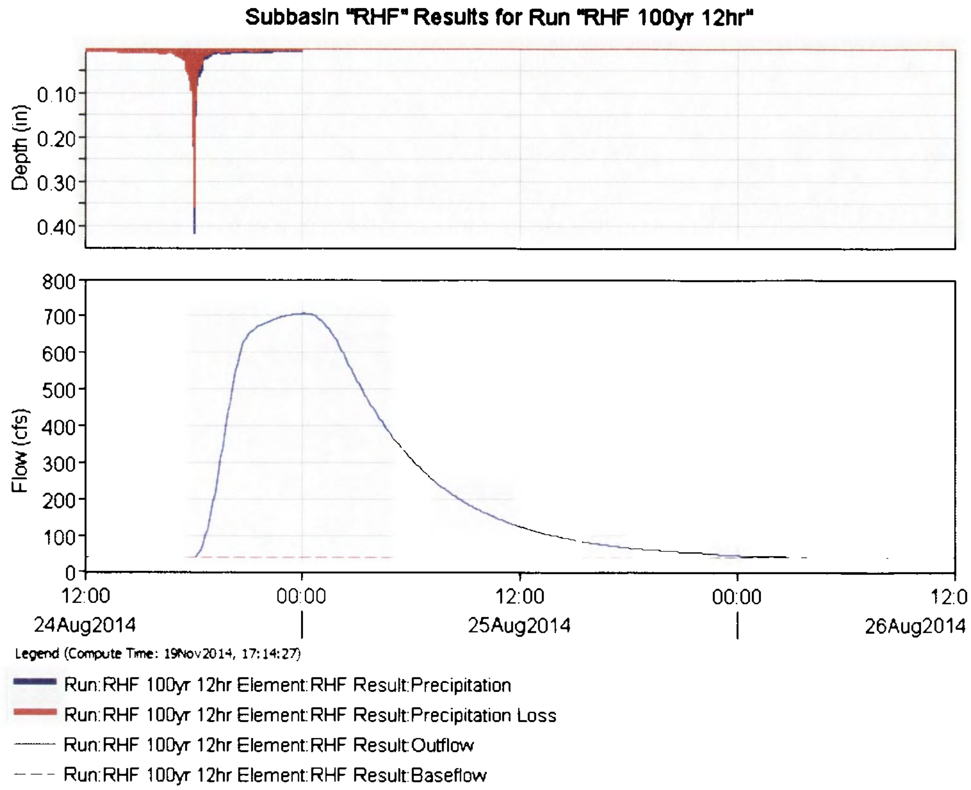


Figure 4. Precipitation and peak flow for 100-yr 12-hr storm.

We next determined the flood plain resulting from a peak flow of 710 cfs. We created a model in HEC-RAS to simulate the peak flow in the streambed. For this model, we ran a steady state simulation. An unsteady state simulation may be more accurate for this project, but due to the lack of experience this was not feasible. The resulting flood plain can be seen in Figure 5.

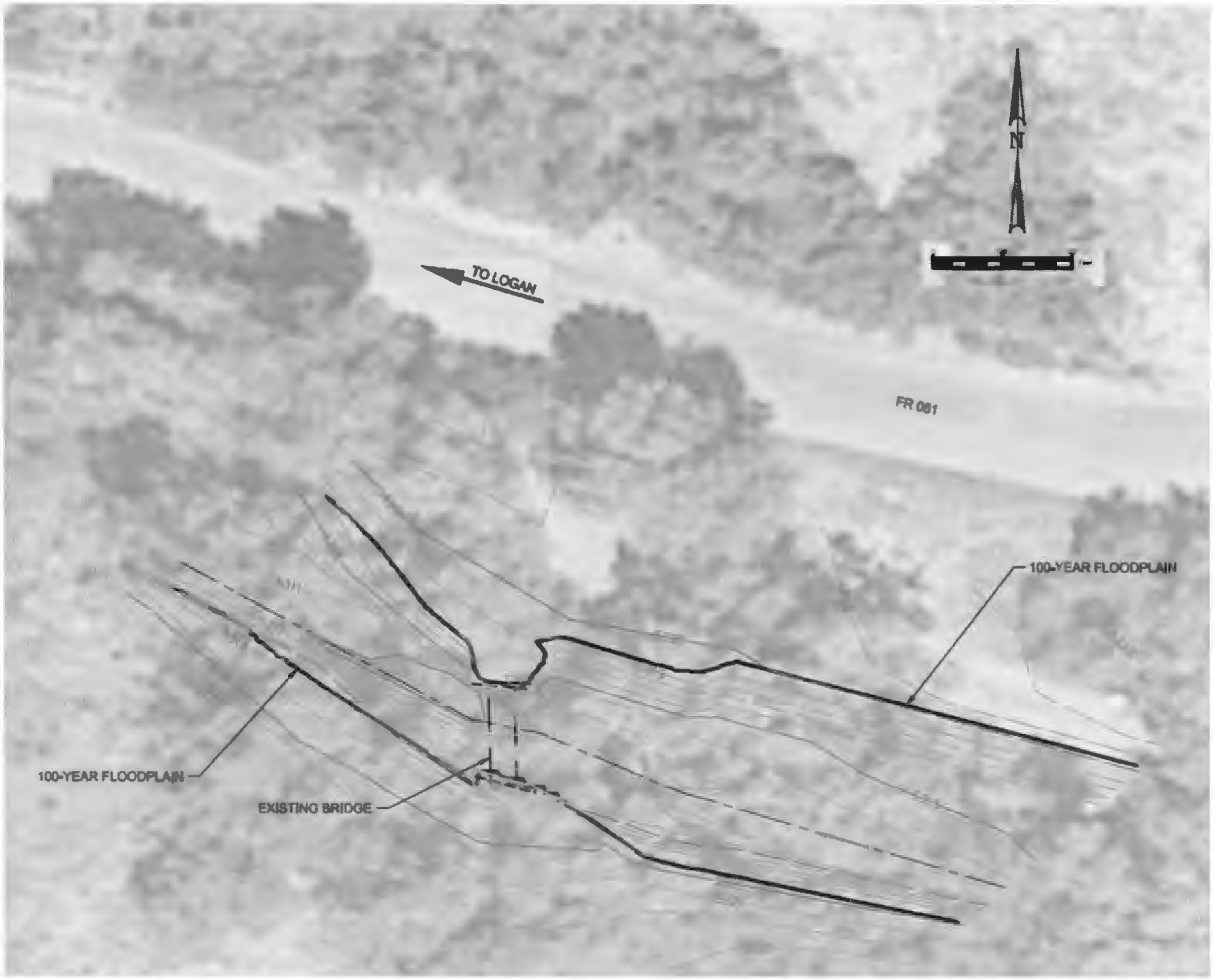


Figure 5. 100-year flood plain.

To establish the accuracy of our peak flow estimate, we took three discharge measurements at Logan River and Right Hand Fork. A discharge cross section was selected on each river based on channel topography, overall water depth, and uniformity of flow. After the two locations were identified rebar was placed on either side of the bank so that a tape measure could be stretched across the river to ensure flow measurements would be captured perpendicular to streamflow. Approximately 10 to 20 verticals were identified within each river channel where velocity and depth measurements were collected. Velocities for the discharge measurements were taken with a Marsh-McBirney velocity meter set at a 25-second averaging interval. The velocity meter was used in conjunction with a top setting rod to help determine the total depth of each vertical within the stream channel.

Discharge measurements were taken during three consecutive months to capture varying flows. After each data collection period the discharge measurements from the Logan River and Right Hand Fork were compared, as shown in Table 3. Flows from the

Logan River were also compared to the USGS gaging station (10109000) located on the Logan River to check accuracy. We found that Right Hand Fork was approximately 9.7% of the flow of Logan River (Table 3). This was well within our estimated peak flow of 710 cfs.

Table 3. Flow Measurements from Logan River and Right Hand Fork.

Date	Logan River Discharge cfs	USGS Gage Station Discharge cfs	Right Hand Fork Discharge cfs	% of Logan River
1/16/2015	65.7	92.0	8.08	12.3%
2/5/2015	135.623	113.000	12.14	9.0%
3/26/2015	119.377	115.000	9.37	7.8%
			Average:	9.7%

The best location for the new bridge was determined to be approximately 25 feet upstream from the current bridge location. This will ensure that the bridge will be above the 100-year flood plain and will minimize added environmental impacts. Figure 6 on the following page shows the position of the proposed bridge.

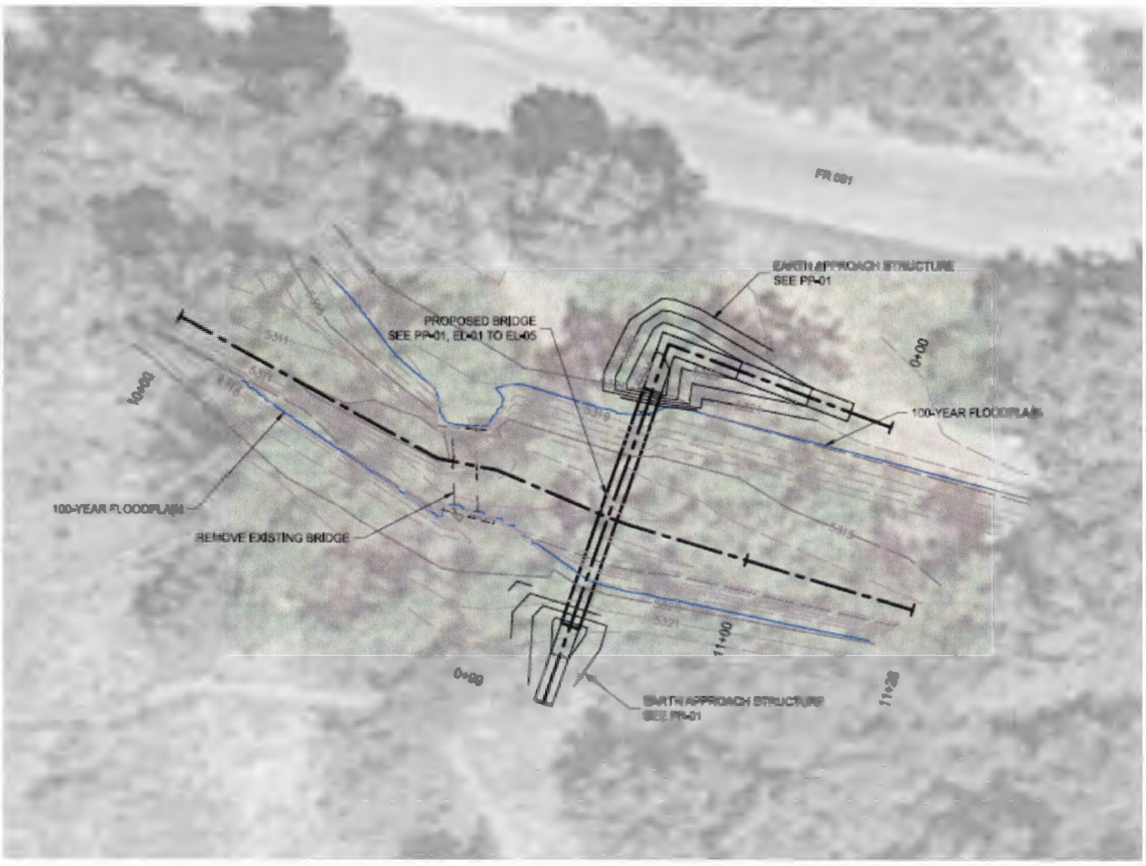


Figure 6: Location of the proposed bridge.

Structural Analysis

Before beginning structural design, we performed preliminary structural analysis under worst case scenarios to determine maximum moment, maximum shear, and reactions on the structure. We used a 40-ft span, which is required at the chosen bridge location to maintain three feet of clearance over the stream surface elevation during the 100-year flood. We determined loads from various combinations. We assumed a wood dead load of 52 psf by calculating the approximate volume of the timber and an assumed density of 30 pcf. Similarly, we assumed a steel dead load of 102 psf by conservatively estimating the weight of the girders and concrete decking (Appendix V (D)). According to AASHTO Design of Pedestrian Bridge Manual, 2009, the location and traffic density of this bridge require a resistance to a pedestrian live load of 90 psf (Section 3.1) and an equestrian live point load of 1.0 kip (Section 3.3). We determined a snow load of 150 psf from the Forest Service PDF document for glulam stringer bridges. Vehicle, wind, and earthquake loads are not a concern for this small, low-use bridge as stated by AASHTO.

With these loads determined, we used LRFD loading combinations to find maximum bending moments and shear to identify the approximate member sizes required for the span used in design. Shear and moment diagrams and tabulated shear and moment diagrams are located in Appendix II (D).

Table 4 contains the tabulated shear and moment values the structure must be designed to resist.

Table 4: Tabulated shears and reactions from load cases used for design.

Timber alternative
40-ft span

	Rxn Left (kips)	Rxn Right (kips)	Max Shear (kips)	Max Moment (kip-ft)
Case 1	27.8	27.8	27.8	277.8
Case 2	19.3	25	25	224.3
Case 3	29.3	27.8	29.3	278.6
Case 4	28.6	28.3	28.6	293.8
Case 5	20.1	25.8	25.8	238.8

Abs Max Shear (kip)	29.3
Abs Max Moment (kip-ft)	293.8

Steel alternative
40-ft span

	Rxn Left (kips)	Rxn Right (kips)	Max Shear (kips)	Max Moment (kip-ft)
Case 1	33.2	33.2	33.2	332.2
Case 2	24.3	30.2	30.2	275.4
Case 3	34.8	33.3	34.8	333
Case 4	34	34	34	348.2
Case 5	25.1	31	31	290.1

Abs Max Shear (kip)	34.8
Abs Max Moment (kip-ft)	348.2

Structural Design

The Forest Service typically constructs pedestrian bridges with timber or steel. Timber is generally the preferred material, since wood costs less than steel and a wooden aesthetic better matches the natural surroundings. Glu-lam bridges are also easier to construct in remote areas since the members weigh less and do not require onsite welding. Glu-lam bridges are limited to a 50-ft maximum span, but this was not a concern for the design project since the span is 40 ft.

We independently prepared a steel bridge alternative and a wood bridge alternative and compared the resulting costs.

Steel Design

Refer to construction drawings in Appendix V.

We used the American Institute of Steel Construction (AISC) Manual as a guide for our design guideline for the steel bridge. We first completed the girder design by selecting an adequate member to resist the required moment from Table 3-10 in the AISC Manual. We then verified the adequacy of the shear capacity using the AISC Specification Chapter G equations. Finally, we checked deflections to satisfy serviceability limit states. According to AASHTO regulation for pedestrian bridges, the maximum deflection due to live loads cannot exceed L/360 where L is the span length (Dead + live load limit states are not required for pedestrian bridges). After iterating through several sections, we determined **two W14x61** girders are adequate for the bridge with a single brace at the mid-span (See Appendix V(E.ii)).

Next, we designed the handrails. According to Forest Service guidelines, a pedestrian-only use bridge must have a 42-inch high handrail, which must also be able to withstand a 200 lb lateral point load and support a distributed load of 50 lb/ft along the entire length of the structure. The handrail must also have a vertical bar spacing of no more than 19 inches. We designed a vertical handrail support post every 4 feet and a vertical spacing of 15 inches for horizontal members. The bars are to be A36 steel, 1" diameter, and 0.25" thickness. Since the loading is perpendicular to the welding axis Chapter J of the AISC Specification allows a 50% increase in capacity. We checked tensile capacity of the base material and the strength of weld and determine a ¼" fillet weld is adequate (See Appendix V(E.ii).

The walking surface shall consist of a 2" metal deck with 2" of concrete for a 4" equivalent depth slab. The slab will be longitudinally flanked by two L5x3x1/4" angles to contain the concrete when poured.

We accounted for axial thermal expansion of the steel girders by designing elastomeric pads to support the girder at one end. We used a 50-durometer Elastomeric Isolator system by Kinetics Noise Control. First, we determined the extent of thermal expansion/contraction expected from temperature variation, which we calculated using the coefficient of thermal expansion of steel. With a maximum temperature differential of $80^{o}F$ from room temperature, we calculated a deflection of 0.257 in. Using two ¼" thick elastomeric pads separated by a steel shim allows for 0.3 in of lateral movement. We next checked the allowable vertical bearing pressure on the pads. To avoid exceeding the allowable pressure, we specified that each girder must bear on an A36 steel plate of dimensions 12”x18”x1/4”, which will rest directly on top of the elastomeric pads. (See Appendix V(E.ii)).

Wood Design

Refer to construction drawings in Appendix V.

We used the National Design Standard (NDS) by the American Wood Council for the timber bridge alternative. The image below is a standard isometric view of a glu-lam bridge prepared by the Forest Service.

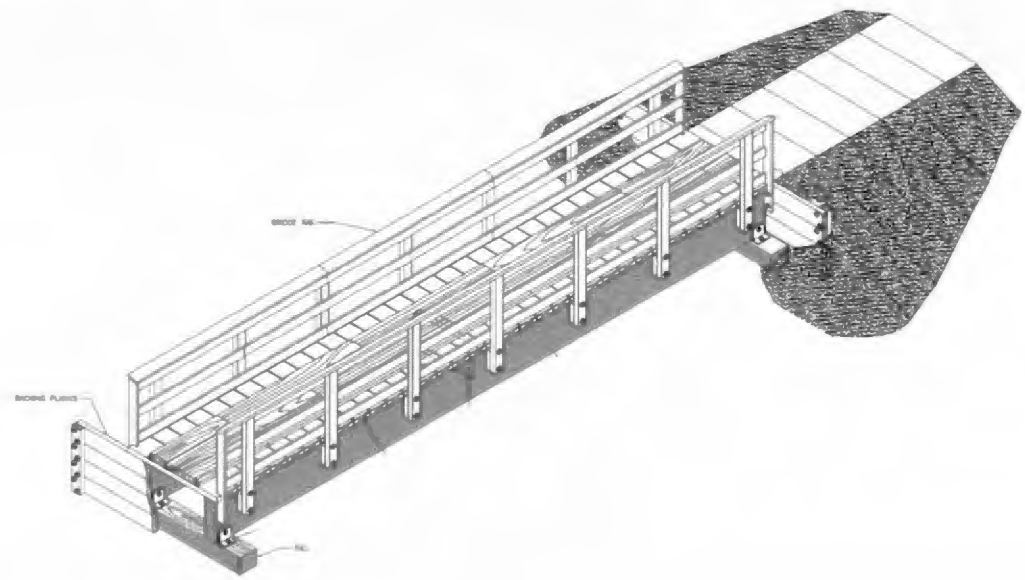


Figure 7. Glued-laminated stringer bridge design used by the Forest Service.

We began the wood design by designing the glu-lam girders with a braced diaphragm at the midspan. According to NDS, the basic moment design equation of bending members is $M' > M$ (M3.3), where M' = adjusted moment capacity = $F_b'*S$. F_b' is the adjusted allowable bending stress, and S is the section modulus about the strong axis. Standard glu-lam material allowable stresses (for example, F_b) are tabulated in NDS Supplement Table 5a. We chose a glu-lam material F24-1.8E (allowable bending stress of 2,400 psi and modulus of elasticity of 1,800,000 psi). We adjusted the allowable bending stress by multiplying by the wet service factor, temperature factor, flat use factor, curvature factor, stress interaction factor, beam stability factor, volume factor, time effect factor, and the LRFD loading coefficient. This is according to the adjusted stress equations in NDS M5.3. Next, we iterated the size of the girder (increasing the section modulus each time) until we reached an allowable moment capacity. We checked the shear, which did not control using a similar procedure (from M3.4). We checked deflections to satisfy serviceability limit states. According to AASHTO regulation for pedestrian bridges, the maximum deflection due to live loads cannot exceed L/360 (Dead + live load limit states are not required for pedestrian bridges). Finally, we checked for allowable bearing stress of the girders at the support. We determined that **two 5-1/8x34-1/2" F2.4-1.8E glu-lam girders** are adequate from the bridge (See Appendix V(Ei)).

Next, we designed the timber handrails from rough sawn lumber. We used the same lateral loading as specified in the steel design: a point load of 200 lb or a distributed load of 50 lb/ft. We used the same spacing as the steel bridge: vertical posts at 4' on center, horizontal posts at 15" on center. The design considerations were similar to the girder design; we adjusted tabulated allowable bending and shear stresses (NDS Supplement Table 4A) for usage conditions and compared the resulting shear and moment capacities to the design shear and moments. We determined **4"x6" rough sawn** posts connected normal to the weak axis are adequate (See Appendix V(Ei)).

The deck is to consist of **3'-6" long 4"x8" rough sawn members** attached directly to the glu-lam girders perpendicular to the length of span with ¼" gaps between each member.

The handrail and decking connections support negligible load compared to the capacity of the fasteners we designed, so we did not perform detailed calculations for the bolts and wood screws. Each vertical handrail post is connected to the girder with two ¾"-16" long bolts separate from the girder by a 2"x6" block. The horizontal handrail components and the decking members are attached with #10x4" wood screws or #10x6" wood screws depending on the width of the side member.

Cost Analysis

The total material cost of the steel alternative (excluding foundations, earthwork, and construction costs) is $9,420.64. The wood alternative was less expensive at $7,173.05.

Foundation Design

Refer to construction drawings in Appendix V.

We next designed the foundation for the bridge. The soil had to be analyzed to find the allowable soil bearing pressure. We were unable to perform a triaxial shear test to determine soil strength, but we assumed a sandy soil with a friction angle of 35° and no cohesion, according to local soil conditions. We performed a bearing capacity analysis using the "Bearing Capacity of Foundations on Top of a Slope" method described in 4.6 of *Principles of Foundation Engineering (Seventh Edition)* by Braja Das. We used this technique since the pads will sit near the top of the river banks.

We found the bearing capacity of the soil using a method for continuous foundations on top of a slope to be 2,960 psf for the critical side (side opposite the road). The dimensions of the footing are 9 ft by 3 ft, which we approximate as continuous. Continuous footings can only displace loads in the perpendicular direction, but a square or rectangular footing can displace loads in all directions and therefore would have more capacity than a continuous footing. This makes the analysis of a 9 ft by 3 ft rectangular pad conservative and acceptable. We used conservative soil properties described above to find a bearing capacity. We used the buoyant unit weight of the soil to simulate the water table at the base of the foundation which occurs during flooding. With these conservative assumptions, we obtained an acceptable ultimate bearing pressure factor of safety of 2.2, comparing the soil's bearing capacity to pressure due to the maximum footing reaction from loading the bridge (See Appendix V(F)). We plan to use a standard Gabion foundation which uses a steel wire cage filled with stone cobbles. A 12"x12" concrete sill beam will sit on top of each foundation pad, and the girders will bear on and connect to the concrete sill. This inexpensive and easy-to-construct alternative will cost $950 for both foundation pads.

We also designed an earth approach structure for the bridge to satisfy ABA accessibility standards. The bridge must be wheel chair assessable, with no approach slope exceeding 12:1. The required slope on the roadway side of the bridge is 42', which is too long for a straight slope, so we designed a 90° bend in the earthworks to provide the needed ramp length. The soil must be well compacted, and natural grass will be permitted to grow on the side slopes to reduce erosion. We estimate that 39 yd^3 of earth fill will be needed to construct the ramp, costing approximately $2,100.

Selected Alternative and Design Cost

We selected a timber alternative for the bridge at Right Hand Fork for three primary reasons.

1. Cost – The timber alternative costs $7,173.05, which is significantly less than the steel alternative cost at $9,420.64.
2. Ease of construction – The timber bridge will be easier to construct since the materials are lighter weight and easier to move to semi-inaccessible locations. Wood does not require any difficult onsite welding or a waiting time for concrete to cure.
3. Aesthetics – The natural finish of a timber bridge better matches the natural aesthetic desired by the Forest Service in remote locations.

The costs of the individual components are listed below. A more detailed cost estimate is located in Appendix V(B).

Table 5: Material costs of timber bridge.

Timber structure	$ 7,173.05
Gabion foundations	$ 950.00
Earth fill	$ 2,100.00
Total	$ 10,223.05

Environmental Concerns

The Right Hand Fork stream contains a pure strain of Bonneville cutthroat trout, a native species of Northern Utah. In past years, fishermen introduced alien species of brown trout that out-compete the Bonneville cutthroat. Forest Service ecologists recognized the need to protect native fisheries and constructed a small weir to block other aquatic species from the Right Hand Fork watershed. Shallow water flows over the weir's crest, with a five-foot elevation drop downstream. The structure is located approximately 100 yards from the Hobbit Caves, making the shallow crest an easy crossing for recreationalists. Climbers and campers began placing wooden pallets across the weir's crest after the bridge was damaged. Ron Vance, Forest Service natural resource manager, is concerned that these foreign objects could harm aquatic life, particularly the Bonneville cutthroat. A new pedestrian bridge would eliminate the need to place obstructions in the stream.

