Canyon Engineering

Steel Bridge at Irving Power Plant

Design Report

For Arizona Public Service

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Dear Mr. Mitchell:

This letter is to inform you that Canyon Engineering has completed the design of the steel bridge crossing Fossil Creek at the Irving Generating Station.

The following report details the proposed design for the bridge. The design is for the temporary purposes of the bridge while decommissioning the Irving Generating station. The report included the hydrology calculations, abutment design, steel design, horizontal alignment and vertical alignment. All related calculations are included in appendices at the end of the report. In addition to the report, the plan set for the bridge is included.

If you have any questions or concerns regarding this work, please feel free to contact Canyon Engineering at our office.

Sincerely,

Kimberly Ashcraft
Project Engineer

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Engineer

Nicholas Yourgules
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Design Report Table of Contents

Executive Summary 1

Introduction 1

Site Location and Access 2

Client Criteria 3

Agency Regulations 4

Hydrology 4

Abutment Design 6

Foundation Design 7

Steel Design 8

Deck Design 10

Horizontal Alignment 10

Vertical Alignment 11

Cross Section 12

Earthwork 13

Truck Turn Around Area 13

Drainage 14

Cost Estimate 14

Project Hours 14

Lessons Learned 14

Conclusion 15

Appendix 16

Executive Summary

Canyon Engineering had designed a bridge crossing at the Irving Generating Station. This bridge was designed to hold an HS20 loading. We have also designed the roadway alignments, earthwork, hydrology, abutment design and steel calculations. The bridge will be set 1 foot above the 100-year 24-hour floodwater depth which is 14.5 feet above the bottom of the channel. The abutments will be mechanically stabilized backfill and the foundation will be a reinforced concrete pad. Both of these were designed to carry the maximum loads caused by an HS20 load. The existing roadway will tie into the bridge to create a safe driving environment. We have determined that the bridge will deflect 3.5 inches; therefore we feel that steel grating for the deck material would be the best option. The main girders will be modified by adding channels to the top and bottom flanges in order to stiffen up the section. Canyon Engineering has completed the design under the budgeted amount of hours and on time.

Introduction

The following document is Canyon Engineering's design report for the bridge across Fossil Creek, located at the Irving power plant. Fossil Creek runs through the Verde Valley in northern Arizona. Since 1909, water from the springs that feed Fossil Creek have been dammed up and diverted into a flume, which runs along the canyon rim before dropping into the Irving power plant. Arizona Public Service (APS) has decided to decommission the power plant in order to restore Fossil Creek to its original condition. Currently at the Irving plant there is an at-grade crossing for vehicles to reach the plant site. The pictures below show the existing at-grade crossing.

Figure 1: Looking north from existing road.

Figure 2: Looking south from plant side.

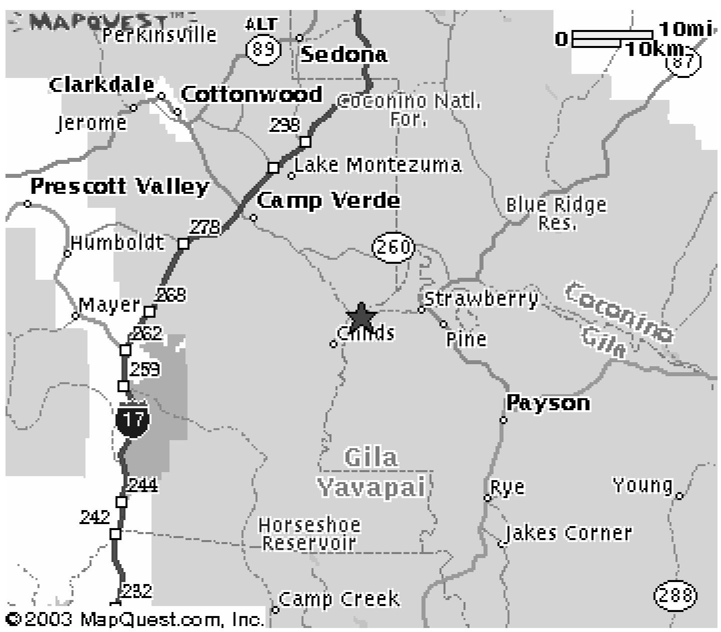
Since most of the water flowing through Fossil Creek has been diverted into the flume, this crossing hasn't posed a problem to vehicles before. With the decommissioning of the plant, the full flow through Fossil Creek will be restored. Heavy machinery will be needed at the plant site to help with the dismantling of the flume and other aspects related to the decommissioning project. To cross at high water, a bridge needs to be built across Fossil Creek that can carry the loads of heavy machinery and be high enough to stay above the high water level.

This is the bridge that Canyon Engineering has designed. Included in this document is the methodology behind all the procedures that went into designing the bridge. These procedures include the steel, hydrology, abutment, foundation, and earthwork calculations and the roadway alignment.

Site Location and Access

The Irving Generating Station is an operating hydroelectric facility located on Fossil Creek in Central Arizona. The project is located entirely on land of the United States, managed by the Forest Service and is part of the Coconino and Tonto National Forests. The project is also on the dividing line between Gila and Yavapai Counties.

The Irving hydroelectric power plant is located approximately 110 miles north of Phoenix, Arizona. The only access to the facility is to use winding gravel roads. One of the roads is approximately 22 miles long and enters off State Highway 260, which is six miles east of Camp Verde. The other access road is approximately 15 miles long and the entrance is off State Highway 87 at Strawberry, Arizona. For design purposes, the access off State Highway 260 will be considered the primary access, as it is topographically more accessible to large vehicles. The access from Strawberry has very sharp turns with steep grades and historically has been subject to shady spots that result in winter icy patches.



