LAS TRANCAS STREAM CROSSING

LAS TRANCAS, PANAMA

TRANCAS ASSOCIATES

IDESIGN FINAL REPORT
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Flexible Buried Steel Bridge for a stream crossing near Las Trancas, Panama

Submitted to:

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Mission Statement:
The mission of Trancas Associates is to design transportation engineering solutions that are sustainable, durable, and economically practical. Trancas Associates is focused on providing the engineering calculations for the design of bridges, culverts, and roads servicing rural communities that will take ownership of these structures.

Purpose:
Trancas Associates (TA) is a group of four Civil Engineering undergraduate students from Michigan Technological University's 2016 International Senior Design Program. In August 2016, TA traveled to the Ngäbe community of Las Trancas in the province of Comarca Ngäbe-Buglé, Panamá in order to survey and collect data on a stream crossing that was causing transportation problems near the community. The proposed flexible buried steel bridge will provide a reliable solution to these problems and keep this route to and from the community open year-round.

Acknowledgements:
Trancas Associates would like to recognize the following people for their time, knowledge, and contributions to this project:

Peace Corps Volunteer -	Frank Dubasik
International Senior Design Advisors -	David Watkins, Ph.D., P.E. Mike Drewyor, P.E. P.S.

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Disclaimer:

This report, titled "Las Trancas Stream Crossing", represents the efforts of undergraduate students in the Civil and Environmental Engineering Department of Michigan Technological University. While the students worked under the supervision and guidance of associated faculty members, the contents of this report ***should not*** be considered professional engineering.

***DO NOT CONSTRUCT THIS BRIDGE UNLESS PLANS HAVE BEEN APPROVED BY A PROFESSIONAL ENGINEER.**

****TRANCAS ASSOCIATES RECCOMMENDS PERFORMING A FINITE ELEMENT ANALYSIS ON STRUCTURAL DESIGN TO CONFIRM STRUCTURAL CAPACITY**

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Executive Summary

This report describes a solution to a problem with a stream crossing affecting a transportation route to the rural community of Las Trancas, Panama. Trancas Associates, a design team, travelled to this problem site in August 2016 in order to collect data for the potential solution. Trancas Associates will detail their proposed solution in this report, including a bridge, channel, and roadway design, as well as a cost estimate and construction schedule.

Trancas Associates collected topographical survey, soil, and hydrologic data while at the stream crossing site for later use in drafting maps and figures used in an analysis of the site. Additionally, Trancas Associates consulted a Peace Corps volunteer living in the community to better understand the needs of the community and their hopes for the project. The results of this data analysis and consultation allowed the team to draw conclusions about the needs and constraints of the project, and create a design that meets them.

The final recommended design is a flexible buried steel bridge. This bridge was selected as it best handled the design constraints and needs of the project compared to other proposed alternatives. The final design was guided by numerous design guides and codes, including *CONTECH Structural Plate Design Guide*, *AASHTO LRFD Bridge Design Specifications*, *Building Code Requirements for Structural Concrete ACI 2014*, and *AISC Steel Construction Manual, 14th ed.*

Trancas Associates intends on providing this final design to the Las Trancas community for consideration. If the community accepts this proposal, an outside organization will be contacted to pursue the project further. Trancas Associates recommends that this outside organization review the data and calculations presented in this report, make arrangements for labor, material, and equipment, and oversee the project's construction. The main organization considered by Trancas Associates for this task is Engineers Without Borders - USA.

1.0 Introduction

The team members of Trancas Associates are Civil Engineering undergraduate students at Michigan Technological University. Through Michigan Tech's International Senior Design (iDesign) program, Trancas Associates traveled on an assessment trip to collect data for a potential vehicle bridge project. Trancas Associates has prepared this report outlining the analysis and design of a potential bridge servicing the remote, rural community of Las Trancas. Las Trancas is in the Comarca Ngäbe-Buglé province of Panama and is located just under 250 miles away from the capital, Panama City, by roadway. Figure 1 displays a map of Panama with Las Trancas' location indicated, as well as the Pan-American Highway that runs between the nearby village of Tolé and Panama City.

Figure 1. Map location of Las Trancas and roadway from Panama City (via Google Maps)

The stream crossing addressed in this project is on an unpaved transportation route leading from the Pan-American Highway into the Las Trancas community. Trucks and other large vehicles use this unpaved roadway for commuting people and delivering resources. This stream crossing is difficult for these vehicles to cross most times of the year, and is often impossible to cross in the peak of the rainy season. The soil in this area of Panama has low infiltration rates and during heavy rainfall, large overland flow accumulates in the stream. The community of Las Trancas has attempted to bridge this problem stream in the past, but those bridges were washed out shortly after their construction. A more permanent structure is needed to keep this transportation route open year round. The project site is visible in Figure 2.

Figure 2. Stream crossing near Las Trancas, Panama

The proposed solution to this problem is a flexible buried steel bridge. This type of bridge can convey large water flow rates, support large truck loadings and withstand the environmental conditions that the area presents. Constructing this type of bridge requires minimal specialized labor, has the greatest ease of mobilization, and requires a minimal amount of time to construct. Trancas Associates is confident that the proposed bridge will be an effective and durable solution that will prove beneficial to the community it will service.

The community would have to secure funding for the proposed project via a grant from the Panamanian government or a non-governmental organization. Grant opportunities for bridge projects of a similar scope usually receive allowances from the Panamanian government in the vicinity of $50,000. Trancas Associates has attempted to limit the cost of this proposed project near this value to have the project be considered feasible. A grant proposal has yet to be drafted by the community, and it is for this reason that there is no set start time proposed for this project. It is recommended that the project be completed during the dry season of the year, which begins in December and runs through mid-March.

The following sections of this report will further discuss the community and its transportation routes, provide an analysis of Trancas Associates' acquired data, and detail the proposed solution. The structural design calculations, the construction plans, and the project schedule and estimate of the flexible buried steel bridge are some key points that will be discussed. The appendices to this report provide more detail to topics referenced in the report body.

2.0 Community & Project Background

This section will provide an overview of the Las Trancas community, describe the transportation routes in the area around Las Trancas, and provide information about the project site.

2.1. Community Background

The community of Las Trancas is inhabited by native Ngäbe Panamanians displaced to the mountainous Comarca Ngäbe-Buglé province by Spanish conquistadors during the colonial period of Panama. These natives were historically a more nomadic people than they are today, travelling from area to area in search of better soils to farm. However, the government has reserved this mountainous area for these natives to permanently reside and call their own.

The community of Las Trancas is divided into two parts, Alto Las Trancas and Bajo Las Trancas, located next to each other along the one roadway through the village. The houses are spread out along the roadway, with large areas of farm land separating households. There is a central location within the village where the school, a satellite phone, and a soccer field are located. This is where special events are held, as Trancas Associates witnessed while staying in the village. Figure 3 displays this village center and some community members of Las Trancas.

Figure 3. Community members and central location of Las Trancas

The community has a population between 1500 and 2000, and is made up of between one hundred and two hundred households. This is a larger village as compared to many in the surrounding area. Socially, men are the primary income providers and farm laborers, while the women stay at home to tend to the daily household tasks. The community is estimated to be 55% women and 45% men, with about 50% of people aged under 15 years old, about 20% of people aged 15-24 years old, and about 30% of community members aged over 24.

The school within the community has approximately five hundred students and provides education at an elementary level through a middle school level. The teachers are mostly from elsewhere in Panama. Many of the students do not go beyond this in their education, although a few commute to outside areas for further education, if they can afford it or receive scholarships.

The community of Las Trancas relies primarily on subsistence farming, although this farming does not support all of the community's hunger needs. This community operates in a feast-or-famine manner, which means that if food is available within the community, it is served in large portions rather than small conservative portions. This method consumes the food supply quickly, leaving the village short on food between crop harvests. This means that some outside food is necessary to support the population during non-harvesting times.

The community does not have the infrastructure needed to support its relatively large population. For example, many houses do not have latrines or water collection systems, and some houses only have earthen floors. There was a previously installed water distribution system within the village that failed and is no longer operational. The roads around this village are also in very poor condition, which will be discussed further in a later section of this report. This village needs more infrastructure beyond the scope of this project to support their population and sustain growth.

There is a Peace Corps volunteer, Frank Dubasik, who has been living in the community since July 2015. Frank was Trancas Associates' main contact within the community, and housed the team during their stay within the village. He answered the team's questions about the community and explained the history of the problem stream crossing. Additionally, Frank provided the team with a community assessment report from his own research to answer any additional pertaining questions following the trip to the community (Dubasik, 2016).

2.2. Transportation Route Overview

Figure 4 displays a map view of key transportation routes in the area around Las Trancas. The town of Tolé is a key town in this area, as it is located along the Pan-American Highway, which is the largest transportation route in the country. All the outside resources to the Las Trancas community are brought in from Tolé. Trucks carry people and goods from Tolé to Las Trancas via an unpaved route, which is the primary route into the village. This route loops through Las Trancas, up to the village of Chichica, and back down to a partially paved stretch of roadway to Tolé. The location of this project is on an unpaved stretch of roadway between Las Trancas and Chichica, which is currently on the secondary route into the village. The primary route into the village takes up to two hours to traverse and is approximately seven miles long. The secondary route to the village is approximately thirteen miles long and takes longer to traverse.

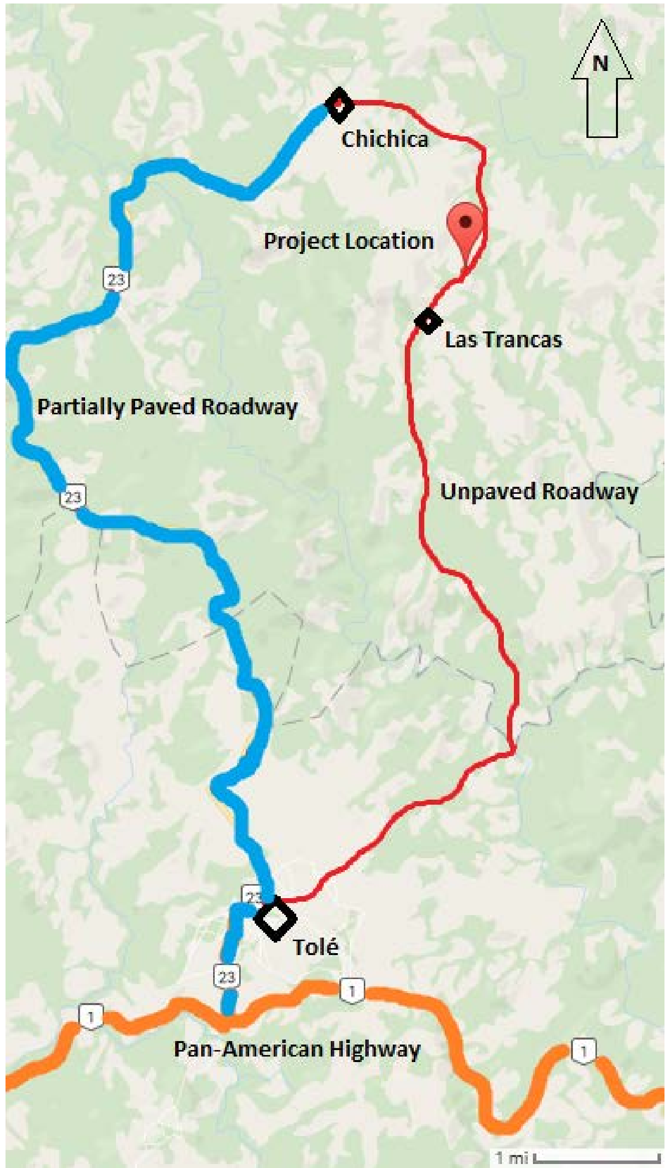


Figure 4. Map detail of transportation routes near Las Trancas (via Google Maps)

The unpaved sections in Figure 4 are in poor condition. The roadbed consists largely of clay and large stones, and has steeply graded uphill and downhill stretches. This makes travel along the roadway exclusive to pickup trucks and other large vehicles with strong suspension systems. The roadway also contributes to large runoff volumes and large buildups of mud due to the low infiltration rate of the soil. This roadway is manageable in the dry season, but becomes much more difficult to traverse in the rainy season. Figure 5 depicts some typical stretches of this roadway.

Figure 5. Unpaved roadway around project location

The roadway between Chichica and Tolé is paved and graded in areas closer to the town of Tolé, and is under construction to be paved in areas near the village of Chichica. Paving this stretch has the potential to make the secondary route into Las Trancas the primary route, due to its shorter unpaved length. Additionally, opening uo this route has the potential to create a paved path into Las Trancas via the current secondary route. A bridge over the problem stream crossing is needed to make this current secondary route a reliable option for transport to and from Las Trancas.

2.3. Project Location

Figure 6. Roadways leading into project location (a) looking north, (b) looking south

Figure 6 displays the roadway leading into the project location. Approaching the project location on either side of the crossing, the roadway splits into two paths. One path leads to a ford through the stream, and the other leads to the remnants of a failed bridge built by community members. The roadbed on the south side of the stream crossing has a steep grade of around twenty percent directly after the two paths merge, and continues to run perpendicular to the stream crossing for a few hundred feet. The roadbed on the north side of the stream crossing has a much lower grade, but turns sharply directly after the stream crossing and then runs parallel with the stream, due to a large soil bank located close to the stream channel. The roadbed at the project location is similar to the rest of the unpaved roadway, consisting of fat clay and large stones.

The village of Las Trancas attempted to create their own solutions to the problem stream crossing in the past. Figure 7 displays the community's most recent attempt at creating a roadway across the stream, as well as what remained of it while Trancas Associates was at the project site. The roadway was constructed using a base of reused concrete culverts and rip-rap, with gravel backfill placed overtop acting as a roadway. This design was constructed out of cheap, readily available components near the project site. This design greatly restricted the flow of the stream and did not withstand the large flows associated with a large rainfall event.

Figure 7. Past attempt at a solution for the problem stream crossing

Currently, vehicles pass the crossing through a ford in the stream when the water level is low enough. It was witnessed that numerous trucks struggled to pass through this ford, losing traction on the steep grade and bottoming out on the sharp grade change leading up to it. Pedestrians can cross through this ford, too, or can cross over on the remaining concrete culverts from the past bridge attempt. However, once the rainy season starts, the water level rises and the stream flows faster, making it impossible for vehicles to pass using this route, and less safe for pedestrians. During the rainy season, only one access route to the village is left for vehicles to use. This single route is unreliable because it can be impassable after large rainfall events. Trancas Associates witnessed one of these large storms during the assessment trip. This rainy season lasts around from mid-March until January. Figure 8 displays this ford through the stream crossing.

Figure 8. Ford path through the stream crossing

The need for a more permanent structure over this stream crossing is clear. A structure is necessary to keep the secondary route into the village of Las Trancas open and accessible to community members during all times of the year. The structure must be able to handle large flow velocities, support moderate single truck loadings, and be resistant to environmental factors surrounding the area in order to be an effective, sustainable, and durable solution. Trancas Associates is confident that the solution proposed in this report will meet these criteria.

3.0 Data Collection Methods, Procedures, and Analysis

This section will outline the data that Trancas Associates collected while at the project location, the procedures followed to collect that data, the data analysis process, and the results of that analysis.

3.1. Surveying & Topographical Mapping

Surveying was performed on site in order to collect the data for topographical mapping. Primarily, level surveying was conducted. The data collection equipment included a GPS, a compass, an abney level, a six-foot carpenter's rule, twenty-five-foot box tape, and a one-hundred-foot-tape. Survey data with the applied correctional factors is compiled in Appendix A.

Work started by setting a control point with a GPS at a central location near the stream where a majority of the topography could be seen. This GPS gave a reliable horizontal base point for tying in our survey to a map location.

Following this, a series of level loops were performed around the site location. Each loop would begin with a team member holding the carpenter's rule plumb over the control point. Another team member would backsight the carpenter's rule with the abney level placed on a straight stick at zero degrees (level). From here, multiple foresights were taken on points of interest at the site. These foresight measurements gave the elevation change from the control point to the critical points of the area. Once the elevation of a point was known, a compass bearing was taken and a horizontal distance was measured with the one-hundred-foot tape. Once the carpenter's rule was too high, too low, or too far away from the level, a new backsight would be set from the most recent foresight and the process would repeat. A loop ended with a final foresight to the control point.

Due to the nature of the instruments used, there was some horizontal and vertical error introduced in the measurements. This error was due to the instrument stick not being perfectly straight, sag in the cloth measuring tape, not having a plumb gage to ensure the instrument stick remained plumb, and the degree of precision with which the equipment can be read. Many small loops were performed rather than larger loops to decrease the overall error in a loop. Correction factors were applied to each loop based on the total error to equally divide the corrections across the whole loop. Figure 9 displays the team performing a level loop.

Figure 9. Level loop surveying on site

Additionally, a Nikon rangefinder, a target, and the compass were used to collect additional data on the less critical points of the site. This was done by having a team member stand over the control point with the rangefinder and shooting a target held plumb by another team member. The target was placed on the instrument stick at the same height as the rangefinder. The slope distance and angle of each rangefinder shot was recorded, as well as the horizontal compass bearing.

Cross sections of the stream were also surveyed at the site location to assist in hydrologic analysis. A foresight would be taken across the stream channel and the hundred-foot tape would be strung horizontally across. The pocket tape was drawn down from the one-hundred-foot tape to different points on the stream channel and the elevation changes were recorded. This was done in four locations, yielding four cross sections for analysis.

All of the recorded surveying data was analyzed and converted into a comprehensive topographic map. The foresight, backsight, bearing, and horizontal distance data was converted into local Cartesian coordinates in a Microsoft Excel spreadsheet. A single local Cartesian coordinate system was obtained by combining individual local coordinate systems. These coordinates were converted into a text file which was imported into AutoCAD Civil3D. This program's analysis tools were used to transfer the point coordinates into a three-dimensional model of the site. Important site features identified in the survey were noted on this model, and each node was connected together to form contour lines. This model was used to create a topographical map of the site and profile views of the stream channel. The topographical map of the site is shown below in Figure 10.

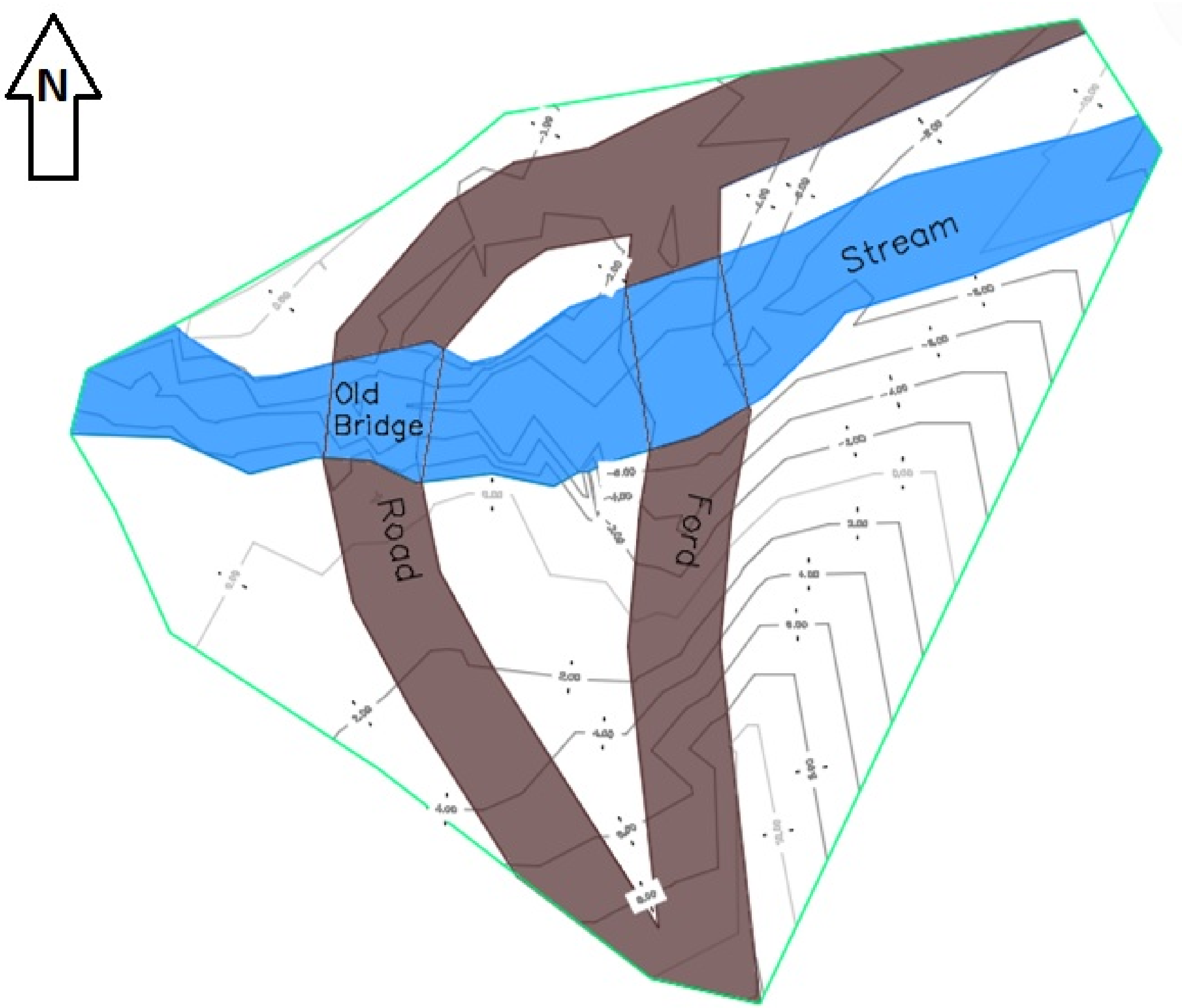


Figure 10. Topographical map of site location with significant features indicated

3.2. Soil Analysis

Soil data was also gathered at the site and analyzed to determine properties for use in design. A visual classification was performed on site since soil test equipment was not available, and a small sample was gathered and tested offsite. This sample was testing using ASTM D2488-09a, (2009) and Reddy, (2002). Tests performed include a dry strength test, a dilatancy test, an odor test, a plasticity test, a soil toughness test, and a moisture condition test. Test procedures on this sample followed the ASTM standards as closely as possible. The only deviation from these test procedures is that a microwave oven was used instead of a conventional oven to heat the sample for the dry strength test. A microwave oven was used due to the lack of access that Trancas Associates had to a conventional oven while staying in Panama City. Test results are visible in Appendix B.

These soil tests allowed Trancas Associates to classify the soil on the ASTM scale, and draw conclusions based on its working properties. The soil was classified as a brown-red fat clay, high plasticity (CH). This soil has poor foundation and drainage properties due to its large, slow-acting settlement and its low permeability. Figure 10 displays the soil sample and one of the bank walls of the stream crossing.

Figure 11. Soil sample and stream bank wall

3.3. Hydrologic Data

Trancas Associates performed a watershed analysis on the upstream area leading into the project site in order to determine maximum flow rates in the channel. This information was critical for properly designing a structure to endure large rainfall events. The NRCS peak discharge method was used to estimate the design flow. This flow value was applied to Manning's equation to ensure sufficient discharge capacity of the channel. These methods are outlined in Wurbs, (2002). All calculations for this process are detailed in Appendix C.

The watershed above the channel was modelled using Google Earth, and was estimated to be a third of a square mile ($0.33mi^2$). This watershed model is visible in Figure 11. The 100-year, 24-hr rainfall event of the Las Trancas area was estimated using data provided in Shamir, (2013), yielding a six-inch, twenty-four-hour design rainfall event. This source analyzed rainfall events at the Panama Canal watershed rather than our project location, but this data was considered the most reliable that was available.

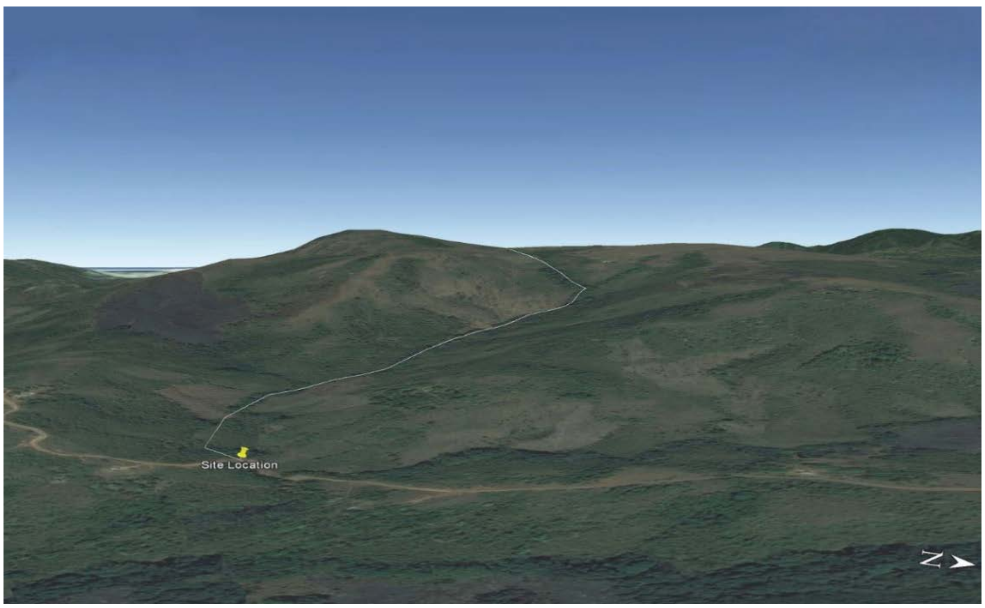


Figure 12. Google Earth watershed model

This design storm and watershed model were used to create a storm hyetograph of the rainfall event using the NRCS Type II rainfall distribution, and then a discharge hydrograph through the channel using a NRCS triangular unit hydrograph. This hydrograph provided a maximum flow rate through the channel at the project location, which was calculated to be 280 cubic feet per second.

The maximum stream depth and velocities were then calculated using Manning's equation with the surveyed cross sections. This yielded information on flow behavior through the channel, and gave insight to channel sizing requirements in design.

4.0 Design Constraints, Assumptions, and Alternatives

This section will outline the constraints adhered to by the final design, the assumptions Trancas Associates made to facilitate calculations and design, and selection of the final design from considered alternative designs.

4.1. Design Constraints

This stream crossing had a number of unique characteristics to it that were used to rank proposed design alternatives on their effectiveness. Key design constraints considered include the site's hydraulic conditions, the remote location of the site, the soil properties, the topography around the site, the cost, and the construction time.

A design will have to be able to resist the damaging effects of the stream at its peak discharge to be feasible. The design will have to safely convey the maximum flow rate as determined in the hydrologic analysis. Additionally, the footings of the alternative would also have to withstand the scour associated with high velocities running through the channel.

The remote geographic location and the condition of the unpaved roadway leading to the project site also serve as constraints. These factors restrict the transport of large sizes and volumes of material into the site. They also provide limited options for heavy equipment use, as only smaller pieces of equipment could be mobilized effectively. Ideally, the roadway leading into this project location would be paved and graded prior to the start of construction on this proposed project. However, this project was estimated and planned in the context that the roads leading to this project location would not be improved under the scope of the project.

Cost was a key constraint considered. Any design alternative would have to minimize labor, material, and equipment costs to keep the costs below $50,000. This is key in increasing the feasibility of receiving a grant to fund this project.

The soil properties of the fat clay surrounding the site also caused some design constraints. These soils often cause large, slow-acting settlements beneath structures built on top them. Any design alternative must be minimally affected by differential settlement of these soils to be effective. Additionally, the low infiltration rate of these soils leads large overland runoff flows. These runoff flows must be considered to protect the roadbed from being washed out.

The topography of the site was also considered. The roadway would have to avoid soil banks surrounding the roadway, particularly the one directly to the north of the stream. Also, the finish grade of the roadway would have to allow vehicles to pass effectively.

Any alternative must be constructed starting at the beginning of the dry season to be effectively constructed. Preferably, construction would end in the same dry season, or shortly after it. This is to minimize the risk that construction will not be slowed, interrupted, or damaged by large rainfall events.

4.2. Design Assumptions

A few assumptions were made for ease of design. First, all soil properties are assumed uniform. By visual analysis of the site, it appeared that this assumption held true, as the soil at surface level was the fat clay type soil defined during Trancas Associates' limited soils testing. However, since extensive soil analysis could not be performed, it must be assumed that this is the case.

Second, Trancas Associates was informed that a supply pit for gravel, rip rap, sand, and cement exists near the village of Chichica. This supply pit was never visited by Trancas Associates, so some assumptions had to be made about it. It was assumed that the location of the supply pit would be about a six-hour trip for a fully loaded supply truck to drive to the project location.

Finally, it was assumed that all steel components within a proposed design could be fabricated and sourced from a Panamanian manufacturer. This was assumed to cut down on manufacturing and shipping costs of the steel as compared to imported steel.

4.3. Design Alternatives & Final Selection

The span of this crossing is quite small, and therefore many different design alternatives were considered as possible solutions. Trancas Associates evaluated the design alternatives against each other to decide upon the best possible final design. A decision matrix of these alternatives is shown in Appendix D. Key criteria for this comparison came from the design constraints. Once this comparison was completed, Trancas Associates modeled and performed a preliminary cost analysis on the highest weighted alternatives. These alternatives included a concrete box culvert, steel truss bridge, wood truss bridge, and flexible buried steel bridge. These calculations and models are in Appendix D as well.

The box culvert was first considered because it has been implemented in other areas in the Comarca near the project site. This alternative was cheap, but due to the need to divert the stream for in-place casting, difficulty of casting using hand mixers, and observed roadbed washouts at other locations, the alternative was not selected.

The steel and wood truss bridge alternatives were also carefully considered. They both could provide high channel clearances as needed and handle large vehicular loads adequately. However, these structures would require large footings that would require extensive excavation and large-volume concrete pours from small hand mixers, and are prone to large differential settlements on the fat clay base material. Piles would have to be driven to prevent large settlements. Stiff bridges such as the wood truss and the steel truss bridge do not tolerate large settlements. Without geotechnical borings, it would be difficult to accurately assess the number of piles and the depth that the piles would need to be driven to prevent any large settlements from occurring. Additionally, mobilizing the trusses to the site would prove difficult. It was for these reasons that these alternatives were not selected.

The flexible buried steel bridge was the highest ranking alternative. This structure is an arch built from several corrugated steel plates bolted together and anchored to concrete spread footings. Overtop this structure, well-graded gravel is backfilled and compacted in 6 – 8 inch lifts. The strength of the compacted gravel distributes the vehicle loads above the structure down through the gravel towards the structure's footings and the gravel at the stream banks. This structure has a wide area underneath that provides capacity for large flow rates. A wide channel width not constrict the flow of the stream, leading to lower flow velocities. Lower flow velocities will reduce the effects of scour on the soil surrounding the footings.

This alternative excelled in criteria that the other alternatives did not. The structure has the highest ease of mobilization due to the size of the individual plates and reliance on soil backfill. These materials are more maneuverable than trusses would be. The structure can support very large top loadings, requires minimal excavation for construction, and is relatively affordable. The foundations are also less affected by differential settlement than the truss alternatives would be. There is no need to drive piles with this design. This lowers the construction time, the construction costs, and the amount of equipment that needs to be brought in to the project site. These structures have a typical service life of around 50 years.

This design was not flawless, though. The high flow velocities of the channel during large rain events can cause scour of the structure's footings. Special considerations must be considered with this structure for channel shaping and footing protection. Also, the final grade of the roadway approaching the channel must be kept to a minimum for vehicles to effectively pass over the structure. A large amount of backfill is needed to achieve this approach grade. Finally, headwalls must be incorporated into the design to prevent the roadbed above the structure from washing out from loadings and drainage.

Despite the special considerations that must be taken into account with this design, the flexible buried steel bridge met the design criteria better than the other design alternatives considered, and was selected as the final design. Once this alternative was selected, specific project design and planning commenced as detailed in the following section of this report.