We found that the proposed location of the new bridge would have the least amount of impact on the ecology of the river. Since the new bridge is near the location of the current bridge, only a limited amount of riparian vegetation would need to be removed. Also, the construction of new trails would not be required. The impact on the environment would be minimal compared to other alternatives.

We attempted to collaborate with the Forest Service to ensure the design is aesthetically and environmentally acceptable, but contact has been limited. Ron Vance, our primary contact with the Logan Ranger District, has not returned any of our emails since the beginning of fall semester 2014. Due to limited contact, NEPA meetings could not be attended.

We also attempted to consult with Forest Service engineers Jason Day, PE, SE, and Oscar Mena, EIT. Mr. Day and Mr. Mena have expressed that they lack the time to continue to work with us further on the project, due to their current work load. Mark Anderson, SE, PE, a structural engineer at Steel Concepts, which is located in Ogden, agreed to be our new professional mentor. The project was completed despite the lack of help from the Forest Service.

EVALUATION

The Right Hand Fork trail bridge will be built due to the failure of the previous bridge. We are using appropriate LRFD and AASHTO design codes to design a more sustainable structure than the current bridge. All design work and surveys will be subject to evaluations. The evaluations include:

- Purchase of proper materials to guarantee that the new bridge can withstand weather conditions.
- Maximum recorded flow rates of the Right Hand Fork creek to determine the height of the bridge span above the river.
- Appropriate location to provide easy access to the public and protection to the ecosystem. It is essential that the bridge be visible from the parking area so recreationalists can see it.
- Annual maintenance requirements for normal wear and tear.

Evaluations of the bridge design will be made by our team and the Forest Service. The bridge will have a 50-year design life.

BUDGET

TEAM EXPENSES

According the Internal Revenue System (IRS), the "standard mileage rates for the use of a car" shall be reimbursed at 56 cents per mile driven for business purposes. Therefore, the total travel reimbursement is currently $100.80 (See Appendix 13.4). All equipment used for the project has been rented for free.

CONCLUSIONS

Right Hand Fork is a beautiful Forest Service area near Logan Canyon that draws crowds to recreational sites. The Hobbit Caves are a recreational site in Right Hand Fork, accessible by a pedestrian bridge, but flooding in 2011 damaged the bridge's abutments. Recreationalists now stack wooden pallets in the creek as an alternative access to the visibly damaged structure. We propose to construct a new bridge with a 50-year service life.

Right Hand Fork is home to the Bonneville cutthroat trout, which is on the Utah Sensitive Species list. Forest Service ecologists feel that it is important to protect the trout by reducing foreign objects (pallets, etc.) in the stream.

The proposed trail bridge project will greatly benefit the public using the area and the local wildlife. The structure will be safer than the current damaged bridge and provide wheelchair accessibility. A new bridge will also promote a positive image of the Forest Service, compared to the sagging bridge that currently spans the creek.

APPENDIX I: GANTT CHARTS

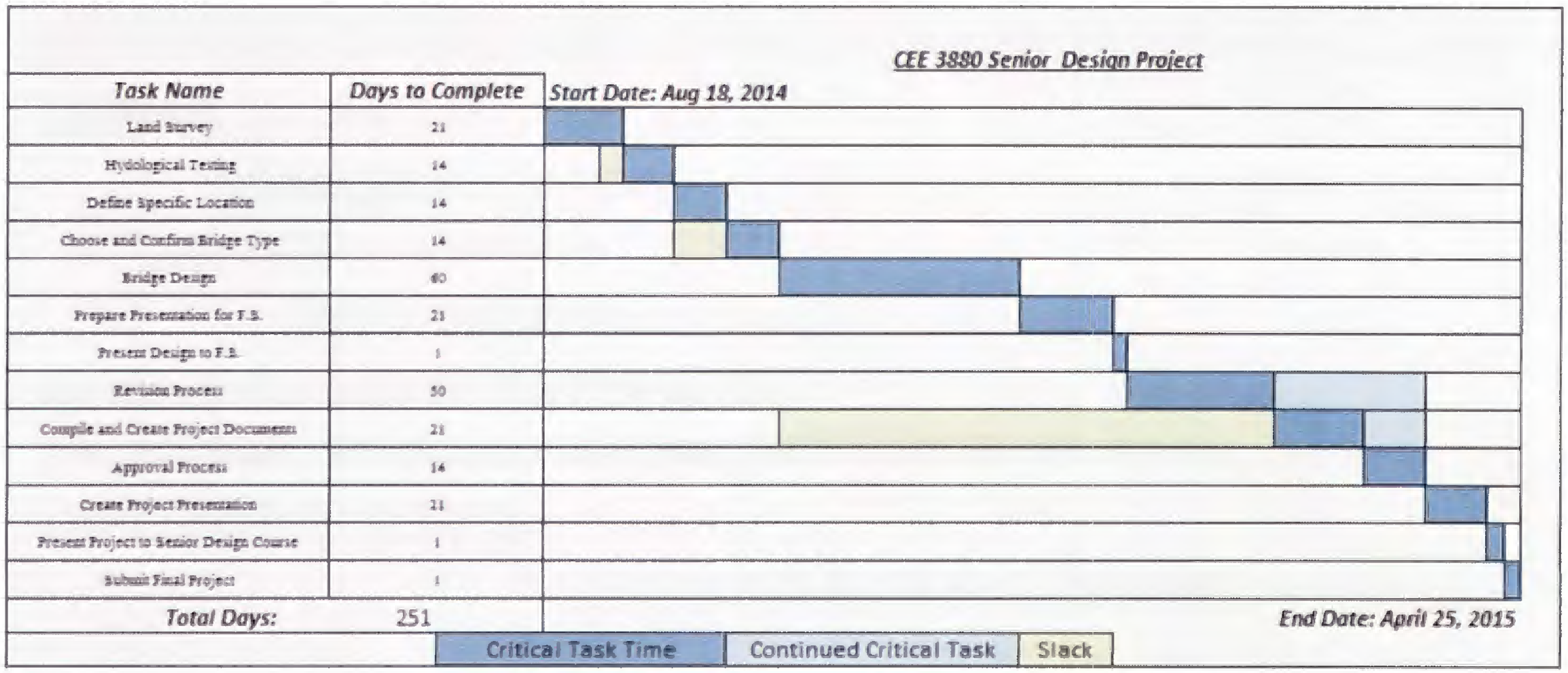


Table 6. Proposed Gantt chart for fall 2014 and spring 2015

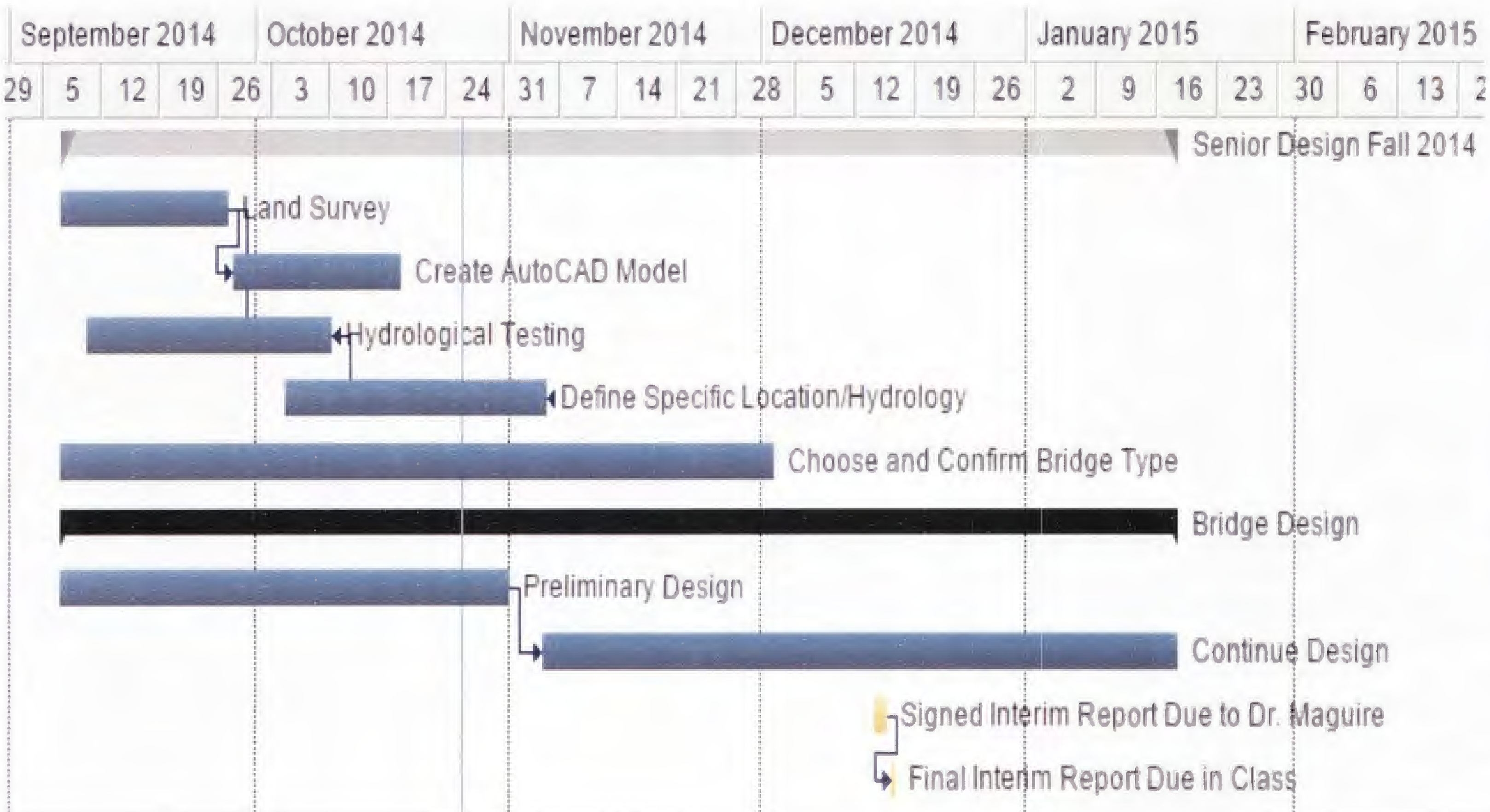


Table 7. Gantt chart for fall 2014

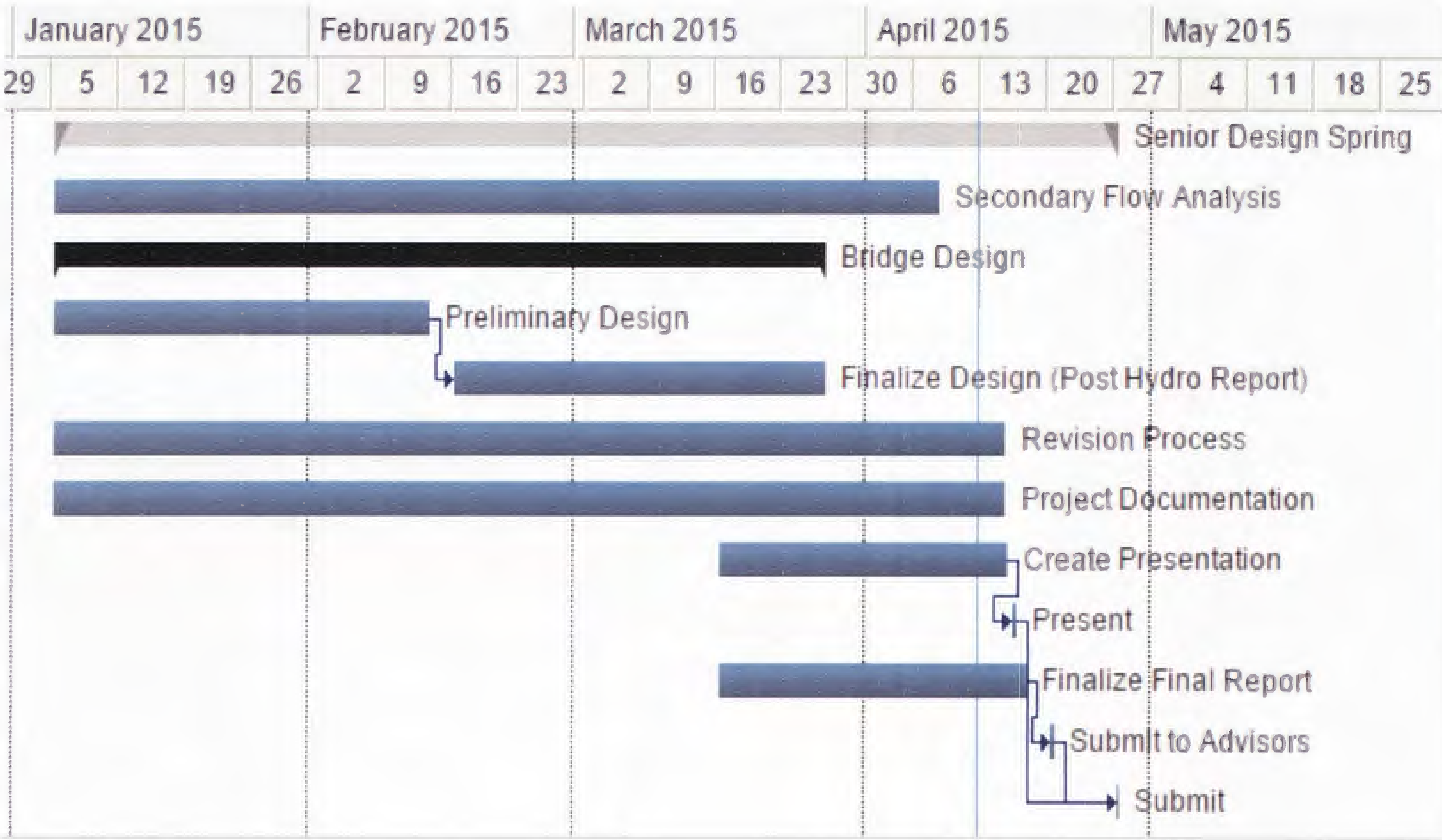


Table 8. Gantt chart for spring 2015

APPENDIX II: DATA

A. Watershed Characteristics Data

Point precipitation frequency estimates (inches)
NOAA Atlas 14 Volume 1 Version 5
Data type: Precipitation depth
Time series type: Partial duration
Project area: Southwest
Location name: Logan, Utah, US*
Station Name: -
Latitude: 41.7804°
Longitude: -111.6357°
Elevation: 5316 ft*
* source: Google Maps

PRECIPITATION FREQUENCY ESTIMATES

by duration for ARI:	1	2	5	10	25	50	100	200
5-min:	0.12	0.15	0.2	0.26	0.34	0.41	0.49	0.59
10-min:	0.18	0.23	0.31	0.39	0.51	0.62	0.75	0.9
15-min:	0.22	0.28	0.39	0.48	0.63	0.77	0.93	1.11
30-min:	0.3	0.38	0.52	0.65	0.85	1.03	1.25	1.49
60-min:	0.37	0.47	0.65	0.8	1.05	1.28	1.54	1.85
2-hr:	0.48	0.61	0.8	0.97	1.24	1.49	1.78	2.12
3-hr:	0.57	0.72	0.91	1.09	1.36	1.61	1.9	2.22
6-hr:	0.8	0.99	1.22	1.43	1.73	1.99	2.27	2.6
12-hr:	1.07	1.32	1.61	1.87	2.25	2.55	2.87	3.22
24-hr:	1.43	1.76	2.15	2.47	2.93	3.3	3.68	4.08
2-day:	1.7	2.1	2.55	2.94	3.48	3.91	4.37	4.84
3-day:	1.9	2.34	2.85	3.29	3.9	4.39	4.9	5.44
4-day:	2.09	2.58	3.16	3.65	4.33	4.87	5.44	6.04
7-day:	2.61	3.23	3.96	4.57	5.43	6.1	6.81	7.55
10-day:	2.99	3.7	4.54	5.22	6.15	6.87	7.62	8.39
20-day:	4	4.95	5.98	6.79	7.86	8.66	9.47	10.27
30-day:	4.89	6.04	7.27	8.25	9.55	10.54	11.53	12.51
45-day:	6.17	7.6	9.06	10.2	11.69	12.79	13.89	14.96
60-day:	7.28	8.97	10.62	11.87	13.45	14.6	15.71	16.79

Date/time (GMT): Thu Sep 11 00:01:20 2014
pyRunTime: 0.0587310791016

Figure 8. Watershed delineation from USGS Streamstats.

Table 9. Land Cover Type and Possible Curve Numbers

Count	Land Cover Type	Percent	Soil Cond	Possible CN	Soil Cond	Possible CN	Soil Cond	Possible CN	Soil Cond	Possible CN
179	Developed, Open Space	0								
41682	Deciduous Forest	58	Fair-B	48	-	73	Fair-C	57	Fair-C	57
10246	Evergreen Forest	14	Fair-B	58	-	77	Fair-C	73	Fair-C	73
74	Mixed Forest	0								
20167	Shrub/Scrub	28	Fair-B	71	-	77	Fair-C	81	Fair-C	63
11	Herbaceuous	0								
19	Hay/Pasture	0								
19	Woody Wetlands	0								

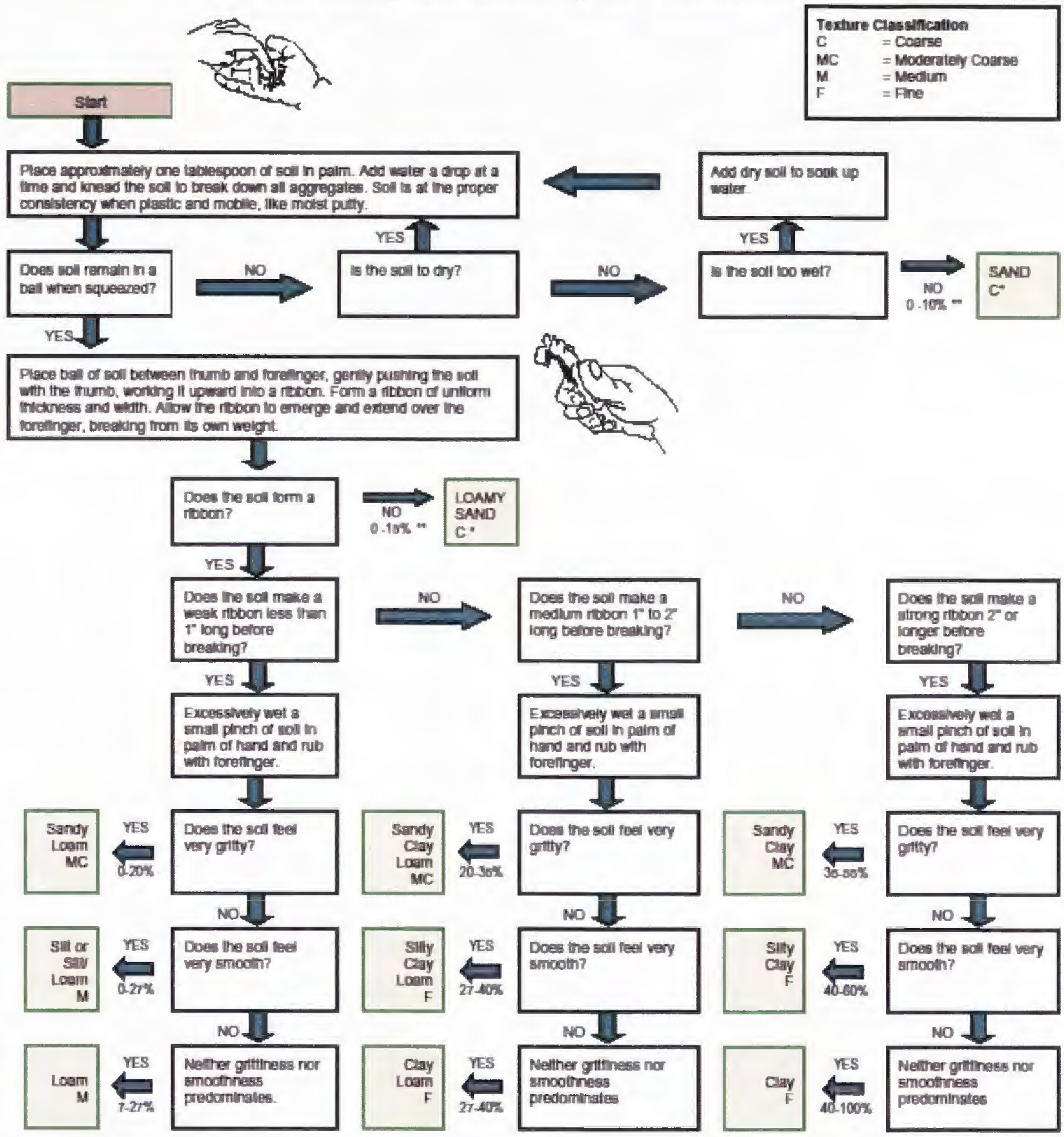
Curve numbers were determined using Table 9-2 from chapter 9 in the National Engineering Handbook.

Table 10. Results of Soils Samples

Location			
N	W	Soil Sample	Soil Type
41.77987	111.63484	1	Silty Clay/Silty Clay loam
41.78009	111.62418	2	Sandy Loam
41.76808	111.62001	3	Loam/ sandy clay loam
41.77497	111.60947	4	Sandy Clay loam/Sandy loam
41.77331	111.60789	5	Sandy Clay/ Sandy Clay loam

The results from the soil samples were used to determine the hydrologic soil group and soil condition for the curve numbers.

Determining Soil Texture by the "Feel Method"



* Sand Particle size should be estimated (very fine, fine, medium, coarse) for these textures. Individual grains of very fine sand are not visible without magnification and there is a gritty feeling to a very small sample ground between the teeth. Some fine sand particles may be just visible. Medium sand particles are easily visible. Examples of sand size descriptions where one size is predominant are; very fine sand, fine sandy loam, loamy coarse sand.

** Cay percentage range.

Modified from: Thien, Steven J., Kansas state University, 1979 Jour. Agronomy education.

B. HEC-HMS Data

Figure 9. Basin model for HEC-HMS model.