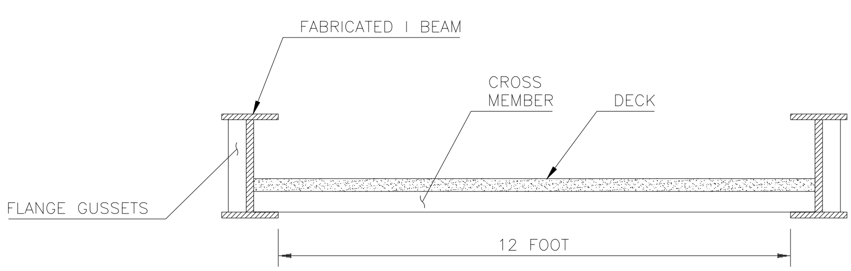
Client Criteria

To begin the process of designing the bridge, the client criteria were obtained from APS. The list is as follows:

• Girder Size	I beam with dimensions (39.5 inches deep, 11.75 inches wide)
• Span	Less than 95 feet
• Width	One traffic lane width (12 feet)
• Loading	HS20
• Abutment Material	Mechanically Stabilized Backfill (MSB)
• Storm Event	100 year 24 hour
• Bridge Decking	Steel or Concrete
• Bridge Height	One foot above 100 year water surface

The girder size was chosen before the design of the project because APS had two large I beams left over from a previous project. By using these large beams, the cost of the project could be reduced. The length of these beams also determined the span. The lengths of the girders were 95 feet and so the span of the bridge would have to be less than that. The width was chosen because of the sight distance across Fossil Creek as well as design speed. The speed of vehicles approaching the bridge will be less than 15 miles per hour because of the dirt road approaching the bridge. At this speed, drivers will be able to look across Fossil Creek and see if they need to yield to oncoming traffic entering the bridge. Therefore, the width of the bridge only needs to be one truck lane width. The HS20 loading was chosen because of heavily loaded tractor-trailer vehicles removing materials from the project site. Mechanically stabilized backfill was chosen for the abutments because it is very strong and will be easily removable when the service life of the bridge is finished. The 100-year 24-hour storm was chosen for multiple reasons. First, it was a documented storm by Northern Arizona University for an upstream location study. Second, it was chosen because it would be agreed to by regulating agencies. The decking options were narrowed to two options of concrete or steel because steel grating could be recycled in future projects and is easily removable and concrete is usually very cost effective. The bridge height was set at one foot above the 100-year water surface elevation because it is a common regulation by many public agencies and would provide a safe elevation during most storms.

In addition to the listed client criteria listed above APS specified a configuration for the bridge. The configurations consisted of the cross members resting on the interior lower flange of the beams spanning between the main girders. This will allow the girders to serve a double purpose as both a safety rail and structural element. Web stiffeners or gussets will be welded in between the outer flanges to provide support for the lower flange to prevent bending from the cross member loadings. A picture of the configuration is shown below.



Agency Regulations

After some lengthy research including calls to Gila County, Yavapai County, Coconino County and several Forest Service offices, it was determined that the only agency regulations that must be abided by are the Forest Service. The county agencies said that they did not have jurisdiction because the project site was on leased Forest Service land. After working our way up the Forest Service ladder we found that the person who had jurisdiction over the bridge was Rich Miller in Albuquerque, New Mexico. He is the engineer in charge of region 3, which includes Coconino National Forest and Tonto National Forest. He informed us that the only special regulations that he would impose is installing a standard Forest Service gate at the south end of the bridge to keep the general public off of the bridge. He also said that the Forest Service would approve the project if it were designed for a 50-year storm because the bridge will be temporary. However since the APS criteria called for the 100-year storm, this event was used for the design.

Hydrology

The purpose of the hydrologic modeling of the stream was to determine the depth of flow through the channel. Once the Fossil Springs diversion dam is removed the stream's perennial flow of 43 cubic feet per second (cfs) will be restored in the channel. Although this flow is relatively small it does produce a depth of flow in the channel of approximately 1.45 feet at the existing low water crossing, therefore requiring a bridge. However, it has also been determined that the bridge must not only stay clear of the water surface for the perennial flow, but must also clear the 100-year 24-hour storm event for the area.

Additionally, the location of the bridge abutments and supports cannot be placed within the floodway for the 100-year event. They can however be placed within the floodplain of the channel. Therefore, it was necessary to determine where the floodplain and floodway boundaries were in the channel for the 100-year event.

Determining the depth of flow required the amount of storm water runoff, in cfs, that would be expected during the 100-year event, and the channel geometry at the proposed bridge location. Additionally, the average slope of the channel near the crossing site and the Manning's roughness coefficient were needed as well. All of these parameters combined would be used to determine the depth of the water at the bridge crossing, which would set the height of the bridge deck.

The amount of storm water runoff was determined by referencing the hydrology study that prepared for APS by Northern Arizona University in November 2002 (Schlinger, Trotta, et.al.). This particular hydrology study utilized the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Centers (HEC) Flood Hydraulic Package HEC-HMS (Hydraulic Modeling System) to produce the peak flows for Fossil Creek for the 2, 5, 10, 25, 50 and 100-year 24-hour storm events. From this hydrologic study, it was determined that the peak flow for the 100-year return period was 6,743 cfs (Schlinger, Trotta, et.al.). It should be noted that this study was only performed on the watershed that extends from the diversion dam, upstream to the surrounding ridgeline. The proposed location of the bridge is located several miles downstream of the dam, indicating that the channel would be servicing a larger watershed and would thus have a higher flow. Therefore, it was necessary to determine the flow at the bridge location using this existing model.