5.0 Final Recommendations

This section will detail our final design recommendations, including the structure loadings, the steel structure design, the footing design, the channel design, the roadbed design, and the headwall design. Additionally, the construction schedule and cost estimate for this design will be discussed. Several textbooks and design manuals were used in this design, and are referenced in section 7.0 of this report. Design drawings and detailing of this final design are shown in Appendix J, which provides more detail on topics outlined in this section of the report. A construction manual that coincides with these final design drawings and outlines basic procedures for proper installation has also been attached as Appendix M.

5.1. Design Recommendations

5.1.1. Loadings

The dead load for this flexible buried steel bridge was based on the weight of the gravel backfilled overtop the structure, the weight of the steel headwalls enclosing this backfill, and the weight of the structure itself. This backfilled gravel is by far the most significant contributor to this dead load.

The live load for this structure was estimated from on-site observations, expected future loadings, and standard loading models presented in *Structural*, (2016). While on-site, it was observed that standard pickup trucks and off-road SUV's were the main vehicles that were traversing the ford through the stream. These pickup trucks were often fully loaded, filled to capacity with people and supplies. The proposed structure was to only allow one vehicle to pass at a time, so therefore the maximum loadings could be represented by one fully loaded pickup or SUV. However, if this roadway were to be open and traversable year-round, it was suspected that larger vehicles would try to cross this structure as well.

The referenced design guide allows users to estimate live loadings on a flexible buried steel bridge based upon standard U.S. highway loadings models. Trancas Associates selected half of an HS-20 highway loading to estimate live loads during structural calculations. In addition to standard-size commuter vehicles, this HS-20 loading model is meant to encompass loadings of large semi-trailer trucks, which would not be able to commute the roads leading to the site. Based on this reasoning and the observations made on-site, half of an HS-20 loading was deemed sufficient to model structural loadings. Loading calculations are provided in Appendix E.

5.1.2. Steel Structure Design

The steel structure was designed using *Structural*, (2016) and *Corrugated*, (2008). *Structural*, (2016) was the primary design guide followed, but *Corrugated*, (2008) was used when needed values for calculations could not be found. These design guides outline the AASHTO Service Loads Design method for flexible buried steel bridges, and provide standard sizing information for corrugated steel plates. The AASHTO design method uses the design loads to compute wall thrust, which is used to compute the structure's buckling capacity and check the flexibility requirements and seam strengths of the corrugated plates. All the calculations for the design of the steel structure were performed in MathCAD and are provided in Appendix E.

The steel structure has a double-radius arch shape, with a span of 23 ft.-5 in., a width of 18 ft.-9 in. and a 9 ft.-6 in. rise. These dimensions best fit the site's topography and the current path of the roadway, and were presented as a standard size structure in *Structural*, (2016). The span of this bridge is long enough to exceed the channel, the width of this bridge allows one vehicle to pass overtop at a time, and the rise of this bridge provided a large head clearance for between the stream and the top of the crown plate. This clearance is important because it protects the bridge from potentially destructive debris that could be carried downstream during large rainfall events. The double radius shape was selected to keep a low bridge profile while allowing the stream to safely flow with a minimal structural rise as compared to a single-radius arch shape.

The corrugations of each plate have a depth of 5.5" with a 15" pitch spacing between them, with a plate thickness of 0.188". These are standard corrugation sizes presented in *Structural*, (2016), designated as 15" x 5.5", gage 7, and sold commercially as BridgeCor steel. This is a deep corrugation size that provides a large amount of strength with a minimal need for backfilled soils. This corrugation depth and thickness is large enough to support top loadings much larger than the estimated loadings with a specified factor of safety.

Standard plate sizes are presented in *Structural*, (2016). Ten 8S plates and five 9S plates are required for this structure, or fifteen plates total. These plates are made from steel conforming to *ASTM A761,* (2009). This ASTM specification also outlines the steel requirements for bolts used to connect the structural plate. These bolts must be of high-strength to avoid shear and tensile failures.

These steel plates will be mobilized using pickup trucks, and the full structure will be assembled on site. There is no need for large, flatbed trucks to transport this material, providing better constructability for the project. This steel structure also does not require skilled labor to construct, the plates just need to be positioned with an excavator and bolted into place.

Design detail drawings for this structure were created in Autodesk AutoCAD 2015, and a model was constructed in Siemens NX. These documents are shown in Appendix H, and provide further detail of this structure.

5.1.3. Footing Design

The footings of the structure were designed using *AASHTO, (*2012) and *Building,* (2014). The design was based around the settlement properties and the bearing strength of the soil on site. *AASHTO,* (2012) was used to calculate the bearing stress of the soil and *Building,* (2014) was used to calculate concrete and rebar requirements. Design calculations were performed in MathCAD and are provided in Appendix F.

The exact soil bearing strength could not be calculated using provided equations due to the limited amount of testing equipment that was available to be used during soil analysis. Therefore, a standard bearing strength for fat clays was selected from a Table C10.6.2.6.1-1 in *AASHTO,* (2012), and is approximately equal to 4000 psf. Spread footings were sized to transfer the load from the bridge to the supporting soil. Trancas Associates found spread footings to be the best footing design given the amount of geotechnical data available.

Normal weight concrete may be used for these footings. A concrete strength of 4000 psi was used in design. The reinforcing rebar was selected to be #6 bars, tied together with #3 stirrups. A detailed drawing of the footing design is shown in Appendix H. Placement of these footings at the site location is also shown in Appendix H. The positioning and pouring these footings is critical to properly attaching the steel structure.

The construction plan is to excavate footing locations, position the rebar, and pour the concrete without forms against the native soil. Pouring against native soil saves on excavation costs and cuts down on required formwork, and preserves some of the surrounding natural soil strength. Riprap will be placed overtop these footings to help protect the surrounding soil from scour.

5.1.4. Headwall & Wingwall Design

A headwall system was implemented in this design to prevent the soil backfilled over the steel structure from being washed out by roadbed drainage. These headwalls are critical to retaining the strength of the bridge because the bridge relies on the interaction between the corrugated steel and the soil for its strength. The headwall design was guided by *Structural*, (2016), *Corrugated*, (2008), *AISC*, (2011), and Coduto, (2011). The calculations for the headwall system are shown in Appendix H.

The materials that were considered for the headwall design included riprap, masonry block, concrete, and corrugated steel. Riprap was not used because the slopes required to place the rip-rap would require a larger width of the steel structure that would not be economically feasible (according to Sturm, 2009). Trancas Associates was also concerned the riprap would not adequately protect the gravel from washout during heavy rainfall events. Masonry block was not selected due to concerns about the effect that differential settlement would have on the block. Cracking would likely result if either the north or south footings settled more than the other. Flexible buried steel bridges perform well under differential settlement, but masonry block does not tolerate large deflections. The use of masonry block would not align with the advantages of a flexible buried steel bridge and would hinder the design. Concrete also cracks under large settlements and would be difficult to cast, so it was not selected. This led Trancas Associates to select a corrugated steel material for the headwall. A 6" x 2", gage 8 corrugation size was determined to be of sufficient strength to withstand the flexibility and strength constraints for headwalls given in *Structural*, (2016).

The corrugated plates were sized so that they could be easily transported to the site and bolted into place. Standard plate sizes were selected from Table 2.62 in *Corrugated*, (2008). The headwall plates must be installed with a foot of overlap between plates in order to achieve a strong connection, ensuring that these plates will not bow out at the seams along the structural plate. These seams are bolted at a maximum of 16 inches.

Anchor rods will have to be installed along the headwall to assist the headwall in resisting pressure from the soil acting upon it. This pressure could topple the headwall. The anchor rod connections were designed using the *AISC*, (2011), and the soil pressure acting on the headwall was determined using methods outlined in Coduto, (2011). These anchor rods will connect the top of the headwall to the crown plate of the steel structure. These rods are at an angle so that they are well below the surface to the soil where they pass beneath the roadway. This will minimize damage to the anchor rods due to traffic loads and will ensure that the rods will not be uncovered by traffic passing over the roadway. The anchor rods are placed at each seam, providing additional support to the connection between the plates. The headwall was attached to the crown plate of the bridge and the footing using angle sections.

Additional horizontal anchor rods are to be placed on two of the seams, further securing the two plates together and providing additional support to the headwall. These were necessary to resist the pressure gradient from the soil that increases with the depth of the headwall. All connections for the headwall were designed as bolted connections to limit the amount of construction expertise and equipment necessary for the assembly of the headwall. A guardrail will also be connected to the top of this headwall for the safety of vehicles and pedestrans traveling across it.

Wingwalls are also needed in addition to the headwall system to protect the roadway from being washed out by drainage on either side of the structure itself. Masonry blocks will serve as these wingwalls. No design manuals had sample designs of these wingwalls, and therefore none of these manuals lead the design. Each side of the stream has a different sized wingwall to match the size and shape of the approaching roadway.

5.1.5. Channel Design

The channel surrounding the structure will require some reshaping and riprap placement in order to better control flow and to protect the footings against scour. The design of the channel was guided by *Riprap*, (1997), Strum, (2009), and Wurbs, (2002). The calculations for the rip-rap placement are shown in Appendix G. This channel design is crucial in protecting the bridge.

The bank walls of the stream must be excavated and reshaped in order to properly place riprap and limit erosion of the banks. The banks are to be excavated at a slope of 3.5:1, and riprap will be placed on the cut sections at a slope of 3:1. The upstream section of the stream currently has tall banks and will require the most excavation. The downstream section of the stream is already cut due to the ford through the stream and will only require placement of riprap and no excavation.

Using the peak flow rate calculated in the hydrologic analysis, the max stream height and velocity through the reshaped channel were found. The maximum stream height was found to be 30 inches and the max stream velocity was found to be 8.4 feet per second. These calculations are shown in Appendix C. The structure and channel will be able to endure this peak stream height and stream velocity.

5.1.6. Roadbed Design

The design of the roadbed at the project site was based on the minimum required height of fill over the top of the steel structure determined in the steel structural design. A two-foot rise of gravel is required overtop the peak of the structure, and the grade across the structure must be level. The grading of the roadway approaching the structure must be respective to this peak roadway height and level grade over the structure. The approaching roadways must be placed to fit the bridge location and skew, and extend far enough to achieve the proper grades. Trancas Associates recommends that the approaching roadways be graded to the specified percent grades given in Appendix J to allow for proper entrance and exit from the bridge.

The roadway will have a lane width of 18.75’ overtop the structure, with the approaching roadways widening to meet this width. The roadbed will also have a five percent crown to allow for effective drainage of rainfall and runoff flow off of the roadbed.

The recommended fill is a well-graded, angular gravel with soil diameters between one and one and a half inches (1” – 1.5”). This gravel would have to be brought to the site and stockpiled until it is needed to continue construction. The gravel will be carefully backfilled and compacted, especially over the steel structure. Lifts of six to eight inches will be placed over the structure at a time, and will be compacted to 90% of the soil’s maximum density before another lift can begin. Soil will be backfilled from the headwalls inward towards the center of the structure, and lifts will occur so that the amount of soil overtop each side of the span remains equal. This is to ensure that the weight overtop the structure will stay evenly distributed during the backfilling process, preventing the steel structure from becoming warped during the process. Backfilling of the roadbed not overtop the structure can be done in larger lifts of eight to twelve inches, again compacted to 90% of the soil’s maximum density.

The backfilling of the gravel overtop the steel structure is the most important step to assuring that the structure achieves full strength. Warping of the steel plate, insufficient compaction, or improper lift sizes can compromise the ability of the structure to handle the loads it was designed for.

5.2. Construction Scheduling and Cost Estimation

Trancas Associates has assembled a construction schedule for potential contractors, outlining individual construction tasks and assigning construction time. This construction schedule is shown in Appendix K. This construction schedule was set to begin at the beginning of January, which is the approximate start of the Panamanian dry season. If all goes as planned, construction will last 59 working days and be completed in mid-March, which is the approximate end to the Panamanian dry season. The schedule duration may fluctuate depending on weather, material availability/transportation, and availability of skilled labor.

First, a work breakdown structure was made to identify and discuss key construction tasks associated with the project. This work breakdown structure is shown in Appendix K as well. Each of these key tasks were added to a Microsoft Project document, and broken into subtasks of specific work to be performed. Each of these subtasks were assigned time durations, required equipment, and predecessor subtasks. These subtasks were then scheduled relative to their time requirements, predecessor task time requirements, and equipment availability on any given day. This assured that the schedule followed a logical order and that equipment was not being over allocated.

The key tasks of the schedule consist of material preparation, mobilization, site preparation, footing installation, steel plate assembly, headwall assembly, wingwall assembly, riprap backfilling, roadbed creation, site repair and clean up/demobilization.

Along with this construction schedule is an accompanying project cost estimate. This cost estimate provides an approximate construction cost, which can serve useful for acquiring a grant and bidding to contractors. This cost estimate considered the construction costs as broken into four groups; material, equipment, labor, and hand tool costs. Hand tools could have been grouped under equipment, but was instead broken into a separate group. This was because these are small pieces of equipment that a contractor is likely to already possess, and therefore could be eliminated entirely from the project cost if this is the case. This cost estimate is found in Appendix L.

Quantities for material pay items were calculated in Appendix I, and were based on the final design requirements. Quantities for labor pay items were determined from the working times presented in the project schedule. Quantities for equipment pay items were based on if a piece of equipment had to be rented or bought outright. Rented equipment quantities were based on the amount of time they were used as determined from the project schedule. Equipment bought outright was quantified per number of the pieces necessary.

The unit costs for material, labor, and equipment were estimated based on the US rate for material and equipment. This was done due to the lack of availability to Panamanian pricing of these materials and equipment. The equipment rates were thought to be remain constant between the US and Panama, while the material and labor were thought to vary slightly from the US rates. Labor rates were estimated to be lower than the standard US rates found, and materials were priced differently based on the manufacturing costs and transportation times from their sourcing locations. These unit cost estimates mainly came from Fortier, (2014) unless otherwise noted in Appendix L.

The final cost estimate was found to be approximately $67,000. The most maneuverable versions of equipment were estimated, for example a small excavator and small concrete hand mixer were selected over larger versions of this equipment. Again, the cost estimate was priced to include all equipment necessary for the project competition, and did not consider the possibility of contractors already owning required equipment. Any materials or unskilled labor that can be donated would help lower the overall cost as well.

5.3. Funding and Maintenance

Funding for this project would come in the form of a grant from the Panamanian Government for a vehicular transportation bridge or a similar grant from a non-governmental organization. Grant opportunities from the Panamanian government for bridge projects of a similar scope usually receive allowances in the vicinity of $50,000. This grant would need to cover the costs of all material, labor, and equipment, as well as some overhead costs. Based on the cost estimate, this project can come close to this typical grant allowance, especially if some of the costs can be reduced as previously discussed.

The design of this structure was made to reduce the amount of maintenance required and protect against possible damage, although it is not immune to the effects of being in service. The steel structure has an average service life of fifty years if routinely inspected and maintained. Any maintenance work should be addressed immediately following a routine inspection if any potential issues are found. Trancas Associates recommends that this structure be fully inspected at least once a year. A community member or outside organization should be tasked with this routine inspection to assure this structure is not incurring any significant damage. Maintenance is not covered under the project estimate.

The gravel roadbed must be checked to assure that compaction percentages stay high, and that the roadbed is not being washed out. Proper gravel levels and compaction are essential to keeping the structure keeping its strength. The steel structure, headwall, and bolted connections should be checked for corrosion and unwanted deflections. These effects could also reduce the strength of the bridge by creating weak spots that could lead to failure. The masonry wingwalls should also be checked for significant displacement and maintained accordingly to ensure the soil remains confined and the roadbed remains intact. The riprap placed along the bank walls may need to be inspected and replaced if significant erosion begins to appear. This stream has been analyzed to have high flow velocitiesthat could damage this riprap and cause damaging scour to the foundations.

6.0 Conclusions & Next Steps

This report has outlined Trancas Associates' plans for a flexible buried steel bridge to be put into service. This bridge will be constructed on the secondary route to the community of Las Trancas, Panama and will allow this transportation route to be kept accessible year-round. This route is crucial to the delivery of necessary goods to the Las Trancas community, and for the commuting of persons in and out of the village. Trancas Associates is confident that the proposed design will best address the constraints of the project, and will meet the needs the community it services. The community will be provided with a copy of this report to decide if this project is favorable, and if it is indeed favorable, Trancas Associates plans to present this report to an outside organization such as Engineers Without Borders. It is hoped that this outside organization will review the presented data and calculations, and oversee this project's execution. Overseeing this project would include assisting the community in applying for a grant, making arrangements for supplying labor, material, and equipment, and overseeing the project's construction. Before any of this occurs, a professional engineer should be consulted to review this report.

7.0 References

AASHTO LRFD Bridge Design Specifications. (2012). American Association of State Highway and Transpiration Officials; Washington, DC. Print.

AISC Steel Construction Manual, 14th ed. (2011). American Institute of Steel Construction; Chicago, IL. Print.

ASTM A761, Standard Specification for Corrugated Steel Structural Plate, Zinc-Coated, for Field-Bolted Pipe, Pipe-Arches, and Arches. (2009). ASTM International; West Conshohocken, PA. Online.

ASTM D2488-09a, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). (2009). ASTM International; West Conshohocken, PA. Online.

Building Code Requirements for Structural Concrete (ACI 318-14). (2014). American Concrete Institute; Farmington Hills, MI. Print.

Coduto, D.P., Yeung, M.C., & Kitch, W.A. (2011). *Geotechnical Engineering: Principles and Practices*. Pearson Education; Upper Saddle River, NJ. Print.

Corrugated Steel Pipe Design Manual, 1st ed. (2008). National Corrugated Steel Pipe Association; Dallas, TX. Print.

Dubasik, Frank. (2016). *Peace Corps Panama Environmental Health Project: Community Analysis and Development Plan (CDAP).* Las Trancas, Panama. Online.

Fortier, Robert, ed. (2014). *RSMeans Heavy Construction Cost Data 2015*, 29th ed. Construction and Consultants; Norwall, MA. Print.

"Panama 1:50,000." Army Map Service of the United States. 3841 II Rio Tabasara. U of Florida: Map and Imagery Library, n.d. N. pag. Print. Rio Tabasara.

Reddy, Krishna. (2002). *Engineering Properties of Soils Based on Laboratory Testing*. Online.

Riprap. (1997). Michigan Department of Transportation: Lansing, MI. 1-17. Online.

Shamir, E, Georgakakos KP, Murphy MJ. (2013). Frequency analysis of the 7-8 December 2010 extreme precipitation in the Panama Canal Watershed. *Journal of Hydrology*. 480:136-148. Online.

Structural Plate Design Guide, 6th ed. (2016). CONTECH Engineered Solutions; West Chester, OH. Print.

Strum, Terry W. (2009). *Open Channel Hydraulics,* 2nd ed. McGraw-Hill Education; New York City, NY. 129-133. Print.

Wurbs, Ralph A., and Wesley P. James. (2002). *Water Resources Engineering*. Prentice-Hall; Upper Saddle River, NJ. Print.

8.0 Appendices

Appendix A:

Survey Data

CP1 has a latitude/longitude of *(8.349420, -81.633281),* with an arbitrary elevation of 0ft, due to inaccuracy of the GPS used in vertical positioning. All given northings, eastings, and elevations in Table A1 are relative to the coordinates of CP1. A CD has been attached to this report that has a Microsoft Excel spreadsheet containing this survey data.

Table A1. Survey Data

Point #	Point Description	Easting (ft)	Northing (ft)	Elevation (ft)
1	Instrument	-8.73965	-2.72477	0.421533
2	Road begin CL at riverbank	-18.4517	4.53803	-0.57256
3	Instrument	-23.3367	-3.89958	-0.5503
4	Road CL	-15.0274	-12.001	1.205185
5	Instrument	-14.1824	-26.2436	2.133478
6	Road CL 2	-7.92514	-24.8642	2.432426
7	Instrument	-5.57937	-39.3379	3.461316
8	Road CL 3	0.98409	-38.8758	3.960463
9	Instrument	10.68166	-41.0512	4.984117
10	Road CL end	18.30181	-63.2402	10.00461
11	Instrument	21.25254	-51.5091	8.131441
12	Ford CL begin	27.56279	-50.3592	8.330389
13	Instrument	14.72978	-33.5006	4.618624
14	Ford CL	23.37031	-38.3074	5.822013
15	Instrument	16.23057	-23.7829	1.853819
16	Ford CL curve	21.33186	-19.8229	1.302899
17	Instrument	18.64044	-5.77033	-3.16755
18	Ford CL	22.79625	-2.29024	-4.11993
19	Instrument	19.15501	6.234968	-7.0974
20	Ford CL end	25.00879	10.12117	-6.99752
21	Instrument	19.10847	3.210224	-5.97612
22	Bank	10.13743	3.037347	-1.87472
23	Instrument	10.46419	-4.66408	-1.00485
24	CP 1	6E-15	-1.5E-14	-3.6E-15
25	Instrument	31.41939	23.3571	-7.82011
26	River CL 2 Begin by falls	38.13109	27.2321	-8.57011
27	River CL 2	32.12845	19.3358	-8.37011
28	River CL 3	26.15381	16.10966	-7.87011
29	River CL 4	20.51878	10.36627	-7.52011
30	River CL 5	11.96912	7.914406	-7.32963
31	Instrument	1.949396	17.57865	-6.41376
32	River CL 6	2.47233	11.60148	-6.86376

33	River CL 7	-5.56954	11.70421	-6.46376
34	River CL 8	-13.6972	11.65805	-6.00398
35	Instrument	-19.2927	8.851138	-5.66901
36	River CL 9	-19.9883	11.64074	-6.26901
37	River CL 10	-25.0506	11.77462	-4.43476
38	Instrument	-48.7381	12.19986	-3.64204
39	River CL 11	-32.8602	11.08957	-4.24204
40	River CL 12	-43.428	13.92521	-4.49204
41	River CL 13	-49.1846	15.37697	-4.39204
42	Fence Begin River CL 14 End	-55.3811	16.3509	-3.99204
43	Top South bank 1 by fence	-57.6879	8.026508	-0.79204
44	Top south bank 2	-44.5839	7.586172	-1.34204
45	Top south bank 3	-34.2574	2.768889	-0.73409
46	Instrument	-10.7455	-2.68046	0.056796
47	Top south bank 4	-25.5957	4.886044	-0.9932
48	Top south bank 5	-12.4625	1.569129	-0.7932
49	Top south bank 6	-2.52425	2.864866	-0.3932
50	Top south bank 7	5.523354	1.127782	-0.08444
51	Instrument	23.94304	-0.4844	-4.87984
52	Top south bank 8	8.770351	3.004925	-5.96763
53	Instrument	27.23727	9.455047	-6.76597
54	Top south bank 9	12.51542	4.096731	-6.81597
55	Top south bank 10	24.3866	7.809214	-6.96597
56	Top south bank 11 CP 2	37.8101	18.19274	-7.15
57	CP 1	22.81471	-4.02285	-4.35
58	Instrument	23.49043	-0.84387	-5.35
59	Instrument	29.53209	9.620604	-6.8
60	CP 2	37.8101	18.19274	-7.15
61	CP 1 cross section south side CP 1	0	0	0
62	CP 1 cross section	-0.34416	2.172914	-0.05
63	CP 1 cross section	-0.93861	5.92613	-6.96667
64	CP 1 cross section	-1.50177	9.481808	-6.55
65	CP 1 cross section	-2.03365	12.83995	-6.3
66	CP 1 cross section edge N bank	-2.50295	15.80301	-6.05
67	CP 1 cross section North side	-2.76889	17.48208	-1.05
68	Road cross section South side CP 3	-17.4374	-0.38585	0.1
69	Road cross section Top of bank	-17.5072	3.613539	-0.95
70	Road cross section	-17.5421	5.613235	-4.28333
71	Road cross section	-17.6119	9.612625	-4.45

72	Road cross section	-17.6399	11.21238	-5.03333
73	Road cross section	-17.6992	14.61186	-4.95
74	Road cross section Top of bank	-17.7673	18.51127	-0.61667
75	Road cross section North side	-17.8039	20.61095	-0.75
76	CP 3	-17.4374	-0.38585	0.1
77	Instrument	-16.873	-3.58648	-0.55
78	CP 1	0	0	0
79	cross section 4 South side	-41.0567	4.29102	-0.8
80	cross section 4	-39.2407	7.855046	-3.76667
81	cross section 4	-38.3327	9.637059	-4.01667
82	cross section 4	-36.9708	12.31008	-4.18333
83	cross section 4 river edge	-35.518	15.1613	-4.43333
84	cross section 4 North Side	-34.7008	16.76511	-1.25
85	cross section 5	-52.7625	7.865761	-1.2
86	cross section 5	-50.5257	11.18191	-3.33333
87	cross section 5	-48.8482	13.66902	-3.33333
88	cross section 5	-46.6114	16.98517	-4.25
89	cross section 5	-44.5983	19.96971	-4.33333
90	cross section 5	-42.5293	23.03715	-0.15
91	Instrument	-38.8516	-3.39907	-0.25
92	CS 4	-41.0567	4.29102	-0.8
93	CS 5	-52.7625	7.865761	-1.2
94	Instrument	-5.3367	38.92076	-2.31093
95	Road North Bank CL	-17.0367	18.65577	-2.51093
96	Road CL 1	-4.37295	33.45508	-2.06093
97	Road CL 2	2.46604	37.68493	-2.26093
98	Road CL 3	12.30038	39.57092	-3.08879
99	Instrument	25.16638	19.80506	-6.72499
100	Road Ford Fork CL	21.30245	44.20097	-3.82499
101	Ford CL 1	21.18092	29.19421	-3.57499
102	Ford CL 2	20.26936	19.97607	-6.42499
103	Ford South Bank Northbound left hand side	16.01048	-9.53118	-1.82091
104	Instrument	11.65399	-4.13523	-1.63148
105	CP 1 END LOOP	6.72E-15	-3.2E-15	0
106	Instrument	21.05967	30.0206	-4.21775
107	Ford Road Bank High point	10.97847	33.29618	-1.06775
108	North Bank 1	7.615501	24.5888	-1.36775
109	North Bank 2	-1.27891	18.14297	-1.86775
110	North Bank 3	-5.36488	17.13248	-1.36775

111	North Bank 4 road NB RH side	-10.5773	20.02211	-1.1378
112	Instrument	-23.4067	5.25126	-3.42044
113	North Bank 5 Road	-20.7558	17.72264	-2.07044
114	North Bank 6	-30.3686	15.57278	-1.87044
115	North Bank 7	-33.589	15.4336	-1.62044
116	North Bank 8	-42.4468	20.66961	0.029559
117	CP 1 End Loop	0	-5.6E-16	0
118	Instrument	-15.782	21.72211	-2.45
119	North Ridge 1	-36.4247	24.99159	1.3
120	North Ridge 2	-25.3987	31.33876	0.15
121	North Ridge 3	-16.8912	37.58337	0
122	North Ridge 4	-8.76735	43.31109	-1.6
123	North Ridge 5	-0.80589	49.88813	-1.4
124	North bank by falls 1	27.77635	35.87508	-2.4
125	North bank by falls 2	24.90891	30.37123	-2.9
126	CP 1	0	0	0
127	Range finder South bank by fence	-34.9139	-2.44142	-0.24434
128	Range finder South bank by fence	-51.9633	-1.8146	-0.72603
129	Range finder South bank by fence	-44.5037	-17.9807	-0.3351
130	Range finder South bank by fence	-27.7745	-25.9002	1.326181
131	Range finder South bank by fence	-17.4963	-35.8727	2.650956
132	Range finder South bank Ridge on hill pushed back	33.06979	-13.3611	4.885761
133	Range finder South bank Ridge on hill pushed back	-36.6342	-9.13394	4.301722
134	Range finder Top of South Ridge	-28.509	-24.7825	9.698906
135	Range finder Top of South Ridge	30.49229	-32.699	10.81654
136	Range finder tree north bank	29.04836	27.08803	-4.73616
137	Range finder road CL north bank	41.13854	52.65493	-4.90696
138	Range finder road CL north bank	74.02355	62.11314	-8.45411
139	Range finder North bank of river by waterfall	54.62178	41.16046	-9.12569
140	Range finder North bank below falls	72.4191	45.25247	-10.1827
141	Range finder River bottom bellow falls	84.89356	45.13871	-12.8289
142	Range finder Road meets ford south bank grass	9.282251	-43.6696	5.639996
143	Range finder Road CL past fork	32.43636	-66.5044	12.24945

Appendix B:

Soil Analysis Results

All test performed in accordance with *ASTM D2488-09a*, (2009) and Reddy, (2002). The only deviation from these test procedures is that a microwave oven was used instead of a conventional oven to heat the soil sample for the dry strength test. Results of testing are presented in Table B1.

Table B1. Soil Analysis Results

Soil color	Red brown
Odor	None
Major soil constituent	Fines
Other soil constituents	Trace coarse gravel (0~5%)
Dry strength test	Medium/High
Dilatancy test	No visual change in sample
Plasticity test	High
Soil Toughness	Medium
Moisture condition	Wet

Appendix C:

Hydrologic Analysis

$Lgth := 4234ft$ Length found on Google Earth. Used GPS coordinates taken while on site. The length was estimated from the coordinate at the project site up to the estimated top of the watershed.

$CN := (0.75 \cdot 79) + (0.25 \cdot 94) = 82.75$ Curve Number for the watershed found on Google Earth. The area was observed to be about 3/4 fair woods with a medium amount of ground cover. And then there is about 25% of bare land that does not have much ground cover.

TABLE 8.3 RUNOFF CN FOR ANTECEDENT MOISTURE CONDITION II

Land use description	Hydrologic condition		Hydrologic soil group A	B	C	D
Fallow, straight row, or bare soil			77	86	91	$\boxed{94}$
Pasture or range	Poor—less than 25% ground cover density		68	79	86	89
	Fair—between 25% and 50% ground cover density		49	69	79	84
	Good—more than 50% ground cover density		39	61	74	80
Brush	Poor—less than 25% ground cover density		48	67	77	83
	Fair—between 25% and 50% ground cover density		35	56	70	77
	Good—more than 50% ground cover density		30	48	65	73
Woods	Poor—less than 25% ground cover density		45	66	77	83
	Fair—between 25% and 50% ground cover density		36	60	73	$\boxed{79}$
	Good—more than 50% ground cover density		25	55	70	77
Farmsteads			59	74	82	86
Fully developed urban areas (vegetation established)						
Lawns, open spaces, parks, golf courses, cemeteries, etc.						
Good condition; grass cover on 75% or more of the area			39	61	74	80
Fair condition; grass cover on 50%–75% of the area			49	69	79	84
Poor condition; grass cover on 50% or less of the area			68	79	86	89
Paved parking lots, roofs, driveways			98	98	98	98
Streets and roads						
Paved with curbs and storm sewers			98	98	98	98
Gravel			76	85	89	91
Dirt			72	82	87	89
Paved with open ditches			83	89	92	93
		Average % impervious				
Commercial and business areas		85	89	92	94	95
Industrial districts		72	81	88	91	93
Residential with average lot size of						
1/8 acre or less		65	77	85	90	92
1/4 acre		38	61	75	83	87
1/3 acre		30	57	72	81	86
1/2 acre		25	54	70	80	85
1 acre		20	51	68	79	84
2 acres		12	46	65	77	82
Developing urban area, newly graded area with no vegetation established			77	86	91	94
Western desert urban areas						
Natural desert landscaping (pervious area only)			63	77	85	88
Artificial desert landscaping			96	96	96	96

Figure C1. NRCS Curve Number Chart (Wurbs, Ralph A., and Wesley P. James)

Slope of the stream using the contour map found for the area. The river length was spilt up into three sections on the typography map for the area and the slope was averaged.

$$Ya := \frac{(630ft - 500ft)}{1188ft} \cdot 100 = 10.943$$

$$Yb := \frac{500ft - 420ft}{2153ft} \cdot 100 = 3.716$$

$$Yc := \frac{420ft - 390ft}{893ft} \cdot 100 = 3.359$$

$$Y := \left(\frac{1188ft}{Lgth}\right) \cdot Ya + \left(\frac{2153ft}{Lgth}\right) \cdot Yb + \left(\frac{893ft}{Lgth}\right) = 5.171$$

The NRCS lag time was determined using the following equation and the gather data.

$$tL := \frac{Lgth^{0.8}(1000 - 9 \cdot CN)^{0.7}}{(1900 \cdot CN)^{0.7} Y^{0.5}}$$

$$tL := 0.86hr$$

8.3.2.1 NRCS lag equation. The NRCS developed the following equation for watersheds with areas of less than about 8 km² (2,000 ac) and CN between 50 and 95 (NRCS, 1985; Haan, Barfield, and Hayes, 1994; McCuen, 1998). Equations 8.3 and 8.4 are English (l in ft) and metric (l in m) versions of the NRCS lag formula.

$$t_L = \frac{l^{0.8}(1{,}000 - 9\mathrm{CN})^{0.7}}{1{,}900\mathrm{CN}^{0.7}Y^{0.5}} \tag{8.3}$$

$$t_L = \frac{l^{0.8}(2{,}540 - 22.86\mathrm{CN})^{0.7}}{1{,}410\mathrm{CN}^{0.7}Y^{0.5}} \tag{8.4}$$

The lag t_L is in hours. The hydraulic length l from the outlet to the most hydraulically remote point in the watershed is in feet (Eq. 8.3) or meters (Eq. 8.4). CN is discussed in Section 8.5.1. Y is the average land slope of the watershed in percent.