Table 11. Detailed Results from 24-hr Simulation of 100-yr 12-hr Storm

Date	Time	Precip (IN)	Loss (IN)	Excess (IN)	Direct Flow (CFS)	Baseflow (CFS)	Total Flow (CFS)
24-Aug-14	12:00				0	40	40
24-Aug-14	12:30	0.04	0.04	0	0	40	40
24-Aug-14	13:00	0.04	0.04	0	0	40	40
24-Aug-14	13:30	0.05	0.05	0	0	40	40
24-Aug-14	14:00	0.05	0.05	0	0	40	40
24-Aug-14	14:30	0.05	0.05	0	0	40	40
24-Aug-14	15:00	0.06	0.06	0	0	40	40
24-Aug-14	15:30	0.05	0.05	0	0	40	40
24-Aug-14	16:00	0.06	0.06	0	0	40	40
24-Aug-14	16:30	0.07	0.07	0	0	40	40
24-Aug-14	17:00	0.06	0.06	0	0	40	40
24-Aug-14	17:30	0.11	0.11	0	0	40	40
24-Aug-14	18:00	0.33	0.33	0	0	40	40
24-Aug-14	18:30	1.04	0.88	0.16	27.8	40	67.8
24-Aug-14	19:00	0.15	0.1	0.05	111.7	40	151.7
24-Aug-14	19:30	0.07	0.05	0.02	243.6	40	283.6
24-Aug-14	20:00	0.08	0.05	0.03	392.4	40	432.4

24-Aug-14	20:30	0.06	0.04	0.02	521.6	40	561.6
24-Aug-14	21:00	0.06	0.03	0.02	599.2	40	639.2
24-Aug-14	21:30	0.06	0.04	0.02	629.9	40	669.9
24-Aug-14	22:00	0.06	0.03	0.02	645.2	40	685.2
24-Aug-14	22:30	0.05	0.03	0.02	656.3	40	696.3
24-Aug-14	23:00	0.05	0.03	0.02	664.5	40	704.5
24-Aug-14	23:30	0.05	0.03	0.02	670.2	40	710.2
25-Aug-14	0:00	0.04	0.02	0.02	673	40	713
25-Aug-14	0:30	0	0	0	669.3	40	709.3
25-Aug-14	1:00	0	0	0	654.5	40	694.5
25-Aug-14	1:30	0	0	0	626.2	40	666.2
25-Aug-14	2:00	0	0	0	585.6	40	625.6
25-Aug-14	2:30	0	0	0	537.4	40	577.4
25-Aug-14	3:00	0	0	0	488.4	40	528.4
25-Aug-14	3:30	0	0	0	443.1	40	483.1
25-Aug-14	4:00	0	0	0	402.1	40	442.1
25-Aug-14	4:30	0	0	0	364.9	40	404.9
25-Aug-14	5:00	0	0	0	331.1	40	371.1
25-Aug-14	5:30	0	0	0	300.4	40	340.4
25-Aug-14	6:00	0	0	0	272.6	40	312.6
25-Aug-14	6:30	0	0	0	247.4	40	287.4
25-Aug-14	7:00	0	0	0	224.5	40	264.5
25-Aug-14	7:30	0	0	0	203.7	40	243.7
25-Aug-14	8:00	0	0	0	184.8	40	224.8
25-Aug-14	8:30	0	0	0	167.7	40	207.7
25-Aug-14	9:00	0	0	0	152.2	40	192.2
25-Aug-14	9:30	0	0	0	138.1	40	178.1
25-Aug-14	10:00	0	0	0	125.3	40	165.3
25-Aug-14	10:30	0	0	0	113.7	40	153.7
25-Aug-14	11:00	0	0	0	103.2	40	143.2
25-Aug-14	11:30	0	0	0	93.6	40	133.6
25-Aug-14	12:00	0	0	0	85	40	125

C. HEC-RAS Data

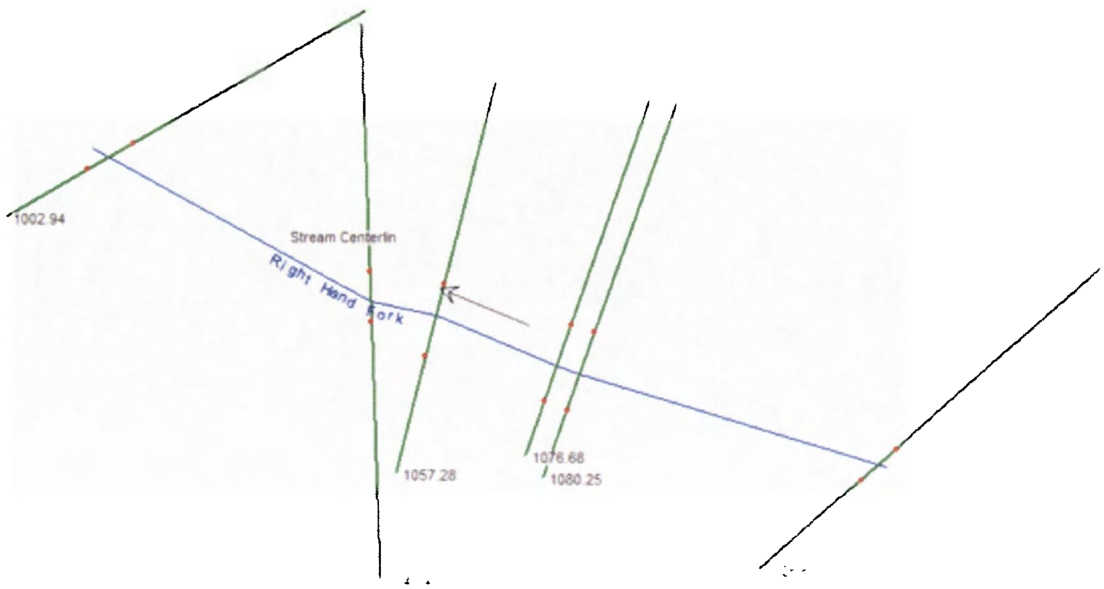


Figure 10. Geometry map for HEC-RAS model.

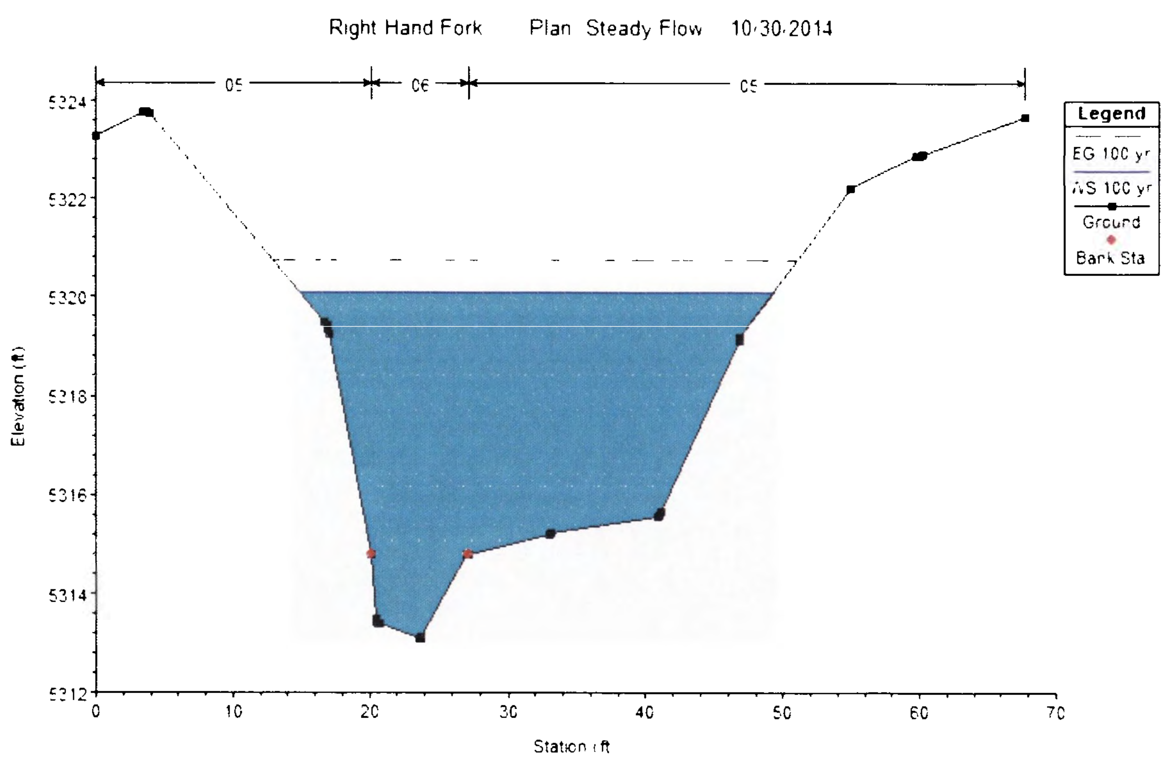


Figure 11. Cross section for station 11+26.61

Table 12. Detailed Results for station 11+26.61

Plan: Steady Flow Right Hand Fork Stream Centerlin RS: 1126.61 Profile: 100 yr

E.G. Elev (ft)	5320.76	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.68	Wt. n-Val.	0.050	0.060	0.050
W.S. Elev (ft)	5320.08	Reach Len. (ft)	48.01	46.36	45.14
Crit W.S. (ft)		Flow Area (sq ft)	10.03	45.29	84.14
E.G. Slope (ft/ft)	0.008686	Area (sq ft)	10.03	45.29	84.14
Q Total (cfs)	900.00	Flow (cfs)	33.06	319.44	547.49
Top Width (ft)	34.40	Top Width (ft)	5.20	7.04	22.17
Vel Total (ft/s)	6.45	Avg. Vel. (ft/s)	3.30	7.05	6.51
Max Chl Dpth (ft)	6.98	Hydr. Depth (ft)	1.93	6.43	3.80
Conv. Total (cfs)	9656.9	Conv. (cfs)	354.8	3427.6	5874.5
Length Wtd. (ft)	45.90	Wetted Per. (ft)	7.73	8.48	23.37
Min Ch El (ft)	5313.10	Shear (lb/sq ft)	0.70	2.90	1.95
Alpha	1.05	Stream Power (lb/ft s)	67.71	0.00	0.00
Frctn Loss (ft)	0.37	Cum Volume (acre-ft)	0.03	0.17	0.12
C & E Loss (ft)	0.00	Cum SA (acres)	0.01	0.03	0.04

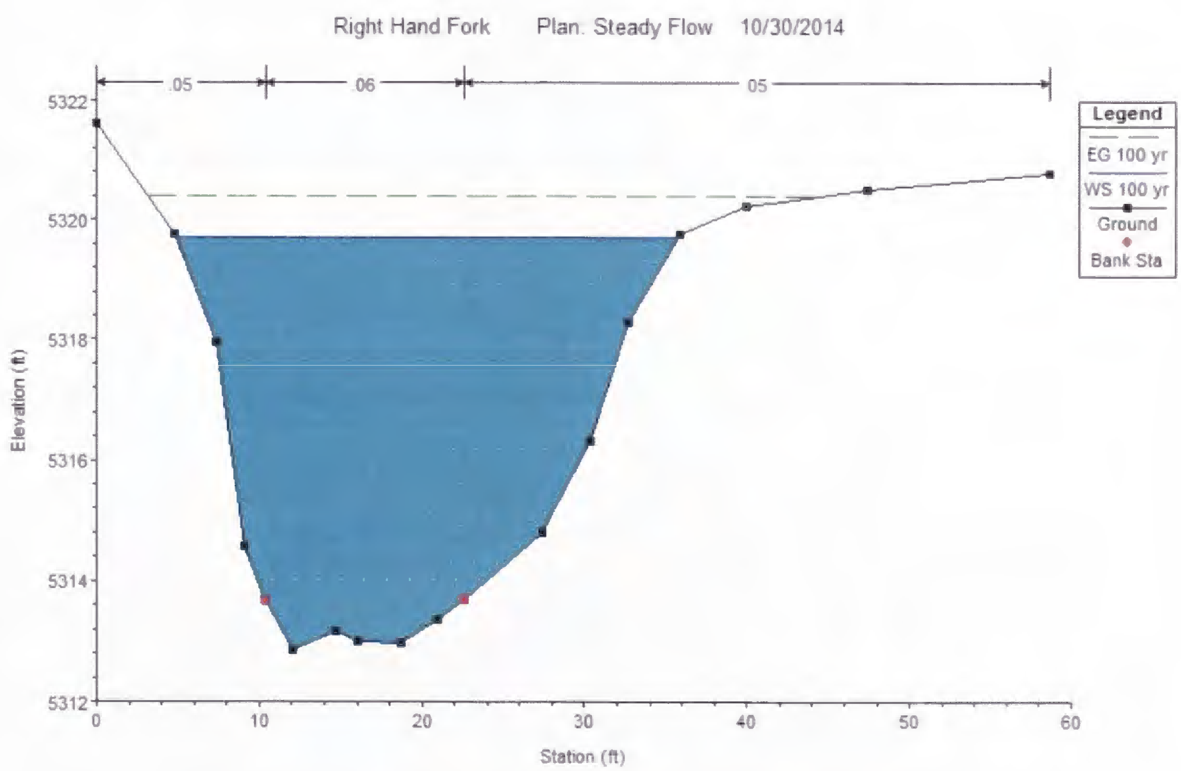


Figure 12. Cross section for station 10+80.25

Table 13. Detailed Results for station 10+80.25

Plan: Steady Flow   Right Hand Fork   Stream Centerlin RS: 1080.25   Profile: 100 yr

E.G. Elev (ft)	5320.39	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.69	Wt. n-Val.	0.050	0.060	0.050
W.S. Elev (ft)	5319.69	Reach Len. (ft)	3.62	3.57	3.54
Crit W.S. (ft)		Flow Area (sq ft)	15.28	79.84	46.27
E.G. Slope (ft/ft)	0.007416	Area (sq ft)	15.28	79.84	46.27
Q Total (cfs)	900.00	Flow (cfs)	58.59	586.90	254.51
Top Width (ft)	30.82	Top Width (ft)	5.45	12.20	13.17
Vel Total (ft/s)	6.37	Avg. Vel. (ft/s)	3.83	7.35	5.50
Max Chl Dpth (ft)	6.84	Hydr. Depth (ft)	2.80	6.54	3.51
Conv. Total (cfs)	10450.9	Conv. (cfs)	680.3	6815.2	2955.4
Length Wtd. (ft)	3.57	Wetted Per. (ft)	8.33	12.48	14.69
Min Ch El (ft)	5312.85	Shear (lb/sq ft)	0.85	2.96	1.46
Alpha	1.10	Stream Power (lb/ft s)	58.65	0.00	0.00
Frctn Loss (ft)	0.03	Cum Volume (acre-ft)	0.02	0.10	0.05
C & E Loss (ft)	0.02	Cum SA (acres)	0.01	0.02	0.02

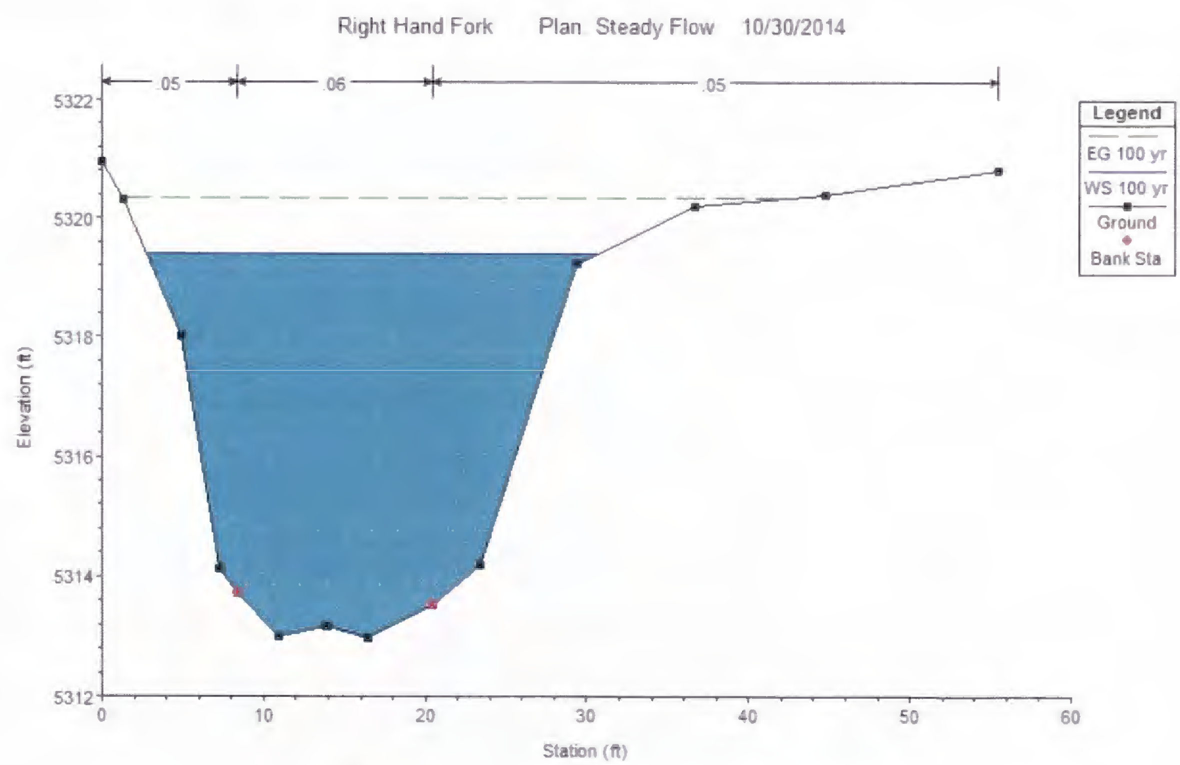


Figure 13. Cross section for station 10+76.68

Table 14. Detailed Results for station 10+76.68

Plan: Steady Flow Right Hand Fork Stream Centerlin RS: 1076.68 Profile: 100 yr

E.G. Elev (ft)	5320.33	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.93	Wt. n-Val.	0.050	0.060	0.050
W.S. Elev (ft)	5319.39	Reach Len. (ft)	19.79	19.40	18.83
Crit W.S. (ft)		Flow Area (sq ft)	15.52	74.75	32.72
E.G. Slope (ft/ft)	0.010527	Area (sq ft)	15.52	74.75	32.72
Q Total (cfs)	900.00	Flow (cfs)	71.72	635.76	192.52
Top Width (ft)	28.02	Top Width (ft)	5.66	12.05	10.31
Vel Total (ft/s)	7.32	Avg. Vel. (ft/s)	4.62	8.51	5.88
Max Chl Dpth (ft)	6.43	Hydr. Depth (ft)	2.74	6.20	3.17
Conv. Total (cfs)	8771.6	Conv. (cfs)	699.0	6196.3	1876.3
Length Wtd. (ft)	19.30	Wetted Per. (ft)	8.32	12.21	12.21
Min Ch El (ft)	5312.96	Shear (lb/sq ft)	1.23	4.02	1.76
Alpha	1.12	Stream Power (lb/ft s)	55.55	0.00	0.00
Frctn Loss (ft)	0.22	Cum Volume (acre-ft)	0.02	0.10	0.05
C & E Loss (ft)	0.03	Cum SA (acres)	0.01	0.01	0.02

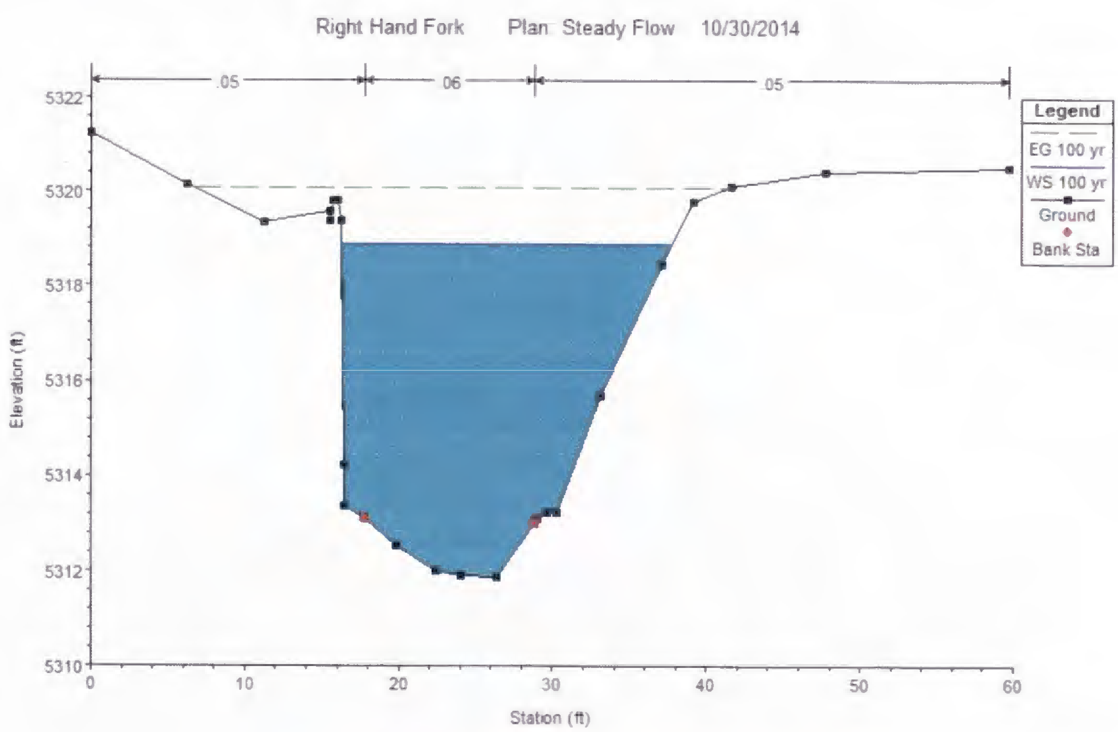


Figure 14. Cross section for station 10+57.28.

Table 15. Detailed Results for station 10+57.28

Plan: Steady Flow Right Hand Fork Stream Centerlin RS: 1057.28 Profile: 100 yr

E.G. Elev (ft)	5320.08	Element	Left OB	Channel	Right OB
Vel Head (ft)	1.21	Wt. n-Val.	0.050	0.060	0.050
W.S. Elev (ft)	5318.87	Reach Len. (ft)	11.22	9.87	9.52
Crit W.S. (ft)		Flow Area (sq ft)	7.76	72.93	28.33
E.G. Slope (ft/ft)	0.012485	Area (sq ft)	7.76	72.93	28.33
Q Total (cfs)	900.00	Flow (cfs)	28.12	693.83	178.05
Top Width (ft)	21.47	Top Width (ft)	1.48	11.05	8.94
Vel Total (ft/s)	8.26	Avg. Vel. (ft/s)	3.62	9.51	6.29
Max Chl Dpth (ft)	7.02	Hydr. Depth (ft)	5.24	6.60	3.17
Conv. Total (cfs)	8054.7	Conv. (cfs)	251.7	6209.6	1593.5
Length Wtd. (ft)	9.99	Wetted Per. (ft)	6.82	11.44	10.88
Min Ch El (ft)	5311.85	Shear (lb/sq ft)	0.89	4.97	2.03
Alpha	1.14	Stream Power (lb/ft s)	59.74	0.00	0.00
Frctn Loss (ft)	0.20	Cum Volume (acre-ft)	0.01	0.06	0.03
C & E Loss (ft)	0.12	Cum SA (acres)	0.00	0.01	0.01

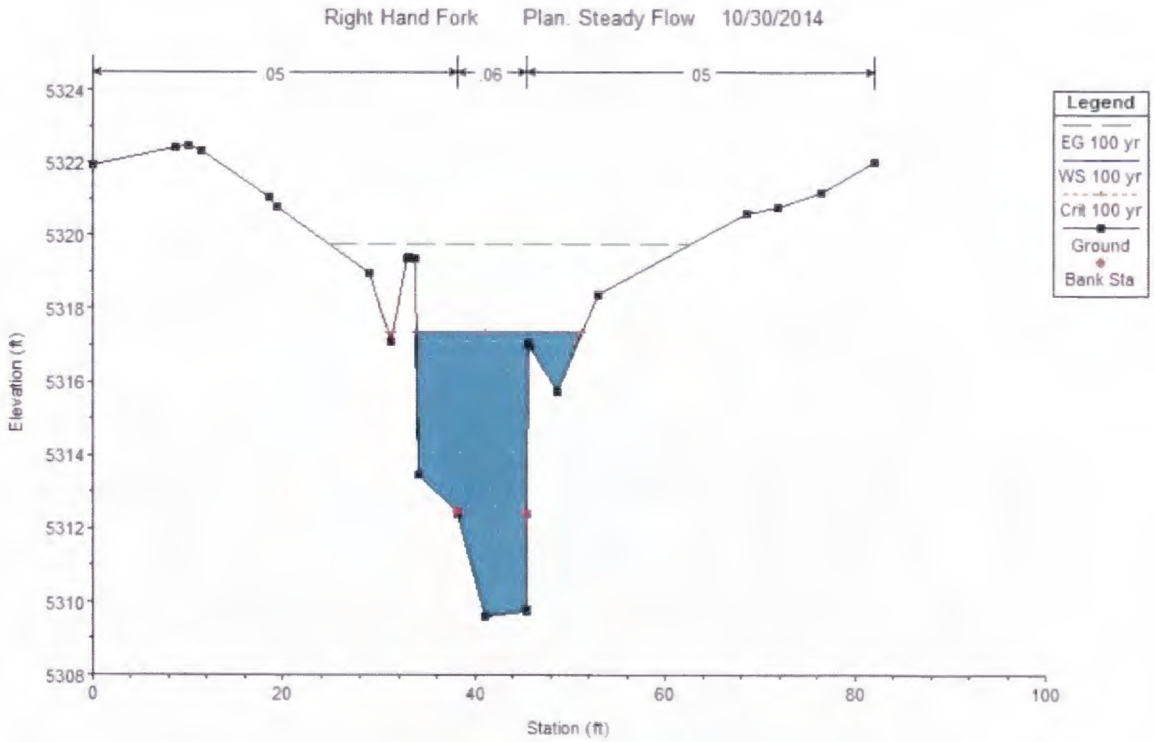


Figure 15. Cross section for station 10+47.41.