As students of Northern Arizona University, we were able to consult with Dr. Paul Trotta and Dr. Charlie Schlinger, two professors and engineers who worked on the hydrology study, regarding how to develop an appropriate method that could be used to determine the peak flow at the bridge location. The recommend procedure required finding a topographic map of the area and locating the boundaries of the watershed that would contribute to the runoff at the bridge crossing. Special attention was given to studying the topography to ensure that there were no distinct variances, such as side canyons, or other channels, which would affect the runoff. Fortunately, no variances existed and the topography appeared to remain uniform for the entire area, and a reasonable watershed was produced for the area. Using AutoCAD, the area of the new basin was determined to be 63.8 square miles. This value is 9 square miles larger than the 54.8 square mile watershed that was used for the NAU hydrology study. For the purposes of our calculations, and for a safety factor, we increased the watershed area to 64.8 square miles, 10 square miles larger than the NAU watershed.

Dr. Trotta and Schlinger recommended that, since the area is homogenous in topography, soil type, and foliage, that the easiest way to determine the flow as the bridge crossing would be to take a ratio of the areas and the flow. The 54.8 square mile watershed that NAU used had a peak flow of 6,743 cfs for the 100-year storm event. By taking a ratio of this area with the area of the watershed for the bridge, and multiplying by the 6,743 cfs flow, a new flow of 7,973 cfs was determined. This is the flow that is expected at the crossing where the proposed bridge location. To simplify calculations, this 7,937 cfs flow was rounded up to 8,000 cfs. (See Hydrology Appendix, Channel Cross Sections) It is felt that this number is representative of the area and will be used in calculating the flow depth of the channel, and in determining the floodway and floodplain at the construction site.

Channel cross-sections located near the existing low water crossing were taken from survey data that was provided by Golden Rule Surveying LLC. Using this information and the aid of AutoCAD, several cross sections were generated, as well as the average channel slope of .0087 ft/ft for the site. (See Hydrology Appendix, Flowmaster) A Manning's coefficient of .05 was selected for this analysis. This coefficient is representative of a natural, stony stream, and is the same roughness coefficient that was determined by NAU for "the scoured bedrock-controlled reaches below the dam" (Schlinger, Trotta, et.al). It is felt that this coefficient is representative of the stream channel where the bridge is to be located.

The water surface elevation for the 8,000 cfs flow was determined using Flowmaster, a hydraulic modeling software developed by Haestad Methods, that models open channel flow. Using the cross section from the survey data, flow rate, Manning's coefficients, and channel slope as the parameters, values for the water surface elevation, top width, and velocity were determined. At the cross section in question, the depth of flow was 11.3 feet, the top width was 154.75 feet, and the velocity was 8.9 feet per second. (See Hydrology Appendix, Flowmaster) The water elevation of 11.3 feet coincides well with reports from APS employees who indicate that the water level has been observed to be about 12 feet deep from the channel bottom.

Because the top width of 154.75 feet for this flow is longer than the 90-foot girders that are to be used, it was necessary to determine how the bridge could be placed in the channel, namely the minimum distance that would be needed between the abutments. We again consulted with Dr. Schlinger, Dr. Trotta, and Justin Ramsey from the College of Engineering at NAU. They informed us that we would be able to place the abutments with the floodplain, but that we were not allowed to enter the floodway of the stream channel. The floodway is determined by erecting two imaginary walls on each side of the channel in Flowmaster, and bringing them together until the water surface elevation rose 1-foot in elevation. The distance between the two walls defined the floodway location. No structures of any kind are allowed in the floodway, but can be placed in the floodplain. Using Flowmaster and this procedure, it was determined that the minimum distance required between the abutments was 76 feet, symmetric about the centerline of the bottom of the channel. This distance would raise the water level exactly 1-foot to a water depth of 12.3 feet (See Hydrology Appendix, Floodway). For safety reasons or team decided that the abutments would be placed 80 feet apart, which is outside of the floodway and will not raise the water surface elevation above the 12.3 foot maximum for this storm event.

As a final step for the hydrology, the elevation of the deck of the bridge was set with respect tot he water surface elevation for the floodway. Again, consulting with Dr. Schlinger, Dr. Trotta, and Rich Miller, we were informed that it is usually standard procedure to place the deck of a bridge at a minimum of 1 foot above the water surface elevation, in this case, 13.3 feet above the channel bottom. Again for safety reasons, a larger value was chosen, placing the bridge deck at a minimum of 14.50 feet above the channel bottom, just over 2 feet above the floodway elevation. (See Drawings Appendix)

Abutment Design

The abutments used in the design of the bridge structure will serve two purposes. The first will be to support the bridge structure itself while keeping it out of the flow path of the stream channel. The second purpose will be to tie in the horizontal and vertical alignment of the bridge into the existing horizontal and vertical alignment of the existing roadway on the north and south side of the channel. The abutments will be placed in the floodplain of the channel but will not encroach into the floodway, as described in the hydrology section of this document. The abutments will be constructed out of mechanically stabilized earth using geosynthetics as the reinforcement for the wall system. Due to the heavy loading that is expected to occur on the abutments and the narrow corridor that the bridge could be placed in, it was determined that a mechanically stabilized earth wall would be used for the abutments. Mechanically stabilized earth is very stable, can carry higher loads, are relatively easy to construct, does not require expensive construction of large retaining walls, and does not have to be extremely large in order to carry large magnitude loads.

The design of the abutments was done by utilizing a report given to us by APS from the Colorado Transportation Institute (CTI). While this report is not a design manual or reference, the design of the abutments was completed in the manner outlined in the report. The CTI timber faced geosynthetic-reinforced wall was used as the basis for this design.