Example 8.1

Estimate the lag time for the watershed of Fig. 8.4. The soil and vegetative characteristics of the watershed are represented by a CN of 80.

Figure C2. NRCS Lag Equation (Wurbs, Ralph A., and Wesley P. James)

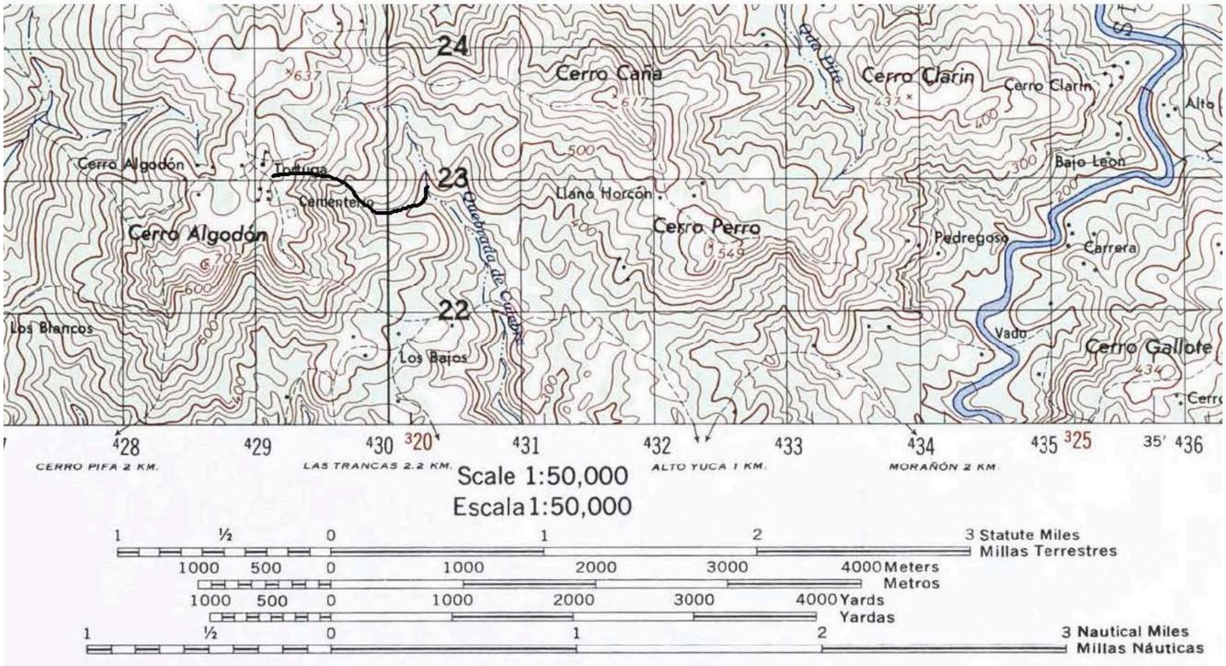


Figure C3. Contour Map of the Project Site ("Panama 1:50,000.")

A map of the Las Trancas area was found and the location of the project site was determined using the GPS coordinates. This contour map was used to estimate the river channel slope within the watershed.

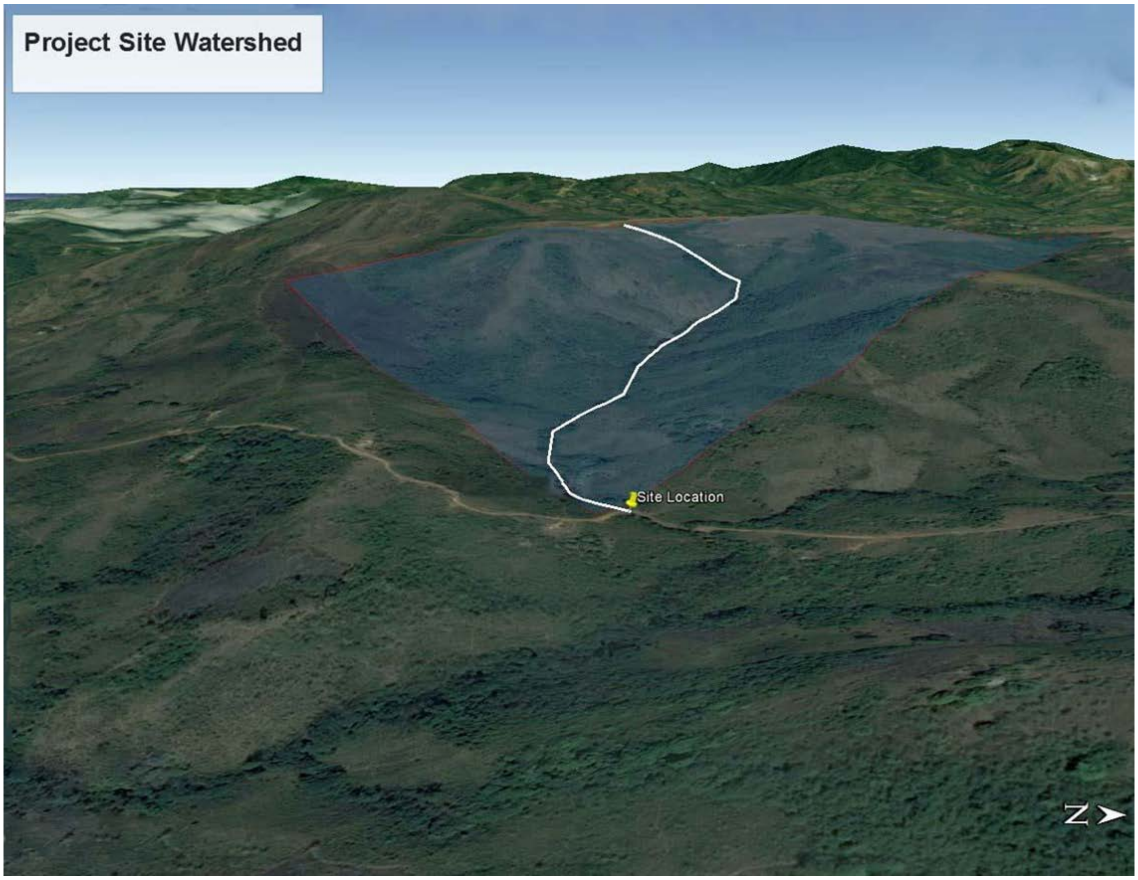


Figure C4. Project Site Approximate Watershed (Google Earth).

The above figure shows the watershed found for the project site on Google Earth. This was used to find the approximate watershed area and river length.

Table C1. 6 in, 100 yr Design Storm

Time (hours)	Time (min)	Phase II	Cumulative Depth, P (in)	Incremental Depth, P (in)	p-0.2S	Runoff Volume (in)	ΔVR (in)		Design Storm		
0.5	30	0.0053	0.03	0.032	-0.39	0	0		6	in	
1	60	0.0108	0.06	0.033	-0.35	0	0		**Curve Number**		
1.5	90	0.0164	0.10	0.034	-0.32	0	0		82.75		
2	120	0.0223	0.13	0.035	-0.28	0	0				
2.5	150	0.0284	0.17	0.037	-0.25	0	0				
3	180	0.0347	0.21	0.038	-0.21	0	0		**S**	2.08	
3.5	210	0.0414	0.25	0.040	-0.17	0	0				
4	240	0.0483	0.29	0.041	-0.13	0	0				
4.5	270	0.0555	0.33	0.043	-0.08	0	0		**tL**	**0.86**	**hr**
5	300	0.0632	0.38	0.046	-0.04	0	0				
5.5	330	0.0712	0.43	0.048	0.01	0	0				
6	360	0.0797	0.48	0.051	0.06	0.00	0.002				
6.5	390	0.0887	0.53	0.054	0.12	0.01	0.004				
7	420	0.0984	0.59	0.058	0.17	0.01	0.007				
7.5	450	0.1089	0.65	0.063	0.24	0.02	0.011				
8	480	0.1203	0.72	0.068	0.30	0.04	0.015				
8.5	510	0.1328	0.80	0.075	0.38	0.06	0.020				
9	540	0.1467	0.88	0.083	0.46	0.08	0.026				
9.5	570	0.1625	0.98	0.095	0.56	0.12	0.034				
10	600	0.1808	1.08	0.110	0.67	0.16	0.044				
10.5	630	0.2042	1.23	0.140	0.81	0.23	0.064				
11	660	0.2351	1.41	0.185	0.99	0.32	0.095				
11.5	690	0.2833	1.70	0.289	1.28	0.49	0.168				
12	720	0.6632	3.98	2.279	3.56	2.25	1.759				
12.5	750	0.7351	4.41	0.431	3.99	2.62	0.377				
13	780	0.7724	4.63	0.224	4.22	2.82	0.198				
13.5	810	0.7989	4.79	0.159	4.38	2.96	0.142				
14	840	0.8197	4.92	0.125	4.50	3.08	0.112				
14.5	870	0.838	5.03	0.110	4.61	3.18	0.099				
15	900	0.8538	5.12	0.095	4.71	3.26	0.086				
15.5	930	0.8676	5.21	0.083	4.79	3.34	0.075				
16	960	0.8801	5.28	0.075	4.86	3.40	0.068				
16.5	990	0.8914	5.35	0.068	4.93	3.47	0.062				
17	1020	0.9019	5.41	0.063	4.99	3.52	0.057				
17.5	1050	0.9115	5.47	0.058	5.05	3.58	0.053				
18	1080	0.9206	5.52	0.055	5.11	3.63	0.050				
18.5	1110	0.9291	5.57	0.051	5.16	3.67	0.047				
19	1140	0.9371	5.62	0.048	5.21	3.72	0.044				
19.5	1170	0.9446	5.67	0.045	5.25	3.76	0.041				
20	1200	0.9519	5.71	0.044	5.29	3.80	0.040				
20.5	1230	0.9588	5.75	0.041	5.34	3.84	0.038				
21	1260	0.9653	5.79	0.039	5.37	3.87	0.036				
21.5	1290	0.9717	5.83	0.038	5.41	3.91	0.035				
22	1320	0.9777	5.87	0.036	5.45	3.94	0.033				
22.5	1350	0.9836	5.90	0.035	5.48	3.97	0.033				
23	1380	0.9892	5.94	0.034	5.52	4.01	0.031				
23.5	1410	0.9947	5.97	0.033	5.55	4.04	0.031				
24	1440	1	6.00	0.032	5.58	4.07	0.029				
							4.0652				

The above table shows the data calculated when designing a 6 in, 100 yr storm (Shamir, E, Georgakakos) on the watershed of the project site.

Table C2. Incremental Hydrograph

Time	Ratio		U.H,										Flow rate (ft^3/s)	
min	t/Tp	Q/Qp	ft3/s			0.06	0.095	0.17	1.76	0.377	0.1984			
0	0.0	0.0	0										0	0.00
30	0.5	0.5	64.82										30	0.00
60	0.9	0.9	129.6										60	0.12
90	1.4	0.789	113.6										90	0.50
120	1.8	0.519	74.67										120	1.23
150	2.3	0.249	35.78										150	2.26
180	2.7		0										180	3.56
210	3.2		0										210	5.11
240	3.6		0										240	6.96
270	4.1		0										270	9.23
300	4.5		0			0.00							300	12.14
330	5.0		0			4.13	0						330	16.30
360	5.4		0			8.27	6.15	0					360	22.87
390	5.9					7.24	12.31	10.9	0				390	34.94
420	6.3			**Tp**	**1.11**	4.76	10.78	21.8	113.98	0			420	152.88
450	6.8			**Qp**	**143.89**	2.28	7.09	19.1	227.96	24.42	0		450	**280.82**
480	7.2					0.00	3.40	12.5	199.69	48.84	12.86		480	277.33
510	7.7					0.00	0	6.01	131.30	42.79	25.72		510	215.03
540	8.1					0.00	0	0	62.92	28.13	22.53		540	139.26
570	8.6					0.00	0	0	0	13.48	14.82		570	65.37
600	9.0				Y=-.6X+	0.00	0	0	0	0	7.10		600	48.82
630	9.5					0.00	0	0	0	0	0		630	40.67
660	9.9					0.00	0	0	0	0	0		660	35.29
690	10.4						0	0	0	0	0		690	31.31
720	10.8							0	0	0	0		720	28.15
750	11.3								0	0	0		750	25.65
780	11.7									0	0		780	23.64
810	12.2										0		810	21.99
												tL	0.86	hr
												A	0.33	mi2
												Tp=D/2 + tL		

The above table shows the incremental hydrograph made for the design storm. It was used to determine the maximum flow rate during a flooding event.

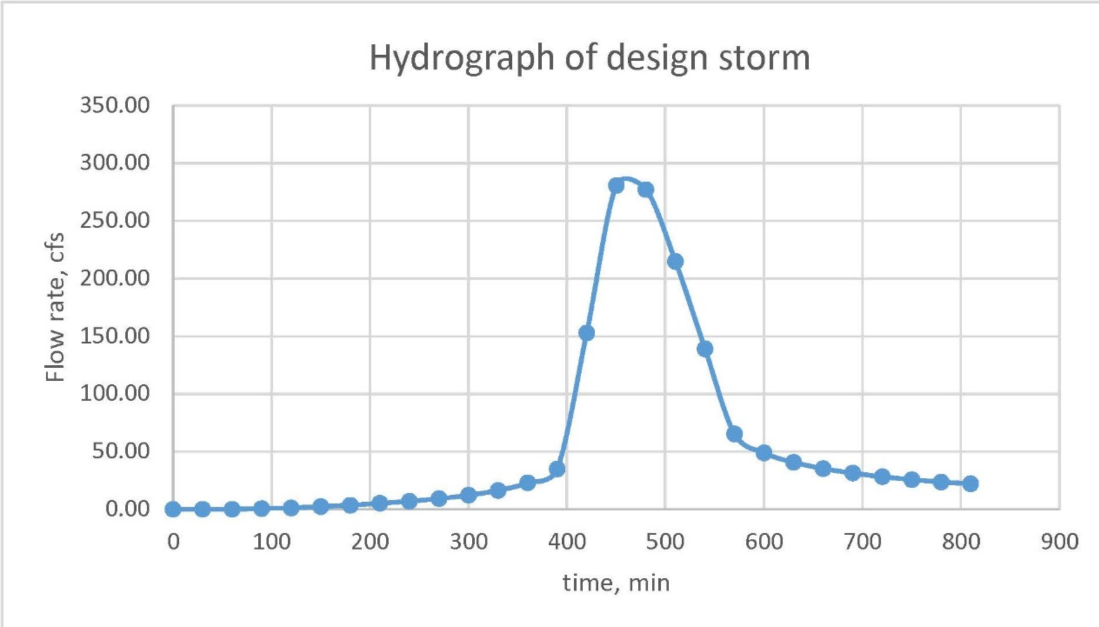


Figure C5. Hydrograph of Design Storm

The above figure displays the calculated flow rates during a flooding event for the project site location.

Maximum Flow Depth Calculations

$r := 102.22\text{in}$

Assume that depth of water is below top of riprap
Side slope of riprap = 1:3

$\alpha := \text{atan}\left(\frac{1}{3}\right) = 0.322$

$w_{footing} := 5\text{ft}$

$w_{foot\_inside} := \frac{w_{footing}}{2} = 30\,\text{in}$

maximum stream height allowable:

$e := r\cdot\sin(\alpha) = 32.325\,\text{in}$

riprap slope will start at center of bridge plate arc.
start with assumption that river maximum height is 20 in.

$h := 20\text{in}$

$b := \frac{h}{\tan(\alpha)} = 5\,\text{ft}$

width of flat section of riprap:

$w_{bridge} := 281\,\text{in}$

$w_{flat} := w_{bridge} - 2\cdot r = 76.56\,\text{in}$

cross sectional area of flow:

$$A := \left[\frac{\left[w_{flat} + \left(w_{flat} + 2\cdot b\right)\right]}{2}\right]\cdot h = 18.967\,\text{ft}^2$$

Manning's n value for channel

$n := 0.05$ (Wurbs, 2002)

length of riprap side slope:

$c := \sqrt{b^2 + h^2} = 63.246\,\text{in}$

wetted perimeter

$P := w_{flat} + 2\cdot c = 203.051\,\text{in}$

Hydraulic Radius

$R_h := \frac{A}{P} = 1.121\,\text{ft}$

Channel slope (from survey data):

$S_s := 0.045$

Iterate using mannings equation to find the flow rate.

Actual flow rate

$$Q_{act} := 281 \frac{ft^3}{s}$$

$$Q := \left(\frac{1.49}{n}\right) \cdot A \cdot R_h^{\frac{2}{3}} \cdot \sqrt{S_s} \cdot 1 \frac{ft^{\frac{1}{3}}}{1s} = 129.377 \frac{ft^3}{s}$$ (Manning's Equation, Wurbs, 2002)

Iteration 2

$$h := 27in$$

$$b := \frac{h}{\tan(\alpha)} = 6.75\,ft$$

width of flat section of riprap:

$$w_{bridge} := 281in$$

$$w_{flat} := w_{bridge} - 2 \cdot r = 76.56\,in$$

cross sectional area of flow:

$$A := \left[\frac{\left[w_{flat} + \left(w_{flat} + 2 \cdot b\right)\right]}{2}\right] \cdot h = 29.543\,ft^2$$

Manning's n value for channel

$$n := 0.05$$ (Wurbs, 2002)

length of riprap side slope:

$$c := \sqrt{b^2 + h^2} = 85.381\,in$$

wetted perimeter

$$P := w_{flat} + 2 \cdot c = 247.323\,in$$

Hydraulic Radius

$$R_h := \frac{A}{P} = 1.433\,ft$$

Channel slope (from survey data):

$$S_s := 0.045$$

Iterate using mannings equation to find the flow rate.

Actual flow rate

$$Q_{act} := 281 \frac{ft^3}{s}$$

$$Q := \left(\frac{1.49}{n}\right) \cdot A \cdot R_h^{\frac{2}{3}} \cdot \sqrt{S_s} \cdot 1 \frac{ft^{\frac{1}{3}}}{1s} = 237.417 \frac{ft^3}{s}$$ (Manning's Equation, Wurbs, 2002)

Continue iterating until you have Q = 281 ft^3/s

$h := 29.286\text{in}$

$b := \frac{h}{\tan(\alpha)} = 7.322\,\text{ft}$

width of flat section of riprap:

$w_{bridge} := 281\text{in}$

$w_{flat} := w_{bridge} - 2\cdot r = 76.56\cdot\text{in}$

cross sectional area of flow:

$A := \left[\left[\frac{\left[w_{flat} + \left(w_{flat} + 2\cdot b\right)\right]}{2}\right]\right]\cdot h = 33.439\cdot\text{ft}^2$

Manning's n value for channel

$n := 0.05$ (Wurbs, 2002)

length of riprap side slope:

$c := \sqrt{b^2 + h^2} = 92.61\cdot\text{in}$

wetted perimeter

$P := w_{flat} + 2\cdot c = 261.781\cdot\text{in}$

Hydraulic Radius

$R_h := \frac{A}{P} = 1.533\cdot\text{ft}$

Channel slope (from survey data):

$S_s := 0.045$

Iterate using mannings equation to find the flow rate.

Actual flow rate

$Q_{act} := 281\,\frac{\text{ft}^3}{\text{s}}$

$Q := \left(\frac{1.49}{n}\right)\cdot A\cdot R_h^{\frac{2}{3}}\cdot\sqrt{S_s}\cdot 1\,\frac{\text{ft}^{\frac{1}{3}}}{1\text{s}} = 281.015\cdot\frac{\text{ft}^3}{\text{s}}$ (Manning's Equation, Wurbs, 2002)

Maximum flow depth is during 100 year design storm:

$h = 29.286\cdot\text{in}$

$V := \frac{Q}{A} = 8.404\,\frac{\text{ft}}{\text{s}}$ (Wurbs, 2002)

This is the flow velocity of the stream.

Appendix D:

Design Alternative Analysis

Table D1 below displays the decision matrix used by Trancas Associates to decide upon alternatives for a preliminary analysis. Each alternative design was scored based on their ability to meet a certain design criterion, then multiplied by an importance factor selected for each criterion to obtain a score. The scores from each criterion were totaled, and an overall score for each alternative was given.

The highest-scoring alternatives were picked to be modeled and roughly estimated, and were compared against each other again to decide upon a final design. The highest ranking alternatives were the flexible buried steel bridge, box culvert, steel truss with wooden decking, and the all-wooden truss bridge. Eliminating the all steel truss, this appendix will display the preliminary analysis of the three alternatives that were not selected for the final design. Total price calculations were determined using approximate quantities and rounded off to an even value upon summing estimated total prices.

Table D1. Decision Matrix

-	Constructability	Cost	Construction Length	Sustainability	Serviceability	Totals
Importance Factor	***9***	***9***	***9***	***6***	***5***	**-**
Box Culvert	5	8	7	5	4	**219**
All Steel Truss	2	2	3	7	6	**116**
Steel Truss with Wooden Deck	4	4	4	7	7	**162**
All Wooden Truss Bridge	5	5	5	4	7	**171**
Flexible Buried Steel Bridge	7	6	7	7	5	**232**

Box Culvert Preliminary Analysis

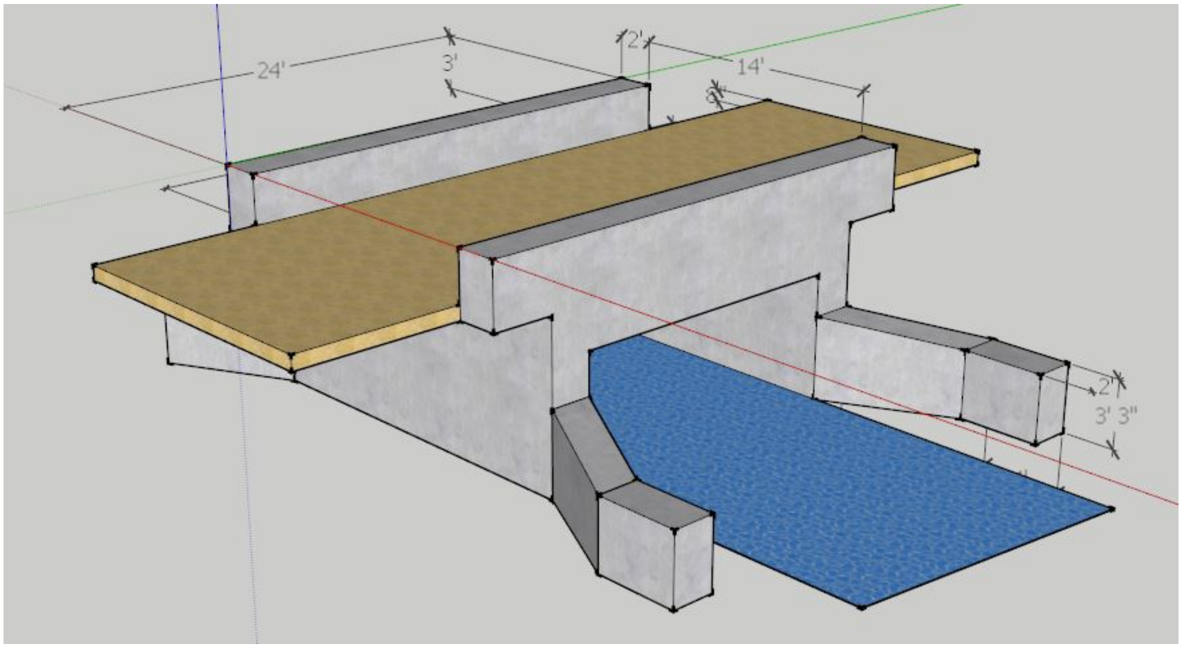


Figure D1. Box Culvert Model

Table D2. Box Culvert Preliminary Estimate

Box Culvert					**Total Incl O& P**
Work Description	**Pay Item**	**Price**	**Quantity**	**Unit**	**Total Price**
Concrete Hand Mix (cft)	4500 psi	$7.51	1596	cft	$11,985.96
8" Gravel Base	Crushed 1"-1/2" stone base, compacted to 4" deep	$11.95	65	syd	$776.75
Grading by hand to match culvert roadway	Fine Grade for slab on grade, Hand Grading	$0.26	119	syd	$30.94
Diverting River for Cast-in-place Culvert	Excavate drainage trench 2 ft wide and 5 ft deep	$16.40	17	cyd	$278.80
Moblization (8 hr)	Small equipment, placed in rear of, or towed by pickup truck	$191.00	4	2 hr	$764.00
Trimming for 4 ft retaining walls	Hand Trimming, bottom of excavation of slopes and sides	$0.97	86	sft	$83.42
				TOTAL	**~$14,000**

Steel Truss Preliminary Analysis

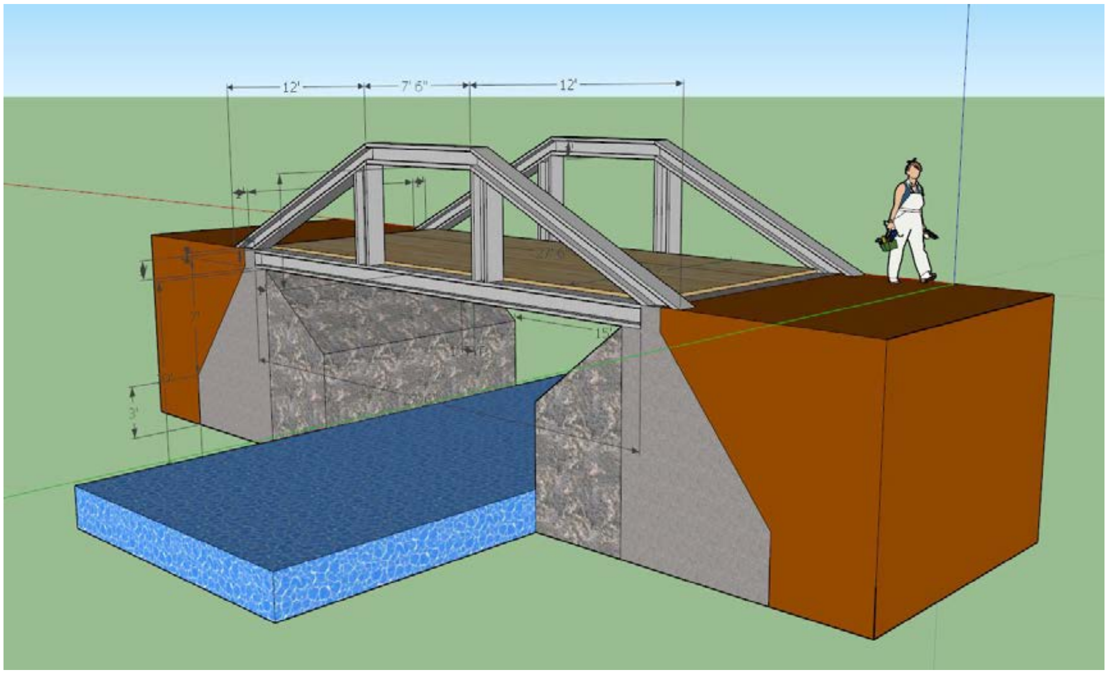


Figure D2. Steel Truss with Wooden Decking Model

Table D3. Steel Truss with Wooden Decking Preliminary Estimate

Steel Truss with Wooden Decking					**Total Incl O& P**
Work Description	**Pay Item**	**Price**	**Quantity**	**Unit**	**Total Price**
Steel Members	W 12 x 26	$49.00	168	lft	$8,232.00
Plate Connections	1/4" thick	$14.85	0.222	sft	$3.30
Steel Worker to make connections	Structural Steel Workers	$52.65	24	hr	$1,263.60
Operator to use Mini-Excavader to lift steel beams	1 Equipment Operator (light)	$48.60	24	hr	$1,166.40
Mini-Excavader for Erection	1-1/2 cyd Capacity	$59.80	24	hr	$1,435.20
Moblization (9 hr)	Equipment hauled on 3-ton capacity towed trailer	$295.00	3	3 hr	$885.00
Abutments	Abutment for Bridge	$535.00	36	cyd	$19,260.00
Operator to use Mini-Excavader to Excavate	1 Equipment Operator (light)	$48.60	8	hr	$388.80
Mini-Excavader for Excavation	1-1/2 cyd Capacity	$59.80	8	hr	$478.40
Reinforcement, Epoxy Coated	#8 to #18	$1,800.00	0.32	ton	$576.00
Paint	1 to to 20 tons	$555.00	1	Each	$555.00
Grading by hand to match culvert roadway	Fine Grade for slab on grade, Hand Grading	$0.26	119	syd	$30.94
Riprap	Machine placed for slope protection	$63.00	41	cyd	$2,583.00
Wooden Roadway	Floors Planks 2" x 6"	$2,850.00	0.039	MBF	$111.15
				TOTAL	**~$37,000**