Table 16. Detailed Results for station 10+47.41

Plan: Steady Flow Right Hand Fork Stream Centerlin RS: 1047.41 Profile: 100 yr

E.G. Elev (ft)	5319.76	Element	Left OB	Channel	Right OB
Vel Head (ft)	2.42	Wt. n-Val.	0.050	0.060	0.050
W.S. Elev (ft)	5317.34	Reach Len. (ft)	39.90	44.47	47.82
Crit W.S. (ft)	5317.34	Flow Area (sq ft)	18.91	52.47	5.37
E.G. Slope (ft/ft)	0.036135	Area (sq ft)	18.91	52.47	5.37
Q Total (cfs)	900.00	Flow (cfs)	185.44	695.88	18.68
Top Width (ft)	18.04	Top Width (ft)	4.89	7.36	5.79
Vel Total (ft/s)	11.73	Avg. Vel. (ft/s)	9.81	13.26	3.48
Max Chl Dpth (ft)	7.76	Hydr. Depth (ft)	3.87	7.13	0.93
Conv. Total (cfs)	4734.5	Conv. (cfs)	975.5	3660.7	98.3
Length Wtd. (ft)	44.80	Wetted Per. (ft)	8.92	11.10	11.10
Min Ch El (ft)	5309.58	Shear (lb/sq ft)	4.78	10.67	1.09
Alpha	1.13	Stream Power (lb/ft s)	82.16	0.00	0.00
Frctn Loss (ft)	1.24	Cum Volume (acre-ft)	0.01	0.05	0.03
C & E Loss (ft)	0.27	Cum SA (acres)	0.00	0.01	0.01

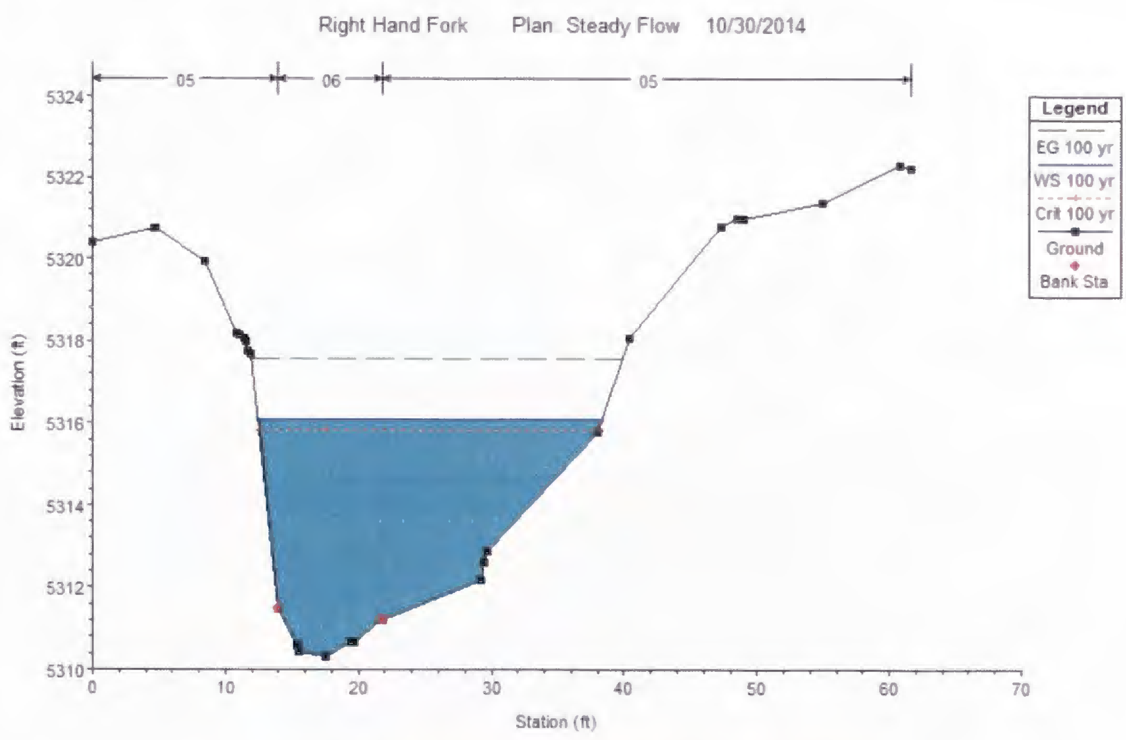


Figure 16. Cross section for station 10+02.94.

Table 17. Detailed Results for station 10+02.94

Plan: Steady Flow Right Hand Fork Stream Centerlin RS: 1002.94 Profile: 100 yr

E.G. Elev (ft)	5317.58	Element	Left OB	Channel	Right OB
Vel Head (ft)	1.51	Wt. n-Val.	0.050	0.060	0.050
W.S. Elev (ft)	5316.07	Reach Len. (ft)			
Crit W.S. (ft)	5315.83	Flow Area (sq ft)	3.36	42.30	49.05
E.G. Slope (ft/ft)	0.022008	Area (sq ft)	3.36	42.30	49.05
Q Total (cfs)	900.00	Flow (cfs)	11.65	461.13	427.22
Top Width (ft)	25.93	Top Width (ft)	1.46	7.84	16.62
Vel Total (ft/s)	9.50	Avg. Vel. (ft/s)	3.47	10.90	8.71
Max Chl Dpth (ft)	5.77	Hydr. Depth (ft)	2.30	5.40	2.95
Conv. Total (cfs)	6066.7	Conv. (cfs)	78.5	3108.4	2879.8
Length Wtd. (ft)		Wetted Per. (ft)	4.82	8.28	17.66
Min Ch El (ft)	5310.30	Shear (lb/sq ft)	0.96	7.02	3.82
Alpha	1.07	Stream Power (lb/ft s)	61.66	0.00	0.00
Frctn Loss (ft)		Cum Volume (acre-ft)			
C & E Loss (ft)		Cum SA (acres)			

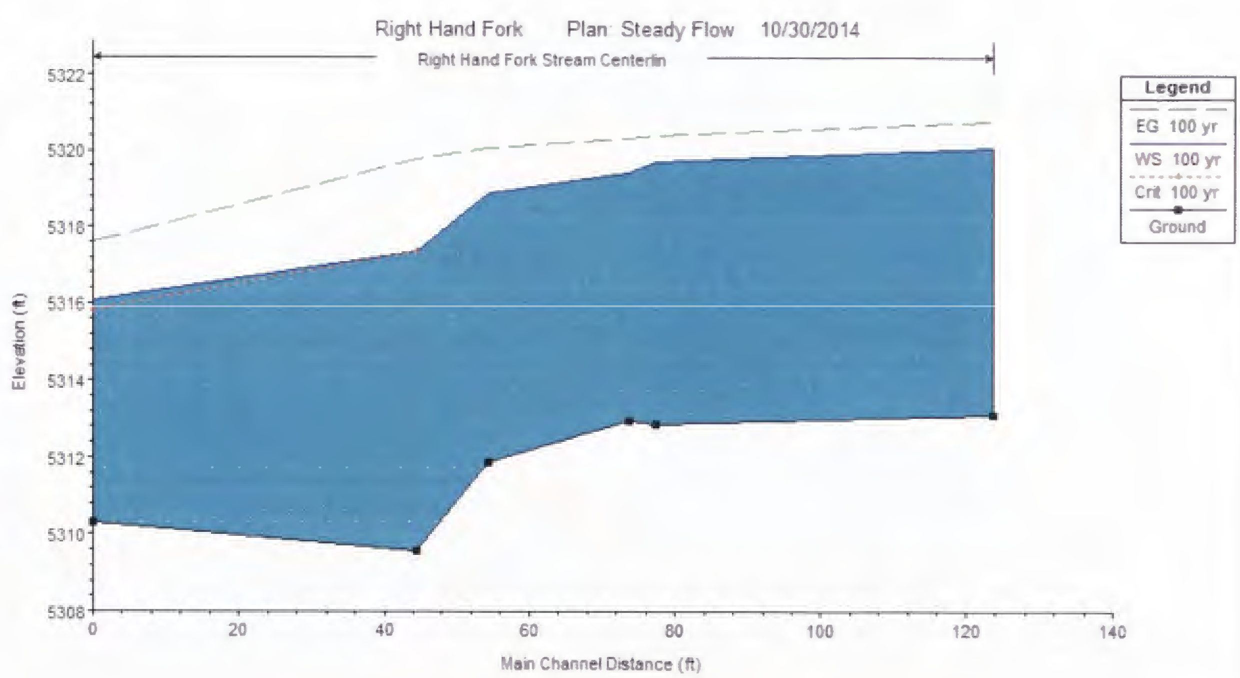


Figure 17. Water surface profile for 100-yr 12-hr storm.

Table 18. Profile Summary

Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
Stream Centerline	1126.61	100 yr	900	5313.1	5320.08		5320.76	0.008686	7.05	139.46	34.4	0.49
Stream Centerline	1080.25	100 yr	900	5312.85	5319.69		5320.39	0.007416	7.35	141.39	30.82	0.51
Stream Centerline	1076.68	100 yr	900	5312.96	5319.39		5320.33	0.010527	8.51	122.99	28.02	0.6
Stream Centerline	1057.28	100 yr	900	5311.85	5318.87		5320.08	0.012485	9.51	109.02	21.47	0.65
Stream Centerline	1047.41	100 yr	900	5309.58	5317.34	5317.34	5319.76	0.036135	13.26	76.75	18.04	0.88
Stream Centerline	1002.94	100 yr	900	5310.3	5316.07	5315.83	5317.58	0.022008	10.9	94.71	25.93	0.83

G. Flow Measurement Data

Logan River

Date 1/16/2015

Time in 1:40 **Time out** 2:05

Distance (ft)	Depth (ft)	Velocity (ft/sec)	Cell	Dis	As a %
0.0	0.00	0.00	0.00	0.00	0.00
4.2	0.00	0.00	2.60	0.00	0.00
5.2	0.40	0.21	1.90	0.16	0.24
8.0	0.57	1.40	2.90	2.31	3.52
11.0	0.81	1.05	3.00	2.55	3.88
14.0	0.51	0.08	3.00	0.12	0.19
17.0	0.83	0.45	3.00	1.12	1.71
20.0	1.10	0.66	3.00	2.18	3.32
23.0	1.00	1.30	3.15	4.10	6.23
26.3	1.20	2.20	2.75	7.26	11.05
28.5	1.05	2.39	2.25	5.65	8.60
30.8	1.33	1.51	2.75	5.52	8.41
34.0	0.68	3.19	2.60	5.64	8.59
36.0	0.78	3.46	2.00	5.40	8.22
38.0	0.83	2.22	2.00	3.69	5.61
40.0	1.05	0.05	2.00	0.11	0.16
42.0	1.10	1.81	2.50	4.98	7.58
45.0	0.84	4.63	3.00	11.67	17.76
48.0	1.02	0.19	3.00	0.58	0.89
51.0	0.50	0.92	3.00	1.38	2.10
54.0	0.52	0.82	3.00	1.28	1.95
57.0	0.20	0.02	2.05	0.01	0.01
58.1	0.00	0.00	1.50	0.00	0.00
60.0	0.00	0.00	0.00	0.00	0.00

Measured Dis **65.7** cfs
USGS Dis 92 cfs

Logan River
Date 2/5/2015

Time in 4:50 **Time out** 5:20

Distance (ft)	Depth (ft)	Velocity (ft/sec)	Cell	Dis	As a %
3.3	0	0			
4.2	0.59	0.23	0.95	0.129	0.20
5.2	0.59	0.29	1.90	0.325	0.49
8	0.79	1.42	2.90	3.253	4.95
11	1.05	2.21	3.00	6.962	10.60
14	0.20	1.38	3.00	0.828	1.26
17	1.00	0.54	3.00	1.620	2.47
20	1.22	0.95	3.00	3.477	5.29
23	1.17	1.88	3.15	6.929	10.55
26.3	1.68	2.73	2.75	12.613	19.20
28.5	1.30	4.19	2.25	12.256	18.66
30.8	1.50	2.86	2.75	11.798	17.96
34	0.93	3.69	2.60	8.922	13.58
36	1.33	3.88	2.00	10.321	15.71
38	1.13	2.63	2.00	5.944	9.05
40	0.77	4.31	2.00	6.637	10.10
42	1.41	2.17	2.50	7.649	11.64
45	1.25	4.99	3.00	18.713	28.49
48	1.30	1.65	3.00	6.435	9.80
51	0.80	0.57	3.00	1.368	2.08
54	0.99	3.04	3.00	9.029	13.74
57	0.50	0.37	2.25	0.416	0.63
58.5	0.00	0.00	1.50	0.000	0.00
60	0.00	0.00	0.00	0.000	0.00

Measured Dis **135.623** cfs
USGS Dis 113 cfs

Logan River

Date 3/26/2015

Time in 4:52 **Time out** 5:10

Distance (ft)	Depth (ft)	Velocity (ft/sec)	Cell	Dis	As a %
3.4	0	0	2.10	0.000	0.00
4.2	0.00	0.00	0.90	0.000	0.00
5.2	0.49	0.02	1.90	0.019	0.03
8	0.78	1.35	2.90	3.054	4.65
11	1.09	2.08	3.00	6.802	10.35
14	0.17	0.68	3.00	0.347	0.53
17	1.00	0.48	3.00	1.440	2.19
20	1.30	0.95	3.00	3.705	5.64
23	1.17	1.91	3.15	7.039	10.72
26.3	1.20	3.08	2.75	10.164	15.47
28.5	1.30	2.80	2.25	8.190	12.47
30.8	1.55	2.59	2.75	11.040	16.81
34	0.93	3.65	2.60	8.826	13.43
36	1.30	4.50	2.00	11.700	17.81
38	1.26	2.59	2.00	6.527	9.94
40	1.21	0.68	2.00	1.646	2.51
42	1.45	2.24	2.50	8.120	12.36
45	1.20	5.01	3.00	18.036	27.46
48	1.32	0.70	3.00	2.772	4.22
51	0.89	0.70	3.00	1.869	2.85
54	1.07	2.31	3.00	7.415	11.29
57	0.53	0.56	2.25	0.668	1.02
58.5	0.00	0.00	1.50	0.000	0.00
60	0.00	0.00	0.00	0.000	0.00

Measured Dis	**119.377**	cfs
USGS Dis	115	cfs

Right Hand Fork

Date 1/15/2015

Time in 5:28 **Time out** 5:40

Distance (ft)	Depth (ft)	Velocity (ft/sec)	Cell	Dis	As a %
0.0	0.00	0.00	0.00	0.00	0.00
1.2	0.00	0.00	0.80	0.00	0.00
1.6	0.15	0.46	0.80	0.06	0.68
2.8	0.53	1.40	1.20	0.89	11.02
4.0	0.68	1.73	1.20	1.41	17.47
5.2	0.53	1.13	1.10	0.66	8.15
6.2	0.34	1.56	0.90	0.48	5.91
7.0	0.59	2.28	0.75	1.01	12.49
7.7	0.58	2.23	0.75	0.97	12.01
8.5	0.61	1.65	1.00	1.01	12.46
9.7	0.50	1.79	1.15	1.03	12.74
10.8	0.39	1.26	1.05	0.52	6.39
11.8	0.22	0.22	1.15	0.06	0.69
13.1	0.00	0.00	1.60	0.00	0.00
15.0	0.00	0.00	0.00	0.00	0.00
			Measured Dis	**8.1**	cfs

Right Hand Fork

Date 2/5/2015

Time in 5:38 **Time out** 5:57

Distance (ft)	Depth (ft)	Velocity (ft/sec)	Cell	Dis	As a %
0					
1.2	0.00	0.00	0.80	0.000	0.00
1.6	0.32	0.85	0.80	0.218	2.69
2.8	0.70	2.19	1.20	1.840	22.77
4	0.90	2.24	1.20	2.419	29.94
5.2	0.70	1.81	1.10	1.394	17.25
6.2	0.47	2.46	0.90	1.041	12.88
7	0.70	1.62	0.75	0.851	10.53
7.7	0.70	2.00	0.75	1.050	13.00
8.5	0.66	1.76	1.00	1.162	14.38
9.7	0.60	1.86	1.15	1.283	15.88
10.8	0.51	1.11	1.05	0.594	7.36
11.8	0.37	0.64	1.15	0.272	3.37
13.1	0.10	0.24	0.80	0.019	0.24
13.4	0.00	0.00	0.45	0.000	0.00
14	0.00	0.00	0.00	0.000	0.00
			Measured Dis	**12.142**	cfs

Right Hand Fork

Date 3/26/2015

Time in 5:47 **Time out** 6:01

Distance (ft)	Depth (ft)	Velocity (ft/sec)	Cell	Dis	As a %
1.2	0.00	0.00	0.80	0.000	0.00
1.6	0.22	0.83	0.80	0.146	1.81
2.8	0.61	1.77	1.20	1.296	16.04
4	0.83	1.96	1.20	1.952	24.16
5.2	0.60	1.99	1.10	1.313	16.26
6.2	0.39	1.67	0.90	0.586	7.25
7	0.61	1.28	0.75	0.586	7.25
7.7	0.70	1.24	0.75	0.651	8.06
8.5	0.55	1.72	1.00	0.946	11.71
9.7	0.50	1.85	1.15	1.064	13.17
10.8	0.40	1.10	1.05	0.462	5.72
11.8	0.26	1.21	1.15	0.362	4.48
13.1	0.03	0.33	0.80	0.008	0.10
13.4	0.00	0.00	0.45	0.000	0.00
14	0.00	0.00	0.00	0.000	0.00
			Measured Dis	**9.372**	cfs

D. Structural Analysis Data

All load cases were calculated using a worst-case scenario span length of 50 feet. Our results are tabulated in the table below. Individual load cases are found after tabulated data.

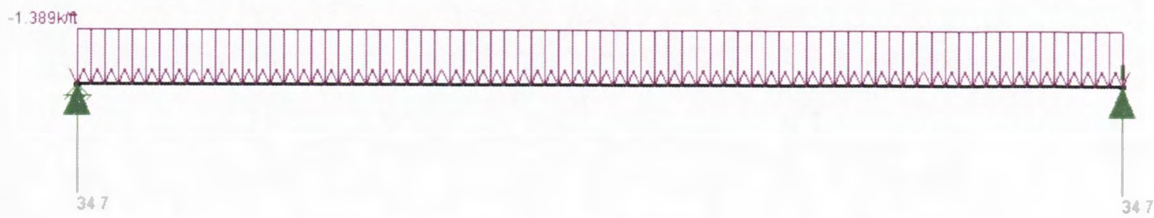
	Rxn Left (kips)	Rxn Right (kips)	Max Shear (kips)	Max Moment (kip-ft)
Case 1	34.7	34.7	34.7	434.1
Case 2	24.1	31.2	31.2	350.4
Case 3	36.3	34.8	34.8	434.9
Case 4	35.5	35.5	35.5	454.1
Case 5	24.9	32	32	368.6

Abs Max Shear (kip)	35.5
Abs Max Moment (kip-ft)	454.1

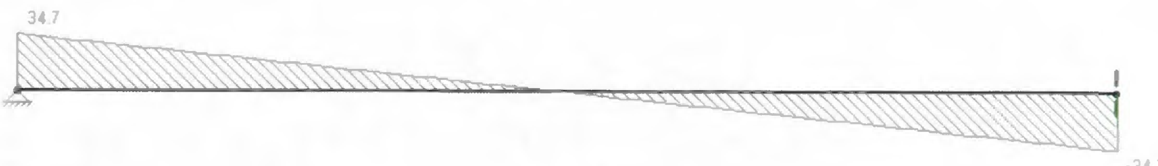
Load Case 1

Loading for D+L+S across entire span.

Loading and Reactions



Shear Diagram



Moment Diagram

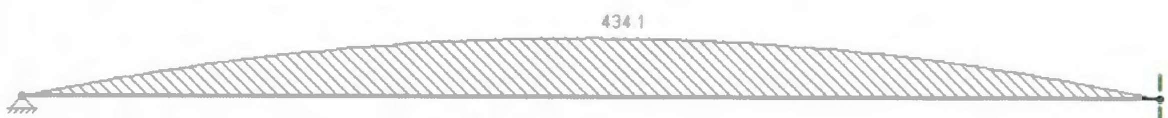
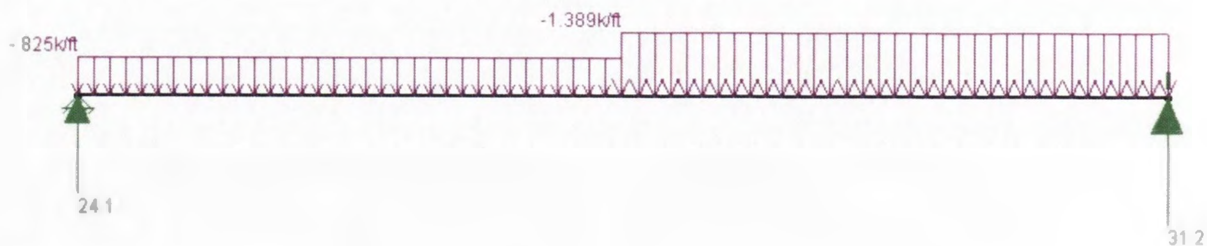


Figure 18. Free Body Diagram, Shear Diagram, Moment Diagram for Load Case 1

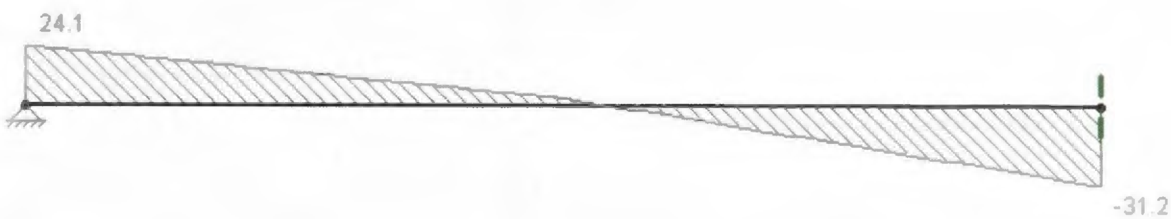
Load Case 2

Loading for D+L on half of span and D+L+S on other half.

Loading and reactions



Shear Diagram



Moment Diagram

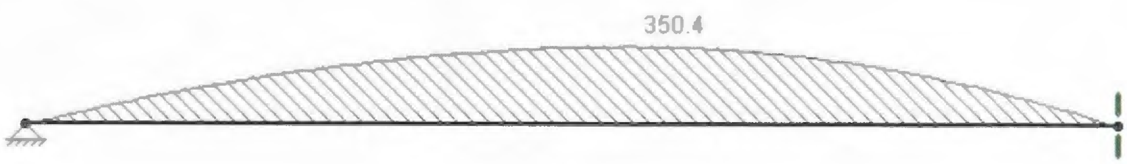
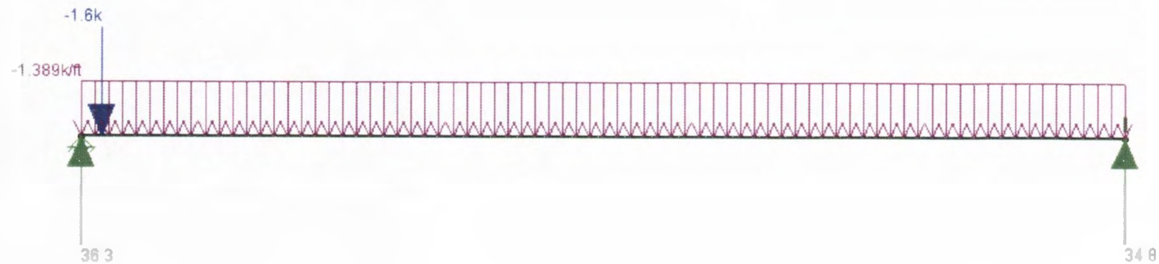


Figure 19. Free Body Diagram, Shear Diagram, Moment Diagram for Load Case 2

Load Case 3

Loading for D+L+S on span and equestrian point load 1.6*EL 1ft from support.