For the design criteria of the abutments, it was determined that it would be a timber faced wall and would be back filled with aggregate base course (ABC) with layers of geosynthetics embedded within the fill. The ABC fill was selected as the fill material for two reasons. The first reason is that ABC is used extensively in heavy construction, especially in roadways. This indicates that it is capable of supporting the high loads that are imparted by traffic, especially large trucks, and would be able to carry the loads of the vehicles that we expect to use this facility. The second reason for the use of ABC is that it is a granular soil type and drains water rather quickly. Since the abutments are in the floodplain of the channel, water will need to drain out of the fill material. Our team consulted Western Technologies Inc., which is a materials testing company located in Flagstaff, Arizona. A geotechnical engineer that is employed with the company provided us with the necessary soil values needed to design the abutments. For our purposes, the following values for the ABC were used: Unit weight (γ) of 135 pounds per cubic foot (pcf), cohesion (c) of 0, friction angle (ϕ) of 36°, and allowable bearing pressure (q_a) of 3,500 pounds per square foot (psf). Additionally, it was assumed that the wall faces would be vertical, the crest of the abutment would be horizontal, and the base of the abutment would be level and resting on the bedrock that is in the channel bottom.

The analysis was performed for an 18-foot high abutment wall. This height is several feet larger than the height of the walls that will be constructed on the north and south sides of the channel. Currently, the southern abutment will be at a height of 15.5 feet from the channel bottom, and the north abutment will be 14.5 feet high with respect to the channel bottom as well. The reason that an 18 foot high wall was analyzed was for purposes of ensuring that the spacing of the geotextile material would not change for heights greater than 14 feet. This would ensure that the wall would still be functional in the event that the wall heights would be increased. The potential for an increase in the wall height is present since the base of the abutment must be placed on a level surface. This would require excavation of the underling soil and existing rock, which would increase the overall height of the abutment. Using this wall height, and the values listed above, it was determined that a geotextile fabric with an ultimate tensile strength of 4,200 pounds per foot or greater should be used in the construction of the abutments. The maximum spacing of the geotextiles was determined to be 0.8 feet throughout the majority of the structure. A spacing of 1.0 foot and greater was determined to be acceptable for soil depths no greater than 6 feet. However, for ease of construction, and because of the size of the timbers that are to be used, it was decided that the spacing of the geosynthetic would be set at 0.5 feet. (See Abutment Appendix)

Construction of the abutments will require the use of the 6” x 6” timbers from the flume that is located on the site, and 2” x 6” boards used for sheathing to connect the 6” x 6” timbers and reinforcing. The geosynthetic material will wrap around the 2” x 6” timber and will be sandwiched between the 6” x 6” timbers. The 2” x 6” and 6’ x 6” will be connected using nails or an equivalent fastening device. The tail of the reinforcing material will then be folded back into the retaining wall. A maximum lift of 0.5 feet will be used between all of the geosynthetic. (See Drawings Appendix, Abutment Detail)

It should be noted that because this abutment wall is very narrow, the timbers will wrap around the sides of the abutments and the geosynthetics will also wrap around the corners of this wall. For instructions on wrapping around a corner, it is recommended that the constructors follow the guidelines of the manufacturer for the reinforcement. As a result of this, layers of reinforcement will overlap in some areas by an order of 2 to 3 layers. This overlapping does not appear to cause any problems in the construction or analysis of the abutment design. (See Drawings Appendix, Abutment Detail) Additionally, because the timber facing will wrap around the face and sides of the abutment, some form of anchoring will be required, such as large spikes or bolts, to connect the wall corners and produce a connected wall structure.

Foundation Design

To transfer the loads of the bridge to the abutments, it was determined that a reinforced concrete pad would be placed at the top of each abutment. Because of the symmetry of the bridge structure, it was determined that only one footing would be designed, but would be constructed twice, one for each end of the bridge. Because of the alignment of the bridge with the roadway, and because of the shape of the abutment face, it was determined that the concrete footing would also be trapezoidal in shape (See Foundation Appendix).

From the steel analysis, it was determined that the footing would be required to support two point loads of 128,000 pounds each. This loading was the result of the dead load of the bridge and a vehicle load equivalent to an HS 20 loading. Using the book, Design of Reinforced Concrete by Jack, C. McCormac as a reference, the concrete footing was designed. It was assumed that the footing would be loaded symmetrically about its center of gravity, and that the beam girders would rest on an elastomeric bearing plate with a maximum width of 8.0” and a minimum length of 8.0”. The center of this bearing plate would be placed symmetrically with respect to the center of gravity of the footing. If a longer bearing plate is used, the center of the plate should also coincide with the center of gravity.

The design of the concrete footing was based on an ultimate strength design as set forth in the reference text. Therefore, it was necessary to make the footing large enough in order to not exceed the allowable soil pressure, resist shear failure, and resist bending failure. From our calculations it was determined that for our loads and allowable soil pressure, that a concrete footing with an area of 84.3 square feet would be required. Because of the width of the bridge and the distance that the girders overhang on the abutments, it was determined that a footing with and area of 126 square feet would be used. This footing will be trapezoidal in shape and will be 18.0 feet long, 7.0 feet wide and 21.0 inches deep. (See Foundation Appendix)

Given the design loads, it was determined that the footing is deep enough to resist a shear failure that would result from the given loads. From the analysis it was determined that the minimum depth of the beam required for shear would be 10.8 inches. The depth of the beam however was governed by the bending moment that was generated from the loading. This bending moment required that the minimum depth of the beam needed to be 17.7 inches. This depth is required based on the minimum amount of reinforcing. When the analysis was performed, it was determined that the amount of reinforcing required was less than the minimum value required by code. Therefore, the minimum value of reinforcing was used, which yielded the minimum footing depth of 17.7 inches. An extra 3.0 inches will be added to the 18.0” to provide cover for the reinforcing, since the footing is exposed to bare earth and possibly a wet environment. Additionally, the footing will be constructed out of 4,000 pounds per square inch (psi) concrete.