All Wooden Truss Bridge Preliminary Analysis

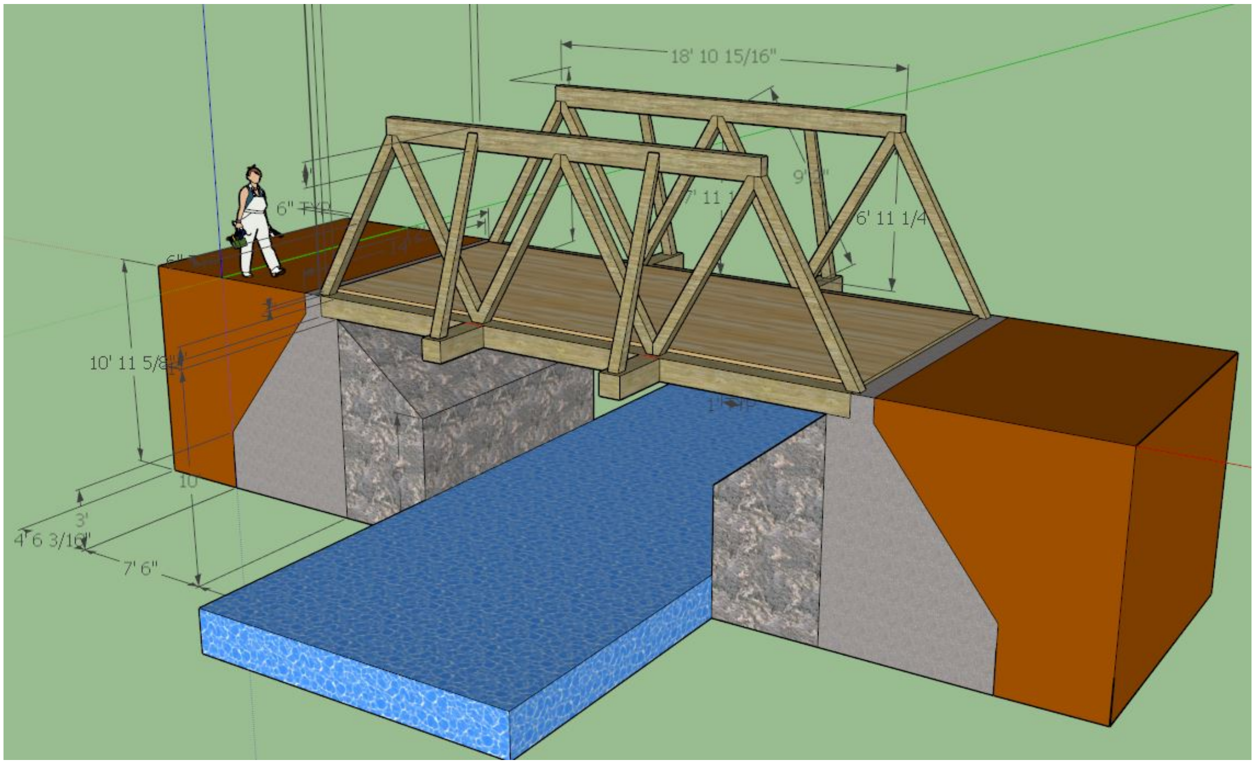


Figure D3. All Wooden Truss Bridge Decking Model

Table D4. All Wooden Truss Bridge Preliminary Estimate

All Wooden Truss Bridge					**Total Incl O& P**
Work Description	**Pay Item**	**Price**	**Quantity**	**Unit**	**Total Price**
Timber Post and Columns	6" x 8"	$14.75	202	ft	$2,979.50
Griders Structural Grade	10" x 16"	$3,800.00	0.111		$421.80
Wooden Roadway	Floors Planks 2" x 6"	$2,850.00	0.039	MBF	$111.15
Riprap	Machine placed for slope protection	$63.00	41	cyd	$2,583.00
Wooden Members connections	Connector Plates, steel, with bolts, straight	$49.00	18	Each	$882.00
Operator to use Mini-Excavader to lift wood beams for connection	1 Equipment Operator (light)	$48.60	24	hr	$1,166.40
Mini-Excavader for Erection	1-1/2 cyd Capacity	$59.80	24	hr	$1,435.20
Abutments	Abutment for Bridge	$535.00	36	cyd	$19,260.00
Grading by hand to match culvert roadway	Fine Grade for slab on grade, Hand Grading	$0.26	119	syd	$30.94
Operator to use Mini-Excavader to Excavate	1 Equipment Operator (light)	$48.60	8	hr	$388.80
Mini-Excavader for Excavation	1-1/2 cyd Capacity	$59.80	8	hr	$478.40
Reinforcement, Epoxy Coated	#8 to #18	$1,800.00	0.32	ton	$576.00
Stain and Varnish	Straight Beams	$4,250.00	0.0176	MBF	$74.80
				TOTAL	**~$31,000**

Appendix E:

Structural Design Calculations

AASHTO Section 12: Design Equations
(Service Load Design)

Defining starting variables:

$psi := 1\frac{lb}{in^2}$ $ksi := 1000\frac{lb}{in^2}$ Defining units for Mathcad calcs

$E := 29000ksi$
$fu := 45ksi$ Steel Properties (Table 6)
$fy := 33ksi$

Span Type: Low profile arch (84A18)

Longitudal Thrust Beams intalled at seams (concrete bracing)

$S := \left(23 + \frac{5}{12}\right)ft = 23.417ft$ S is span of the bridge, standard size from (Table 74)

$R := \left(9 + \frac{3}{12}\right)ft = 9.25ft$ R is rise of the bridge, standard size from (Table 74)

$\frac{R}{S} = 0.395$ Check: Ratio must be >= 0.3 to simulate to round pipe calcs (S = D)

Area of Lower radius arch

$r_1 := 102.22in$

$h_1 := 92.002in$

$$A_{bot} := \left[\frac{\left(\pi \cdot r_1^{\,2}\right)}{2}\right] - r_1^{\,2} \cdot acos\left(\frac{h_1}{r_1}\right) + h_1 \cdot r_1 \cdot sin\left(acos\left(\frac{h_1}{r_1}\right)\right) + 76.56in \cdot h_1 = 158.636ft^2$$

Area of upper arch

$r_2 := 190.058in$

$$h_s := \sqrt{\left(r_2 - r_1\right)^2 - \left[\frac{(76.56in)}{2}\right]^2} = 79.058in$$

$h_2 := h_1 + h_s = 171.06in$

$$A_{top} := acos\left(\frac{h_2}{r_2}\right) \cdot r_2^{\,2} - h_2 \cdot r_2 \cdot sin\left(acos\left(\frac{h_2}{r_2}\right)\right) = 14.722ft^2$$

Flow area

$FA := A_{top} + A_{bot} = 173.358ft^2$

$C = 15" \times 5.5"$ C is corrugation sheet size of steel, sold commercially as BridgeCor Steel

$H := 2ft$ H is height of cover (fill) over top of structure
Selected minimum amount as determined by (Table 74)

Backfill Density: 90% of standard proctor density is specified conforming to AASHTO T 180

$LL := 0.5 \cdot 850 \frac{lbf}{ft^2} = 2.951\, psi$ LL determined by 0.5 H20 Loading & 2' of cover (Table 1)

$\psi := 133.7 \frac{lbf}{ft^3}$ Unit weight of cover, using general for GW soil (Preferred Fill)
(http://www.geotechdata.info/parameter/soil-dry-unit-weight.html)

Calculations:

$DL := \psi \cdot H = 1.857\, psi$ Dead Load

$P := 1.2DL + 1.6LL = 3.22 \times 10^4 \frac{ft}{s^2} \cdot \frac{lb}{ft^2}$ Total Load, Facotred more although not specified to in equations

$Ts := P \cdot \frac{S}{2} = 3.77 \times 10^5 \frac{lb}{s^2}$ Wall Thrust

$fa := \frac{fy}{2} = 1.65 \times 10^4 \cdot psi$ allowable wall stress, with a FOS = 2 introduced

$A\_req := \frac{Ts}{fa} = 22.851 \frac{ft}{s^2} \cdot \frac{in^2}{ft}$ **required plate area**

Steel Plate Properties (From other design manual since none listed for this shape in current)

$t := 0.188 \cdot in$ (Table 7.2) Thickness of selected plate **(GAGE 7)**

$r := 1.950 \cdot in$ (Table 7.2) Radius of Gyration of selected plate

$A := 3.088 \frac{in^2}{ft}$ (Table 7.2) Cross Sectional Area of Selected Plate

$I := 978.64 \cdot 10^{-3} \frac{in^4}{in}$ (Table 7.2) Moment of Inertia of Selected Plate

AREA REQUIRED FROM THIS PLATE IS LOWER THAN AREA OF PLATE, OKAY!

$k := 0.22$ (Contech pg. 14) $S = 23.417\, ft$

$\frac{r}{k} \cdot \sqrt{\frac{24E}{fu}} = 91.861\, ft$ > S, so use following equation

$$fcr := fu - \frac{fu^2}{48E} \cdot \left(\frac{k \cdot S}{r}\right)^2 = 4.354 \times 10^4 \cdot psi$$

$$\frac{fcr}{2} = 2.177 \times 10^4 \cdot psi \qquad fa = 1.65 \times 10^4 \cdot psi$$

$fcr > fa$

CRITICAL BUCKLING STRENGTH IS MORE THAN ALLOWABLE WALL STRESS, OKAY!

$$FF\_req := \frac{S^2}{E \cdot I} = 2.782 \times 10^{-3} \cdot \frac{in}{lb} \qquad FF\_arch := 0.01 \frac{in}{lb}$$ (Table 7.5)

FLEXIBILITY FACTOR IS LESS THAN FLEXIBILTY FACTOR FOR SHAPE

$$SS\_req := Ts \cdot 3 = 1.131 \times 10^6 \frac{lb}{s^2}$$ **required seam strength** required with FOS = 3 introduced

$$SS := 102000 \frac{lb}{ft}$$ (Table 7.4b in other manual, ASTM A796) 4.8 bolts per foot

SEAM STRENGTH REQUIRED IS LESS THAN SEAM STRENGTH OF GAGE

Bolts used are 3/4" diameter – high strength bolts, meeting ASTM A449.

Bolts and nuts also used for connecting arch plates to receiving angles and structural reinforcement to structural plates.

Footing Reaction

$$[(R + H) \cdot S - FA] \cdot \psi = 12.044 \cdot \frac{kip}{ft}$$ Weight of soil on arch

$$R_{dl} := [(R + H) \cdot S - FA] \cdot \frac{\psi}{2} = 6.022 \cdot \frac{kip}{ft}$$ Vertical reaction @ spring line due to soil load

$$R_{ll} := \frac{32000 lbf}{(8ft + 2H)} = 8.58 \times 10^4 \frac{lb}{s^2}$$ Vertical reaction @ spring line due to live load (ch. 7 pg 404) Assume one lane

$$R_{total} := 1.2R_{dl} + 1.6R_{ll} = 11.493 \cdot \frac{kip}{ft}$$

Appendix F:

Footing Design Calculations

Footing Design

$R := 8.688\frac{kip}{ft}$

$\lambda := 1.0$ (ACI 19.2.4.2)

$f_y := 60ksi$ (use grade 60 steel)

$\rho_{concrete} := 150\frac{lbf}{ft^3}$

$f'c := 4000psi$

Allowable soil-bearing pressure (service limit state)

$D_{allow} := 4\frac{kip}{ft^2}$ (AASHTO LRFD BDS, Table C10.6.2.6.1-1)

Footing is considered shallow (ACI 13.1.1)

$E_c := 57000 \cdot 1psi^{0.5} \cdot \sqrt{f'c} = 3.605 \times 10^6 \cdot psi$ (ACI 19.2.2.1b)

Fine and coarse aggregate must meet the requirements of ASTM C33.

Concrete shall meet all other requirements specified in ACI Chapter 19.

$Width_{footing} := \frac{R}{D_{allow}} = 2.172 \cdot ft$

$Width_{footing} := 5ft$

This is the minimum width of the footing. To protect from overturning and sliding and estimation errors for soil compression strength use 5 ft as width.

Check bearing capacity including concrete self weight.

$R = 8.688 \cdot \frac{kip}{ft}$

$D_{soil} := \frac{R}{Width_{footing}} + \rho_{concrete} \cdot 2ft = 2.038 \cdot \frac{kip}{ft^2}$

2.038 kif/ft^2 < 4 kip/ft^2 so the bearing capacity is good.

Check Shear.

$d_3 := 0.375in$ (ACI App A)

$d_6 := 0.75in$ (ACI App A)

$d_6 := 0.75in$ (ACI App A)

$b_w := Width_{footing} = 5\,ft$

$cover := 3in$ (ACI Table 20.6.1.3.1)

$h_{foot} := 2ft$ $f'c = 4 \times 10^3\,psi$

$d := h_{foot} - (cover + d_3 + d_6) = 1.656\,ft$

$V_c := 2 \cdot \lambda \cdot \sqrt{f'c} \cdot b_w \cdot d \cdot \left(1\frac{lbf}{in^2}\right)^{0.5} = 150.841 \cdot kip$ Per foot of footing

$\Phi_v := 0.75$ (ACI Table 21.2.2)

$\Phi_v \cdot V_c = 113.13 \cdot kip$

$V_u := R \cdot b_w = 43.44 \cdot kip$

ΦVc > Vu OK

Stirrup bend diameter

$d_{bend} := 4d_3 = 1.5 \cdot in$

Maximum rebar spacing

$c_c := cover = 3 \cdot in$

$f_s := \left(\frac{2}{3}\right) \cdot f_y = 4 \times 10^4\,psi$

$s_1 := 15in \cdot \left[\frac{(40000psi)}{f_s}\right] - 2.5 \cdot c_c = 7.5 \cdot in$ (ACI Table 24.3.2)

$s_2 := 12 \cdot \left(\frac{40000}{f_s}\right) \cdot 1\frac{lbf}{in} = 12 \cdot in$

$s := s_1 = 7.5 \cdot in$

Appendix G:

Channel Design Calculations

Using MDOT Specifications to properly size the riprap for the stream's conditions.

Rip Rap

$DepthOfFlow_{max} := 18in$

$DepthOfFlow_{max} = 1.5 \cdot ft$

$So := 0.045$

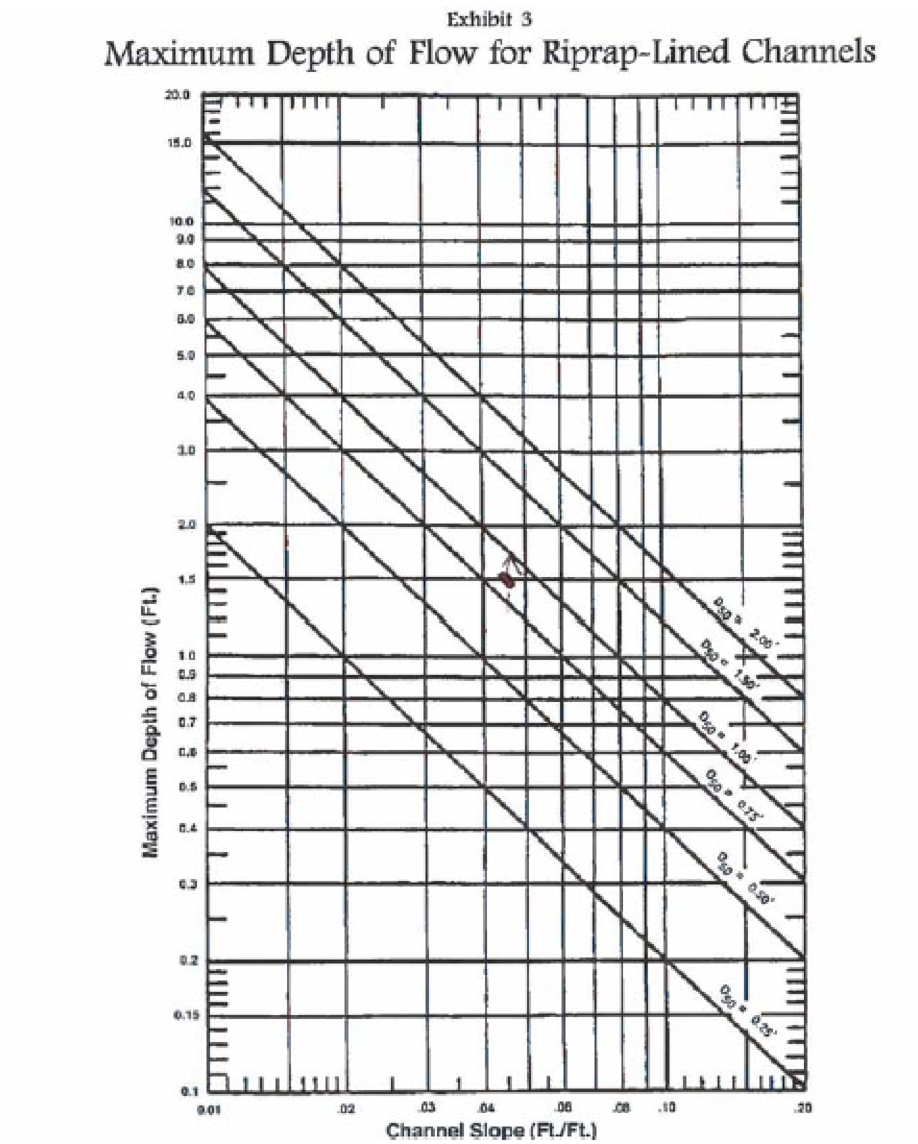


Figure G1. Maximum Depth of Flow for Riprap Lined Channels ("Riprap").

$d_{50} := 12in$

This means between 3/4 in and 12 in. But rounded up to 12 in to size it larger

Using Exhibit 4 to determine the boundary shear around the wetter perimeter of the channel (K1)

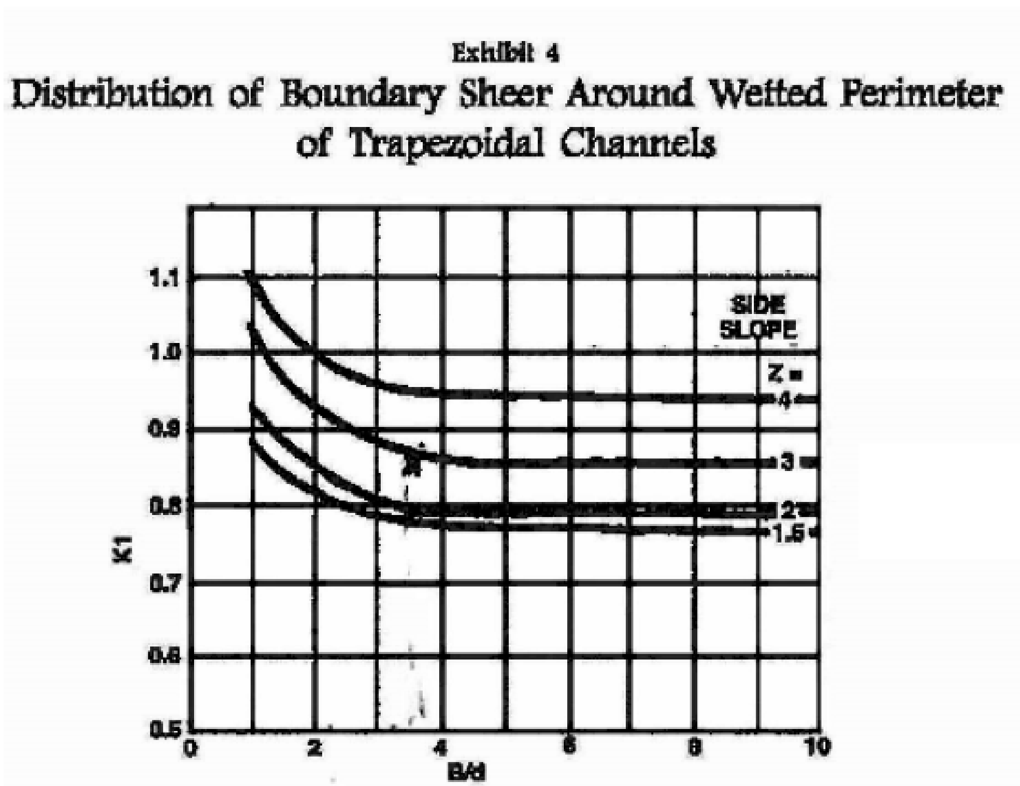


Figure G2. Distribution of Boundary Sheer Around Wetted Perimeter of Trapezoidal Channels ("Riprap")

$b := 18.5\text{ft}$ b is the channel width

$d := 5.03\text{ft}$ d is the channel depth

$\frac{b}{d} = 3.678$

$K1 := 0.85$

Using Exhibit 5 to determine the angle of repose for the to be placed riprap

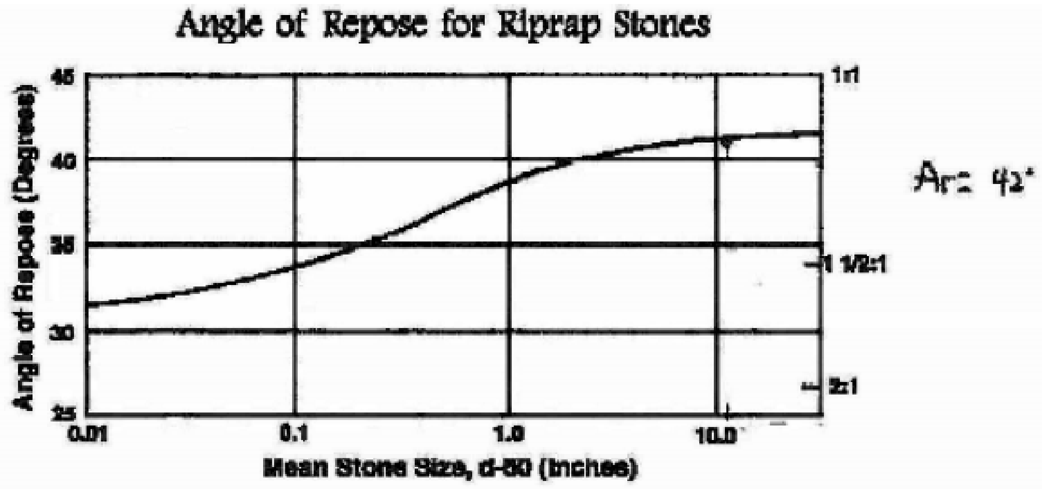


Figure G3. Angle of Repose for Riprap Stones ("Riprap")

$Ar := 42$

Using Exhibit 6 to determine the ratio of the critical shear on the bottom of the channel (K2)

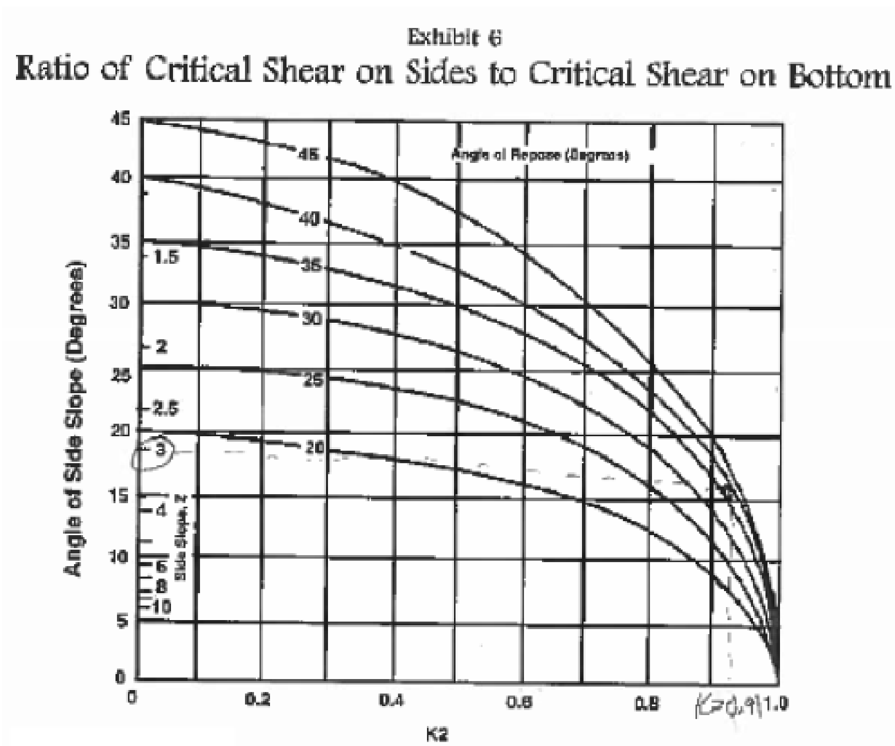


Figure G4. Ratio of Critical Shear on Sides to Critical Shear on Bottom ("Riprap")

$K2 := 0.91$

$$d_{50act} := d_{50} \cdot \frac{K1}{K2} = 11.209 \cdot in$$

Using the figure below to confrim the side slope based on the angle of repose

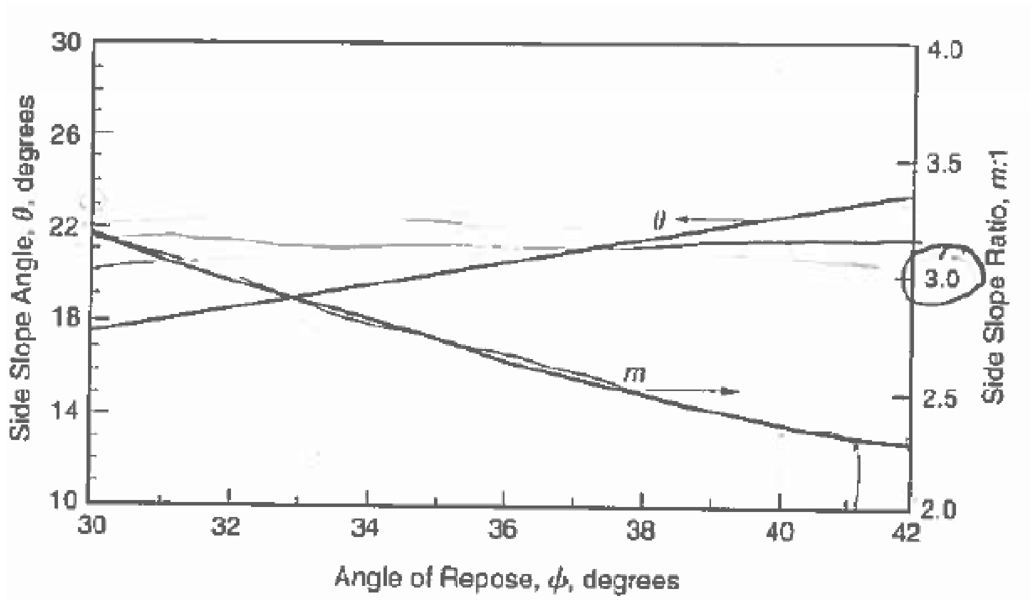


Figure G5. Side Slope of the Riprap along the River Bank Walls (Strum)

Side slope is confirmed to be 3:1

Using table 1 to determine the mean size of the riprap

$$d_{50} := 12in$$

The mean diameter is increased to 14 in. This is to increase the riprap unit weight to add extra protect from scour for the riprpa under the bridge plate that is protecting the footings. Increasing the mean diameter of the riprap also accounts for the roughness of the hydrolgic data. This also allows for more protect of river bank and footing protection from high flooding flow rates.

Table 1
Size of Typical Riprap Stones

Weight (lbs)	Mean Spherical Diameter (in)	Typical Rectangular Shape Length (in)	Width, Height (in)
50	10	18	6
100	13	21	7
150	14	24	8
300	18	30	10
500	22	36	12
1000	27	45	15
1500	31	52	17
2000	34	57	19
4000	43	72	24
6000	49	83	28
8000	54	90	30

Figure G6. Size of Typical Riprap Stones ("Riprap")

$$d_{50} := 14in$$

Rip Rap

South side of the stream

Finding the area of the riprap placement around the stream

Used the hatched area of the riprap on page J-3 of the Design Drawings

$SW_{RR} := 11ft$

$SL_{RR} := 15ft + 21ft + 24ft + 13ft$

North side of the stream

$NW_{RR} := 10ft$

$NL_{RR} := 21ft + 21ft + 8ft + 17ft + 10ft$

$NArea_{RR} := SW_{RR} \cdot SL_{RR} = 89.222 yd^2$

$SArea_{RR} := NW_{RR} \cdot NL_{RR} = 85.556 yd^2$

Area found of each side of the to be placed riprap.

$RRAreaTotal := NArea_{RR} + SArea_{RR} = 174.778 yd^2$

The total syd of the river bank walls and channel to have riprap placed

$AvgDepth := 0.75ft$

$RRVolTotal := RRAreaTotal \cdot AvgDepth = 43.694 yd^3$

Finding total volume of the needed riprap

$RRTonTotal := RRVolTotal \cdot 150 \frac{lb}{ft^3} = 88.481 ton$

Finding the total weight of the riprap

Appendix H:

Headwall Design Calculations

Headwall calculations

$$\gamma_{gravel} := 133.7\frac{lb}{ft^3}$$

$$\rho_{water} := 62.4\frac{lb}{ft^3}$$

$$\gamma_{tot} := \gamma_{gravel} + \rho_{water} = 0.113\frac{lb}{in^3}$$

h is the depth of the soil from the top of the headwall to the 15" x 5 1/2" structural plate of the bridge as a function of the distance along the headwall.

$$h_0 := 135in$$

$$h_{42} := 82.52in$$

$$h_{78} := 46.83in$$

$$h_{114} := 31.83in$$

$$g := 9.80665\frac{m}{s^2} = 32.174\frac{ft}{s^2}$$

Use load factors from ASCE 7 section 2.3a. P = 1.4 D

$$P_{0\_max} := 1.4g \cdot h_0 \cdot (\rho_{water} + \gamma_{gravel}) = 21.448 psi$$

$$P_{42\_max} := 1.4g \cdot h_{42} \cdot (\rho_{water} + \gamma_{gravel}) = 13.111 psi$$

$$P_{78\_max} := 1.4g \cdot h_{78} \cdot (\rho_{water} + \gamma_{gravel}) = 7.44 psi$$

$$P_{114\_max} := 1.4g \cdot h_{114} \cdot (\rho_{water} + \gamma_{gravel}) = 5.057 psi$$

This is the pressure of water where the bridge intersects the headwall at the overlap of the headwall plates. The headwall will be braced at these sections of overlap.

$W_{bridge} := 18.75ft$ width of the bridge

Anchor rods will be conected to the headwall 12" on center below the top surface of the headwall.

conection at 114"

$$d_{conec} := 12in$$

$$h_{c114} := h_{114} - d_{conec} = 19.83 in$$

$b_{c114} := 9ft = 108 \cdot in$

$l_{r114} := \sqrt{b_{c114}^{2} + h_{c114}^{2}} = 109.805 \cdot in$

Tributary area supported by rod 114.

$w_{trib} := \frac{(3ft + 9ft)}{2} = 72 \cdot in$ this is the width of the tributary area

$h_{l\_114} := h_{78} = 46.83 \cdot in$

$h_{r\_114} := 2ft = 24 \cdot in$

$h_{eff114} := \frac{\left(h_{l\_114} + h_{r\_114}\right)}{2} = 35.415 \cdot in$ this is the effective depth of the plate

$P_u := 0.7 \cdot h_{eff114}^{2} \cdot \gamma_{tot} \cdot g \cdot w_{trib} = 49.817 \, ft^{2} \cdot psi$

Effective height

$a := h_{eff114} - 12in = 23.415 \cdot in$

Actual Height

$a_{114} := h_{114} - 12in = 19.83 \cdot in$

Actual rod length

$l_{114} := \sqrt{a_{114}^{2} + b_{c114}^{2}} = 109.805 \cdot in$

Anchor rods will be numbered from bottom to top.

$$\boxed{F_{1\_h\_114} := \frac{\left(0.7 \cdot h_{eff114}^{3} \cdot \gamma_{tot} \cdot g \cdot w_{trib}\right)}{3 \cdot a} = 3.617 \cdot kip}$$

$\theta := atan\left(\frac{b_{c114}}{a}\right) = 1.357$

$$\boxed{F_{1\_114} := \frac{F_{1\_h\_114}}{\sin(\theta)} = 3.701 \cdot kip}$$

$$\boxed{F_{0\_114} := \frac{\left[0.7 \cdot h_{eff114}^{2} \cdot \gamma_{tot} \cdot g \cdot w_{trib} \cdot \left(a - \frac{h_{eff114}}{3}\right)\right]}{a} = 3.557 \cdot kip}$$

$A_{eff} := h_{eff114} \cdot w_{trib} = 17.707 \, ft^{2}$

$P_u = 7.174 \cdot kip$

$F_R := F_{1\_h\_114} + F_{0\_114} = 7.174 \cdot kip$

Pu = Fr good.

These are the forces in tension supported by anchor rods 42a and 42h. Rods will be offset, they are not on the same corrugation, so the angled rod can pass the horizontal rod.

conection at 78"

$d_{conec} := 12in$

$h_{c78} := h_{78} - d_{conec} = 34.83 \cdot in$

$b_{c78} := 9ft = 108 \cdot in$

$l_{r78} := \sqrt{b_{c78}^{\ 2} + h_{c78}^{\ 2}} = 113.477 \cdot in$

Tributary area supported by rod 78.