Loading and Reactions



Shear Diagram

Moment Diagrams

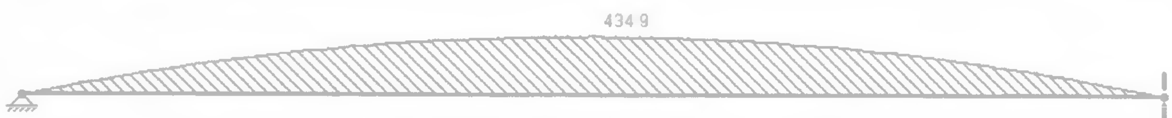
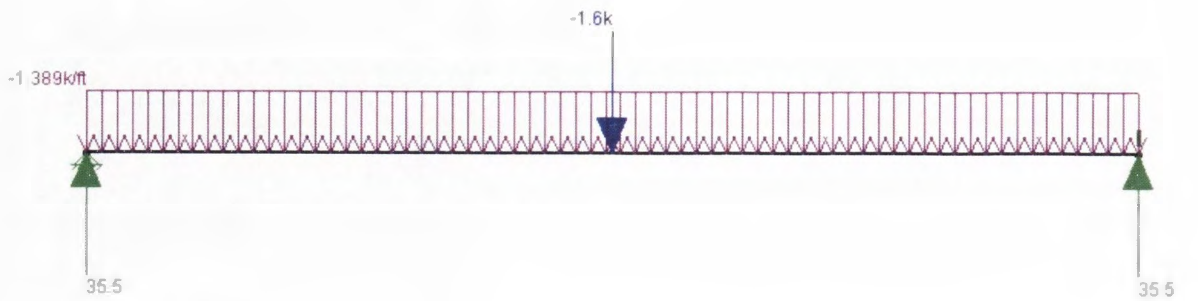


Figure 20. Free Body Diagram, Shear Diagram, Moment Diagram for Load Case 3

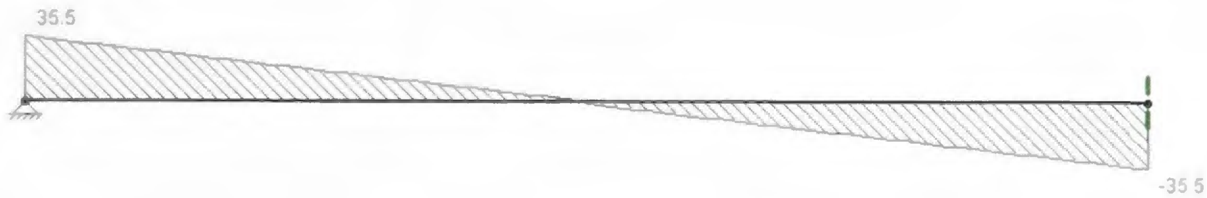
Load Case 4

Loading for D+L+S on span and equestrian point load 1.6*EL at CL.

Loading and Reactions



Shear Diagram



Moment Diagram

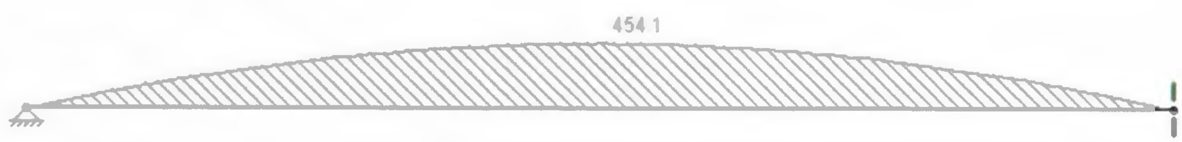


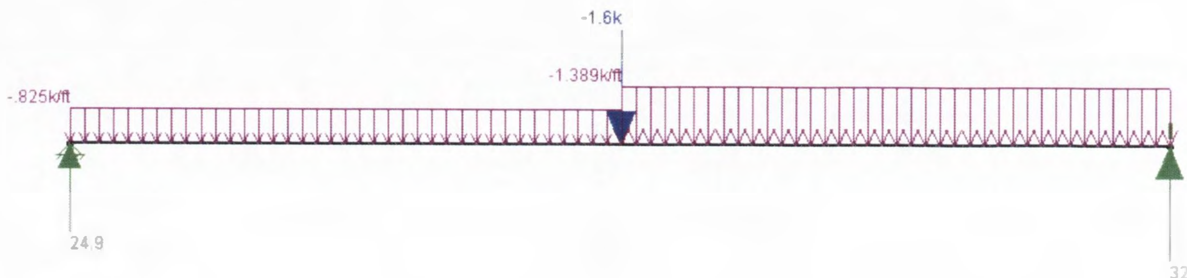
Figure 21. Free Body Diagram, Shear Diagram, Moment Diagram for Load Case 4

Load Case 5

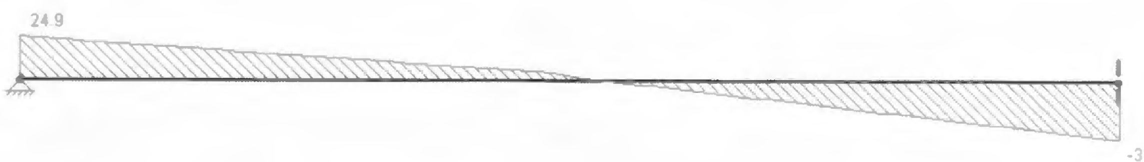
Loading for D+L on half of span, equestrian point load 1.6*EL at CL, and D+L+2*S on other half.

Situation: Snowplow plows half of bridge and sits in middle with other half supporting all snow.

Loading and Reactions



Shear Diagram



Moment Diagram

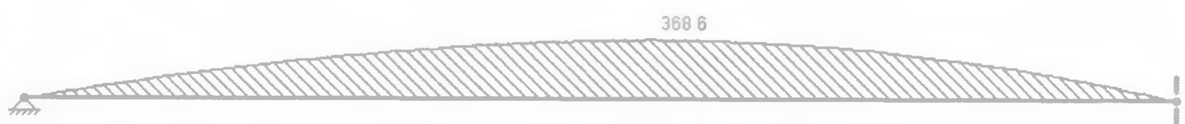


Figure 22. Free Body Diagram, Shear Diagram, Moment Diagram for Load Case 5

APPENDIX III: PHOTOS

The following are additional photos associated with the project that could not reasonably be included in the main body of the proposal. This is the shallow portion of the stream created by the fish barrier that recreationalists sometimes encumber with pallets or other objects to cross the stream.

Figure 23. Fish barrier.

This is a possible location for the new trail bridge located fifty yards west of the current bridge.

Figure 24. Possible bridge location

This image is an additional view of the damage done to the current structure. The timber stringers are subjected to significant torsion, making the structural characteristics unpredictable.

Figure 25. View of current bridge.

This image is an additional view of the shear fracture of the bridge abutment caused by 2011 runoff flooding.

Figure 26. Footing shear failure.

This image is a sample of the stream bed condition of the site. This was helpful in determining an appropriate Manning's coefficient.

Figure 27. View of Stream Bed Conditions

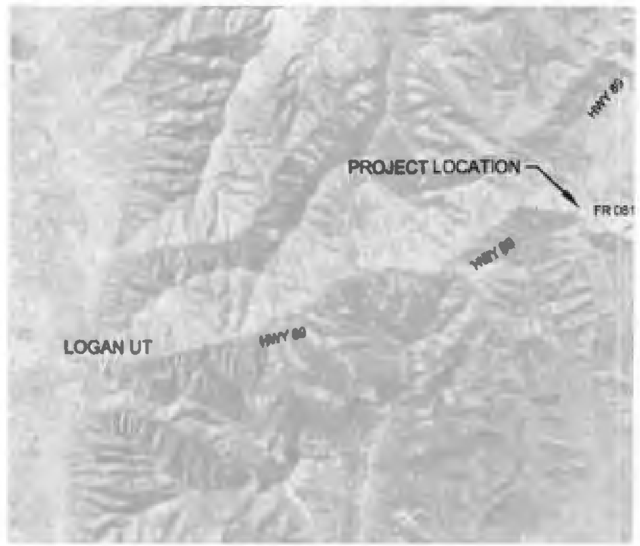
A Total Station was used to determine the topography of the river.

Figure 28. Surveying Equipment

APPENDIX IV: CONSTRUCTION DRAWINGS

UINTA-WASATCH-CACHE NATIONAL FOREST
RIGHT HAND FORK PEDESTRIAN BRIDGE
CACHE COUNTY, UT
2015

PROJECT NO.	SHEET NO.
4880-01	1



VICINITY MAP

Stream Spanners
USU COLLEGE OF ENGINEERING
CEE 4880

APPROVAL

RECOMMENDED FOR APPROVAL:

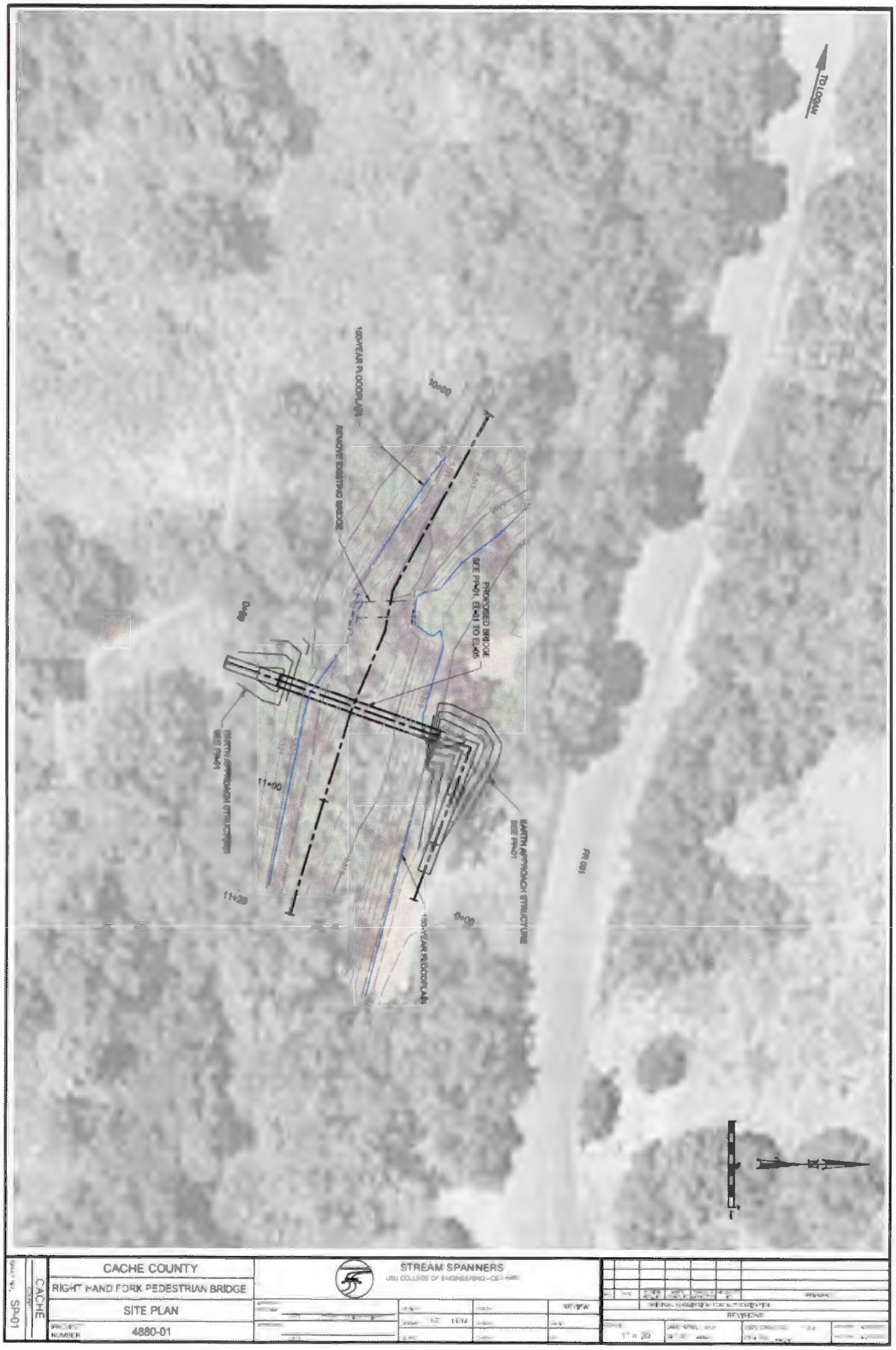
ENGINEER DATE

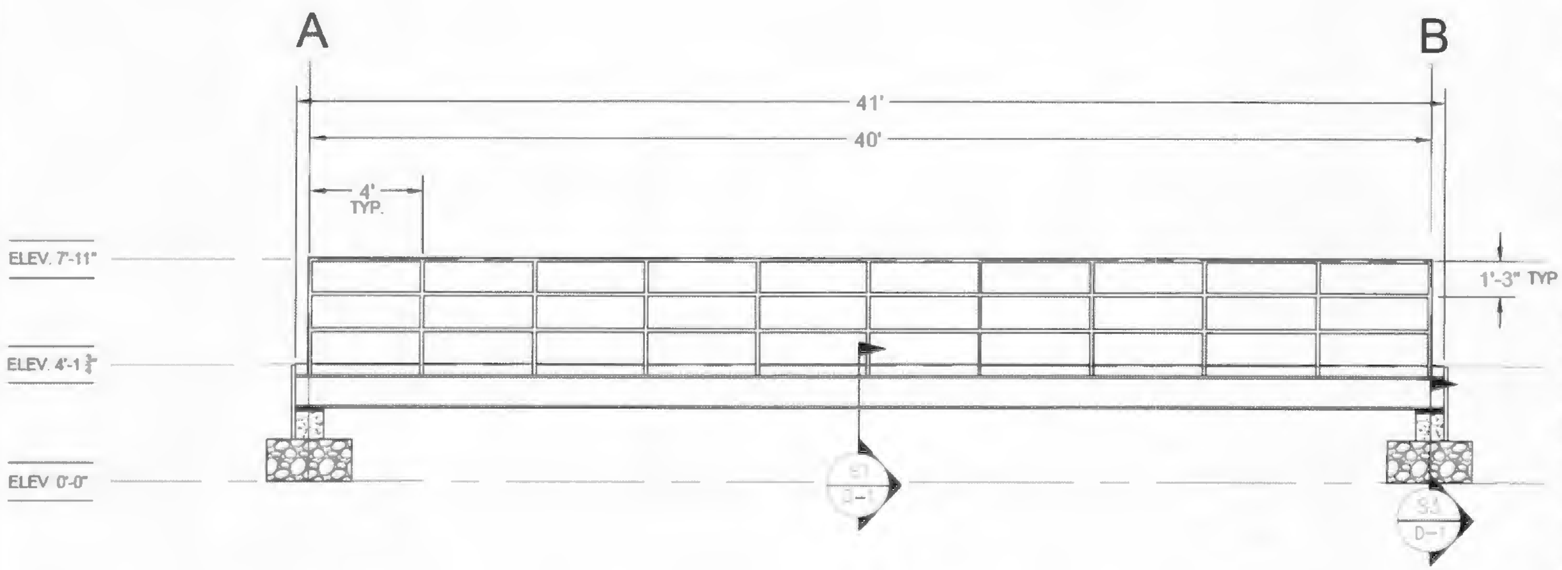
APPROVED:

CLIENT DATE

INDEX TO SHEETS

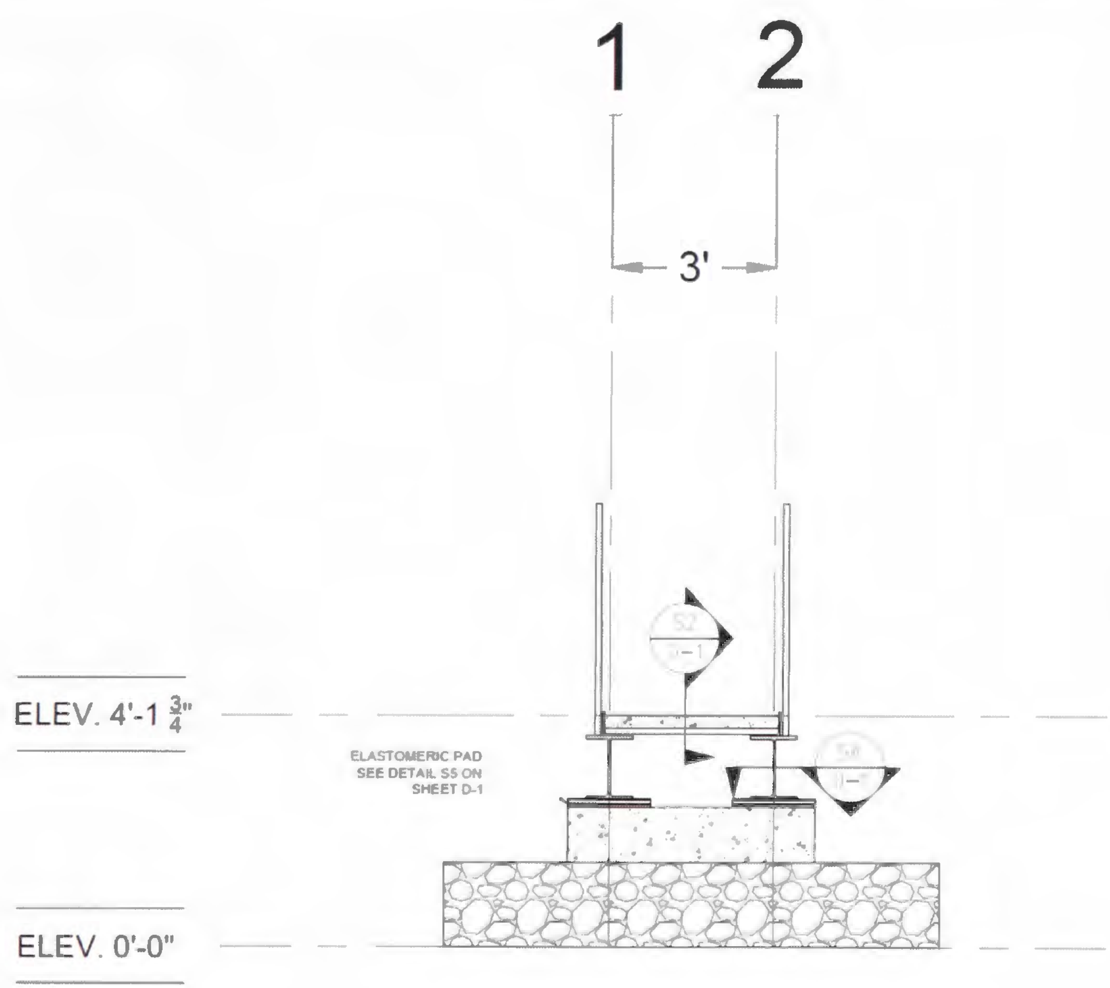
SHEET #	SHEET DESCRIPTION
1	TITLE SHEET
SP-01	SITE PLAN SHEET
PP-01	PLAN AND PROFILE SHEET
EL-01 to EL-06	ELEVATION SHEETS
DT-01 to DT-02	DETAIL SHEETS


TO LOGAN
10+00
100-YEAR FLOODPLAIN
REMOVE EXISTING BRIDGE
PROPOSED BRIDGE
SEE PP-01, EL-01 TO EL-05
EARTH APPROACH STRUCTURE
SEE PP-01
11+00
11+20
CACHE COUNTY
RIGHT HAND FORK PEDESTRIAN BRIDGE
SITE PLAN
PROJECT NUMBER 4880-01
STREAM SPANNERS
USU COLLEGE OF ENGINEERING
CACHE
SP-01
1" = 20'