For simplicity in construction the same size of reinforcing will be used throughout the footing. All reinforcing will be #9 bars rated at 60,000 psi. Having the same size bars throughout the footing will eliminate having to obtain different sizes of reinforcing, and will prevent any errors associated with placing the wrong reinforcing steel in the appropriate areas. From the concrete analysis, it was determined that longitudinal steel would be needed in both the top and bottom of the footing along the 18-foot dimension. This steel will be placed 3.0 inches from the final edge of the concrete surface. This longitudinal reinforcing is needed to resist the positive bending moment at the center of the footing of 380,000 foot pounds, and the negative bending at the footing ends of 66,300 foot pounds. There will be six number 9 bars placed 1.0 foot on center and symmetric about the centerline of the footing. (See Drawings Appendix, Abutment Detail) In the short direction (the 7.0 foot length), reinforcing will be needed in the bottom of the footing to resist the positive bending of 166,000 foot pounds. For simplicity in construction, it was determined that seventeen number 9 bars would be placed at 1.0 foot on center, symmetric about the centerline of the footing, on the bottom of the footing. (See Drawings and Foundation Appendix) While reinforcing along the 7 foot dimension was not required, as negative bending was not present, it was determined that temperature reinforcing would be required. Again for simplicity and the fact that it does meet the code requirements, it was determined that seventeen number 9 bars would be placed 1.0 foot on center with respect to the centerline. These bars would be placed at a distance of 3.0 inches below the top of the footing and will resist cracking or failure that would result from temperature changes. (See Drawings and Foundation Appendix)

Steel Design

In calculating the loading and design for the steel, the Load and Resistance Factor Design Method was used. This method was used because it is what we were taught in classes and had the most experience in using.

The two main girders that support the bridge were scrap steel from the West Phoenix Power Plant. To start the design of the bridge, the section properties needed to be calculated for these girders. Before calculating the properties, the exact dimensions were obtained from Page Steel, who is holding the girders for APS. The dimensions are as follows:

- $d = 39\ {}^{7}/_{16}$”
- $t_w = 3/4$”
- $b_f = 11\ {}^{13}/_{16}$”
- $t_f = 1\ 1/4$”

From these dimensions, the different section properties could be calculated. The section properties calculated included the moment of inertia, section modulus, area, radius of gyration and the plastic section modulus. (See Steel Appendix, Girders) These calculations were carried out for both the X and Y-axis of the girder.

The loading for the bridge was to be an HS20 load. APS specified this in the criteria given to us before the project started. An HS20 loading is a standard AASHTO (American Association of State Highway and Transportation Officials) load that consists of two 32,000 pound axles and one 8,000 pound axle. These loads correspond to a tractor trailer. The spacing of these loads is 14 feet for the distance between the 8,000 and the first 32,000 pound load. The distance between the two 32,000 pound loads could range from 14 to 30 feet. To get the maximum loading of the bridge from these loads, the spacing for all point loads was set at 14 feet. AASHTO also specified a 640 pound per lane width loading. Since the bridge has only one lane, this load was uniformly distributed across the deck for calculations. (See Steel Appendix, Loads)

In order to calculate the maximum moment that the girders would encounter, the center of gravity of the wheel loads needed to be determined. The maximum moment would occur when the center of gravity of the wheel loads was placed over the center of the bridge span. The center of gravity of the wheel loads was found to be 4.66 feet past the first 32,000 pound load. This was placed at exactly the center of the span to produce the maximum moment. The maximum moment was calculated for both factored and unfactored loads. The factored loads were used to determine the ultimate strength of the girders, while the unfactored loads were used to determine the maximum deflection of the girders. An impact load was also added on to the truck loads as specified by AASHTO. The bridge was assumed to be a simple span for all the calculations.

The maximum shear was calculated by finding where the loading produced the maximum end reactions. To make the process easier, Microsoft Excel was used to calculate the end reactions for different placements of the loads. Calculations were written that changed the location of the loads to see where the maximum reactions would occur. The table listing the values that were obtained is in the Appendix. (See Steel Appendix, Loads) The end reactions were then used in the design of the abutments and footing for the bridge to be attached to.

The maximum moment was determined by placing the center of gravity of the truck load directly over the center span of the bridge. Once the maximum factored load was determined, it was checked against the yield moment of the girders. Since the moment diagram was very close to the diagram for a uniformly loaded beam, the calculated maximum moment was used to determine what uniform load would give the same maximum moment. With this uniform load, the maximum deflection was calculated and found to be approximately 5 inches. (See Steel Appendix, Deflection) Since this amount of deflection is unacceptable for the bridge, the section properties of the girders needed to change. After speaking with Bill Mancini, a structural professor at NAU, channels were added to the top and bottom flange. With these channels, the deflection was cut down to 3.5 inches. This gives a deflection of l/320 which after speaking with the client is acceptable. With the channels added to the girders, the maximum moment that the bridge can carry is increased. The channels that were chosen are MC 10x41.1. Adding these channels onto the girders, the loads need to be recalculated to account for the increased dead load. The section properties were also recalculated to account for the channels.

The cross beams were determined by first finding the maximum loading conditions for the beam. The maximum loading conditions include two 16,000 pound point loads, placed 6 feet apart and the uniform 640 pound per lane load specified by AASHTO. Also, the dead load from the steel decking was taken into consideration. The section was found by calculating the ultimate moment and then determining the plastic section modulus. This number was compared to values listed in the LRFD Steel Manual and a section was chosen. Originally, a W12x16 was found to be sufficient to support the deck. When the connection was designed, it was determined that a section with a larger web thickness was needed, so a W12x35 was chosen. (See Steel Appendix, Crossbeams)

It was decided that welded connections would be easier to construct than bolted connections to attach the cross beams to the main girders. In order to ensure that the deck load was transferred directly into the plane of the web of the girders, double angles are welded from the web of the cross beam to the web of the girder. In designing this connection, the minimum web thickness of the cross beams were inadequate with the W12x35. Since the horizontal and vertical alignments had already been computed, it was decided to stay with a W12 shape and increase the size to get the correct web thickness. This is how the deck supports were determined to be W12x35. The angles used in the connection were determined to be L3x3x$^3/_8$. (See Steel Appendix, Attachments)