$w_{trib} := \frac{(3ft + 3ft)}{2} = 36 \cdot in$ this is the width of the tributary area

$h_{l\_78} := h_{42} = 82.52 \cdot in$

$h_{r\_78} := h_{114} = 31.83 \cdot in$

$h_{eff78} := \frac{\left(h_{l\_78} + h_{r\_78}\right)}{2} = 57.175 \cdot in$ this is the effective depth of the plate

$P_u := 0.7 \cdot h_{eff78}^{\ 2} \cdot \gamma_{tot} \cdot g \cdot w_{trib} = 9.349 \cdot kip$

Effective rod height

$a := h_{eff78} - 12in = 45.175 \cdot in$

Actual rod height and length

$a_{78} := h_{78} - 12in = 34.83 \cdot in$

$l_{78} := \sqrt{a_{78}^{\ 2} + b_{c78}^{\ 2}} = 113.477 \cdot in$

Anchor rods will be numbered from bottom to top.

$$\boxed{F_{1\_h\_78} := \frac{\left(0.7 \cdot h_{eff78}^{\ 3} \cdot \gamma_{tot} \cdot g \cdot w_{trib}\right)}{3 \cdot a} = 3.944 \cdot kip}$$

$\theta := atan\left(\frac{b_{c78}}{a}\right) = 1.175$

$$\boxed{F_{1\_78} := \frac{F_{1\_h\_78}}{\sin(\theta)} = 4.275 \cdot kip}$$

$$F_{0\_78} := \frac{\left[0.7 \cdot h_{eff78}^{2} \cdot \gamma_{tot} \cdot g \cdot w_{trib} \cdot \left(a - \frac{h_{eff78}}{3}\right)\right]}{a} = 5.405 \cdot kip$$

$A_{eff} := h_{eff78} \cdot w_{trib} = 14.294\,ft^2$

$P_u = 9.349 \cdot kip$

$F_R := F_{1\_h\_78} + F_{0\_78} = 9.349 \cdot kip$

Pu = Fr good.

conection at 42". This will have 3 anchor rods supporting the headwall. One angled as in anchor rods 78 and 114 and two horizontal, spaning the width of the bridge.

$d_{conec} := 12in$

$h_{c42} := h_{42} - d_{conec} = 70.52 \cdot in$

$b_{c42} := 9ft = 108 \cdot in$

$l_{r42} := \sqrt{b_{c42}^2 + h_{c42}^2} = 128.985 \cdot in$

Tributary area supported by rod 42.

$h_{l\_42} := h_0 = 135 \cdot in$

$h_{r\_42} := h_{1\_78} = 82.52 \cdot in$

$h_{eff42} := \frac{(h_{l\_42} + h_{r\_42})}{2} = 108.76 \cdot in$ this is the effective depth of the plate

$P_{42\_max} := 1.4g \cdot h_{eff42} \cdot (\rho_{water} + \gamma_{gravel}) = 17.279 \cdot psi$

$\gamma_{tot} := \rho_{water} + \gamma_{gravel} = 0.113 \cdot \frac{lb}{in^3}$

$w_{trib} := \frac{(3ft + 3.5ft)}{2} = 39 \cdot in$ this is the width of the tributary ar

Anchor rods will be numbered from bottom to top.

$h_{ceff42} := h_{eff42} - 12in = 96.76 \cdot in$

Effective height

$a := 30in$ This is the distance from the bridge connection to the first anchor rod

$b := 25in$ this is the distance between anchor rod 1 and anchor rod 2.

$d := 12in$

$c := h_{eff42} - (a + b + d) = 41.76 \cdot in$

Actual height

$$b_{42} := \frac{(b \cdot h_{42})}{h_{eff42}} = 18.968 \cdot in$$

$$c_{42} := \frac{(c \cdot h_{42})}{h_{eff42}} = 31.685 \cdot in$$

$$d_{42} := 12in$$

$$a_{42} := h_{42} - (b_{42} + c_{42} + d_{42}) = 19.867 \cdot in$$

Actual rod length

$$l_{42} := \sqrt{(a_{42} + b_{42} + c_{42})^2 + b_{c42}^2} = 128.985 \cdot in$$

$$\boxed{F_{3\_h\_42} := \frac{\left[1.4\gamma_{tot} \cdot g \cdot \left[\left(\frac{c}{2}\right) + (d)\right]^2 \cdot w_{trib}\right]}{2} = 3.349 \cdot kip}$$

Horizontal component

$$\theta := \mathrm{atan}\left(\frac{b_{c42}}{h_{c42}}\right) = 0.992$$

$$\boxed{F_{3\_42} := \frac{F_{3\_h\_42}}{\sin(\theta)} = 4 \cdot kip}$$

Force in Anchor rod 2. rod will be placed at 40 in above anchor rod 1.

$$F_{2\_42} := \left[\frac{\left[1.4\gamma_{tot} \cdot g \cdot \left[\left(\frac{c}{2}\right) + \left(\frac{b}{2}\right)\right]^2 \cdot w_{trib}\right]}{2}\right] + 1.4\gamma_{tot} \cdot g \cdot \left[\left(\frac{c}{2}\right) + (d)\right] \cdot w_{trib} \cdot \left[\left(\frac{c}{2}\right) + \left(\frac{b}{2}\right)\right] = 10.253 \cdot kip$$

$$\boxed{F_{1\_42} := \left[\frac{\left[1.4\gamma_{tot} \cdot g \cdot \left[\left(\frac{a}{2}\right) + \left(\frac{b}{2}\right)\right]^2 \cdot w_{trib}\right]}{2}\right] + 1.4\gamma_{tot} \cdot g \cdot \left[\left(\frac{b}{2}\right) + (c + d)\right] \cdot w_{trib} \cdot \left[\left(\frac{b}{2}\right) + \left(\frac{a}{2}\right)\right] = 13.633 \cdot kip}$$

Rod will be placed at 30 in above bridge.

$$\boxed{F_{0\_42} := \left[\frac{\left[1.4\gamma_{tot} \cdot g \cdot \left(\left(\frac{a}{2}\right)\right)^2 \cdot w_{trib}\right]}{2}\right] + 1.4\gamma_{tot} \cdot g \cdot w_{trib} \cdot \left(d + c + b + \frac{a}{2}\right) \cdot \left(\frac{a}{2}\right) = 9.411 \cdot kip}$$

This is the force in the connecton at the base of the head wall where it connects to the base.

Check force

$A_{t42} := w_{trib} \cdot h_{eff42} = 29.456 \cdot ft^2$

$P_{resultant} := \dfrac{P_{42\_max}}{2} = 8.64\,psi$ this is the resultant pressure.

$h_p := \dfrac{h_{l\_42} + h_{r\_42}}{6} = 36.253 \cdot in$ this is the height of the resultant pressure

$F_R := \left(F_{3\_h\_42} + F_{2\_42} + F_{1\_42} + F_{0\_42}\right) = 36.647 \cdot kip$

$F_{soil} := A_{t42} \cdot P_{resultant} = 36.647 \cdot kip$ reactions good.

These are the forces in tension supported by anchor rods 42a and 42h. Rods will be offset, they are not on the same corrugation, so the angled rod can pass the horizontal rod.

Connection 0

$d_{conec} := 12in$

$h_{c0} := h_0 - d_{conec} = 123 \cdot in$

Tributary area supported by rod 42.

$h_{eff0} := h_0 = 135 \cdot in$

$P_{0\_max} := 1.4g \cdot h_{eff0} \cdot \left(\rho_{water} + \gamma_{gravel}\right) = 21.448 \cdot psi$

$\gamma_{tot} := \rho_{water} + \gamma_{gravel} = 0.113 \cdot \dfrac{lb}{in^3}$

$w_{trib} := \dfrac{(3.5ft)}{2} = 21 \cdot in$ this is the width of the tributary area

Anchor rods will be numbered from bottom to top.

$e := 30in$

$d := 36in$ This is the distance from the bridge connection to the first anchor rod 2

$c := 24in$ this is the distance between anchor rod 1 and anchor rod 2.

$b := 16in$

$a := h_0 - (b + c + d + e) = 29 \cdot in$

$$\boxed{F_{4\_0} := \frac{\left[\gamma_{tot} \cdot g \cdot \left[\left(\frac{d}{2}\right) + (e)\right]^2 \cdot w_{trib}\right]}{2} = 2.745 \cdot kip}$$

Force in Anchor rod 4. rod will be placed at a + b + c + d above the bridge.

$$\boxed{F_{3\_0} := \left[\frac{\left[\gamma_{tot} \cdot g \cdot \left[\left(\frac{d}{2}\right) + \left(\frac{c}{2}\right)\right]^2 \cdot w_{trib}\right]}{2}\right] + \gamma_{tot} \cdot g \cdot \left[\left(\frac{d}{2}\right) + (e)\right] \cdot w_{trib} \cdot \left[\left(\frac{d}{2}\right) + \left(\frac{e}{2}\right)\right] = 4.847 \cdot kip}$$

$$F_{2\_0} := \left[\frac{\left[\gamma_{tot} \cdot g \cdot \left[\left(\frac{c}{2}\right) + \left(\frac{b}{2}\right)\right]^2 \cdot w_{trib}\right]}{2}\right] + \gamma_{tot} \cdot g \cdot \left[\left(\frac{c}{2}\right) + (d + e)\right] \cdot w_{trib} \cdot \left[\left(\frac{c}{2}\right) + \left(\frac{b}{2}\right)\right] = 4.194 \cdot kip$$

$$F_{1\_0} := \left[\frac{\left[\gamma_{tot} \cdot g \cdot \left[\left(\frac{b}{2}\right) + \left(\frac{a}{2}\right)\right]^2 \cdot w_{trib}\right]}{2}\right] + \gamma_{tot} \cdot g \cdot \left[\left(\frac{b}{2}\right) + (c + d + e)\right] \cdot w_{trib} \cdot \left[\left(\frac{b}{2}\right) + \left(\frac{a}{2}\right)\right] = 5.858 \cdot kip$$

$$F_{0\_0} := \left[\frac{\left[\gamma_{tot} \cdot g \cdot \left(\left(\frac{a}{2}\right)\right)^2 \cdot w_{trib}\right]}{2}\right] + \gamma_{tot} \cdot g \cdot \left[\left(\frac{a}{2}\right) + (b + c + d + e)\right] \cdot w_{trib} \cdot \left(\left(\frac{a}{2}\right)\right) = 4.415 \cdot kip$$

Rod will be placed at 31.5 in above bridge.

This is the force in the connecton at the base of the head wall where it connects to the base.

Check force

$A_{t42} := w_{trib} \cdot h_{eff42} = 15.861 \cdot ft^2$

$P_{resultant} := \frac{P_{42\_max}}{2} = 8.64\,psi$ this is the resultant pressure.

$h_p := \frac{h_{l\_42} + h_{r\_42}}{6} = 36.253 \cdot in$ this is the height of the resultant pressure

$F_R := 1.4 \cdot \left(F_{3\_0} + F_{2\_0} + F_{1\_0} + F_{4\_0} + F_{0\_0}\right) = 30.884 \cdot kip$

$F_{soil} := A_{t42} \cdot P_{resultant} = 19.733 \cdot kip$ reactions good.

Headwall plate Calculations

Use seam strength of plates to determine gage of steel.

$SS := 66000 \frac{lbf}{ft}$ From bridge design calcs.

Use 8 gage 6 x 2 corrugation. 4 bolts per foot except at anchor rod connections. (Contech, table 2).

$SS := 81000 \frac{lbf}{ft}$ (contech, table 2)

Anchor rod conections

Bolts used are 3/4" diameter meeting ASTM A449 specifications.

$F_n := 90ksi$ (AISC Table J3.2 group A) assume threads exposed. this is the tensile strength of the 3/4" bolt.

$d_b := 0.75in$

$$A_b := \pi \cdot \frac{d_b^{\,2}}{4} = 0.442 \cdot in^2$$

$R_n := F_n \cdot A_b = 39.761 \cdot kip$ (AISC J3-1) this is the nominal capacity of a bolt in pure tension.

Loads

$F_{0\_114} = 3.557 \cdot kip$

$F_{0\_78} = 5.405 \cdot kip$

$F_{0\_42} = 9.411 \cdot kip$

$F_{hmax} := F_{1\_42} = 13.633 \cdot kip$

$F_{0\_0} = 4.415 \cdot kip$

$F_{1\_42} = 13.633 \cdot kip$

$F_{1\_0} = 5.858 \cdot kip$

$F_{2\_42} = 10.253 \cdot kip$

$F_{2\_0} = 4.194 \cdot kip$

$F_{3\_0} = 4.847 \cdot kip$

$F_{4\_0} = 2.745 \cdot kip$

$\Phi := 0.75$

$\Phi \cdot R_n = 29.821 \cdot kip$ ΦRn > Fhmax so the bolts are sufficently strong in pure tension.

Check bolts in combined shear and tension

horizontal component

$F_{1\_h\_114} = 3.617 \cdot kip$

$F_{1\_h\_78} = 3.944 \cdot kip$

$F_{3\_h\_42} = 3.349 \cdot kip$

resultant

$F_{1\_114} = 3.701 \cdot kip$

$F_{1\_78} = 4.275 \cdot kip$

$F_{3\_42} = 4 \cdot kip$

verticlal component for shear calcs

$$F_{1\_v\_114} := \sqrt{F_{1\_114}^{\,2} - F_{1\_h\_114}^{\,2}} = 0.784 \cdot kip$$

$$F_{1\_v\_78} := \sqrt{F_{1\_78}^{\,2} - F_{1\_h\_78}^{\,2}} = 1.65 \cdot kip$$

$$F_{3\_v\_42} := \sqrt{F_{3\_42}^{\,2} - F_{3\_h\_42}^{\,2}} = 2.187 \cdot kip$$

Connection at 42" controls. Use this for calculations.

$F_{nt} := F_n = 90 \cdot ksi$ (AISC Table J3.2)

$F_{nv} := 54ksi$ (AISC Table J3.2)

$f_{rv} := \frac{F_{3\_v\_42}}{A_b} = 712.851 \frac{1}{ft^2} \cdot kip$ (AISC J3-1)

$F'_{nt} := 1.3 \cdot F_{nt} - \left(\frac{F_{nt}}{\Phi \cdot F_{nv}}\right) \cdot f_{rv} = 1.06 \times 10^5 \, psi$ (AISC J3-3a)

$R_n := F'_{nt} \cdot A_b = 46.829 \cdot kip$ (AISC J3-2)

$\Phi \cdot R_n = 35.122 \cdot kip$

$F_{rmax} := F_{2\_42} = 10.253 \cdot kip$

ΦRn > Frmax so the bolts in tension shear bearing are good.

Anchor rod diameter at anchor rod 42 anchor rod #1 and #2.

$\Phi := 0.90$

$A_{ar\_req} := \frac{F_{hmax}}{\Phi \cdot F_n} = 0.168 \cdot in^2$

$d_{ar\_req} := \sqrt{A_{ar\_req} \cdot 4 \cdot \pi} = 1.454 \cdot in$ $d_{bl} := 1.5in$

Use 1.5 inch rod for connections at 42.

$F_{rmax2} := F_{1\_0} = 5.858 \cdot kip$

$\Phi := 0.90$

$A_{ar\_req} := \frac{F_{rmax2}}{\Phi \cdot F_n} = 0.072 \cdot in^2$

$d_{ar\_req} := \sqrt{A_{ar\_req} \cdot 4 \cdot \pi} = 0.953 \cdot in$

Use 1 inch rod at all other connections.

$d_{bs} := 1.0in$

Calculate minimum thickness of the plates for connection due to prying action.

Use A36 steel for plate

$F_u := 58ksi$ $F_y := 36ksi$ (AISC table 2-5)

Use four bolts per plate with 1 in anchor rod

$T := \frac{F_{hmax}}{4} = 3.408 \cdot kip$

$\Phi := 0.90$

$b_s := 6in$ this is the distance between corrugations

$b := \left(\frac{b_s}{2}\right) - \frac{d_{bs}}{2} = 2.5 \cdot in$

$b' := b - \frac{d_b}{2} = 2.125 \cdot in$

$p := 2 \cdot b = 5 \cdot in$

$t_{min} := \sqrt{\frac{(4 \cdot T \cdot b')}{\Phi \cdot p \cdot F_u}} = 0.333 \cdot in$

$t := \left(\frac{3}{8}\right) in$ use 3/8 in plate for thickness

$d_{e\_min} := 1.0in$ (AISC table J3.4) This is the minnimum distance from the bolt center to the edge of the plate.

$d_e := 1.5in$ Use 1.5 in

$w_{pl} := b_s + 2 \cdot d_e = 9 \cdot in$ This is the outside width of the square plate.

check strength of plate in shear.

$A_{pl} := w_{pl} \cdot t = 3.375 \cdot in^2$

$R_n := 0.60 \cdot F_y \cdot A_{pl} = 72.9 \cdot kip$ (AISC J4-3)

$\Phi := 1.0$

$\Phi \cdot R_n = 72.9 \cdot kip$ This is the failure strength of the plate subjected to shear upon yeilding. It is greater than all axial loads so the plate is good in yeilding for shear.

Check shear rupture.

find smallest net area of plate.

$d_b = 0.75 \cdot in$

$d_{bh} := d_b + 0.125in = 0.875 \cdot in$ (AISC specifies 1/16 inch for fitting bolt in the hole and 1/16 inch for damage to material when drilling, AISC Table J3.3)

$d_{ah} := d_{bl} + 1.25in = 2.75 \cdot in$

$l_{n1} := 2 \cdot 1.5in + 2\sqrt{(3in)^2 + (3in)^2} - \left(2 \cdot d_{bh} + d_{ah}\right) = 6.985 \cdot in$

$l_{n2} := w_{pl} - 2 \cdot d_{bh} = 7.25 \cdot in$

plath 1 including the hole for the anchor rod is the shortest. use path 1.

$A_{nv} := l_{n1} \cdot t = 2.619 \cdot in^2$

$R_n := 0.60 \cdot F_u \cdot A_{nv} = 91.158 \cdot kip$ (AISC J4-4)

$\Phi := 0.75$

$\Phi \cdot R_n = 68.368 \cdot kip$

This is larger than all loads experienced by the connected anchor rods therefore the plate is good in shear rupture.

Plate is good.

Calculate slotted anchor rod hole length.

$t = 0.375 \cdot in$

$d_{bl} = 1.5 \cdot in$ Diameter of large anchor rod

$d_{hl} := d_{bl} + \left(\frac{1}{16}\right) in = 1.562 \cdot in$ Diameter of large hole

$d_{bs} = 1 \cdot in$ Diameter of small anchor rod

$d_{hs} := d_{bs} + \left(\frac{1}{16}\right) in = 1.063 \cdot in$ Diameter of small hole

Top connection at 114"

$\theta_{114} := atan\left(\frac{a_{114}}{b_{c114}}\right) = 0.182$

$o_{114a} := t \cdot tan(\theta_{114}) = 0.069 \cdot in$

$o_{114b} := \frac{d_{hs}}{cos(\theta_{114})} = 1.08 \cdot in$

$l_{slot\_114} := o_{114a} + o_{114b} = 1.149 \cdot in$

use lslot = 1.25 in for slot 114.

Top connection at 78"

$\theta_{78} := atan\left(\frac{a_{78}}{b_{c78}}\right) = 0.312$

$o_{78a} := t \cdot tan(\theta_{78}) = 0.121 \cdot in$

$o_{78b} := \frac{d_{hs}}{cos(\theta_{78})} = 1.116 \cdot in$

$l_{slot\_78} := o_{78a} + o_{78b} = 1.237 \cdot in$

Use lslot = 1.25 in for slot 78

Top connection at slot 42

$$\theta_{42} := \operatorname{atan}\left(\frac{a_{42} + b_{42} + c_{42}}{b_{c42}}\right) = 0.578$$

$$o_{42a} := t \cdot \tan(\theta_{42}) = 0.245 \cdot in$$

$$o_{42b} := \frac{d_{hs}}{\cos(\theta_{42})} = 1.269 \cdot in$$

$$l_{slot\_42} := o_{42a} + o_{42b} = 1.514 \cdot in$$

Use lslot = 1.625 in for slot 78

Crown Plate connections:

For anchor rod connections on the crown plate, use the length of slotted conections used on the head wall.

Use L6x4x3/4. 6 in in length

check shear.

Bolts used are 3/4" diameter meeting ASTM A449 specifications.

$F_n := 90ksi$ (AISC Table J3.2 group A) assume threads exposed. this is the tensile strength of the 3/4" bolt.

$d_b := 0.75in$

$$A_b := \pi \cdot \frac{d_b^{\ 2}}{4} = 0.442 \cdot in^2$$

$R_n := F_n \cdot A_b = 39.761 \cdot kip$ (AISC J3-1) this is the nominal capacity of a bolt in pure tension.

Loads

Check bolts in combined shear and tension

horizontal component for shear clacs

$F_{1\_h\_114} = 3.617 \cdot kip$

$F_{1\_h\_78} = 3.944 \cdot kip$

$F_{3\_h\_42} = 3.349 \cdot kip$

resultant

$F_{1\_114} = 3.701 \cdot kip$

$F_{1\_78} = 4.275 \cdot kip$

$F_{3\_42} = 4 \cdot kip$

verticlal component for tension calcs

$$F_{1\_v\_114} := \sqrt{F_{1\_114}^{\ 2} - F_{1\_h\_114}^{\ 2}} = 0.784 \cdot kip$$

$$F_{1\_v\_78} := \sqrt{F_{1\_78}^{2} - F_{1\_h\_78}^{2}} = 1.65 \cdot kip$$

$$F_{3\_v\_42} := \sqrt{F_{3\_42}^{2} - F_{3\_h\_42}^{2}} = 2.187 \cdot kip$$

Connection at 42" controls. Use this for calculations.

$F_{nt} := F_n = 90 \cdot ksi$ (AISC Table J3.2)

$F_{nv} := 54 ksi$ (AISC Table J3.2)

$f_{rv} := \dfrac{F_{1\_h\_78}}{A_b} = 8.927 \cdot ksi$ (AISC J3-1)

$F'_{nt} := 1.3 \cdot F_{nt} - \left(\dfrac{F_{nt}}{\Phi \cdot F_{nv}}\right) \cdot f_{rv} = 97.162 \cdot ksi$ (AISC J3-3a)

$R_n := F'_{nt} \cdot A_b = 42.925 \cdot kip$ (AISC J3-2)

$\Phi \cdot R_n = 32.193 \cdot kip$

$F_{rmax} := F_{2\_42} = 10.253 \cdot kip$

ΦRn > Frmax so the bolts in tension shear bearing are good.

Use 2 bolts to prevent twisting of angle.

Appendix I:

Construction Items Quantity Calculations

Gravel Placement

Calculating the total amount of gravel needed to be placed onto the steel plated after it is assembled to the concrete footings

$L1a := 36.5ft$ $B2 := 60.1ft$

$L1b := 10ft$

$H1 := 7.83ft$ $H2 := 13.83ft$

$$Area1 := \left(\frac{L1a + L1b}{2}\right)\cdot H1 = 182.048\cdot ft^2$$

$$Area2 := \frac{0.5\cdot(B2\cdot H2)}{2} = 207.796\cdot ft^2$$

$$GravelTotal := (Area1 + Area2)\cdot 14ft = 202.141\cdot yd^3$$

$$GravelTon := GravelTotal\cdot 133.7\frac{lb}{yd^3} = 13.513\cdot ton$$

$$TL1 := \left(L1a^2 + H1^2\right)^{0.5} = 37.33\cdot ft \quad TL2 := L1b \quad TL3 := \left(B2^2 + H2^2\right)^{0.5} = 61.671\cdot ft$$

$$TotalLength := TL1 + TL2 + TL3 = 109.001\cdot ft$$

$$TotalSA := TotalLength\cdot 14ft = 1526.016\cdot ft^2$$

Gravel Spreading

Excavation Removal

Excavation of the foundations and of the channel

$2(20ft\cdot 7.5ft\cdot 5ft) = 55.556\cdot yd^3$ Foundations

$\frac{(23ft\cdot 9ft\cdot 20ft)}{2} = 76.667\cdot yd^3$ Channel Excavation

Concrete Footing

$CF := 2(20ft\cdot 2ft\cdot 5ft) = 14.815\cdot yd^3$

Placed concrete footings

Channel Sloping

The area to be sloped along the north and south sides of the stream channel

$NW_{CS} := 10ft$

$SW_{CS} := 12ft$

$NL_{CS} := 10ft + 17ft + 5ft + 27ft$

$SL_{CS} := 15ft + 27ft + 17ft + 6ft$

$Area_N := NW_{CS}\cdot NL_{CS} = 65.556\cdot yd^2$

$Area_S := SW_{CS}\cdot SL_{CS} = 86.667\cdot yd^2$

$Area_{Total} := Area_N + Area_S = 152.222\cdot yd^2$

Site Restoration

$SRWS := 7ft$
$SRLS := 15ft + 27ft = 42 \cdot ft$
$SRWN := 3ft$
$SRLN := 10ft + 17ft + 5ft = 32 \cdot ft$

$NTotal := SRWS \cdot SRLS = 32.667 \cdot yd^2$

$STotal := SRWN \cdot SRLN = 10.667 \cdot yd^2$

$Total := NTotal + STotal = 43.333 \cdot yd^2$

Grass seeding to be placed in the areas above the riprap to help with erosion control

Masonry Wall

South Side

$SS := (5.5ft \cdot 16ft \cdot 10in) = 2.716 \cdot yd^3$

North Side

$NSL1 := 3.5ft + 12ft$

$NSH1 := 3.5ft$

$NSL2 := 5ft + 13ft + 9ft + 10ft$

$NSH2 := 3ft$

$NS1 := NSL1 \cdot NSH1 \cdot 10in = 1.674 \cdot yd^3$

$NS2 := NSL2 \cdot NSH2 \cdot 10in = 3.426 \cdot yd^3$

$Curb := 2(29ft + 35ft) \cdot 10in \cdot 6in = 1.975 \cdot yd^3$

$NS := NS1 + NS2 + Curb = 7.076 \cdot yd^3$

Volume of concrete blocks to be used in constructing the side masonry walls to contain the gravel

Toyota Hilux Cab Size

Calculations for the transportation of material needed for construction

$L := 2160mm = 7.087ft$

$W := 1464mm = 4.803ft$

$D := 405mm = 1.329ft$

$Vol := L \cdot W \cdot D = 1.675yd^3$

Dump Truck

$TruckVol := 12yd^3$

$AggVol := 20yd^3$

$69.2\frac{lb}{ft^3} \cdot 18ton = 1.81 \times 10^7 \frac{kg^2}{m^3}$

$SandVol := 20yd^3$

$ConSack := 125$

$RipRapVol := 44yd^3$

$Price := 1.72\frac{¤}{ft^3} = 60.741\frac{¤}{m^3}$

$1680\frac{kg}{m^3} = 2831.732\frac{lb}{yd^3}$

$AggVol \cdot Price = 928.8¤$

$GravelVol := 202yd^3$

$\frac{GravelVol}{TruckVol} = 16.833$	17 Trucks	6 hr Pit
$\frac{SandVol}{TruckVol} = 1.667$	2 Truck	6 hr Pit
$\frac{RipRapVol}{TruckVol} = 3.667$	4 Truck	6 hr Pit
$\frac{ConSack}{9} = 13.889$	14 Pickup Truck	6 hr Pit
$\frac{AggVol}{TruckVol} = 1.667$	2 Pick Truck	6 hr Pit
Plates + Bolts	2 Pickup Truck	8 hr Panama City
Rebar	2 Pickup Truck	6 hr Pit
Falsework	1 Pickup Truck	6 hr Pit

Steel Crown Plate

$$LargeCorrugatedPlate := 3000\frac{¤}{ton}$$

Crown Plate
Corrugated 15.5 in x 5 in

10 8S Plates
5 9S Plates

$$PL8S := 454lb$$
$$PL9S := 507lb$$

$$TotalPL8S := 10\cdot PL8S$$
$$TotalPL9S := 5\cdot PL9S$$

$$LargeSteelPlate := LargeCorrugatedPlate\cdot(TotalPL8S + TotalPL9S) = 10612.5¤$$

Steel Headwall Plate

$$SmallCorrugatedPlate := 2000\frac{¤}{ton}$$

Headwall Plate
Corrugated 6 in x 2 in

$$N := 2 + 2 + 2 = 6$$ Number of Plates needed

$$PlateWeight := 272lb$$

$$TotalWeight := PlateWeight\cdot N = 1632\, lb$$

$$Price := SmallCorrugatedPlate\cdot TotalWeight = 1632¤$$

Steel Footing Plate

$$PlateWeight := \frac{272lb}{10ft} = 27.2\cdot\frac{lb}{ft}$$

Footing Plate
Corrugated 6 in x 2 in

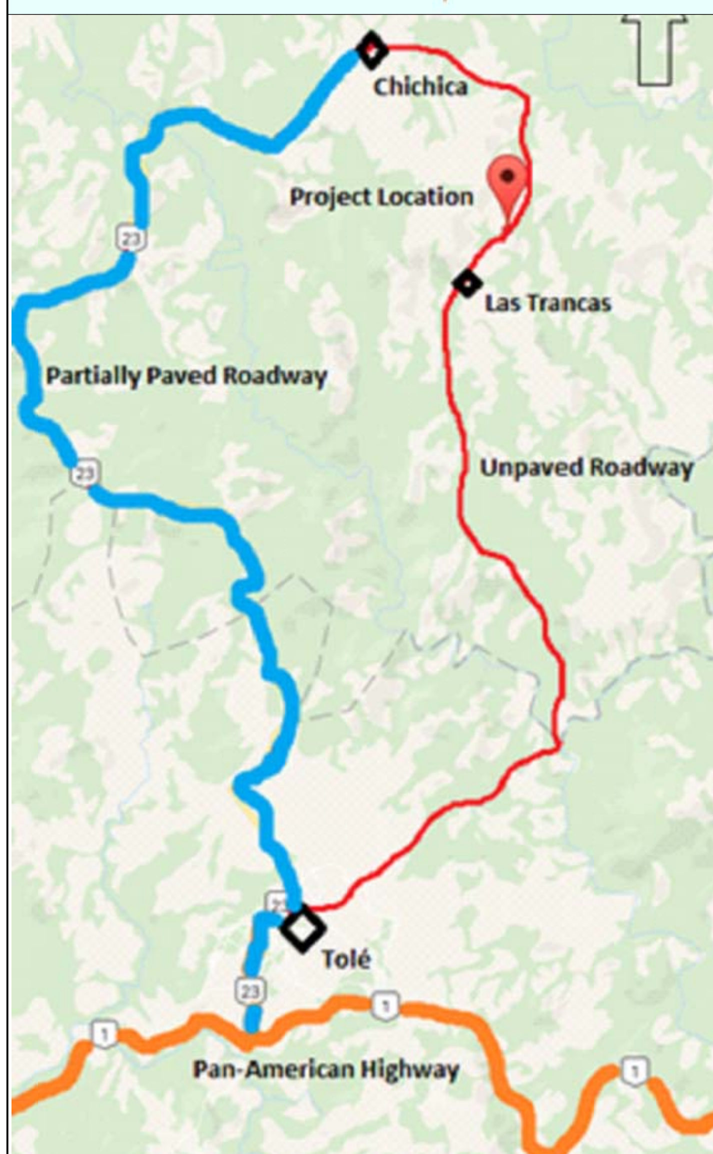
$$FootingPlateWeight := 18.75ft\cdot PlateWeight = 510\, lb$$