STEEL BRIDGE ELEVATION - LINE 1
LINE 2 SIMILAR

STREAM SPANNERS
NTS
CACHE COUNTY
RIGHT HAND FORK PEDESTRIAN BRIDGE
ELEVATION SHEET
4880-01
CACHE
EL-01



STEEL BRIDGE ELEVATION - LINE A
LINE B SIMILAR

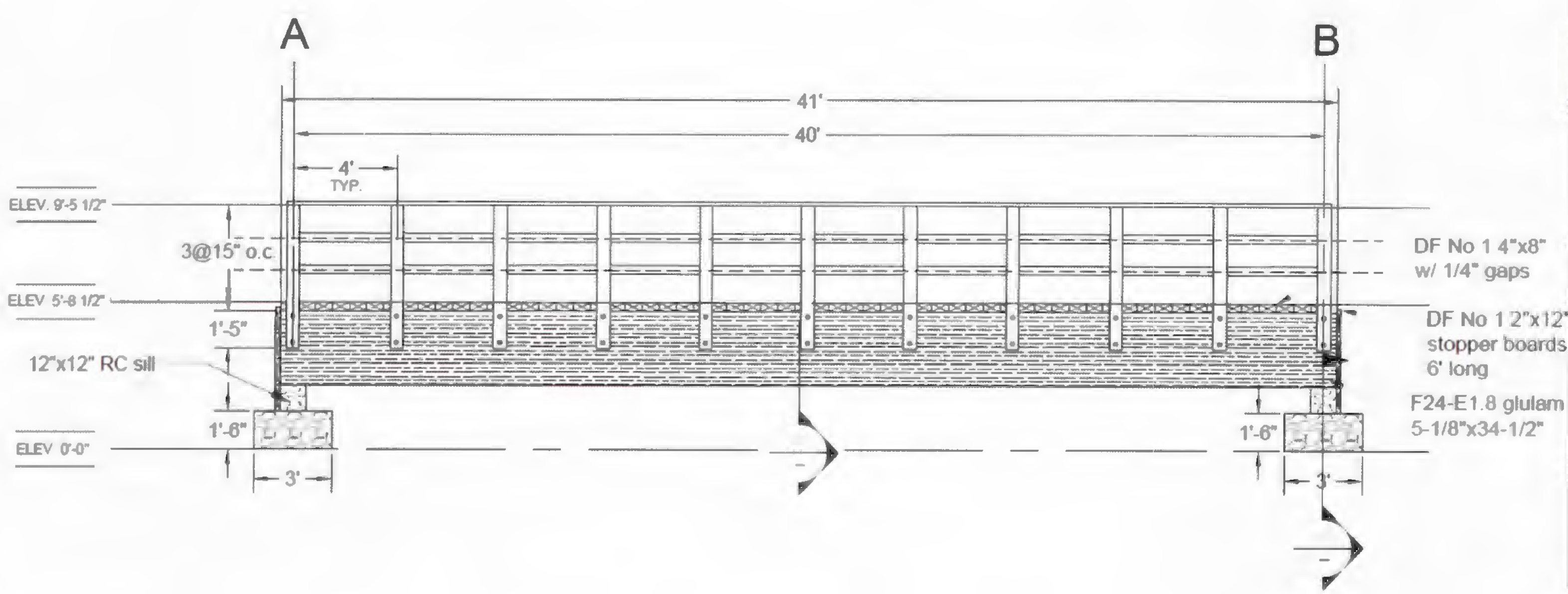
STREAM SPANNERS

CACHE COUNTY
RIGHT HAND FORK PEDESTRIAN BRIDGE
ELEVATION SHEET
4880-01

NTS

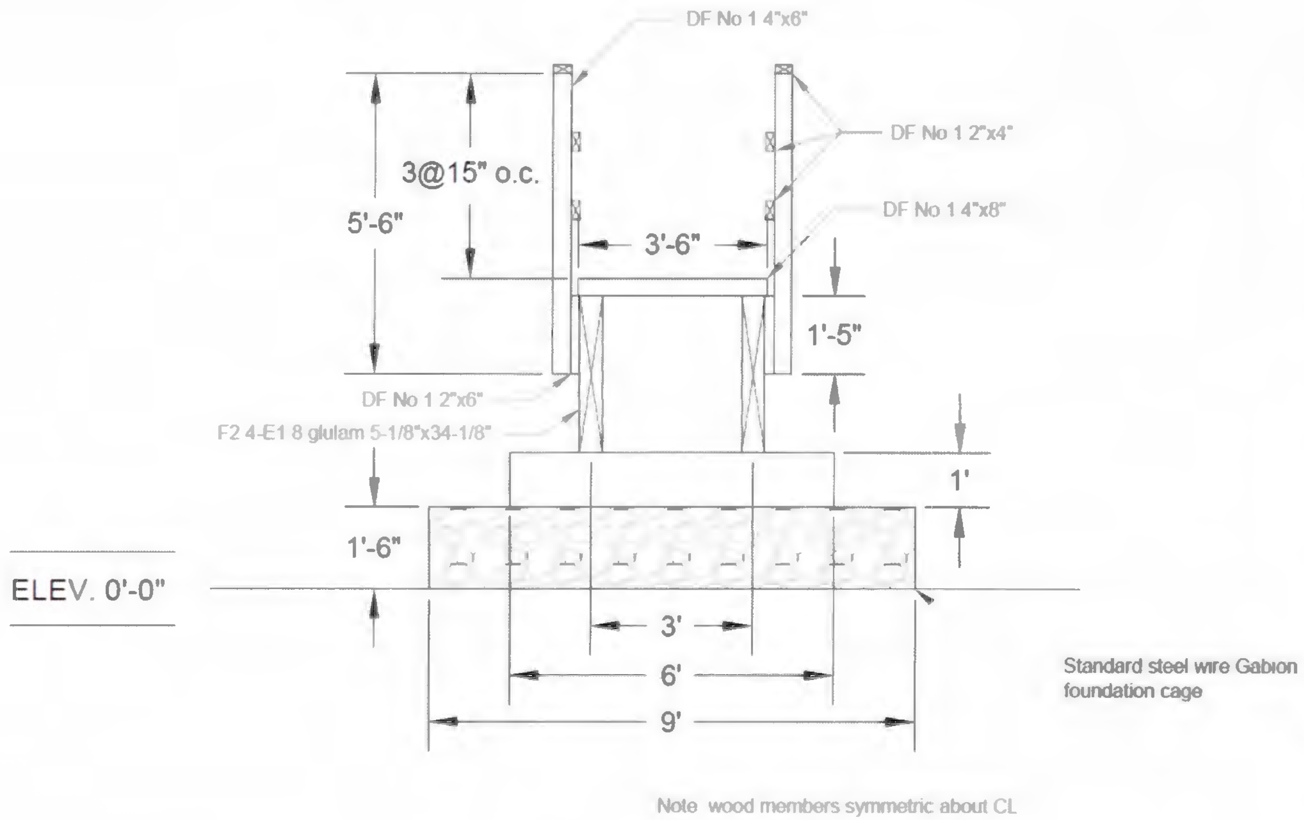
CACHE

EL-02



WOOD BRIDGE ELEVATION - LINE 1
LINE 2 SIMILAR

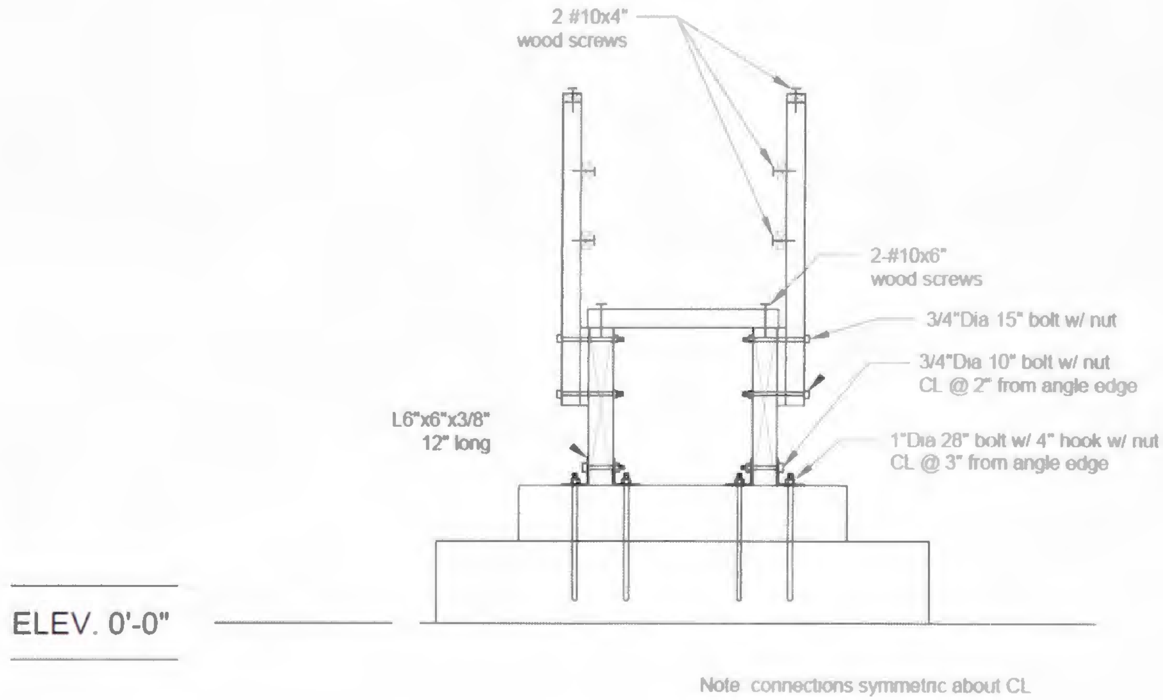
STREAM SPANNERS
NTS
CACHE COUNTY
RIGHT HAND FORK PEDESTRIAN BRIDGE
ELEVATION SHEET
4880-01
CACHE
EL-03



WOOD BRIDGE ELEVATION - LINE A
LINE B SIMILAR

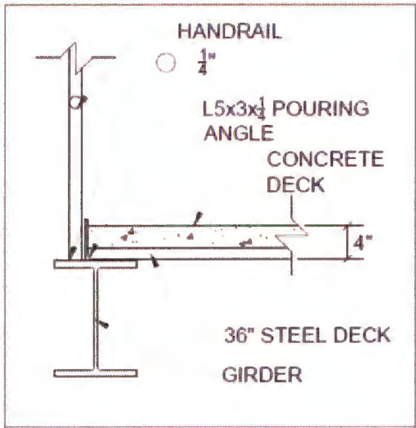
STREAM SPANNERS

CACHE COUNTY
RIGHT HAND FORK PEDESTRIAN BRIDGE
ELEVATION SHEET
4880-01
NTS
CACHE
EL-04

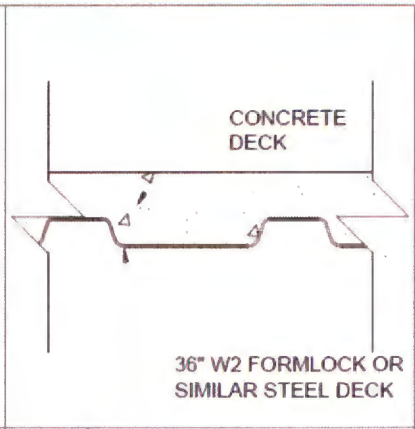


WOOD BRIDGE ELEVATION - LINE A
LINE B SIMILAR

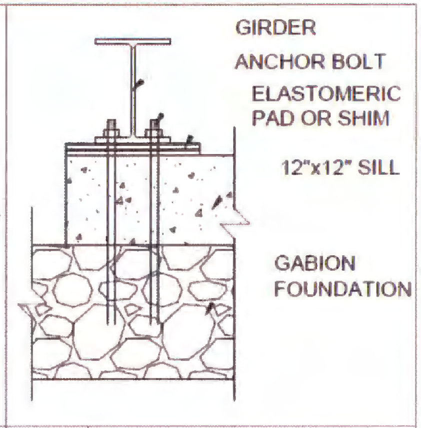
CACHE COUNTY
RIGHT HAND FORK PEDESTRIAN BRIDGE
ELEVATION SHEET
4880-01
STREAM SPANNERS
NTS
CACHE
EL-05



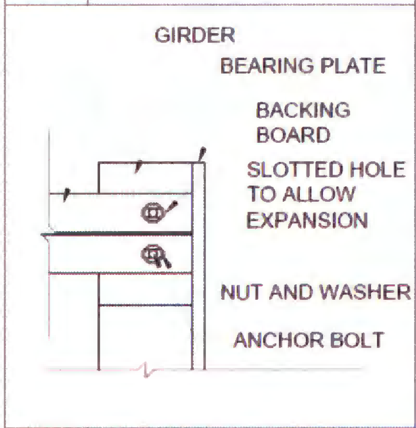
S1 BRIDGE SPAN SECTION CUT



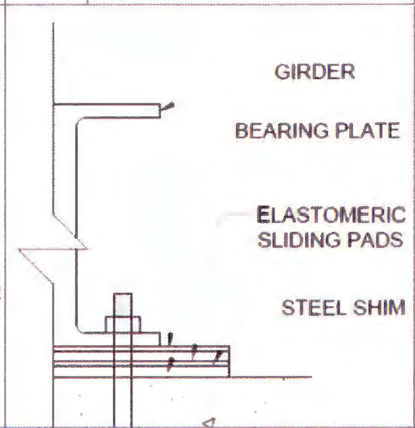
S2 BRIDGE DECK



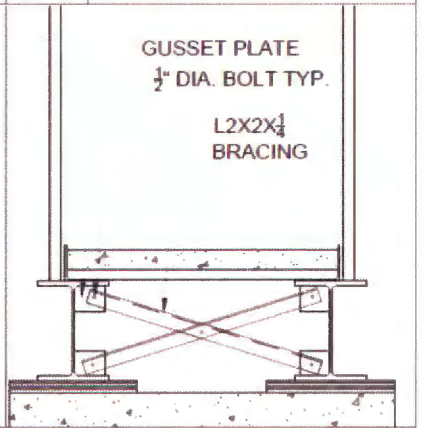
S3 FOUNDATION ATTACHMENT



S4 FOUNDATION CONNECTION



S5 ELASTOMERIC PAD



S6 BRACING DETAIL

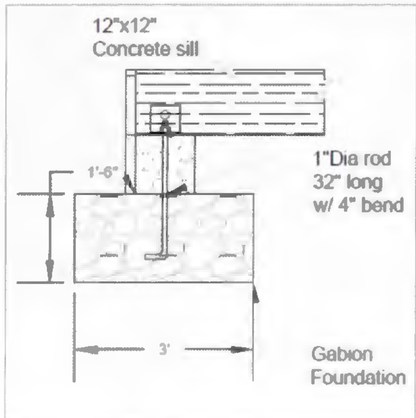
STRUCTURAL DETAILS

STREAM SPANNERS

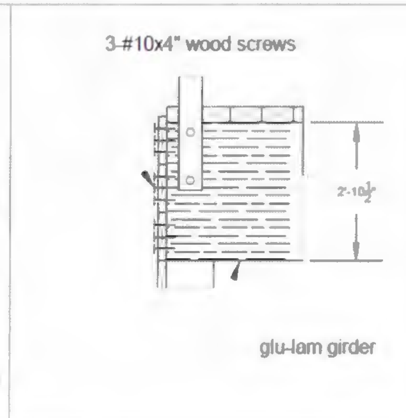
CACHE COUNTY
RIGHT HAND FORK PEDESTRIAN BRIDGE
DETAIL SHEET
4880-01

NTS

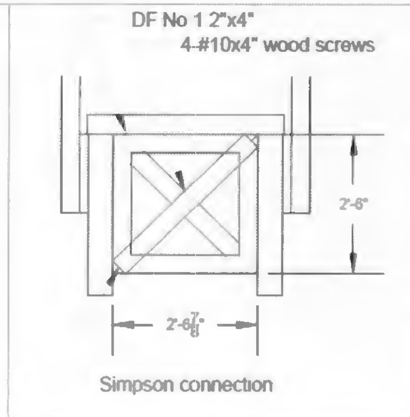
CACHE COUNTY
DT-01



W1 GABION FOUNDATION



W2 WOOD ABUTMENT BACKING



W3 DIAPHRAM

STRUCTURAL DETAILS

STREAM SPANNERS

CACHE COUNTY

RIGHT HAND FORK PEDESTRIAN BRIDGE

DETAIL SHEET

4880-01

NTS

CACHE

DT-02

APPENDIX V: DETAILED CALCULATIONS

A. Travel reimbursement

Cost per mile = $0.56/mile

Miles to date = 9 trips*20 miles = 180 miles

$$Current\ Reimbursement = cost\ per\ mile * miles$$

$$= \$0.56 * 180 = \$\mathbf{100.80}$$

B. Bridge Costs

Table 19: Bridge cost estimates.

Steel Option:						
Member:	Number/Length:		Pricing:		Weight/Area/Vol:	Cost
Bridge Girders:	2 W14x61 @	41	$ 0.75	per pound	5002	$ 3,751.50
Handrail:	2 @	40	$ 58.00	per foot	NA	$ 4,640.00
Perimeter Angle:	2 L5x3x1/4 @	41	$ 0.75	per pound	574	$ 430.50
Bracing Angle:	2 L5x3x1/4 @	10	$ 0.75	per pound	140	$ 105.00
Concrete Deck:	4" x 3.5' Slab @	41	$ 100.00	per yd^3	1.77	$ 177.14
Anchor Bolts:	16 @	2	$ 6.00	per piece	NA	$ 96.00
Decking:	14 pieces @	3.5 "	$ 1.50	per ft^2	147	$ 220.50
						$ 9,420.64

Wood Option:						
Member:	Number/Length:		Pricing:		Weight/Area/Length:	Cost
Bridge Girders:	2 - 36" Glulam @	41	$ 40.00	per foot	82	$ 3,280.00
Decking:	62 4x8 @	3.5	$ 8.50	per foot	217	$ 1,844.50
Posts:	22 - 4x6 Posts @	5	$ 6.25	per foot	110	$ 687.50
Railing:	4 2x4 Rails @	41	$ 2.25	per foot	164	$ 369.00
Hardware:	44 3/4" Bolts @	16	$ 12.90	per piece	44	$ 567.60
Hardware 2:	10 3/4" Bolts @	10	$ 7.35	per piece	10	$ 73.50

Screws	132 Screws @	4	$ 12.15	per 50	3	$ 36.45
Screws 2:	248 screws @	6	$ 17.75	per 50	6	$ 106.50
Angle	6x6x3/8 @	8	$ 1.75	per lb	64	$ 112.00
Anchor Bolts:	16 @	2	$ 6.00	per piece	NA	$ 96.00
						$ 7,173.05

General Costs:					
Item:		Pricing:			Cost:
Concrete Sill:	1x1x4.5	$100	per yd^3	0.167	$ 33.33
Earth Fill:	$150 delivery	$50	per yd^3	39	$ 2,100.00
Gabion Found:	$100 per Cage	$50	per yd^3	6	$ 1,050.00
					$ 3,183.33

Total for Each Option:	
Steel:	$ 12,603.98
Wood:	$ 10,356.38

C. Calculation of Weighted Curve Numbers

Given: 58% of watershed has a curve number of 57, 14% of watershed has a curve number of 73, and 28% of the watershed has a curve number of 81.

$$CN = \frac{58\% * 57 + 14\% * 73 + 28\% * 81}{100\%} = 66$$

D. Structural Analysis

i. Wood

Loading Calculations for a 40-foot span timber bridge

Dead Load

For a one foot section a four foot wide bridge.

Stringers $W_s := 2 \cdot 1\,ft \cdot 3\,ft \cdot \frac{8}{12}\,ft \cdot 30\,\frac{lbf}{ft^3}$

$W_s = 120\,lbf$

Decking $W_d := \left(4\,ft \cdot 1\,ft \cdot \frac{4}{12}\,ft + 3\,ft \cdot 1\,ft \cdot \frac{2}{12}\,ft\right) \cdot 30\,\frac{lbf}{ft^3}$

$W_d = 55\,lbf$

Railing $W_r := \left(5\,ft \cdot \frac{4}{12}\,ft \cdot \frac{6}{12}\,ft + 3 \cdot 1\,ft \cdot \frac{2}{12}\,ft \cdot \frac{6}{12}\,ft\right) \cdot 30\,\frac{lbf}{ft^3}$

$W_r = 32.5\,lbf$

$DL := \frac{\left(W_s + W_d + W_r\right)}{4\,ft \cdot 1\,ft}$ $DL = 51.875\,psf$

Pedestrian Live Load	$PLL := 90\,psf$	AASHTO 3.1
Equestrian Live Load	$ELL := 1\,kip$	AASHTO 3.3
Snow Load	$SL := 150\,psf$	Tbl 1A FS GLStringerBridge

Vehicle, wind, and earthquake loads are trivial compared to the other loads and can be omitted.

Linearize Loads

Dead Load	$DL := DL \cdot 4\,ft$	$DL = 207.5\,\frac{lbf}{ft}$
Pedestrian Live Load	$PLL := PLL \cdot 4\,ft$	$PLL = 360\,\frac{lbf}{ft}$
Equestrian Live Load		
Snow Load	$SL := SL \cdot 4\,ft$	$SL = 600\,\frac{lbf}{ft}$

Loading Combinations

1.4*D $\quad$ $1.4\ DL = 290.5\ \frac{lbf}{ft}$

1.2*D + 1.6*L + 0.5*(S or R) $\quad$ $1.2\ DL + 1.6\ PLL + 0.5\ SL = 1125\ \frac{lbf}{ft}$

1.2*D + 1.6*L $\quad$ $1.2\ DL + 1.6\ PLL = 825\ \frac{lbf}{ft}$

1.2*D + 1.6*(S or R) + 0.5*(L or W) $\quad$ $1.2\ DL + 1.6\ SL + 0.5\ PLL = 1389\ \frac{lbf}{ft}$

Therefore, use $\quad$ $w_u := 1389\ \frac{lbf}{ft}$

Case 5 Loading (with double snow load due to plow)

1.2*D + (2)*1.6*(S or R) + 0.5*(L* or W) $\quad$ $1.2\ DL + 2\ 1.6\ SL + 0.5\ PLL = 2349\ \frac{lbf}{ft}$

For the loading cases analyzed, the following reactions, max shears, and max moments are obtained.

40 ft span				
Timber Alternative				
Load Case	**Rxn 1 (kips)**	**Rxn 2 (kips)**	**Max shear (kips)**	**Max Moment (kip-ft)**
1	27.8	27.8	27.8	277.8
2	19.3	25	25	224.3
3	29.3	27.8	29.3	278.6
4	28.6	28.6	28.6	293.8
5	20.1	25.8	25.8	238.8

Abs Max Shear	29.3	kips
Abs Max Moment	293.8	kip-ft

Example of caluculations for Load Case 1

Reactions on each side of bridge, considering wu distributed evenly across the top

Length of span $\quad$ $L := 40\ ft$

$R_y := \frac{w_u\ L}{2}$ $\quad$ $R_y = 27.78\ kip$

Maximum shear

$V_{max} := R_y$ $\quad$ $V_{max} = 27.78\ kip$

Maximum moment at midspan

$M_{max} := \frac{w_u\ L^2}{8}$ $\quad$ $M_{max} = 277.8\ kip\ ft$

Use maximum absolute reaction of all load cases to find ultimate load on foundation.

$$F_{ymax} = 1[illegible] \, kip$$

The ultimate bearing pressure for a [illegible]ftX[illegible]ft gabion foundation is

$$F_u = \frac{F_{ymax}}{[illegible] \, ft \cdot [illegible] \, ft} \qquad F_u = [illegible] \, \frac{kip}{ft^2}$$

ii. Steel

Loading Calculations for a 40-foot span steel bridge

Dead Load - 2 W14x48 girders

For a one foot section a four foot wide bridge.

Stringers

$W_s = 2 \cdot 48 \dfrac{lbf}{ft} \cdot 1\ ft$

$W_s = 96\ lbf$

Assume a maximum moment of 350 kip-ft to be supported by two girders. From the AISC Manual Table 3-10, section W14x48 has an allowable moment of 196 kip-ft with an unbraced length of 20 ft. Two W14x48 girders have an allowable moment of 392 kip-ft.

$E := 29000\ ksi$

$I_x := 484\ in^4$

Decking

Assume 2 inch metal deck with 2.25 inches of concrete for an average depth of 3.25 inches.

$W_d := 1\ ft \cdot 4\ ft \cdot \dfrac{3.25}{12}\ ft \cdot 150 \dfrac{lbf}{ft^3}$

$W_d = 162.5\ lbf$

Railing $W_r := 150\ lbf$ Assumption

$DL := \dfrac{\left(W_s + W_d + W_r\right)}{4\ ft \cdot 1\ ft}$ $DL = 102.125\ psf$

Pedestrian Live Load	$PLL := 90\ psf$	AASHTO 3.1
Equestrian Live Load	$ELL := 1\ kip$	AASHTO 3.3
Snow Load	$SL := 150\ psf$	Tbl 1A FS GLStringerBridge

Vehicle, wind, and earthquake loads are trivial compared to the other loads and can be omitted.

Linearize Loads

Dead Load	$DL := DL \cdot 4\ ft$	$DL = 408.5 \dfrac{lbf}{ft}$
Pedestrian Live Load	$PLL := PLL \cdot 4\ ft$	$PLL = 360 \dfrac{lbf}{ft}$
Equestrian Live Load	n/a	
Snow Load	$SL := SL \cdot 4\ ft$	$SL = 600 \dfrac{lbf}{ft}$

Loading Combinations

1.4*D	$1.4 \cdot DL = 571.9 \dfrac{lbf}{ft}$
1.2*D + 1.6*L + 0.5*(S or R)	$1.2 \cdot DL + 1.6 \cdot PLL + 0.5 \cdot SL = 1366.2 \dfrac{lbf}{ft}$
1.2*D + 1.6*L	$1.2 \cdot DL + 1.6 \cdot PLL = 1066.2 \dfrac{lbf}{ft}$
1.2*D + 1.6*(S or R) + 0.5*(L or W)	$1.2 \cdot DL + 1.6 \cdot SL + 0.5 \cdot PLL = 1630.2 \dfrac{lbf}{ft}$

Therefore, use $w_u := 1630.2 \frac{lbf}{ft}$

$ELL := ELL \cdot 1.6$

$ELL = 1.6\ kip$

Case 5 Loading (with double snow load due to plow)

1.2*D + (2)*1.6*(S or R) + 0.5*(L* or W) $1.2 \cdot DL + 2 \cdot 1.6 \cdot SL + 0.5 \cdot PLL = 2590.2 \frac{lbf}{ft}$

Example of caluculations for Load Case 1

Reactions on each side of bridge, considering wu distributed evenly across the top

Length of span $L := 40\ ft$

$R_y := \frac{w_u \cdot L}{2}$ $R_y = 32.604\ kip$

Maximum shear

$V_{max} := R_y$ $V_{max} = 32.604\ kip$

Maximum moment at midspan

$M_{max} := \frac{w_u \cdot L^2}{8}$ $M_{max} = 326.04\ kip\ ft$

Example of caluculations for Load Case 4

Reactions on each side of bridge, considering wu distributed evenly across the top and a 1.0-kip equestrian load at the midspan.

Length of span $L := 40\ ft$

$R_y := \frac{w_u \cdot L}{2} + \frac{ELL}{2}$ $R_y = 33.404\ kip$

Maximum shear

$V_{max} := R_y$ $V_{max} = 33.404\ kip$

Maximum moment at midspan

$M_{max} := \frac{w_u \cdot L^2}{8} + \frac{ELL \cdot L}{4}$ $M_{max} = 342.04\ kip\ ft$

Check deflection serviceability limit states according to AASHTO Design of Pedestrian Bridge 2009 5.

Live load

$$\Delta_{LL} < \frac{L}{360}$$

$$\Delta_{LL} = \frac{5 \cdot PLL \cdot L^4}{384 \cdot E \cdot I_x} + \frac{1.0\ kip \cdot L^3}{48 \cdot E \cdot I_x}$$

$$\Delta_{LL} = 1.6415\ in \quad > \quad \frac{L}{360} = 1.3333\ in$$

Therefore, 2 W14x48 are not acceptable according to servicibility limit states.

Try a larger section.

Dead Load - 2 W14x61 girders

For a one foot section a four foot wide bridge.

Stringers

$$W_s = 2 \cdot 61 \frac{lbf}{ft} \cdot 1\ ft$$

$$W_s = 122\ lbf$$

Assume a maximum moment of 350 kip-ft to be supported by two girders. From the AISC Manual Table 3-10, section W14x61 has an allowable moment of 297 kip-ft with an unbraced length of 20 ft. Two W14x61 girders have an allowable moment of 594 kip-ft.

$$E = 29000\ ksi$$

$$I_x = 640\ in^4$$

Decking

Assume 2 inch metal deck with 2.25 inches of concrete for an average depth of 3.25 inches.

$$W_d = 1\ ft \cdot 4\ ft \cdot \frac{3.25}{12}\ ft \cdot 150 \frac{lbf}{ft^3}$$

$$W_d = 162.5\ lbf$$

Railing $W_r = 150\ lbf$ Assumption

$$DL := \frac{\left(W_s + W_d + W_r\right)}{4\ ft \cdot 1\ ft} \qquad DL = 108.625\ psf$$

Pedestrian Live Load	$PLL = 90\ psf$	AASHTO 3.1
Equestrian Live Load	$ELL = 1\ kip$	AASHTO 3.3
Snow Load	$SL = 150\ psf$	Tbl 1A FS GLStringerBridge

Vehicle, wind, and earthquake loads are trivial compared to the other loads and can be omitted.