The first part in designing the gussets was to determine where they needed to be placed along the main girder. The maximum lateral unbraced length was determined (See Steel Appendix, Bracing) and the placement of the gussets was based off of that dimension. It was decided to use a distance half of the calculated maximum distance. This provides a factor of safety for the loading conditions on the bridge. The spacing used in the design is 8 feet. Steel plates will be used to provide the lateral bracing. The outside dimensions of the plates are determined by the distance between the top and bottom flanges and the width of the flanges. To determine the thickness required for the plates, the load that each plate needed to counteract was found. Once the load was found, the plate was modeled as a column. This method was suggested to us by Bill Mancini. The moment was found and then the section modulus was calculated. From the section modulus, the thickness of the plate was determined. (See Steel Appendix, Bracing)

The channels will be welded intermittently as diagramed on the detail sheets. (See Drawings Appendix, Steel Details) The largest welds consist of the full penetration butt welds that will splice the pieces of the main girder together along with the extension pieces on the ends. Although welding is expensive, we felt that it would be easier when constructing the bridge on location.

The anchor bolts were designed by consulting the AASHTO Manual. AASHTO specified that for a span of 50 – 100 feet, two anchor bolts were required. These bolts would be 1 1/4 inch diameter and extend at least 12 inches into the foundation. These bolts were checked against the maximum end reactions and it was determined that four bolts on each end of the girder was necessary for the loading conditions. On the southern end of the bridge, the anchor bolts will create a fixed connection with the foundation. To take into consideration any thermal expansion that might happen, the bolt holes on the southern end will be slot holes. These slots will allow the bridge to expand without causing excess stress on the anchor bolts. (See Drawings Appendix, Steel Details)

Deck Design

Since the bridge will be deflecting 3.5 inches, it was determined that steel grating for a deck material would be the best option. Concrete was considered as an option, but it's not flexible and would crack as the bridge deflected. Steel grating would be able to bend while maintaining its structural integrity and therefore felt to be the best option. The specifications used in designing the deck material came from Amico-Klemp. The grating that was used in the design was 19-H-166. This was based on the design charts provided by the manufacturer. This grating was chosen based on the largest point loads that would occur on the bridge. The design charts also specified the support spacing to hold the designed loads. This spacing was 2 feet and is what determined the cross beam support distances. The specifications for attaching the grating to the cross beams was also specified by the manufacturer. (See Decking Appendix)

Horizontal Alignment

The horizontal alignment of the bridge at Fossil Creek will tie the bridge deck into the existing dirt road that approaches the Irving generating station. This will be done in an effort to accommodate the heavy machinery and trucks that are expected to enter and exit the facility. The bridge will be a straight, approximate 90-ft long span, which will traverse the majority of the channel. Grading and earthwork will be required to align the bridge with the existing dirt roads heading into and out of the plant.

The bridge will cross the channel slightly downstream of the existing concrete crossing that is located in the streambed. The horizontal alignment will contain a Y-intersection from Fossil Creek road, which will connect the bridge roadway, Fossil Creek Road and the existing road that traverses the channel. This intersection will allow use of the existing roadway while the bridge is being built and for use when the storm flows are not present. It will tie into Fossil Creek Road from the south and terminate in the existing road at the Irving station on the north.

Two horizontal curves will be used to realign the existing roadway with the bridge. Given the steep terrain of the area, it was determined that vehicles, namely the large construction vehicles, will not be able to enter the plant site by use of the bridge if traveling from the town of Strawberry, Arizona. Trucks attempting to enter the facility from this route would not be able to turn and line up with the bridge. It is recommended that all large construction vehicles enter from the town of Camp Verde, Arizona. It is from this direction that the design of the horizontal alignment for the bridge will be based.

Two horizontal curves were utilized in the roadway realignment, one on each end of the structure. Each curve has a 70-foot radius and ties into the existing roadway. The American Association of State Highway and Transportation Officials (AASHTO) manuals were used as a reference in determining this radius. Based on their information, we found that for large trucks, such as the type that are expected to use this structure, that a minimum turning radius of 45 feet was required. This radius is the minimum needed to prevent obstructions from entering the wheel path of the vehicle and will provide easier turning. Utilizing a radius of 70 feet will therefore help prevent turning difficulty for the vehicles. Additionally, the use of the larger radius allows the trucks to be more in line with the bridge throughout the turning operations.

The first horizontal curve is located on the south side of Fossil Creek. It is joined into the roadway that connects Fossil Creek Road and the low water crossing that enters into the Irving generating station. (See Drawings Appendix) This proposed horizontal curve begins at the point of tangency of the curved roadway section coming into the facility, creating an "S"-shaped curve or reverse curve. It should be noted that in the design of a highway, this would not be allowed. The reasoning for this is that for high-speed roadways where superelevation is incorporated into the curved sections, a minimum distance is required to transition from one superelevation extreme to the other. Therefore, the point of tangency of one curve cannot be the point of curvature for the other. However, this principle is applied to highways, where superelevation is required to counteract the centrifugal forces experienced when traversing through a curve at high speeds. Given the terrain of the area leading into the bridge, speeds greater than 5-10 miles per hour are not expected, and the use of superelevation is not necessary. Therefore, the reverse curve can be utilized in this manner.

The second curve is located on the north side of Fossil Creek on the Irving generating station side. (See Drawings Appendix) This horizontal curve simply ties into the roadway from the low water crossing and enters the APS generating station. Again, no superelevation was incorporated into this horizontal curve. Additionally, on the generating side of the bridge, the horizontal alignment is in-line with the bridge for approximately 80 feet, which does not present any problems in aligning the vehicles for use of the structure.