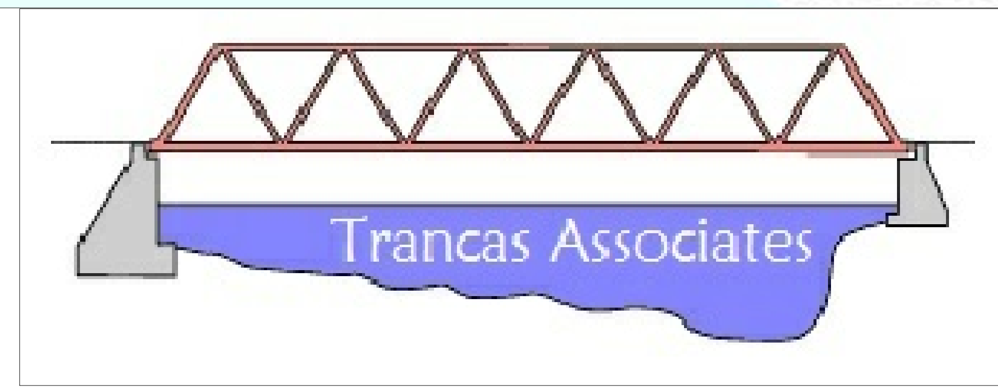
$$Price := SmallCorrugatedPlate\cdot FootingPlateWeight = 510¤$$

Appendix J:

Final Design Drawings and Detailing

(Attached)





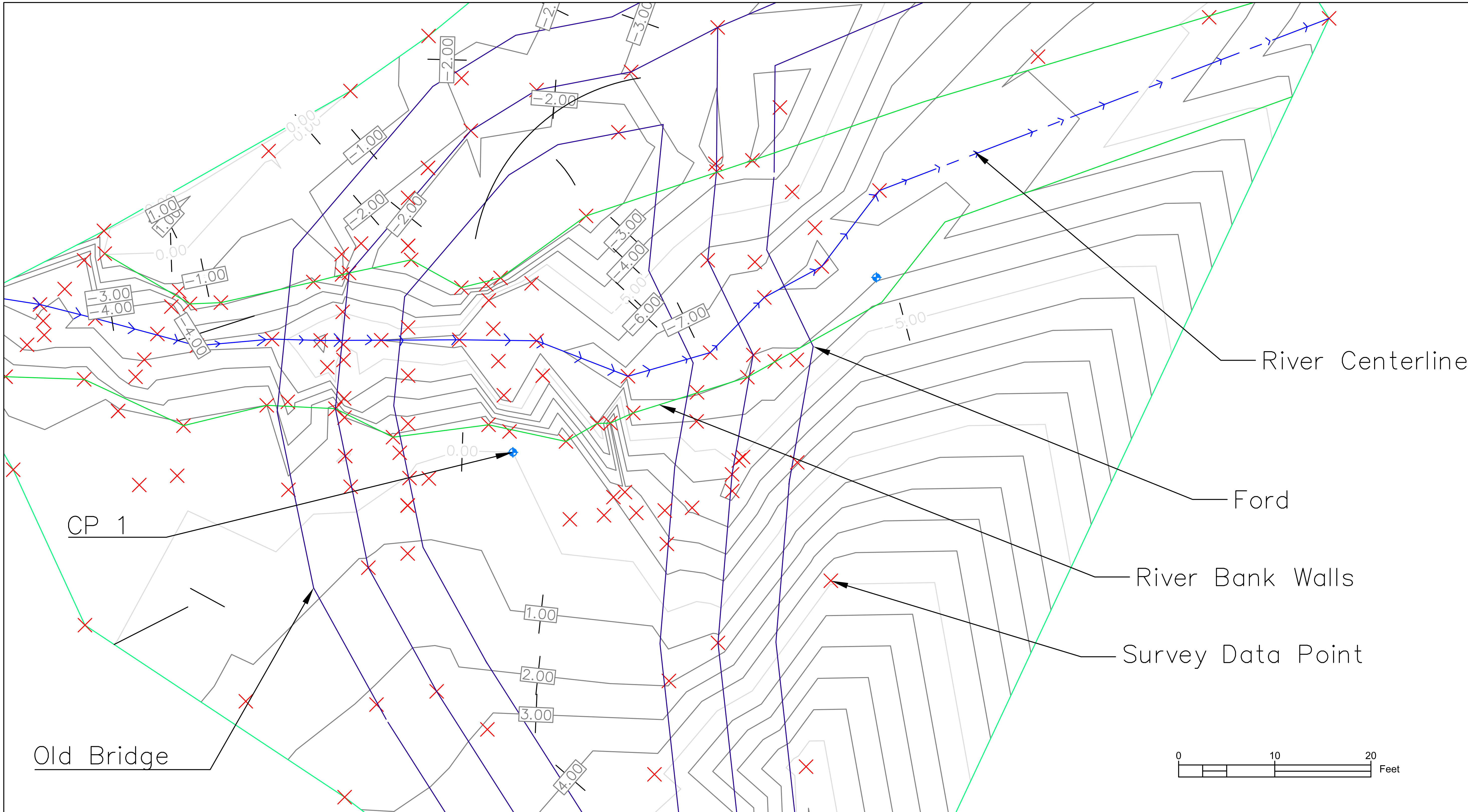
Trancas Associates

INDEX OF SHEETS

PROJECT: LAS TRANCAS STREAM CROSSING

DRAWING: Title Page	ORGANIZATION: TRANCAS ASSOCIATES	
PLOT DATE: 12/8/2016	SCALE: N/A	
FILENAME:sitesketchCivil3Da.dwg	SHEET: 1 of 18	J-1

Trancas Associates
1400 Townsend Drive
Houghton, MI 49931



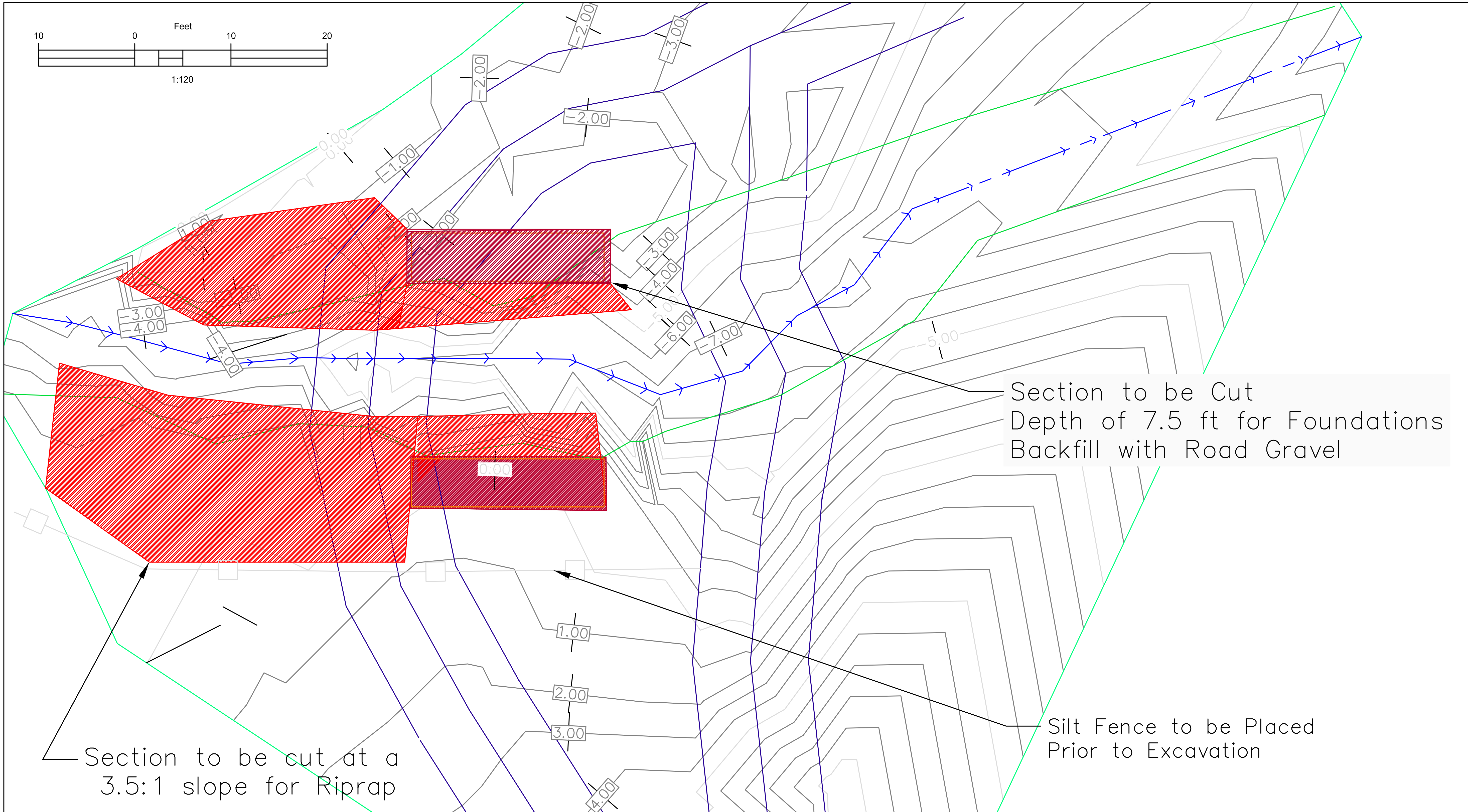
Contour interval Lines follow
1 ft increments from CP 1

N

PROJECT: LAS TRANCAS STREAM CROSSING	
DRAWING: Contour Map	ORGANIZATION: TRANCAS ASSOCIATES
PLOT DATE: 12/8/2016	SCALE: 1" = 20'
FILENAME: sitesketchCivil3Da.dwg	SHEET: 2 of 18

Trancas Associates

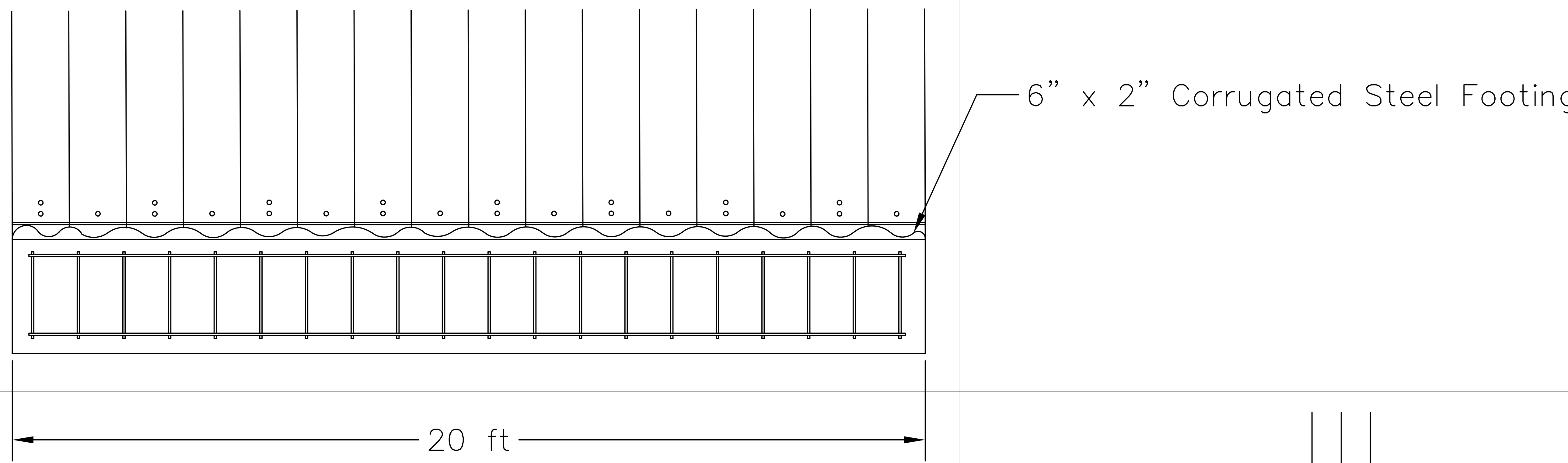
J-2

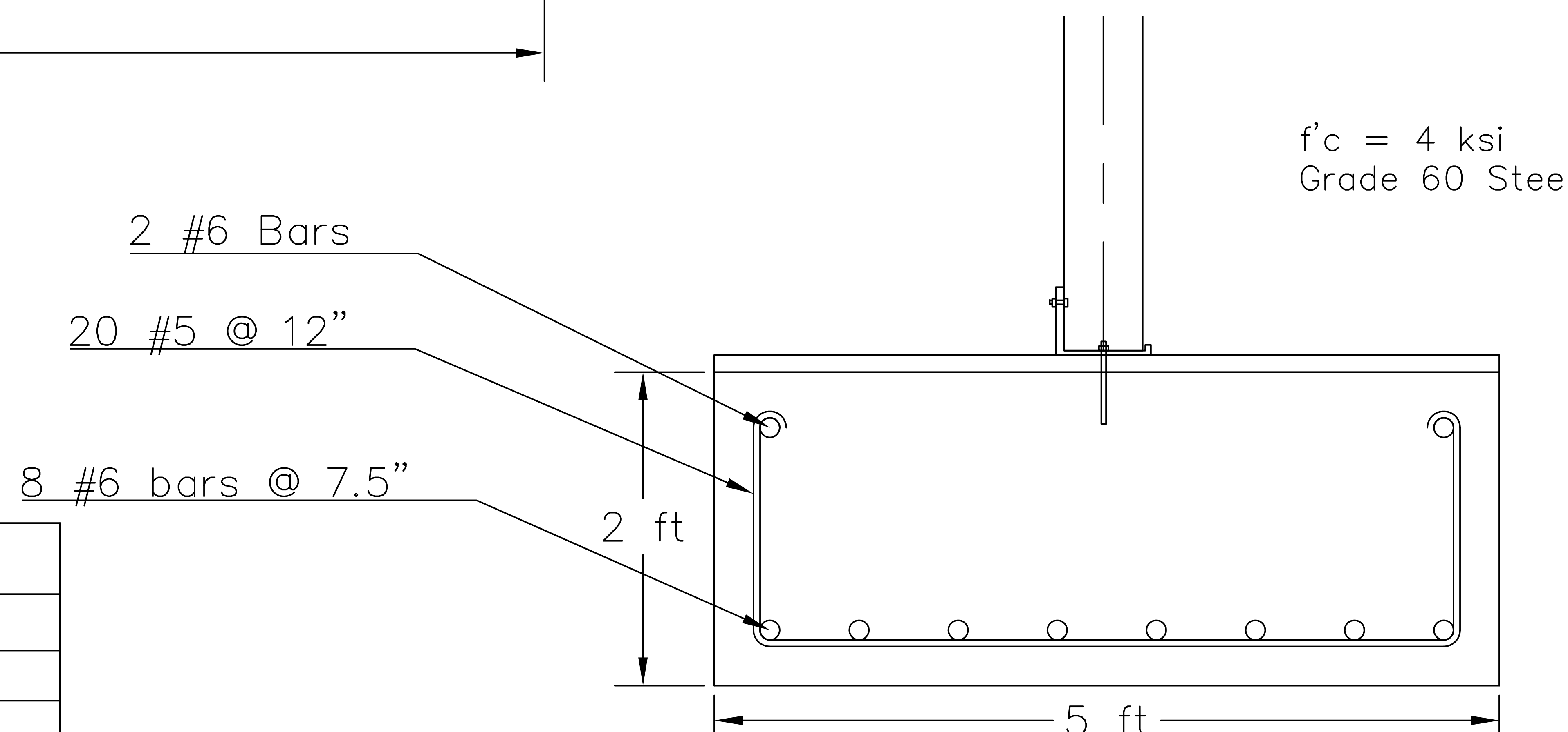


Construction Items	
River Bank Cut	30 cyd
Foundation Cut	56 cyd
Channel Sloping	155 syd
Silt Fence	73 ft

PROJECT: LAS TRANCAS STREAM CROSSING

DRAWING: Excavation Cut Sheet	ORGANIZATION: TRANCAS ASSOCIATES	Trancas Associates
PLOT DATE: 12/8/2016	SCALE: 1" = 10'	
FILENAME:sitesketchCivil3Da.dwg	SHEET: 3 of 18	J-3

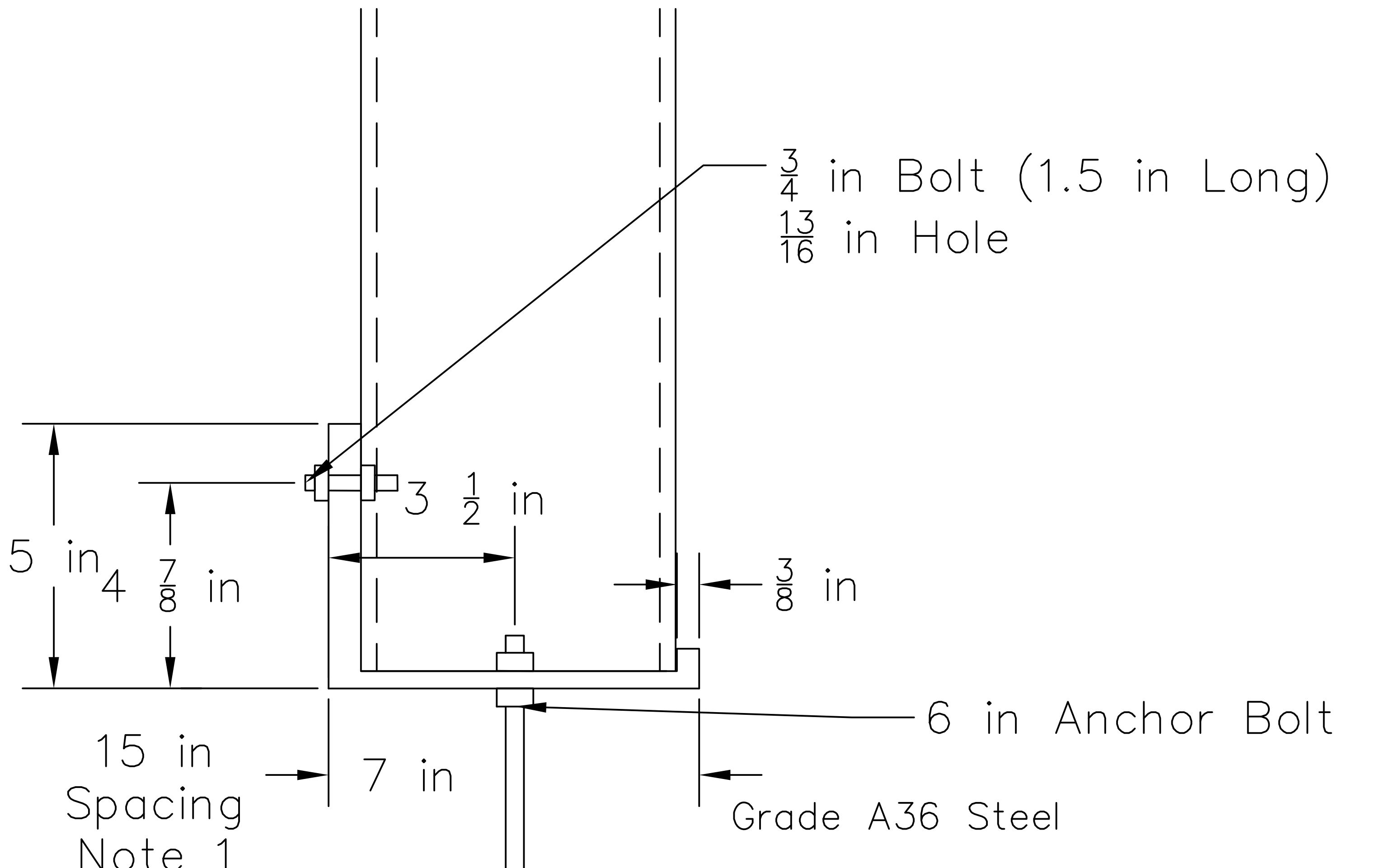


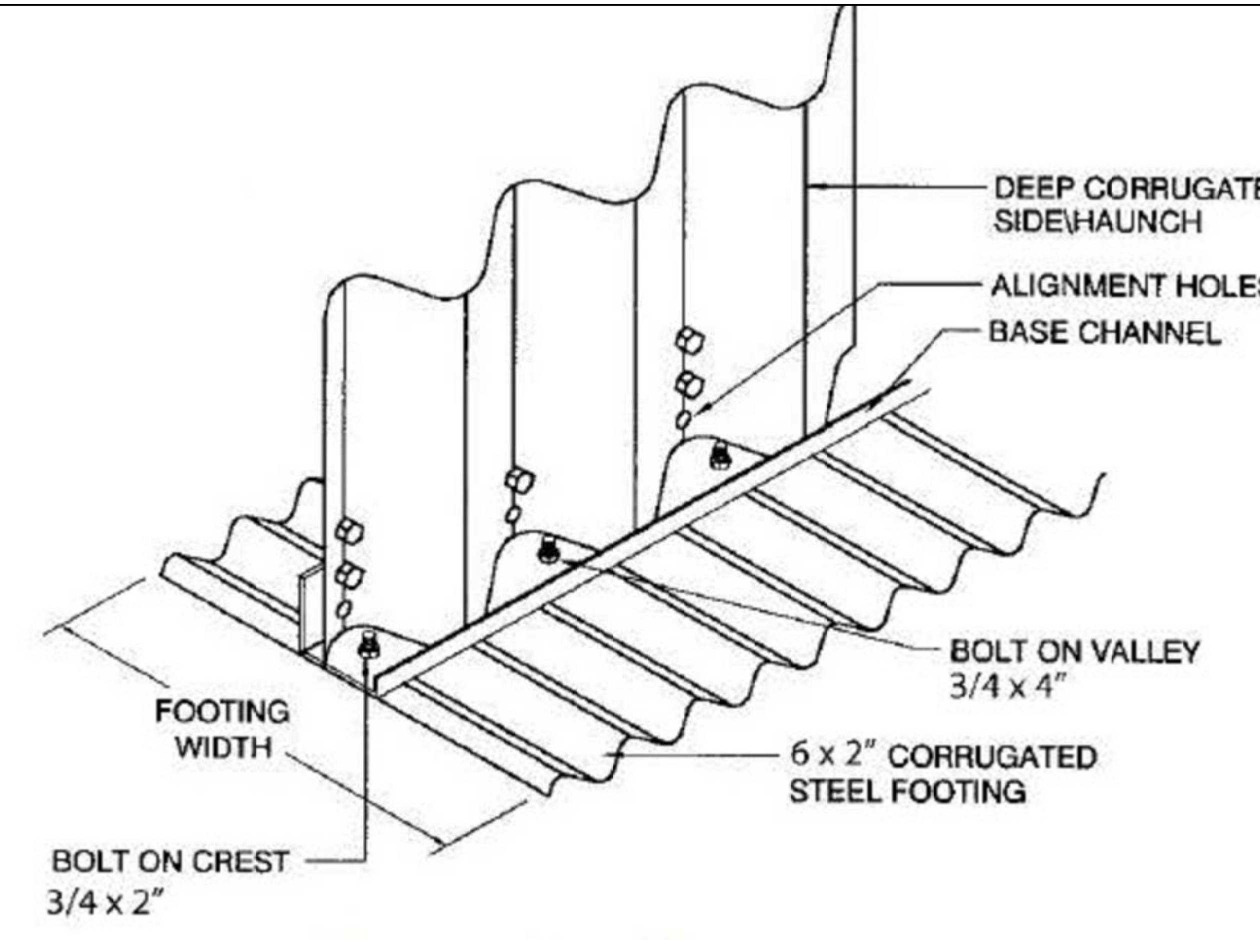


Construction Items	
Concrete Footing	15 cyd
Metal Footing	18.75 Ft
#5 Stirrups Rebar	170 lb
#6 Reinforced Rebar	295 lb
Concrete Sacks	78 Each
Aggregate	540 cft
Sand	20 cyd
Steel Rebar Wiring Roll	1 Each

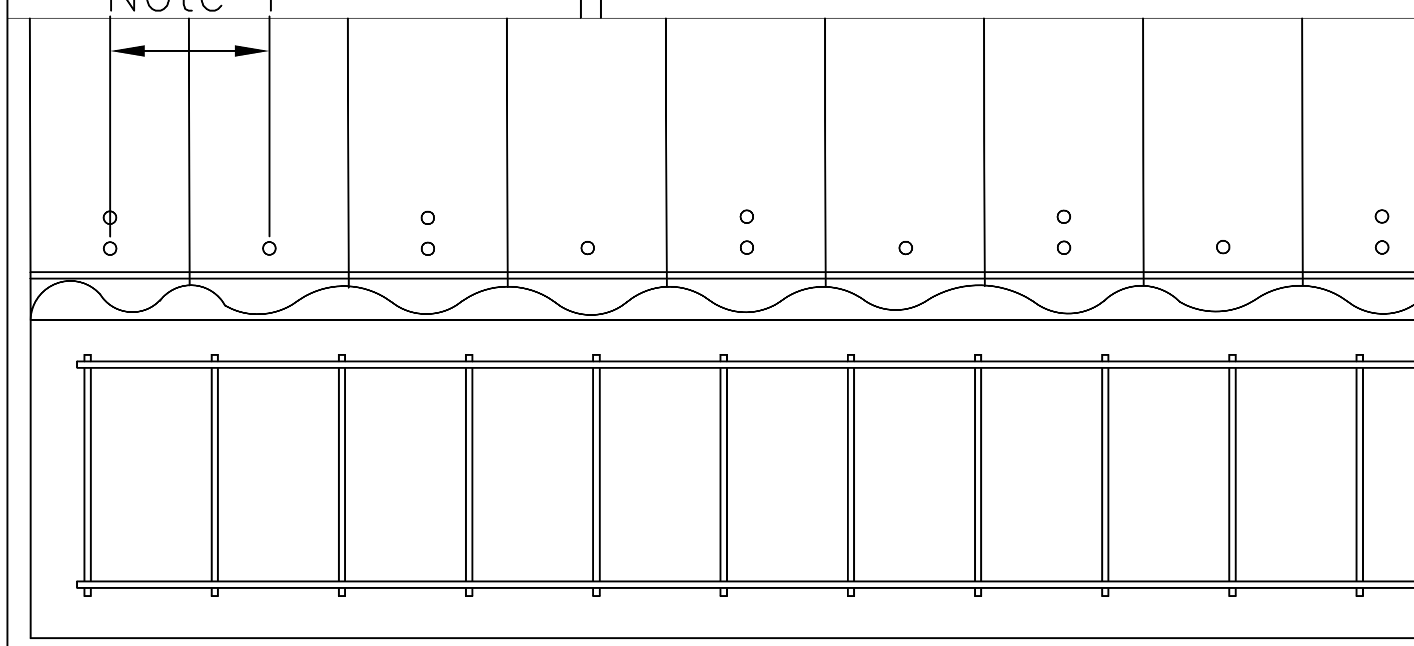
PROJECT: LAS TRANCAS STREAM CROSSING

DRAWING: Footing Details	ORGANIZATION: TRANCAS ASSOCIATES	Trancas Associates
PLOT DATE: 12/8/2016	SCALE: N/A	
FILENAME:sitesketchCivil3Da.dwg	SHEET: 4 of 18	J–4



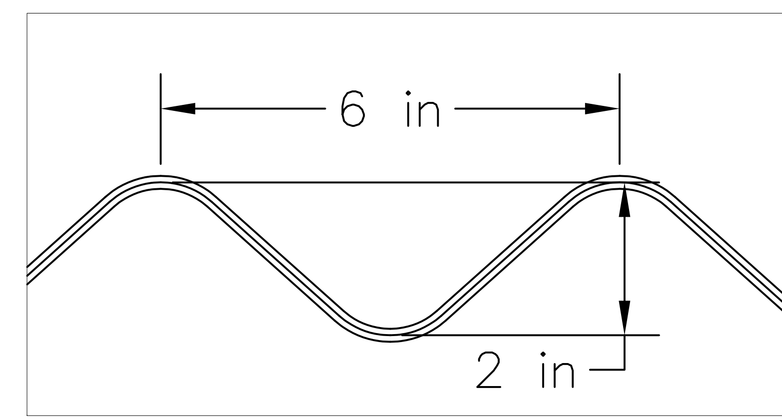


Detail – Typical Steel Footer



Steel Footer
18.75 ft Length
5 ft Wide

15 Dia $\frac{3}{4}$"
6" Length
Anchor bolts



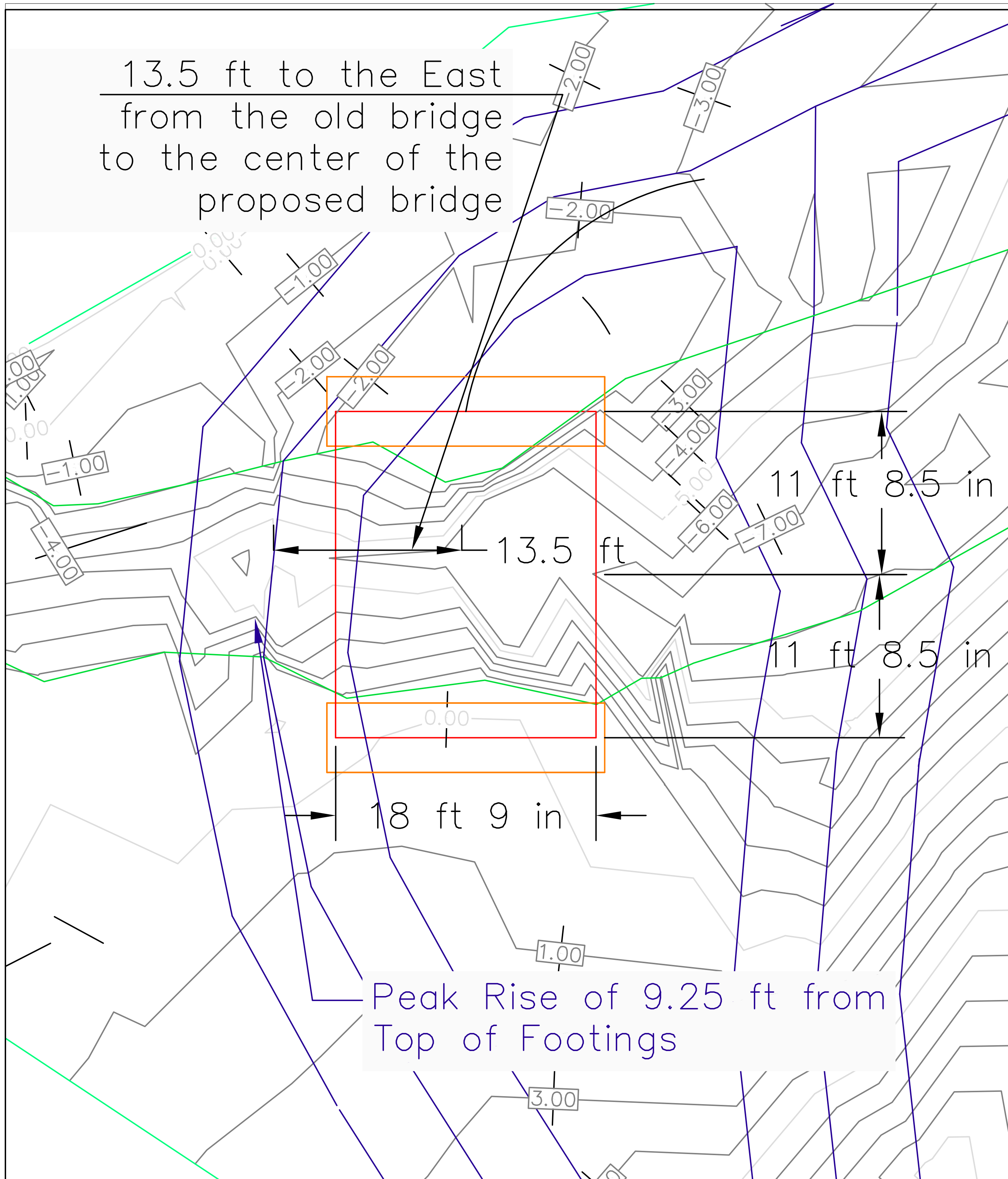
6 in x 2 in Corrugation
Thickness Gauge 8 (0.17 in)
ASTM A761
3 oz per sft of galvanized coating