Linearize Loads

Dead Load	$DL := DL \cdot 4\ ft$	$DL = 434.5\ \frac{lbf}{ft}$
Pedestrian Live Load	$PLL := PLL \cdot 4\ ft$	$PLL = 360\ \frac{lbf}{ft}$
Equestrian Live Load		
Snow Load	$SL := SL \cdot 4\ ft$	$SL = 492.126\ \frac{1}{m}\ \frac{lbf}{ft}$

Loading Combinations

1.4*D	$1.4 \cdot DL = 608.3\ \frac{lbf}{ft}$
1.2*D + 1.6*L + 0.5*(S or R)	$1.2 \cdot DL + 1.6 \cdot PLL + 0.5 \cdot SL = 1397.4\ \frac{lbf}{ft}$
1.2*D + 1.6*L	$1.2 \cdot DL + 1.6 \cdot PLL = 1097.4\ \frac{lbf}{ft}$
1.2*D + 1.6*(S or R) + 0.5*(L or W)	$1.2 \cdot DL + 1.6 \cdot SL + 0.5 \cdot PLL = 1661.4\ \frac{lbf}{ft}$

Therefore, use $w_u := 1661.4\ \frac{lbf}{ft}$

$ELL := ELL \cdot 1.6$

$ELL = 1.6\ kip$

Case 5 Loading (with double snow load due to plow)

1.2*D + (2)*1.6*(S or R) + 0.5*(L* or W) $\quad 1.2 \cdot DL + 2 \cdot 1.6 \cdot SL + 0.5 \cdot PLL = 2621.4\ \frac{lbf}{ft}$

Example of caluculations for Load Case 1

Reactions on each side of bridge, considering wu distributed evenly across the top

Length of span $\quad L := 40\ ft$

$R_y := \frac{w_u \cdot L}{2} \qquad R_y = 33.228\ kip$

Maximum shear

$V_{max} := R_y \qquad V_{max} = 33.228\ kip$

Maximum moment at midspan

$M_{max} := \frac{w_u \cdot L^2}{8} \qquad M_{max} = 332.28\ kip\ ft$

Example of calculations for Load Case 4

Reactions on each side of bridge, considering wu distributed evenly across the top and a 1.0-kip equestrian load at the midspan.

Length of span $L = 40\ ft$

$$R_y = \frac{w_u \cdot L}{2} + \frac{ELL}{2} \qquad R_y = 34.028\ kip$$

Maximum shear

$$V_{max} = R_y \qquad V_{max} = 34.028\ kip$$

Maximum moment at midspan

$$M_{max} = \frac{w_u \cdot L^2}{8} + \frac{ELL \cdot L}{4} \qquad M_{max} = 348.28\ kip\ ft \quad < \phi Mn = 594\ \text{kip-ft}$$

Therefore, two W14x61 girders have adequate moment capacity.

Check deflection serviceability limit states according to AASHTO Design of Pedestrian Bridges 2009 5.

Live load

$$\Delta_{LL} < \frac{L}{360}$$

$$\Delta_{LL} = \frac{5 \cdot PLL \cdot L^4}{384 \cdot E \cdot I_x} + \frac{1.0\ kip \cdot L^3}{48 \cdot E \cdot I_x}$$

$$\Delta_{LL} = 1.2414\ in \quad < \quad \frac{L}{360} = 1.3333\ in$$

Dead and live load deflection limit states are specificed by the code.

Therefore, the deflection limit state is adequate.

Check shear capacity according to AISC Manual Spec Chapter G (Shear).

$V_{max} = 34.8\ kip$ <- maximum shear with a 1-kip point load acting 1 ft from the support.

Each girder carries half the max shear. Therefore,

$$V_{max} = \frac{1}{2} \cdot V_{max} \qquad V_{max} = 17.4\ kip$$

Girder properties

W14x61 A992 steel

$F_y = 50\ ksi$

$E = 29000\ ksi$

$A_w = 17.9\ in^2$

$ht_w = 30.4$

AISC G2.1a

$h/t_w = 30.4 \qquad 2.24\sqrt{\frac{E}{F_y}} = 53.9464$

Therefore,

$C_v = 1.0$

$\phi = 1.0$

$\phi V_n = \phi\, 0.6\, F_y\, A_w\, C_v \qquad \phi V_n = 5[illegible]7\ kip$

$V_{max} = 17.4\ kip \qquad \phi V_n = 5[illegible]7\ kip$

Therefore, the section has adequate shear capacity.

Therefore, use 2W14x61 girders for the bridge main span.

Use maximum absolute reaction of all load cases to find ultimate load on foundation

$F_{ymax} = 34.[illegible]\ kip$

The ultimate bearing pressure for a [illegible]ftx[illegible]ft caisson foundation is

$F_u = \frac{F_{ymax}}{[illegible]\ ft \cdot [illegible]\ ft} \qquad F_u = 1.[illegible]\ \frac{kip}{ft^2}$

E. Structural Design

i. Wood Design

LRFD timber beam design according to the American Wood Council

For a glulam brige with two girders spanning 40 feet and a diaphram brace at 20 feet.

The basic moment design of bending members is (M3.3):

$M' \geq M$

M' = adjusted moment capacity = Fb'*S

M = bending moment

The basic moment design of shear members is (M3.4):

$V' \geq V$

V' = adjusted shear capacity = 2/3*(Fv')*A

V = shear force

From the load combinations, the maximum moment and shear are:

$M = 293.8$ kip-ft

$V = 29.3$ kips

But, this is a two-girder system, so each system must only resist half the shear and moment. Therefore,

$M = \frac{M}{2}$ $\qquad$ $M = 146.9$ kip-ft

$V = \frac{V}{2}$ $\qquad$ $V = 14.65$ kips

The adjusted shear and moment stress equations are (M5.3):

$F_{b'} = F_b \cdot C_{Mb} \cdot C_t \cdot C_L \cdot C_v \cdot C_{fu} \cdot C_c \cdot C_I \cdot 2.54 \cdot 0.85 \cdot \lambda$

$F_{v'} = F_v \cdot C_{Mv} \cdot C_t \cdot 2.70 \cdot 0.80 \cdot \lambda$

Use Douglas Fir-Larch timber 24F-1.8E. From Table 5a,

$F_b = 2400$ psi

$F_v = 265$ psi

$E_{min} = 0.95 \cdot 10^6$ psi

$E = 1.8 \cdot 10^6$ psi

The length of the span is:

$L = 40$ ft

The cross sectional properties are:

$b = 5.125$ in (where b < 10.75 in)

$d = 34.5$ in input

The corresponding area and section modulus taken from Table 1C (Supplement) are:

$A = 176.8$ in^2 input

$S = 1017$ in^3 (about the strong axis)

$I = 17540 \quad in^4$ (about the strong axis)

The adjustment factors in equations M5.3 are:

Wet service factor (5A):

$C_{Mb} := 0.8$

$C_{Mv} := 0.875$

Temperature factor (5.3.4):

See Table 2.3.3

$C_t := 0.8$ (dry, 100F < T < 125F)

Flat use factor (5.3.7):

$C_{fu} := 1.0$ (Girders are placed such that lamination joints are horizontal)

Curvature factor (5.3.8):

$C_c := 1.0$

Stress interaction factor (5.3.9):

$C_I := 1.0$

Beam Stability Factor (5.3.5):

Choose the smaller of C_L and C_v

See section 3.3.3

$l_u := 20 \cdot 12$ in (Girders braced at mid-span)

$l_u = 240$ in

$\frac{l_u}{d} = 6.9565 < 7$ check Therefore, $l_e := 2.06\, l_u$

$l_e = 494.4$ in

$R_B := \sqrt{\frac{l_e \cdot d}{b^2}}$ $\qquad R_B = 25.4833$

$F_{bE} := \frac{1.20\, E_{min}}{R_B^{\,2}}$ $\qquad F_{bE} = 1755.4766$ psi

$F_{bstar} := F_b\, C_{Mb}\, C_c\, C_I$ $\qquad F_{bstar} = 1920$ psi

$$C_L := \frac{\left(1 + \frac{F_{bE}}{F_{bstar}}\right)}{1.9} - \sqrt{\left(\frac{\left(1 + \frac{F_{bE}}{F_{bstar}}\right)}{1.9}\right)^2 - \frac{F_{bE}}{0.95\, F_{bstar}}}$$ $\qquad C_L = 0.778$

Volume factor (5A):

$L = 40$ ft

$d = 34.5$ in

$b = 5.125$ in

$x = 10$ (For all species but Southern Pine)

$$C_V = \left(\frac{21}{L}\right)^{\frac{1}{x}} \cdot \left(\frac{12}{d}\right)^{\frac{1}{x}} \cdot \left(\frac{5.125}{b}\right)^{\frac{1}{x}} \qquad C_V = 0.8436 < 1.0$$

Choose the smaller of the volume factor or the stability factor.

$$C_{VL} = \min\left(\left(C_V \; C_L\right)\right) \qquad C_{VL} = 0.778$$

Time effect factor (2.3.7):

See N.3.3

$\lambda = 0.8$

The adjusted shear and moment stress are:

$$F_{b'} = F_b \cdot C_{Mb} \cdot C_t \cdot C_{VL} \cdot C_{fu} \cdot C_c \cdot C_I \cdot 2.54 \cdot 0.85 \cdot \lambda \qquad F_{b'} = 2063.9982 \text{ psi}$$

$$F_{v'} = F_v \cdot C_{Mv} \cdot C_t \cdot 2.70 \cdot 0.80 \cdot \lambda \qquad F_{v'} = 320.544 \text{ psi}$$

The factored moment is:

$$M' = \frac{F_{b'} \cdot S}{12 \cdot 1000}$$

$M' = 174.9238$ kip-ft $>$ $M = 146.9$ kip-ft

Ok.

The factored shear is:

$$V' = \frac{\frac{2}{3} \cdot F_{v'} \cdot A}{1000}$$

$V' = 37.7815$ kip $>$ $V = 14.65$ kip-ft

Ok.

Check deflection serviceability limit states
according to AASHTO Design of Pedestrian Bridge 2009 5.

Live load

$$\Delta_{LL} < \frac{L}{360}$$

$PLL = \frac{360}{12}$ $\qquad$ $PLL = 30$ lb/in $\qquad$ (Linearized live load)

$L = 40 \cdot 12$ $\qquad$ $L = 480$ in $\qquad$ (Span length)

$$\Delta_{LL} = \frac{5 \cdot PLL \cdot L^4}{384 \cdot E \cdot I} + \frac{1000 \cdot L^3}{48 \cdot E \cdot I}$$

$\Delta_{LL} = 0.7298$ < $\frac{L}{360} = 1.3333$ in

Ok.

Check bearing according to NDS

Reaction at support

$R = V \cdot 1000$ $R = 14650$ lbs

$A_{br} = 5.125 \cdot 12$ $A_{br} = 61.5$ in^2

The bearing stress is

$f_{Cb} = \frac{R}{A_{br}}$ $f_{Cb} = 238.2114$ psi

The allowable bearing stress is (from Table 5C, 24F-1.8E)

$F_{Cb} = 650$ psi

The adjusted bearing stress is

$C_{Mbr} := 0.53$ (5A)

$C_t = 0.8$

$C_b = 1.0$

$F_{Cb'} := F_{Cb} \cdot C_{Mbr} \cdot C_t \cdot C_b \cdot 1.67 \cdot 0.90$ $F_{Cb'} = 414.2268$ psi

$F_{Cb'} = 414.2268$ psi > $f_{Cb} = 238.2114$ psi

Ok.

I tried the following girder sizes by inputing the cross sectional dimensions and determined the following.

Try 5.125x24 No good.

Try 5.125x33 No good.

Try 5.125x36 Ok

Try 5.125x34.5 Ok

The calculations shown are for a 5.125" x 34.5" girder.

Handrail design

The handrails shall be designed to withstand a horizontal point load of 200 lb or a distributed load of 50 lbs/ft.

$P = 200$ lb

$w = 50$ lb/ft

Choose a vertical post spacing of 4 ft, so that:

$w \cdot L = 4 \cdot 50 = 200$ ft

The length of a vertical post from the top edge to the bolt in the girder is:

$L = 15 \cdot 3 + 4 + 2.5$ in $\qquad L = 51.5$ in

The maximum shear and moment from loading are:

$M = P \cdot L \qquad MM = 10300$ lb-in

$V = P \qquad VV = 200$ lb

The basic moment design of bending members is (M3.3):

$M' \geq M$

M' = adjusted moment capacity = Fb' * S

M = bending moment

The basic moment design of shear members is (M3.4):

$V' \geq V$

V ' = adjusted shear capacity = 2/3 * (Fv') * A

V = shear force

The adjusted shear and moment stress equations are (M4.3):

$F_{b'} = F_b \cdot C_F \cdot 2.54 \cdot 0.85 \cdot \lambda$

$F_{v'} = F_v \cdot 2.88 \cdot 0.75 \cdot \lambda$

Use Double Fir-Larch No. 1 timber. From Table 4A:

$F_b = 1000$ psi

Compression stress perpendicular to grain:

$F_c = 625$ psi

Assume the shear stress is equal to Fc:

$F_v = F_c$

$F_v = 625$ psi

The cross sectional properties are:

$b = 4$ in

$d = 6$ in

The corresponding area and section modulus taken from Table 1B (Supplement) are:

$A = 19.25 \ \ in^2$

$S = 11.23 \ \ in^3$ (about the weak axis)

The adjustment factors in equations M4.3 are:

Size factor (4A):

6-in width and 4-in depth for Fb

$C_F := 1.3$

Time effect factor (4.3.16):

See N.3.3

$\lambda = 0.8$

The adjusted bending and shear stresses are:

$F_b' = F_b \, C_F \, 2.54 \, 0.85 \, \lambda$ $\qquad F_b' = 2245.36 \ \ psi$

$F_v' = F_v \, 2.88 \cdot 0.75 \, \lambda$ $\qquad F_v' = 1080 \ \ psi$

The factored moment is:

$M' = F_b' \cdot S$

$M' = 25215.3928 \ \ lb\text{-}in > M = 10300 \ \ lb\text{-}in$

Ok.

The factored shear is:

$V' = \frac{2}{3} F_v' \, A$

$V' = 13860 \ \ lb > V = 200 \ \ lb$

Ok.

Therefore, a 4"x6" rail post is adequate for the bridge, requiring a total length of

$L = 2.5 + 12 + 2.5 + 4 + 15 \cdot 3$ $\qquad L = 66 \ \ in$

Place 2"x4" bars horizontally at 15 inch vertical increments on the rail post.

ii. Steel Design

Loading Calculations for a 40-foot span steel bridge

Dead Load - 2 W14x48 girders

For a one foot section a four foot wide bridge.

Stringers

$$W_s = 2 \cdot 48 \frac{lbf}{ft} \cdot 1\ ft$$

$$W_s = 96\ lbf$$

Assume a maximum moment of 350 kip-ft to be supported by two girders. From the AISC Manual Table 3-10, section W14x48 has an allowable moment of 196 kip-ft with an unbraced length of 20 ft. Two W14x48 girders have an allowable moment of 392 kip-ft.

$$E = 29000\ ksi$$

$$I_x = 484\ in^4$$

Decking

Assume 2 inch metal deck with 2.25 inches of concrete for an average depth of 3.25 inches.

$$W_d = 1\ ft \cdot 4\ ft \cdot \frac{3.25}{12}\ ft \cdot 150 \frac{lbf}{ft^3}$$

$$W_d = 162.5\ lbf$$

Railing $W_r = 150\ lbf$ Assumption

$$DL = \frac{\left(W_s + W_d + W_r\right)}{4\ ft \cdot 1\ ft} \qquad DL = 102.125\ psf$$

Pedestrian Live Load	$PLL = 90\ psf$	AASHTO 3.1
Equestrian Live Load	$ELL = 1\ kip$	AASHTO 3.3
Snow Load	$SL = 150\ psf$	Tbl 1A FS GLStringerBridge

Vehicle, wind, and earthquake loads are trivial compared to the other loads and can be omitted.

Linearize Loads

Dead Load	$DL = DL \cdot 4\ ft$	$DL = 408.5 \frac{lbf}{ft}$
Pedestrian Live Load	$PLL = PLL \cdot 4\ ft$	$PLL = 360 \frac{lbf}{ft}$
Equestrian Live Load	n/a	
Snow Load	$SL = SL \cdot 4\ ft$	$SL = 600 \frac{lbf}{ft}$

Loading Combinations

1.4*D	$1.4 \cdot DL = 571.9 \frac{lbf}{ft}$
1.2*D + 1.6*L + 0.5*(S or R)	$1.2 \cdot DL + 1.6 \cdot PLL + 0.5 \cdot SL = 1366.2 \frac{lbf}{ft}$
1.2*D + 1.6*L	$1.2 \cdot DL + 1.6 \cdot PLL = 1066.2 \frac{lbf}{ft}$
1.2*D + 1.6*(S or R) + 0.5*(L or W)	$1.2 \cdot DL + 1.6 \cdot SL + 0.5 \cdot PLL = 1630.2 \frac{lbf}{ft}$

Therefore, use $w_u := 1630.2 \frac{lbf}{ft}$

$ELL := ELL \cdot 1.6$

$ELL = 1.6\ kip$

Case 5 Loading (with double snow load due to plow)

1.2*D + (2)*1.6*(S or R) + 0.5*(L* or W) $\quad 1.2 \cdot DL + 2 \cdot 1.6 \cdot SL + 0.5 \cdot PLL = 2590.2 \frac{lbf}{ft}$

Example of caluculations for Load Case 1

Reactions on each side of bridge, considering wu distributed evenly across the top

Length of span $\quad L := 40\ ft$

$R_y := \frac{w_u \cdot L}{2} \qquad R_y = 32.604\ kip$

Maximum shear

$V_{max} := R_y \qquad V_{max} = 32.604\ kip$

Maximum moment at midspan

$M_{max} := \frac{w_u \cdot L^2}{8} \qquad M_{max} = 326.04\ kip\ ft$

Example of caluculations for Load Case 4

Reactions on each side of bridge, considering wu distributed evenly across the top and a 1.0-kip equestrian load at the midspan.

Length of span $\quad L := 40\ ft$

$R_y := \frac{w_u \cdot L}{2} + \frac{ELL}{2} \qquad R_y = 33.404\ kip$

Maximum shear

$V_{max} := R_y \qquad V_{max} = 33.404\ kip$

Maximum moment at midspan

$M_{max} := \frac{w_u \cdot L^2}{8} + \frac{ELL \cdot L}{4} \qquad M_{max} = 342.04\ kip\ ft$

Check deflection serviceability limit states
according to AASHTO Design of Pedestrian Bridge 2009 5.

Live load

$$\Delta_{LL} < \frac{L}{360}$$

$$\Delta_{LL} = \frac{5 \cdot PLL \cdot L^4}{384 \cdot E \cdot I_x} + \frac{1.0\,kip \cdot L^3}{48 \cdot E \cdot I_x}$$

$$\Delta_{LL} = 1.6415\,in \quad > \quad \frac{L}{360} = 1.3333\,in$$

Therefore, 2 W14x48 are not acceptable according to servicibility limit states.

Try a larger section.

Dead Load - 2 W14x61 girders

For a one foot section a four foot wide bridge.

Stringers

$$W_s = 2 \cdot 61 \frac{lbf}{ft} \cdot 1\,ft$$

$$W_s = 122\,lbf$$

$$E = 29000\,ksi$$

$$I_x = 640\,in^4$$

Assume a maximum moment of 350 kip-ft to be supported by two girders. From the AISC Manual Table 3-10, section W14x61 has an allowable moment of 297 kip-ft with an unbraced length of 20 ft. Two W14x61 girders have an allowable moment of 594 kip-ft.

Decking

Assume 2 inch metal deck with 2.25 inches of concrete for an average depth of 3.25 inches.

$$W_d = 1\,ft \cdot 4\,ft \cdot \frac{3.25}{12}\,ft \cdot 150 \frac{lbf}{ft^3}$$

$$W_d = 162.5\,lbf$$

Railing $W_r = 150\,lbf$ Assumption

$$DL := \frac{\left(W_s + W_d + W_r\right)}{4\,ft \cdot 1\,ft} \qquad DL = 108.625\,psf$$

Pedestrian Live Load	$PLL = 90\,psf$	AASHTO 3.1
Equestrian Live Load	$ELL = 1\,kip$	AASHTO 3.3
Snow Load	$SL = 150\,psf$	Tbl 1A FS GLStringerBridge

Vehicle, wind, and earthquake loads are trivial compared to the other loads and can be omitted.

Linearize Loads

Dead Load	$DL = DL \cdot 4\ ft$	$DL = 434.5 \frac{lbf}{ft}$
Pedestrian Live Load	$PLL = PLL \cdot 4\ ft$	$PLL = 360 \frac{lbf}{ft}$
Equestrian Live Load		
Snow Load	$SL = SL \cdot 4\ ft$	$SL = 492.126 \frac{1}{m} \frac{lbf}{ft}$

Loading Combinations

1.4*D	$1.4 \cdot DL = 608.3 \frac{lbf}{ft}$
1.2*D + 1.6*L + 0.5*(S or R)	$1.2 \cdot DL + 1.6 \cdot PLL + 0.5 \cdot SL = 1397.4 \frac{lbf}{ft}$
1.2*D + 1.6*L	$1.2 \cdot DL + 1.6 \cdot PLL = 1097.4 \frac{lbf}{ft}$
1.2*D + 1.6*(S or R) + 0.5*(L or W)	$1.2 \cdot DL + 1.6 \cdot SL + 0.5 \cdot PLL = 1661.4 \frac{lbf}{ft}$

Therefore, use $w_u = 1661.4 \frac{lbf}{ft}$

$ELL = ELL \cdot 1.6$

$ELL = 1.6\ kip$

Case 5 Loading (with double snow load due to plow)

1.2*D + (2)*1.6*(S or R) + 0.5*(L* or W) $1.2 \cdot DL + 2 \cdot 1.6 \cdot SL + 0.5 \cdot PLL = 2621.4 \frac{lbf}{ft}$

Example of caluculations for Load Case 1

Reactions on each side of bridge, considering wu distributed evenly across the top

Length of span $L = 40\ ft$

$R_y = \frac{w_u \cdot L}{2}$ $R_y = 33.228\ kip$

Maximum shear

$V_{max} = R_y$ $V_{max} = 33.228\ kip$

Maximum moment at midspan

$M_{max} = \frac{w_u \cdot L^2}{8}$ $M_{max} = 332.28\ kip \cdot ft$

Example of calculations for Load Case 4

Reactions on each side of bridge, considering wu distributed evenly across the top and a 1.0-kip equestrian load at the midspan.

Length of span $L := 40\,ft$

$R_y := \frac{w_u \cdot L}{2} + \frac{ELL}{2}$ $R_y = 34.028\,kip$

Maximum shear

$V_{max} := R_y$ $V_{max} = 34.028\,kip$

Maximum moment at midspan

$M_{max} := \frac{w_u \cdot L^2}{8} + \frac{ELL \cdot L}{4}$ $M_{max} = 348.28\,kip\,ft$ < φMn = 594 kip-ft

Therefore, two W14x61 girders have adequate moment capacity.

Check deflection serviceability limit states according to AASHTO Design of Pedestrian Bridges 2009 5.

Live load

$\Delta_{LL} < \frac{L}{360}$

$\Delta_{LL} := \frac{5 \cdot PLL \cdot L^4}{384 \cdot E \cdot I_x} + \frac{1.0\,kip \cdot L^3}{48 \cdot E \cdot I_x}$

$\Delta_{LL} = 1.2414\,in$ < $\frac{L}{360} = 1.3333\,in$

Dead and live load deflection limit states are specificed by the code.

Therefore, the deflection limit state is adequate.

Check shear capacity according to AISC Manual Spec Chapter G (Shear).

$V_{max} := 34.8\,kip$ <- maximum shear with a 1-kip point load acting 1 ft from the support.

Each girder carries half the max shear. Therefore,

$V_{max} := \frac{1}{2} \cdot V_{max}$ $V_{max} = 17.4\,kip$

Girder properties

W14x61 A992 steel

$F_y := 50\,ksi$

$E := 29000\,ksi$

$A_w := 17.9\,in^2$

$ht_w := 30.4$

[illegible]

h/t_w = [illegible] $\quad$ [illegible] $\sqrt{\frac{E}{F_y}}$ = [illegible]

Therefore

C_v = [illegible]

ϕ = [illegible]

$\phi V_n = \phi \cdot 0.6 \cdot F_y \cdot A_w \cdot C_v$ $\quad$ ϕV_n = [illegible] kip

V_{max} = [illegible] kip $\quad$ ϕV_n = [illegible] kip

Therefore, the section has adequate shear capacity

Therefore, use [illegible] girders for the bridge main span.

Use maximum absolute reaction of all load cases to find ultimate load on foundation

F_{umax} = [illegible] kip

The ultimate bearing pressure for a [illegible] pylon foundation is

$F_u = \frac{F_{umax}}{[illegible]\,ft \cdot [illegible]\,ft}$ $\quad$ F_u = [illegible] $\frac{kip}{ft^2}$

Lundgreen, Jeffrey

Handrail Calculations Senior Design

According to the Forest Service guidlines a handrail must be 42 inches high.

It must also be able to withstand a 200 lb point load at supports or a distributed load of 50 lb/ft along the entire structure.