While superelevation is not incorporated into the design, a 2% grade will be used to aid in the drainage of the roadway in the event of a storm. This constant 2% grade will be used on the proposed horizontal alignment only in areas where earthwork is required. This grade will not be incorporated into the bridge deck, which will be kept at a 0% grade. This 2% slope in the roadway will be placed so that the east edge of the roadway is higher than the west side, thus allowing any drainage to enter the stream channel downstream of the bridge.

Vertical Alignment

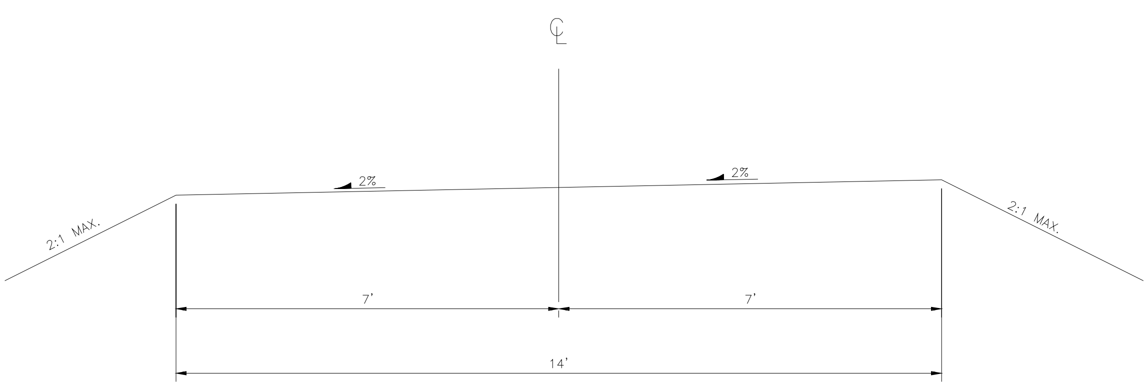
The vertical alignment of the bridge is dictated by the water elevation flowing through the channel. The deck of the bridge will be located above the expected water level that is expected to occur with the 100-year storm event. In this case, it is located approximately 2 feet above the water surface elevation that is expected to occur in the floodway of the channel.

With the minimum height of the bridge deck from the bottom of the channel determined, the vertical curves were chosen in order to connect the bridge with the existing roadway alignment. From the survey data, it was found that the approach toward the bridge on the south side of the site was fairly steep, having a downgrade of 11.0%. Additionally, the grade of the existing road on the north side has an upgrade of 8.5%. While this grade is not as steep as the approaching grade, it is still fairly steep. One of the main concerns with this steep grade is that since the bridge will be nearly flat across the channel, all of the vehicles will need to be able to pass over it without having the vertical curves too short. If the curves are too short, trucks and other vehicles will transition from one grade to another too quickly, and will either "bottom out", high center, or jackknife. For this reason a 40-foot long sag vertical curve is used at both ends of the bridge. This length of curve is flat enough to accommodate the trucks, for the speed that they are expected to cross the bridge, without causing any difficulty.

The vertical alignment of the deck will be such that the southern end of the deck will be placed 0.9 feet higher than the north end, i.e. a 1% downgrade. This downgrade was done for two reasons. First, because of the steep grade of the existing road coming into the bridge from the south side, having this 1% down grade decreased the length of the vertical curve that will be required and will flatten it out. The second reason this downgrade was incorporated was to assist in drainage off of the bridge. We are investigating the option of using steel grating for the bridge deck, which does not require provisions for drainage as the water can simply pass through the grating. If however the grating is not feasible and another decking material is required, such as reinforced concrete, the provision of the 1% downgrade will assist in draining water off of the structure and will eliminate the possibility of ponding occurring on the bridge.

Cross Section

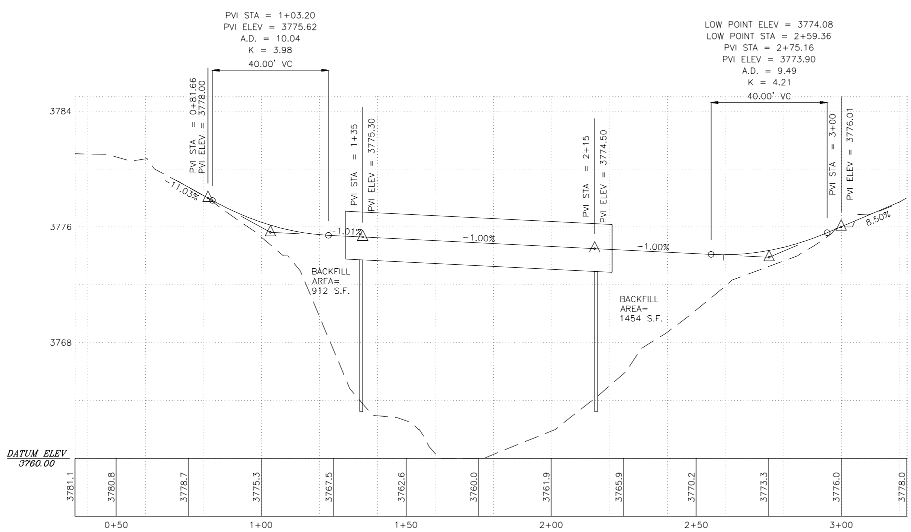
The cross section has been chosen because of low level traffic on the site as well as the easy removal after decommissioning of the facility. The existing Fossil Creek road that the bridge ties into is currently a dirt road at the site. The remoteness and low level speed of the vehicles entering the site do not warrant any pavement or superelevation for the cross sections. The cross section selected has a 2.00% slope that falls from the right edge of the road to the left. This will allow for surface water to drain toward the downstream side of the bridge, which will keep from adding to the discharge calculated for the bridge height. The side slopes for the edges of the cross section were chosen as a 2:1 maximum slope to keep erosion of the surface at a minimum.