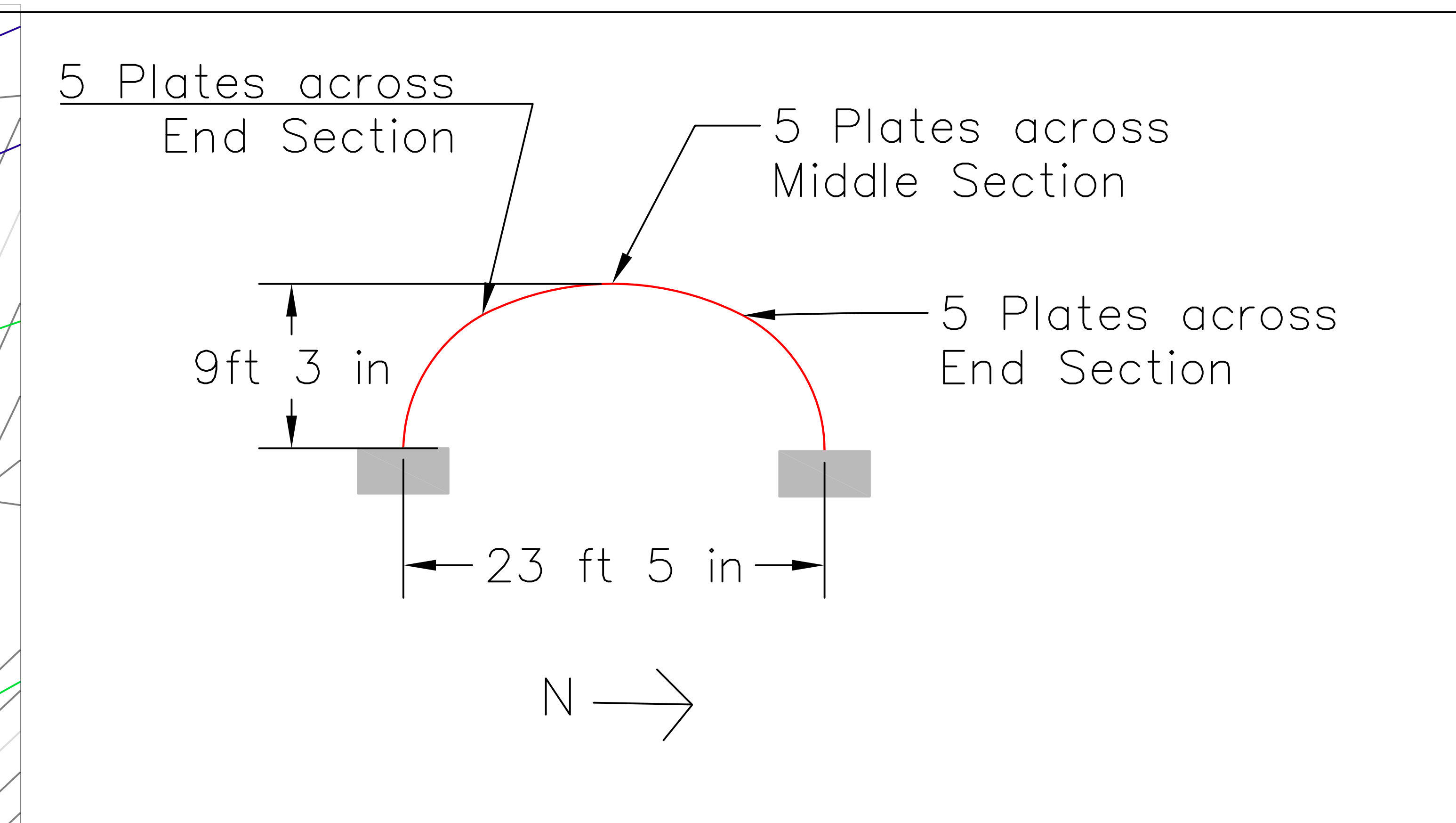
Construction Items	
Steel Base Unbalanced Channel 5" x 7"	38 ft
Anchor Bolts	15 Each
Anchor Bolt Epoxy	43 oz

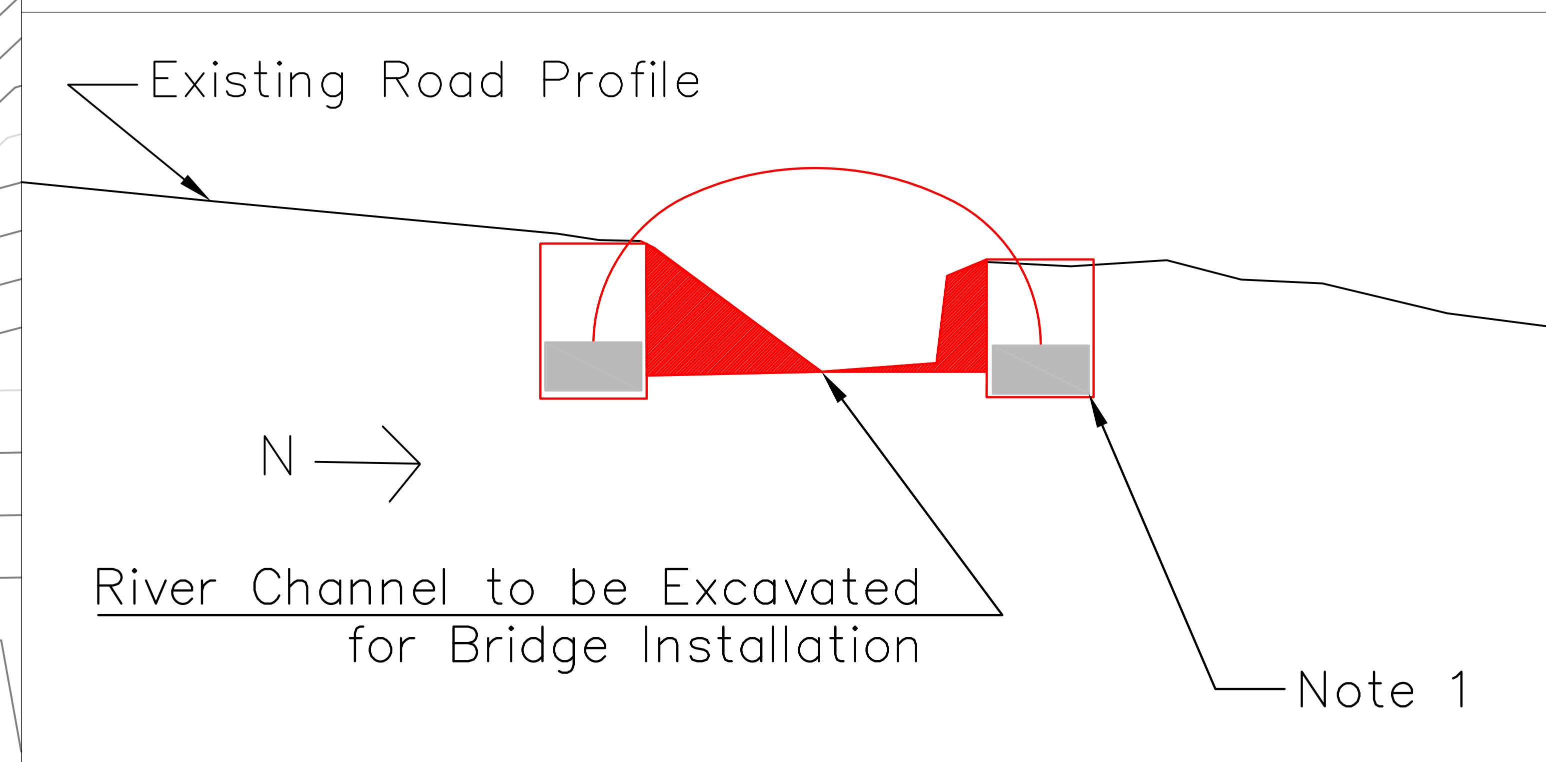
Note 1
15 in spacing with the anchor bolts and the connecting bolts on the base channel

PROJECT: LAS TRANCAS STREAM CROSSING

DRAWING: Footing Connection Details	ORGANIZATION: TRANCAS ASSOCIATES	Trancas Associates
PLOT DATE: 12/8/2016	SCALE: N/A	
FILENAME:sitesketchCivil3Da.dwg	SHEET: 5 of 18	J–5







Construction Items

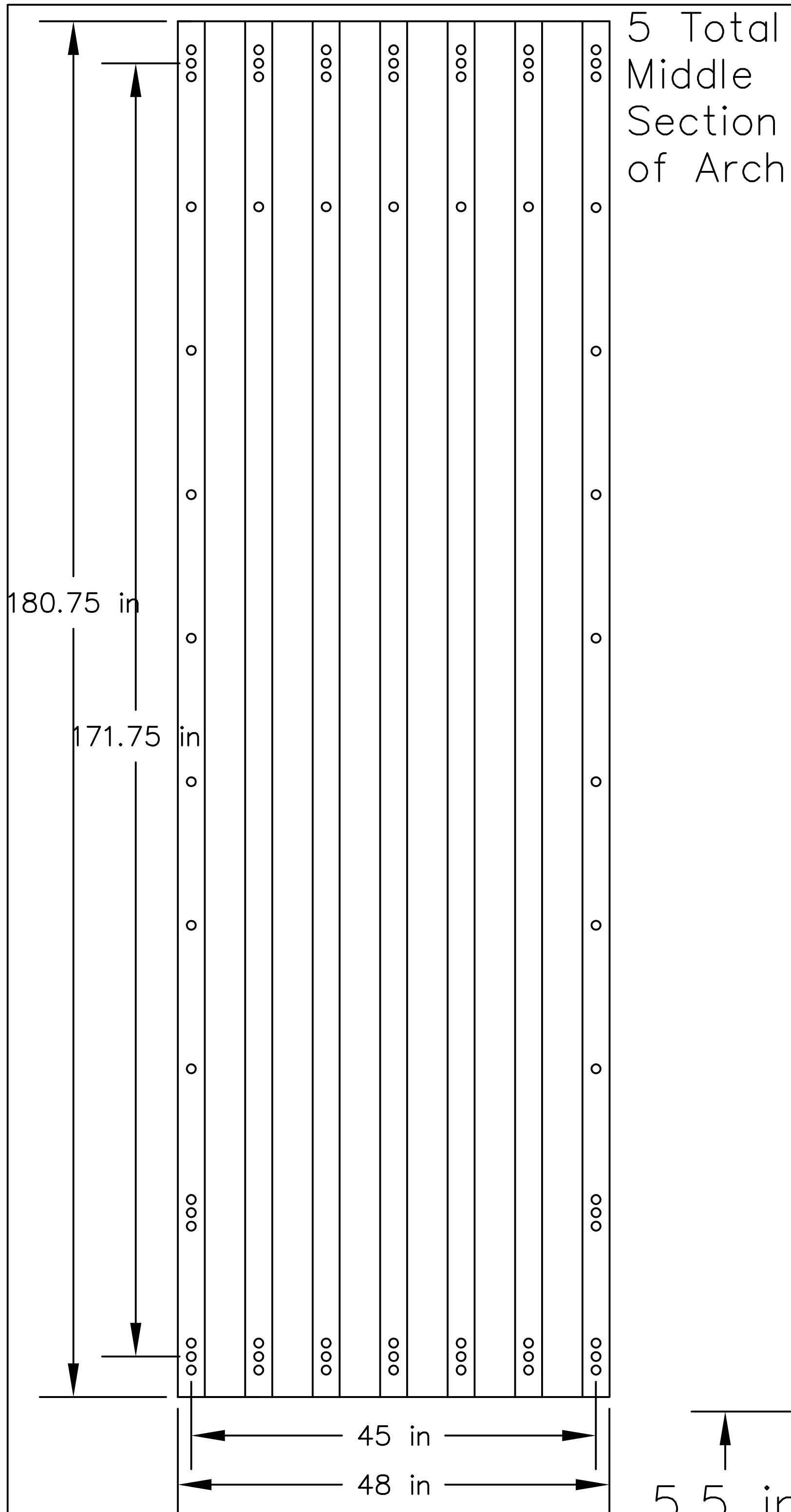
Channel Excavation	77 cyd
Falsework (2" x 4")	189 ft
Screws	5 lb

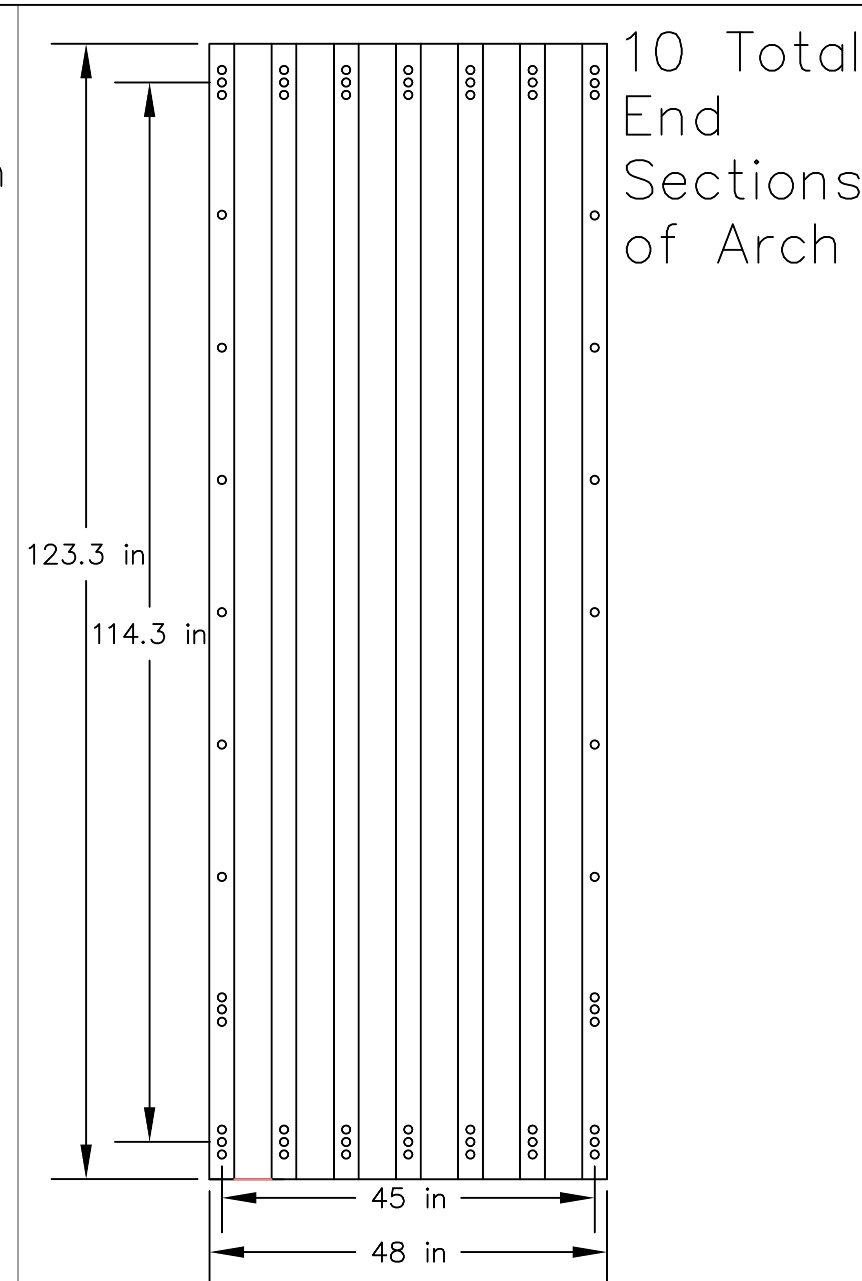
Note 1

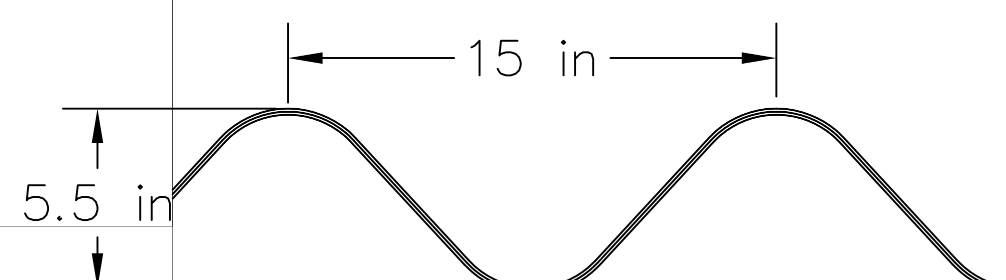
Footings to be a depth of about 7.5 ft.
Final top elevation grades should be equal

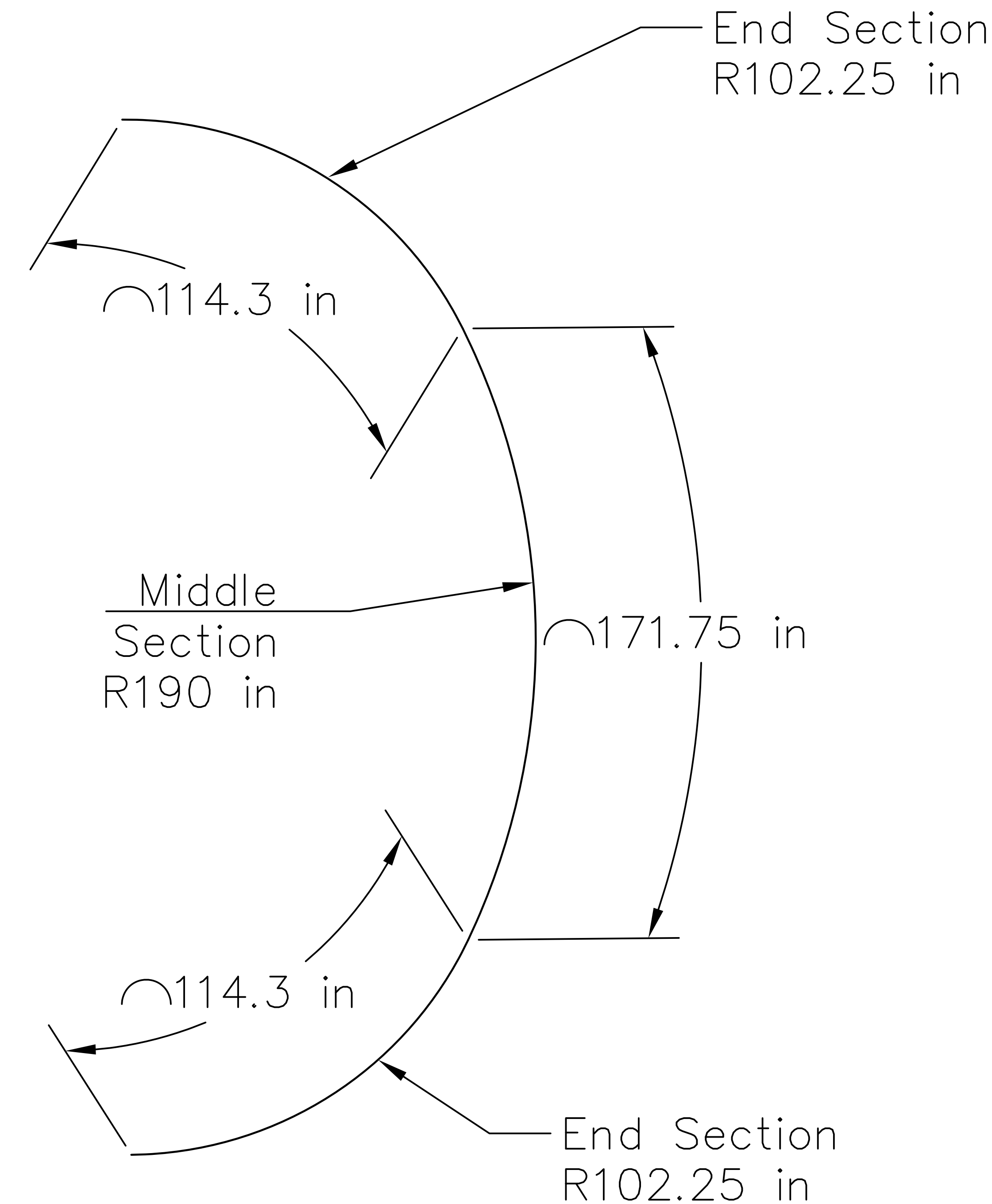
PROJECT: LAS TRANCAS STREAM CROSSING

DRAWING: Crown Plate Placement	ORGANIZATION: TRANCAS ASSOCIATES	Trancas Associates
PLOT DATE: 12/8/2016	SCALE: 1" = 10'	
FILENAME:sitesketchCivil3Da.dwg	SHEET: 6 of 18	J-6









Crown Plate Dimensions

15 in x 5.5 in Corrugation
Gauge 7 (0.188 in) Thickness
Plates shall conform to ASTM A761
3 oz per square foot of galvanized coating
This plates have a 1 $\frac{1}{2}$ in lip in order for the plates to over lap when connecting together and $\frac{13}{16}$ in holes

Construction Items	
15 in x 5.5 in Corrugated Metal Plate	7075 lb

PROJECT: LAS TRANCAS STREAM CROSSING

DRAWING: Crown Plate	ORGANIZATION: TRANCAS ASSOCIATES	Trancas Associates
PLOT DATE: 12/8/2016	SCALE: N/A	
FILENAME:sitesketchCivil3Da.dwg	SHEET: 7 of 18	J-7

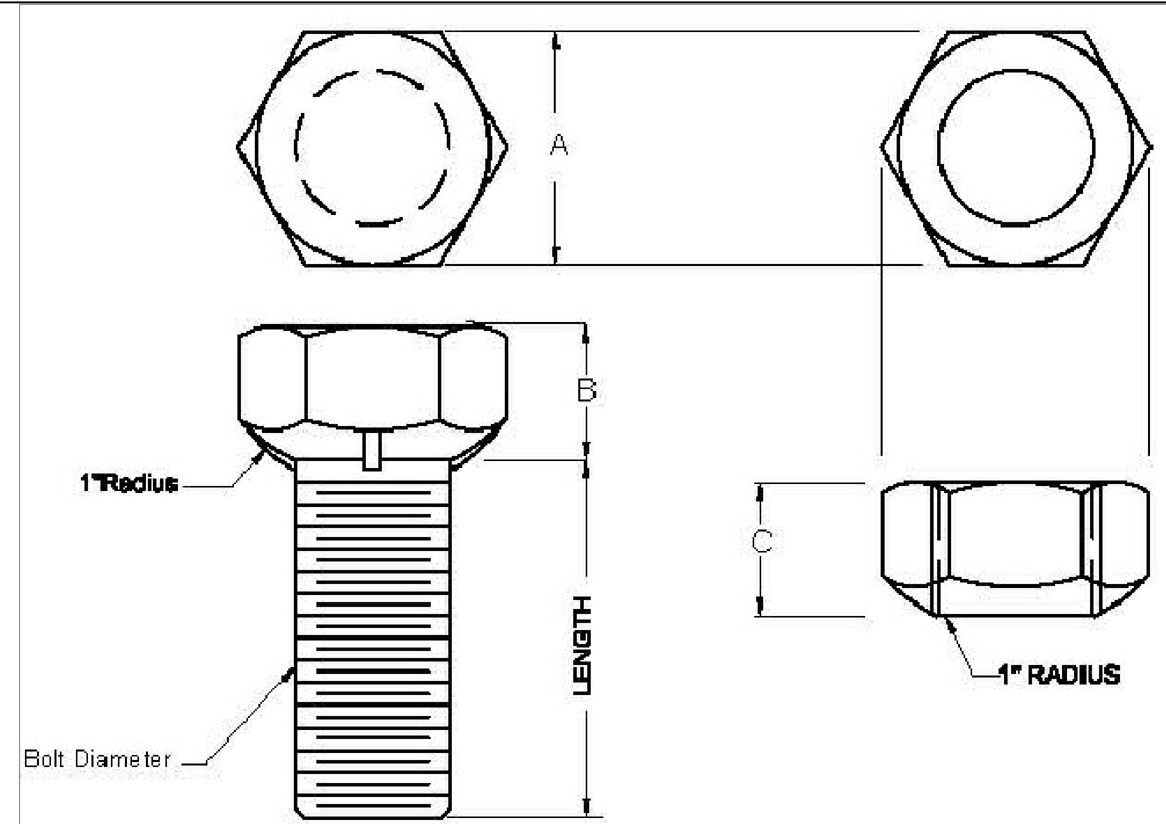
Bolting

If the plates are well aligned, the torque applied with a power wrench need not be excessive. Bolts should be torque initially to a minimum 100 foot pounds and a maximum 300 foot pounds. A good plate fit is far better than high torque.

Hot-dipped galvanized, specially heat-treated $\frac{3}{4}$ in diameter steel bolts, meeting ASTM A449 specifications, are typically used to assemble structural plate sections. The underside of the bolt head is uniformly rounded and does not require special positioning.

In addition, the underside of the bolt head is ribbed to prevent bolt head rotation while tightening. Unlike conventional bolts, once the nut is finger tight, final tightening can usually be accomplished by one worker.

The longer connecting bolts (2" Long) are to be used when the 3 plates are to be connected.



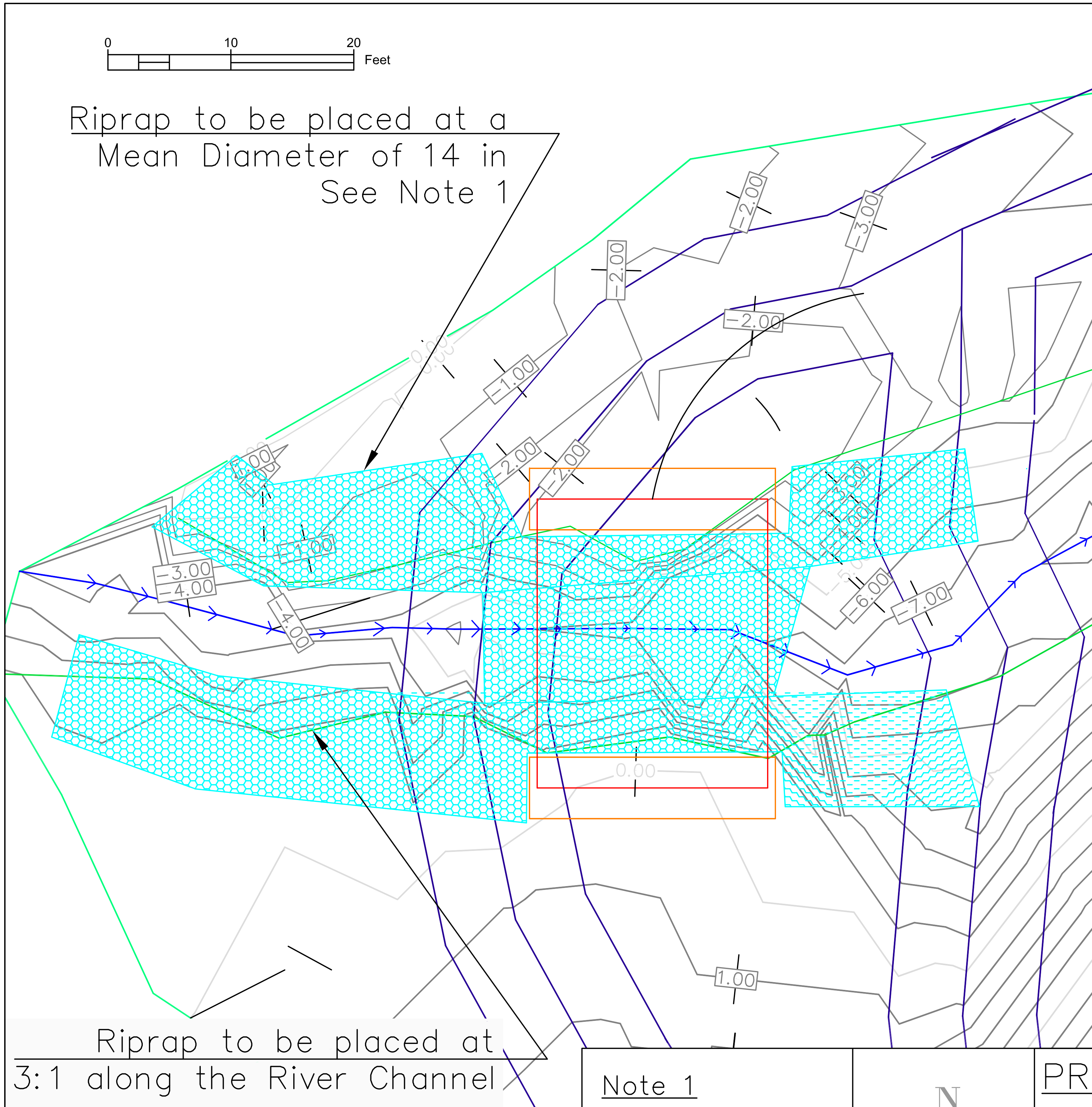
Typical Bolt and Nut

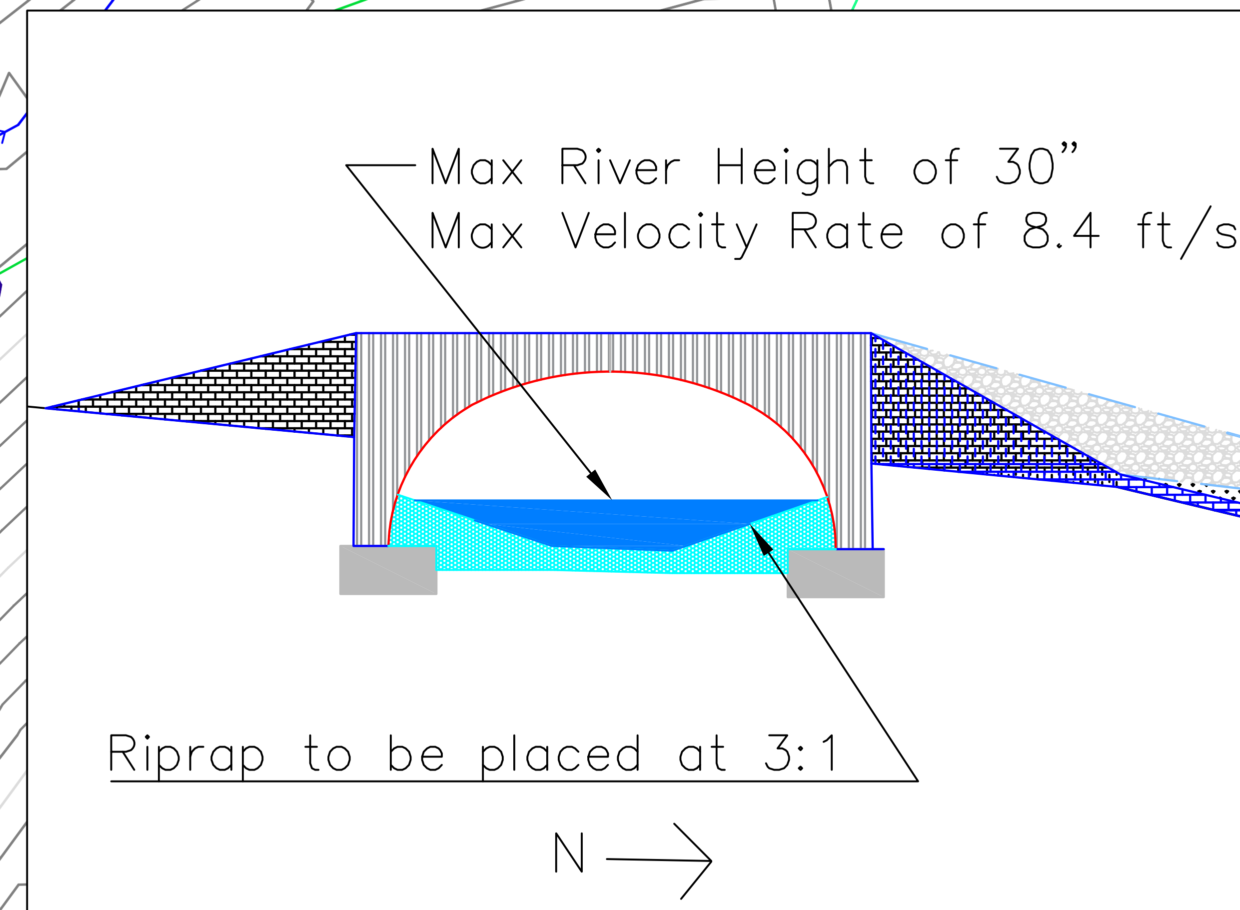
Diameter (Inches)	A (Inches)	B (Inches)	C (Inches)
3/4	1 1/4	5/8	13/16