$L = 40 \ (ft)$

$w = 50 \ \left(\frac{lb}{ft}\right)$

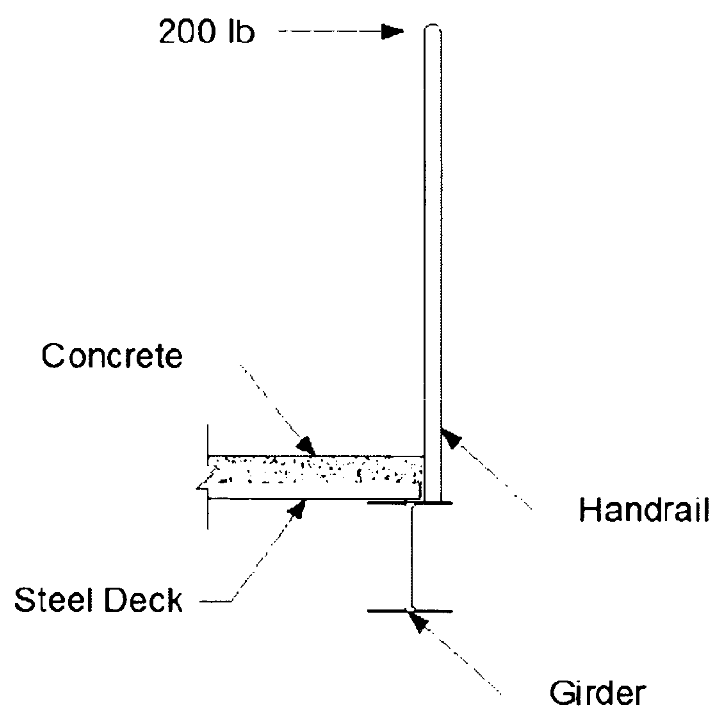
Assuming a support every 4 ft

$s = 10$ number of supports

$\frac{L \cdot w}{s} = 200 \ \left(\frac{lb}{support}\right)$

Therefore we will use a 200 lb force for our analysis and design.

$F = 200 \ (lb)$

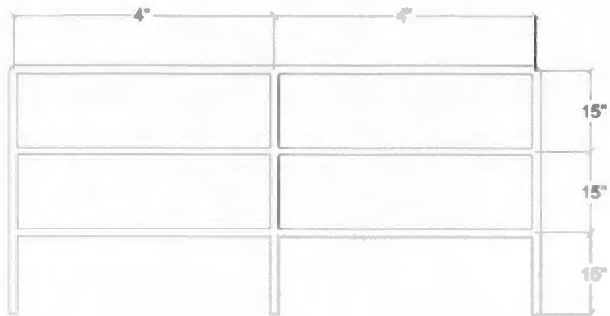


The Handrail must have vertical spacing no larger than 19 inches so in order to create equal spacing we will make our handrail 45 in and space our horizontal bars at 15 in.

Dimensions of steel bar.

$d = 1.5 \quad (\mathrm{in})$

$h = 45 \quad (\mathrm{in})$



To find the shear force that the rail supports need to resist at the weld we will use the following equations. These equations come from the Design considerations for welds section on page 8-12.

Shear caused from reaction force.

$$f_v = \frac{F}{2 \cdot \pi \cdot \frac{d}{2}} \qquad f_v = 42.4413 \left(\frac{\mathrm{lb}}{\mathrm{in}}\right)$$

Force caused from Moment.

$$f_b = \frac{F \cdot h \cdot \frac{d}{2}}{\pi \left(\frac{d}{2}\right)^3} \qquad f_b = 5092.9582 \left(\frac{\mathrm{lb}}{\mathrm{in}}\right)$$

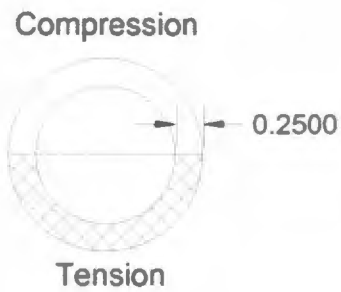
Total force on weld

$$f_t = \sqrt{f_b^{\,2} + f_v^{\,2}} \qquad f_t = 5093.135 \left(\frac{\mathrm{lb}}{\mathrm{in}}\right)$$

Convert to Kips

$$f_t = f_t \frac{1}{1000} \left(\frac{\mathrm{lb}}{\mathrm{kip}}\right) \qquad f_t = 5.0931 \left(\frac{\mathrm{Kip}}{\mathrm{in}}\right)$$

Because the Shear force is so minimal we can treat the weld as if it is all coming from the force caused by the moment. This means that half of the round section will be in compression and half in tension. The half in tension will carry all of the load. See the figure below.



Our force would be resisted by a fillet weld. We need to check both the weld and the base material.

Weld

Due to the load being at 90 degrees from the axis the code allows a 50% increase in capacity as found in the design considerations for welds on page 8-9. The equation used is equation J2-5 in the welds section of section J in the spec.

$r := .75$ (in) $\quad t_{EFF} := \frac{3}{16}$ (in)

$F_{EXX} := 70$ (ksi) $\quad L_{EFF} := \frac{2\,\pi\,r}{2} \quad A_e := L_{EFF}\,t_{EFF}$

$A_e = 0.4418$ (in^2) $\quad \Phi := .75$

$R_{nw} := \Phi\,0.60\,F_{EXX}\,A_e\,1.5$ $\quad \boxed{R_{nw} = 20.8744}$ $\boxed{(kips)}$ $\quad$ (This capacity controlls.)

Base Material

Solving for tensile yielding.

$\Phi := .9 \quad F_y := 36$ (ksi) $\quad A_g := \frac{\pi\,r^2}{2} \quad A_g = 0.8836$ (in^2)

$R_{nb} := \Phi\,F_y\,A_g \quad R_{nb} = 28.6278$ (kips)

Solving for tensile rupture.

$A_e := A_g \quad \Phi := 0.75 \quad F_u := 50$ (ksi)

$R_{nr} := \Phi\,F_u\,A_e \quad R_{nr} = 33.134$ (kips)

Using the lowest capacity and dividing by the length of the weld we get.

$$\frac{R_{nw}}{2 \cdot L_{EFF}} = 4.4[illegible] \quad \left(\frac{Kip}{in}\right) \qquad f_t = 5.0931 \left(\frac{Kip}{in}\right)$$

Therefore we need more weld.

Solve for throat width of the Weld

$$t_w = \frac{f_t}{.6 \cdot .75 \cdot .707 \cdot F_{EXX}} \qquad t_w = 0.2287 \quad (in)$$

Use 1/4 in weld.

$$r = .75 \quad (in) \qquad t_{EFF} = \frac{4}{16} \quad (in)$$

$$F_{EXX} = 70 \quad (ksi) \qquad L_{EFF} = \frac{2 \cdot \pi \cdot r}{2} \qquad A_e = L_{EFF} \cdot t_{EFF}$$

$$A_e = 0.589 \quad \left(in^2\right) \qquad \phi = .75$$

$$R_{nw} = \phi \cdot (.6) \cdot F_{EXX} \cdot A_e \cdot 1.5 \qquad \boxed{R_{nw} = 27.8325} \; (kips)$$

This capacity controls.

$$\boxed{\frac{R_{nw}}{2 \cdot L_{EFF}} = 5.9062} \left(\frac{Kip}{in}\right) \quad \boxed{>} \quad \boxed{f_t = 5.0931} \left(\frac{Kip}{in}\right)$$

Okay use 1/4 in. fillet weld.

Calculations for elastomeric bearing pads

Use 50 duromter Elastomeric Isolators by Kinetics Noise Control

Dimensions:

18"x18"x1/4"

Allows static deflections up to 0.15 in
and a vertical pressure of 60 psi

Thermal expansion

Coefficient of thermal expansion (steel): $\alpha := 6.7 \cdot 10^{-6}$ /oF

Maximum expected temperature variation: $\Delta T := 70 - (-10)$

$\Delta T = 80$ oF

Length of girder: $L := 40 \cdot 12$

$L = 480$ in

$\varepsilon = \alpha \cdot \Delta T$ and $\varepsilon = \frac{\delta}{L}$

Therefore, $\delta = \varepsilon * L = \alpha * \Delta T * L$

$\delta := \alpha \cdot \Delta T \cdot L$

$\delta = 0.2573$ in

Use 2 pads with a steel shim

$0.15 \cdot 2 = 0.3$ in $>$ $\delta = 0.2573$ in ok

Check allowable vertical pressure $\sigma_b := 60$ psi

Maximum reaction (for both girders): $R_{max} := 29.3$ kips

Reactions on one girder: $R_{max} := \frac{R_{max}}{2} \cdot 1000$

$R_{max} = 14650$ lbs

Width of concrete sill: $b_s := 12$ in

Width of W14x61: $b_w := 8$ in

Critical width of elastomeric pad: b_c

The allowable bearing pressure is:

$$\sigma_b = \frac{R_{max}}{A_b} = \frac{R_{max}}{b_s \cdot b_c}$$

Rearranging to find the critical width,

$$b_c = \frac{R_{max}}{b_s \sigma_b}$$ $b_c = 20.3472$ in

The isolators come in 18-in squares,
and 18 in is close to 20.3 in.
Therefore, use pads of dimensions:

12"x18"

The girders will bear on a steel plate of dimensions

12"x18"x1/4"

to distribute the girder load everly over the pad.

F. Foundation Design

Preliminary Foundation Analysis

From Principles of Foundation Engineering, Das, the bearing capacity of a soil on the top of a slope is calculated as follows.

Bearing Capacity of Foundations on Top of a Slope

In some instances, shallow foundations need to be constructed on top of a slope. In Figure 4.14, the height of the slope is H and the slope makes an angle β with the horizontal. The edge of the foundation is located at a distance b from the top of the slope. At ultimate load, q_u, the failure surface will be as shown in the figure.

Meyerhof (1957) developed the following theoretical relation for the ultimate bearing capacity for *continuous foundations*:

$$q_u = cN_{cq} + \tfrac{1}{2}\gamma B N_{\gamma q} \qquad (4.37)$$

For purely granular soil, $c = 0$, thus,

$$q_u = \tfrac{1}{2}\gamma B N_{\gamma q} \qquad (4.38)$$

Again, for purely cohesive soil, $\phi = 0$ (the undrained condition); hence

$$q_u = cN_{cq} \qquad (4.39)$$

where c = undrained cohesion

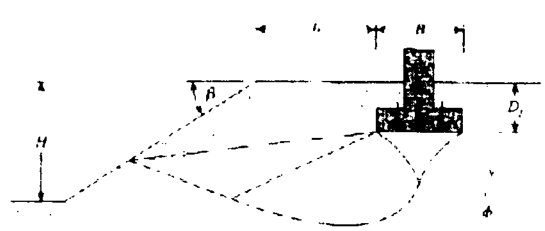


Figure 4.14 Shallow foundation on top of a slope

This is a preliminary analysis, and we have limited data, so we made several assumptions. All assumptions are conservative and account for worst case scenarios.

Side A

Assumptions

Length of span: $L_s = 40\ ft$

Max reaction on footing: $R = 36.3\ kips$???

Footing dimensions: $B = 3\ ft$

$L = 9\ ft$

$D_f = \frac{2}{3}\ ft$

Slope dimensions: $b = \frac{(50 - 35.9)}{2} - 1.5$ $b = 5.55\ ft$ (see hand drawing of elevation view for estimated dimensions)

$b := 5$

$\beta = 40$ degrees

$H = 7\ ft$

Soil properties $\gamma = 115\ pcf$ (assume the soil is purely granular and c'=0)

$c' = 0$

$\varphi = 35$ degrees

$\gamma_w = 62.4\ pcf$

The required pressure acting on the soil from the bridge is

$$q_{req} = \frac{R}{B \cdot L} \qquad q_{req} = 1.3444\ ksf$$

The ultimate bearing capacity on the soil is calculated as follows

$$qu = \frac{1}{2} \cdot \gamma \cdot B \cdot N_{\gamma q}$$

Assume the water table rises to the bottom of the foundation. Therefore, use the buoyant unit weight of soil for calculating bearing capacity.

$$\gamma' = \gamma - \gamma_w \qquad \gamma' = 52.6\ pcf$$

The correction factor Nγq is determined from Figure 4.15.

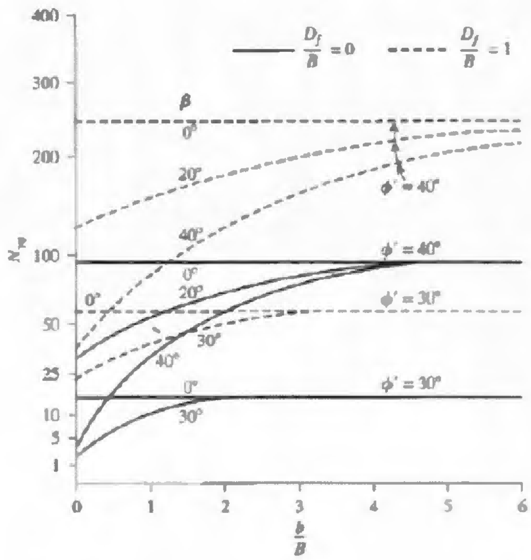


Figure 4.15 Meyerhof's bearing capacity factor $N_{\gamma q}$ for granular soil (c' = 0)

$\frac{b}{B} = 1.6667$

$\frac{D_f}{B} = 0.2222$

$\varphi = 35$

$\beta = 40$

For $\varphi=30$ and $\beta=40$

$N_{\gamma q1} = 15$

For $\varphi=40$ and $\beta=40$

$N_{\gamma q2} = 60$

Taking an average to find the correction factor $\varphi=35$

$N_{\gamma q} = \frac{(N_{\gamma q1} + N_{\gamma q2})}{2}$ $\quad N_{\gamma q} = 37.5$

The ultimate bearing capacity of the soil is

$q_u = \frac{1}{2}\ \gamma'\ B\ N_{\gamma q}\left(\frac{1}{1000}\right)$ $\quad q_u = 2.9588$ ksf

The factor of safety for the footing is

$FS = \frac{q_u}{q_{req}}$ $\quad FS = 2.2007$

Side B

Assumptions

Length of span: $L_s =$ [illegible] ft

Max reaction on footing: $R := 36.3$ kips ???

Footing dimensions: $B =$ [illegible] ft

$L =$ [illegible] ft

$D_f =$ [illegible] ft

Slope dimensions: $d = \frac{[illegible]}{[illegible]}$ [illegible] $d = 5.55$ ft   see hand drawing of elevation view for estimated dimensions

$\beta =$ [illegible] degrees

$H =$ [illegible] ft

Soil properties: $\gamma =$ [illegible] pcf   assume the soil is purely granular and $c' =$ [illegible]

$c' =$ [illegible]

$\phi =$ [illegible] degrees

$\gamma_w =$ [illegible] pcf

The required pressure acting on the soil from the bridge is

$$q_{req} := \frac{R}{B \cdot L} \qquad q_{req} = 1.3444 \text{ ksf}$$

The ultimate bearing capacity of the soil is calculated as follows

$$q_u = \frac{1}{2} \cdot \gamma' \cdot B \cdot N_{\gamma q}$$

Assume the water table rises to the bottom of the foundation. Therefore, use the buoyant unit weight of soil for calculating bearing capacity.

$\gamma' := \gamma - \gamma_w \qquad \gamma' = 52.6$ pcf

The correction factor Nγq is determined from Figure 4.15.

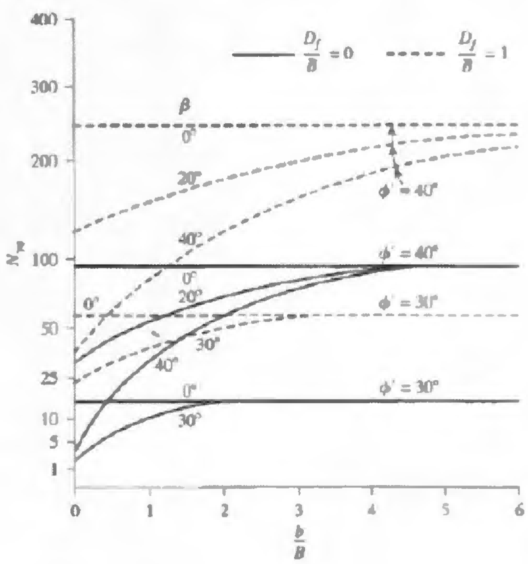


Figure 4.15 Meyerhof's bearing capacity factor for granular soil ($c' =$

$\frac{b}{B} = 1.3333$

$\frac{D_f}{B} = 0.6667$

$\varphi = 30$

$\beta = 30$

For $\varphi=30$ and $\beta=30$

$N_{\gamma q1} = 30$

For $\varphi=40$ and $\beta=30$

$N_{\gamma q2} = 80$

Taking an average to find the correction factor $\varphi=35$

$N_{\gamma q} := \frac{\left(N_{\gamma q1} + N_{\gamma q2}\right)}{2}$ $\qquad N_{\gamma q} = 55$

The ultimate bearing capacity of the soil is

$q_u := \frac{1}{2} \gamma' B N_{\gamma q} \left(\frac{1}{1000}\right)$ $\qquad q_u = 4.3395$ ksf

The factor of safety for the footing is

$FS := \frac{q_u}{q_{req}}$ $\qquad FS = 3.2277$

APPENDIX VI: SPECIAL SUMMARY DOCUMENTATION

CHANGES AS PER PROFESSIONAL MENTOR'S RECOMMENDATIONS

- Our professional mentor, Mark Anderson, is an S.E. and a P.E. We added S.E. to his titles throughout the document.
- In the problem statement, we more appropriately stated the beginning of the construction period "as soon as late summer 2015," rather than our previous statement of "will be constructed beginning in late summer 2015."
- We included the values of the bearing capacity (7500 psf) of the soil in the "Foundation Design" section.
- Under "Evaluation," we explicitly stated that we are using LRFD and AASHTO standards for the new structure.

CONSTRAINTS CONSIDERATIONS SUMMARY

HEALTH AND SAFETY

The current condition of the pedestrian bridge at Right Hand Fork poses risks to pedestrian users. Stream crossings with pallets are also dangerous. The requirements placed on the new bridge will promote safe crossings. The location of the new bridge will be surveyed to provide easy access to the general public. It will be built with materials designed to withstand designated AASHTO LRFD loadings and various adverse weather conditions. The bridge will be built at a suitable height above the river to prevent abutment erosion. The structure will also be accessible to offer safe crossing for individuals with physical disabilities. During the construction phase, all workers will follow Occupational Safety and Health Administration (OSHA) standards. This will ensure a safe work environment that will help avoid accidents.

CONSTRUCTIBILITY

The Right Hand Fork pedestrian bridge is a relatively small structure with a braced, two girder design. Access to the northern side of the stream is unobstructed, since the bank is adjacent to the dirt road. Machine access to the southern bank will prove more difficult since the stream banks are quite steep. Therefore, a Gabion foundation design is suitable for this location since it can easily be installed without the use of heavy machine equipment. Each glu-lam girder weighs approximately 1750 lbs, which is reasonable for a crew of ten workers to place across the stream. Therefore, no crane will be needed. The rest of the timber materials are quite easy to manipulate and move with one or two workers. A dump truck will be required to haul the earth fill, but the crew will be able to place and compact the fill with shovels and a compactor onsite.

Construction should not take longer than two weeks, nor require more than five to ten workers.

ENVIRONMENTAL

Protection of native plant and animal life in Right Hand Fork is the primary environmental constraint. Also, protection of water quality and stability were considered in the design. Construction will not require heavy equipment that could harm the ecosystem. We considered multiple locations for the bridge to ensure the protection of native wildlife while meeting accessibility requirements. The proposed location will not destroy any trees, and approximately 1,500 ft^2 of grassy slope will be removed to allow for the foundations and the earth approach structure. The bridge will allow the public to cross the stream without disturbing the Bonneville cutthroat trout's habitat.

SOCIAL

Accessibility is the primary social constraint. In order to meet this constraint, the bridge design meets ABA accessibility standards. This mandates that the bridge must have a minimum 3-ft width, and the proposed design has a 3.5-ft width. Also, the bridge must be wheelchair accessible, and approach ramps must not exceed a slope of 1:12, which we designed in the earth approach structure. Therefore, the bridge meets ABA requirements.

SUSTAINABILITY

The foundation design of the structure is resistant to scour since we placed the abutments higher on the stream bank. The bridge has a standard 50-year service life that will not likely be damaged during future flooding events since the design provides 3 feet of clearance above the 100-yr 12-hr storm water level. The bridge will be constructed out of timber to meet these constraints. A hydrological study provided the necessary knowledge of the required height of the bridge span.

ENGINEERING TOOLS SUMMARY

Table 20. Engineering Tools Summary

Software Name	Version	Manufacturer
HEC-HMS	4.0	US Army Corps of Engineers
AutoCAD Civil 3D	19.1	Autodesk, Inc.
SMathStudio	0.97.5346	

Excel	15.0	Microsoft
MATLAB	8.1	MathWorks, Inc.
RISA-2D	12	RISA Technologies, LLC
HEC-RAS	4.1	US Army Corps of Engineers
ArcGIS	10.2	ESRI

PROFESSIONAL RESPONSIBILITY AND CONDUCT SUMMARY

Table 21. Expected Professional Standards used in Design

Organization	Number	Name
American Society of Civil Engineers, ASCE	7-10	Minimum Design Loads for Buildings and Other Structures
American Association of State Highway and Transportation Officials, AASHTO	6^{th} Edition, 2013	Bridge Design Specifications
American Institute of Steel Construction	14^{th}	Steel Construction Manual
National Design Specification	2012	Wood Design Manual
Natural Resources Conservation Service	Part 630	National Engineering Handbook

Table 22. Expected Identification Number and Name of Government Regulations

Organization	Number	Name
Architectural Barriers Act, ABA	401	Accessible Routes
Forest Service Trail Accessibility Guidelines, FSTAG	7.4	Technical Provisions

POTENTIAL RISKS

WATER DAMAGE

There is a possibility of water damage on the structure during high runoff seasons as evidenced by the abutment failure of the current bridge in 2011. For this reason, we will design the new bridge with a longer deck span, which will place the abutments at a higher elevation that will not choke the stream's flow. Additionally, Forest Service regulations require that all bridge spans have a three foot clearance over the 100-year flood water level.

EXCEEDING DESIGN LOADS

There exists an improbability of actual loads on the bridge exceed factored design loads, causing structural damage. These circumstances are unlikely since the design will satisfy safety codes for loading. AASHTO loading for wind, snow, and earthquake loads will be considered. The majority of the traffic loads (humans) will be small compared to capacity. The design will also safely support the weight of ATVs if misused.

VANDALISM

The Hobbit Caves are in a fairly remote location and somewhat infrequently visited. Remote areas can attract delinquent behavior. Therefore, vandalism is a potential risk the bridge faces.

ASPECTS TO BE REVIEWED BY PROFESSIONALS

ENVIRONMENTAL IMPACT

We will attend NEPA meetings to consult with Forest Service professionals about environmental impacts associated with the design. Specifically, ecologists will assess the impact to the Bonneville cutthroat trout strain and any trees that need to be removed.

STRUCTURAL SOUNDNESS

Forest Service engineers Jason Day and Oscar Mena will assist us in the bridge design process. Their expertise will help ensure the bridge is properly designed according to Forest Service, ASCE, AASHTO, and ABA standards. Finally, Dr. Marc Maguire, structural engineering professor and our faculty advisor, will review all designs before the project is finalized.

FLOOD PLAIN

Due to the lack of experience in modeling watersheds, many assumptions were made in determining the 100-year flood plain. Therefore, it is important the results from HEC-HMS and HEC-RAS should be doubled checked to ensure the bridge will be above the flood plain.

CONDUCT OF PROJECT MANAGEMENT

SCHEDULING

Ren Gibbons will create and oversee the team's scheduling. The schedule will be structured to meet all deadlines required for the Senior Design course and the Forest Service. We will follow the Gantt charts created for the project.

DECISION MAKING

Major decision making will be done as a group during face-to-face meetings. Minor decisions may be proposed by a group member via email if other members do not object.

RESOLVING DISPUTES

Disputes will be resolved by the team's dispute resolver, Kedric Curtis. Persistent disputes will be discussed during group meetings and each side will express their opinion. Reasonable action will then be taken by the group under Kedric's lead.

CHECKING RESULTS

Redundant verification of each team member's contribution is required. Any research, studies, or design work completed by any individual will be checked by at least one other group member. Next, these results will be reviewed by our professional mentor and faculty advisor.

ENSURE A QUALITY PRODUCT

Ensuring a quality product is the result of the previously mentioned four points of Conduct of Project Management. If these regulations are kept, we will be confident that our bridge design for Right Hand Fork will be effective and economically feasible.