TYPICAL ROADWAY SECTION
NOT TO SCALE

Earthwork

Earthwork calculations were conducted by using a variation of the average end area method. The length of backfill in the abutments was so small that a cross section was cut through the center of the bridge to reveal a representative cross section of the backfill area. The cross section is shown below. Two triangles were drawn encompassing the backfill area on each side of the bridge. This area was determined by using AutoCAD. The area was divided by 4 because of the exaggerated vertical scale and then multiplied by the 22 foot width for the average road width. Next, the volume was divided by 27 to convert the area from cubic feet into cubic yards. The number revealed was 482 cubic yards of borrow that needed to be imported. For conservative estimation as well as ease of ordering this number was rounded to 500 cubic yards.



Truck Turn Around Area

The truck turn around area posed some difficulty as well as ease with the project. The survey for the area of our project that was done by Golden Rule Surveying did not encompass the truck turn around area. Therefore, we asked APS if they would be doing any surveys of the area. They said that the area would be surveyed but unfortunately after our design was due. This caused us some difficulty because we had to arrange for survey equipment and survey it ourselves. Our original intention was to design an area that would be conducive for large trucks to turn around in by designing grades, earthwork, and drainage. However, upon surveying the area we found that the grades were already traversable and that the drainage already worked. Therefore, the only issues with the truck turn around area involved removal of a small masonry decorative barrier wall, a basketball hoop, and some barbed wire fencing. Once these items are removed the truck turn around area would almost quadruple in size and would be sufficient for its intended purpose.

Drainage

Drainage for the project was a major concern. The roadway cross section was chosen to drain the water downstream from the bridge to minimize flow under the span. Also the deck was chosen because it would eliminate ponding on the bridge and cause an increased loading during precipitation events. Then the bridge was sloped to cause water to run off the bridge in case another decking material is chosen. The truck turn around area already has adequate drainage and so will remain in its current condition. Overall, the topographic relief in the area will provide adequate drainage for the site. No regulations were considered for the discharge hydrograph from the site because the post-development drainage will be the same as the pre-development drainage since the amount of impervious landscape will not change.

Cost Estimate

A cost estimate for the project has been conducted. The estimated project cost is about $175,000.00 with a 10 percent contingency. The contingency has been added due to the remote location of the project. The cost estimate was conducted by obtaining unit costs for materials from manufacturers and contractors. Two cost estimates were conducted. The first estimate was conducted by calling Tiffany Construction and asking what a unit cost for placement and compaction of backfill costs and construction of structural concrete. After answering our questions they forwarded our inquiry to Contech Inc. This company builds many bridges and gave us a unit cost for a bridge similar to ours in dollars per square feet. With the addition of these costs we had a close approximation to what it would cost to construct the bridge. The second cost analysis was conducted by taking the costs of Tiffany construction along with material costs for the steel and adding a labor cost to the steel. In addition, the removal of items from the truck turn around area was added. The estimates for the cost were similar so we chose to use the more defined cost estimate. (See Cost Estimate Appendix)

Project Hours

Progress on this project did not correlate well with the schedule. There were many reasons why we had a tough time abiding by the scheduled hours from our proposal The first reason is that the tasks that were listed on the schedule were unrealistic because we did not have a clear picture of what would have to be done to complete the project. One example of this was how long it would take to complete the steel design. On the schedule, we thought it would only take 24 days. Our project took much longer for the steel design than anticipated. However, on the other hand the truck turn around area took a much shorter time than anticipated. When we began this project we did not have a good “feeling” for how long tasks would take. Thankfully, our actual hours were under our scheduled hours and our project was completed by the due date. One reason for that was that we built time into the beginning of the schedule for proposal approval. Our proposal was accepted the semester before so we did not have to spend time submitting the proposal in the spring semester. A graph showing our actual hours in relation to the scheduled hours is included in the appendix. (See Project Hours Appendix)

Lessons Learned

This semester has taught us very much. When we started this semester we did not know how to manage a project. In previous classes we always had explicit instructions from our professors on what was due and when. However in this class we had to control our own progress and manage the project effectively. If we had a better schedule, our management of the project would have been much easier. If we had our milestones placed effectively, we could have enforced our progress better and made a better assessment of how much time would be needed to finish. However, we now have a much better idea of how much time certain tasks take. Therefore we could generate much better schedules for future projects of a similar type. We also have a much better ability to research within the engineering community to find out information that we need. We also have a much better knowledge of what public agencies may be involved in projects. This project required us to make a lot of phone calls to public agencies and design professionals to learn methods and regulations for our designs. One aspect of this project that set it apart from other classes is that the answer that we determined could not be checked in a textbook. Our designs had to be checked with common sense and alternative methods of calculation. If our results corresponded with tabulated data for similar steel or with observed data then we felt comfortable with the answer. Another aspect that set this class apart is that it seemed to transform our perspective from being a student into being a design professional. The reason for this is that if our calculations or assumptions were wrong there could be serious consequences for this project. In previous classes if we were wrong the only consequence was that we would miss points on our grade. This time lives could be at stake with our design. Our feeling of designer responsibility was greatly heightened during the project progress. Overall this class helped to open our eyes to issues and concerns that a design professional deals with every day that students do not experience until they enter the field.

Conclusion

Canyon Engineering has completed the design for the bridge over Fossil Creek at the Irving Generating Station. The different aspects of the design included hydrology of the area, horizontal and vertical alignment with the existing roadway, the substructure including the abutments and foundation, and the superstructure, which included the steel and decking. The bridge was designed according to the given client criteria and various code requirements. This project was completed on time and under budget.