Construction Items	
Connecting Bolts (2" Long)	90
Connecting Bolts (1.5" Long)	150

PROJECT: LAS TRANCAS STREAM CROSSING

DRAWING: Crown Plate Connections	ORGANIZATION: TRANCAS ASSOCIATES	Trancas Associates
PLOT DATE: 12/8/2016	SCALE: N/A	
FILENAME:sitesketchCivil3Da.dwg	SHEET: 8 of 18	J-8





Construction Items	
Riprap	90 ton
Riprap Placing	175 syd
Silt Fence	73 Ft

Note 1

Mean diameter of 14 in means 50% of the riprap is to be less than 14 in and 50% of the riprap is to be greater than 14 in

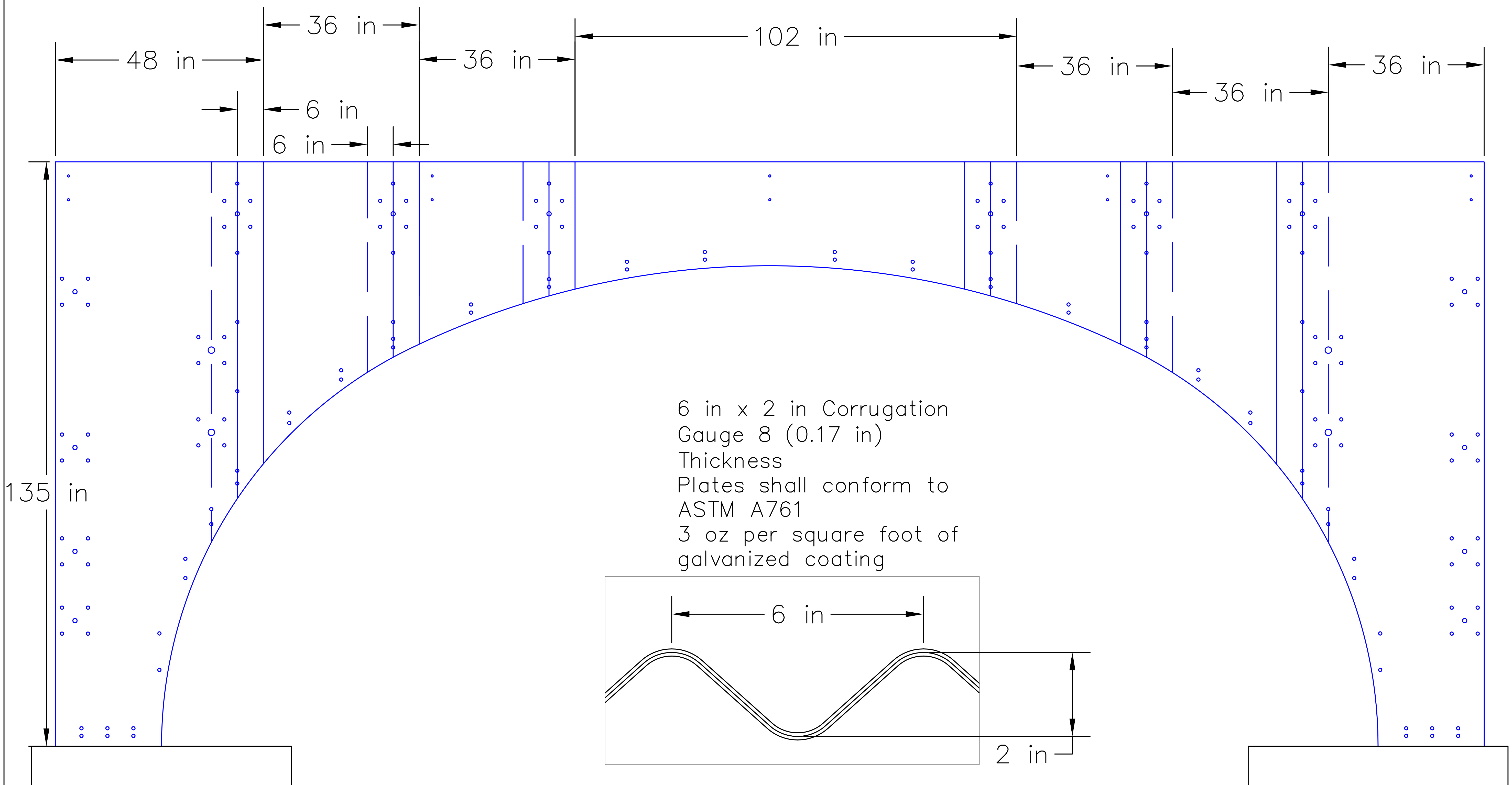
N

PROJECT: LAS TRANCAS STREAM CROSSING

DRAWING: Riprap Placement	ORGANIZATION: TRANCAS ASSOCIATES
PLOT DATE: 12/8/2016	SCALE: 1" = 10'
FILENAME:sitesketchCivil3Da.dwg	SHEET: 9 of 18

Trancas Associates

J-9



Construction Items

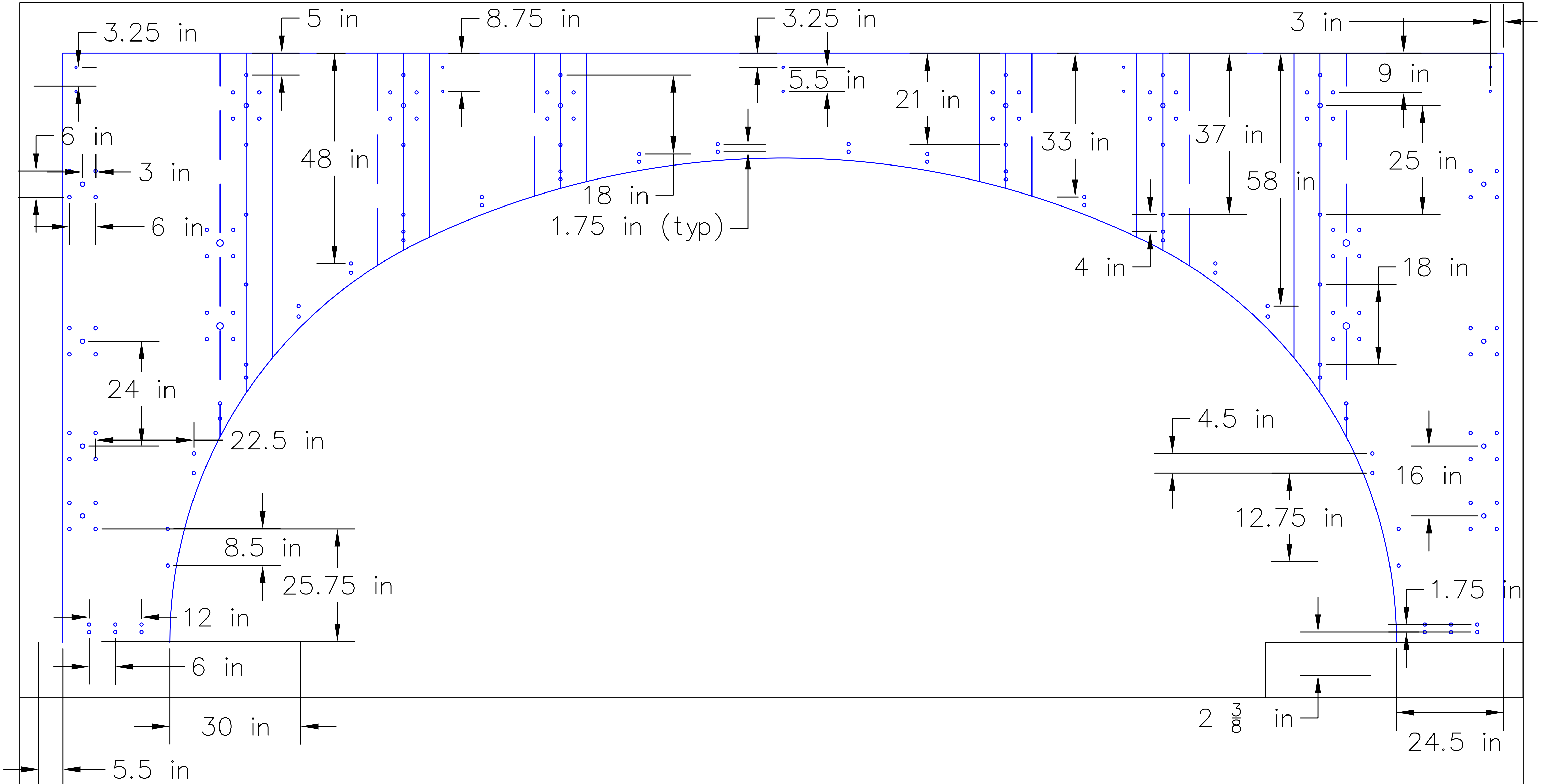
6 in x 2 in Corrugated Metal Plate	1632 lbs

These plates have a 1 1/2 in lip in order for the plates to overlap when connecting together

Holes to be 13/16 in

PROJECT: LAS TRANCAS STREAM CROSSING

DRAWING: Headwall Lengths	ORGANIZATION: TRANCAS ASSOCIATES	Trancas Associates
PLOT DATE: 12/8/2016	SCALE: N/A	
FILENAME:sitesketchCivil3Da.dwg	SHEET: 10 of 18	J-10



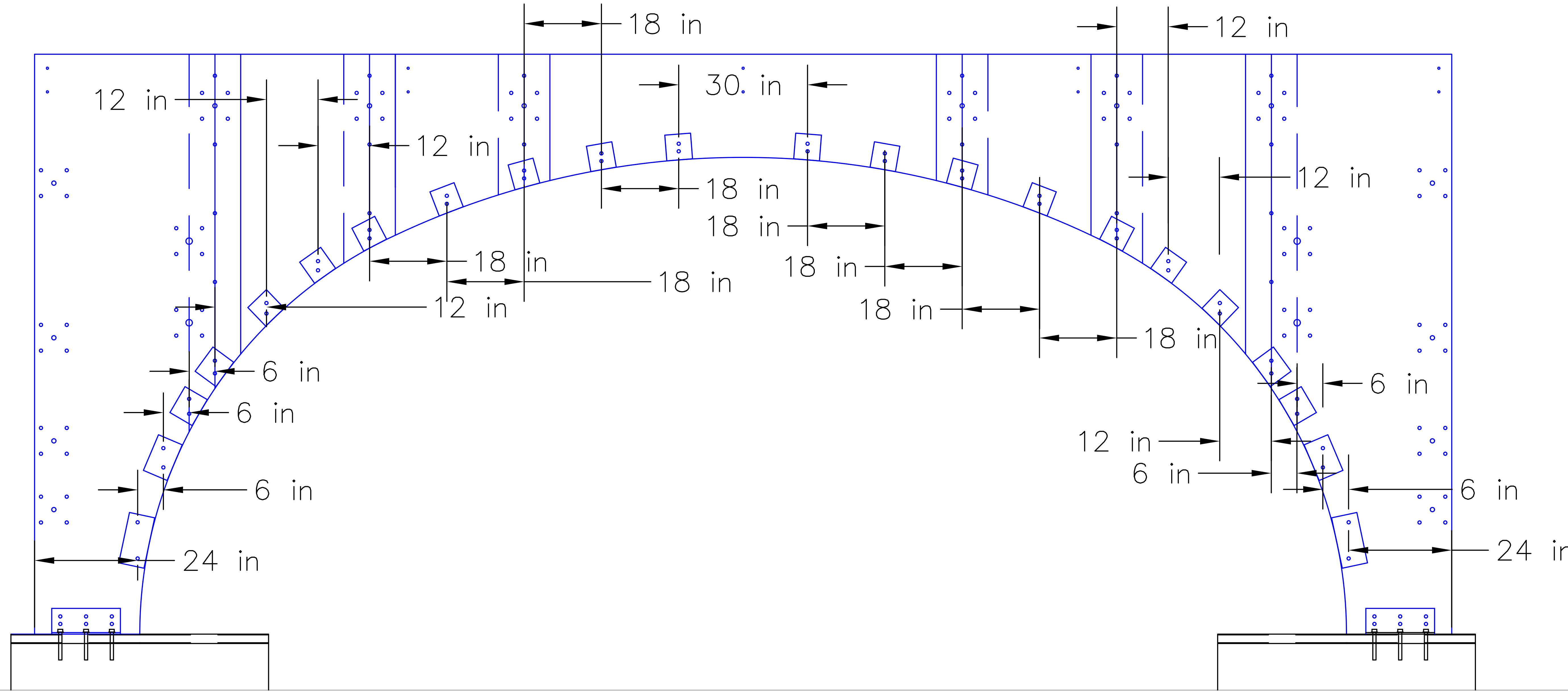
Construction Items	
Connecting Bolts ($\frac{3}{4}$ in Dia, 1.5")	36 Each

All plate holes are $\frac{13}{16}$ in

Holes to be punched during fabrication

PROJECT: LAS TRANCAS STREAM CROSSING

DRAWING: Headwall Bolt Holes	ORGANIZATION: TRANCAS ASSOCIATES	Trancas Associates
PLOT DATE: 12/8/2016	SCALE: N/A	
FILENAME:sitesketchCivil3Da.dwg	SHEET: 11 of 18	J-11

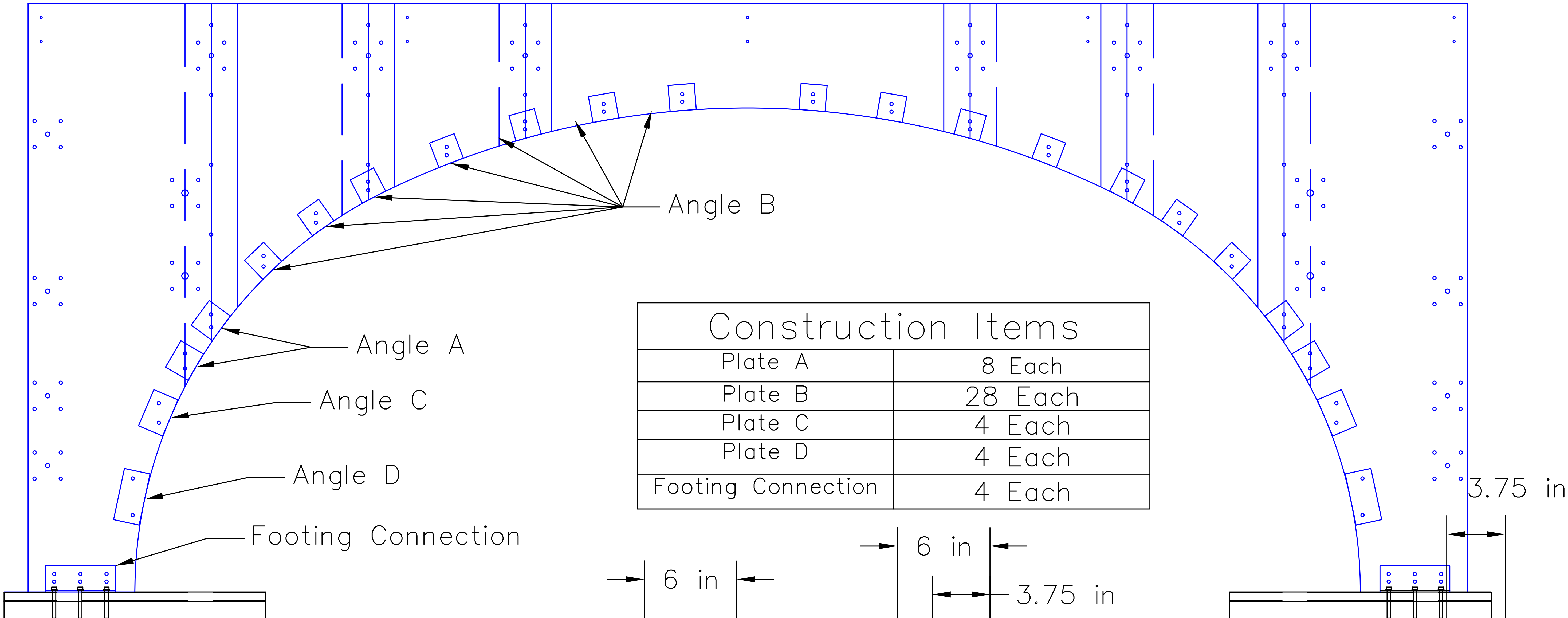


Construction Items	
Connecting Bolts (Dia $\frac{3}{4}$ in, 2" Length)	112 Each
Connecting Bolts (Dia $\frac{3}{4}$ in, 1.5" Length)	88 Each
Anchor Bolts (Dia $\frac{3}{4}$ in, 6" Length)	12 Each

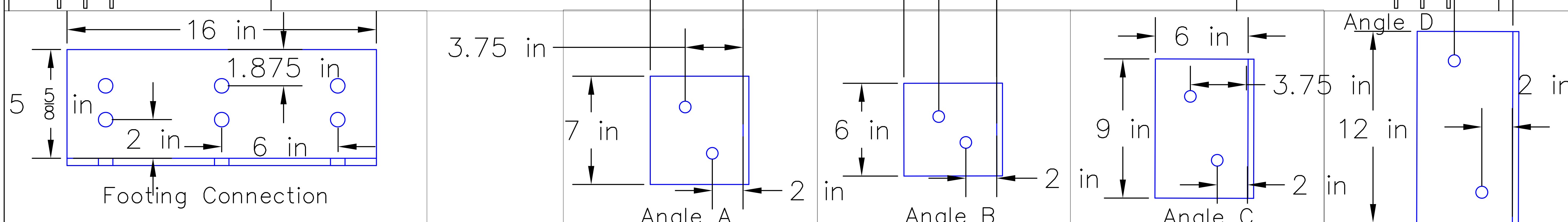
Angles thickness of $\frac{3}{8}$ in
Plate holes of $\frac{13}{16}$ in
All plates are A36 Grade

PROJECT: LAS TRANCAS STREAM CROSSING

DRAWING: Connections Spacing	ORGANIZATION: TRANCAS ASSOCIATES	Trancas Associates	
PLOT DATE: 12/8/2016	SCALE: N/A		
FILENAME: sitesketchCivil3Da.dwg	SHEET: 12 of 18	J-12	



Construction Items	
Plate A	8 Each
Plate B	28 Each
Plate C	4 Each
Plate D	4 Each
Footing Connection	4 Each

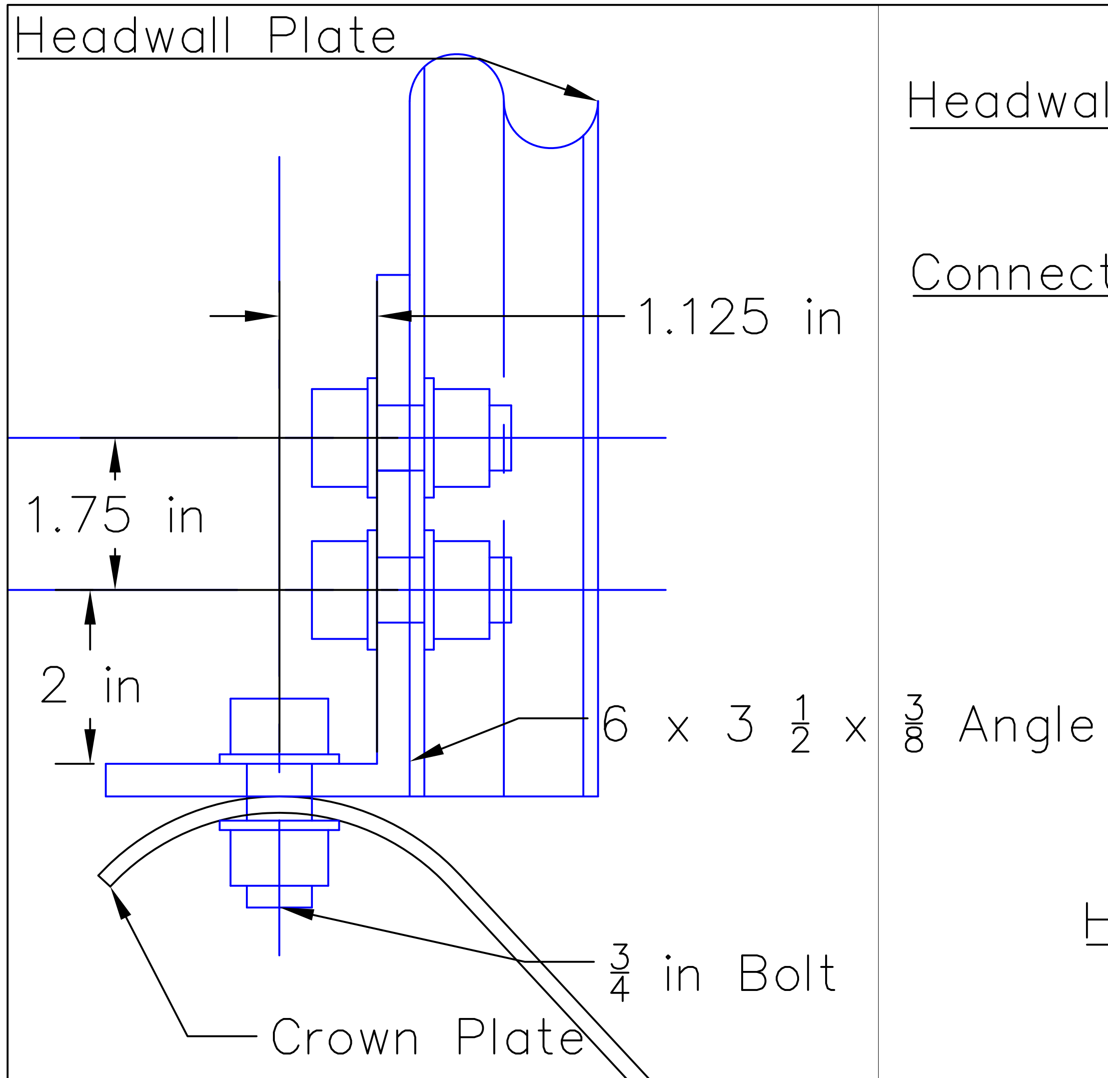


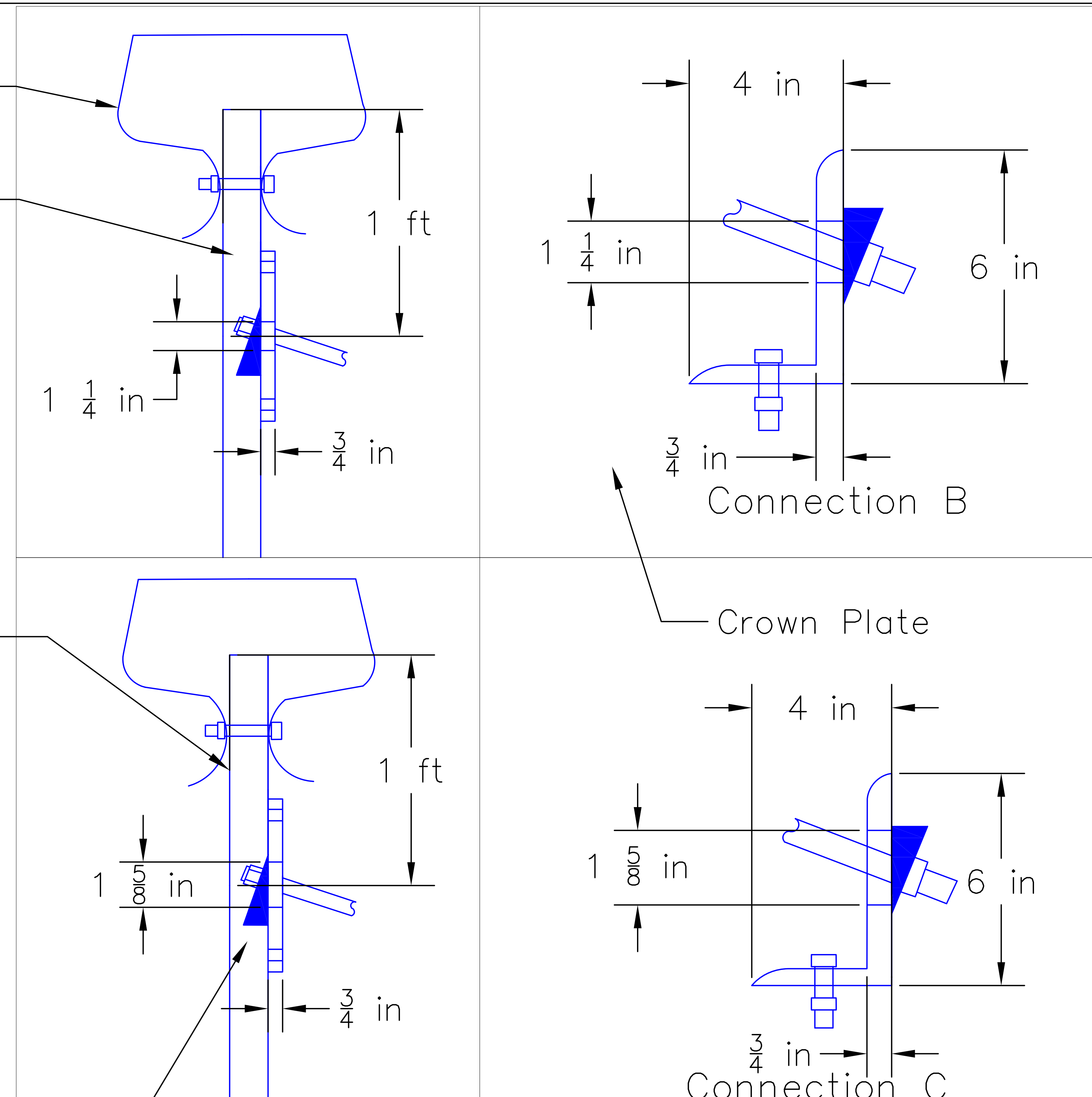
Construction Items	
Steel Plates	23.5 sft

The depth of all the angles are 3.5 in
Depths are to connect to the crown plate underneath

PROJECT: LAS TRANCAS STREAM CROSSING

DRAWING: Angle Connections	ORGANIZATION: TRANCAS ASSOCIATES	Trancas Associates
PLOT DATE: 12/8/2016	SCALE: N/A	
FILENAME:sitesketchCivil3Da.dwg	SHEET: 13 of 18	J-13





Connection C

Connection from Headwall to Crown Plate

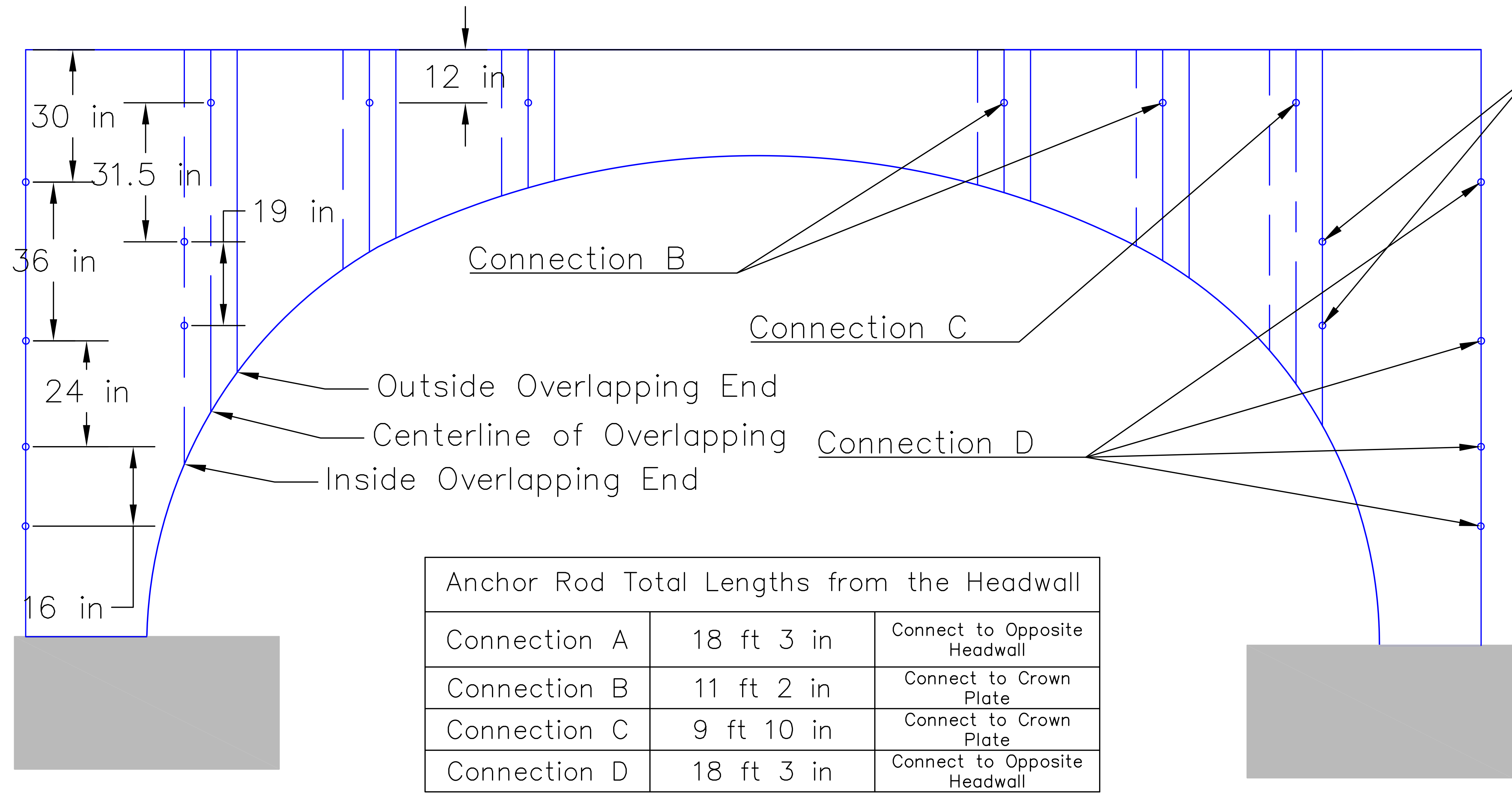
Angle Steel Plates	
Connection B	12 Each
Connection C	8 Each

Steel Plates to be A36 Grade

Construction Items	
Steel Plates	20.25 sft
Angle Steel Plates	2 sft
Headwall Cap	25 Ft
Bevel Washers (Dia ¾ in)	484 Each

Bevel Washers are to be F436 Grade

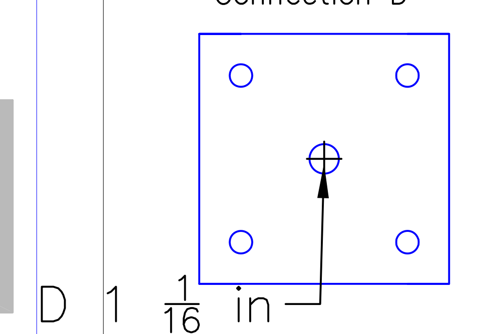
PROJECT: LAS TRANCAS STREAM CROSSING		
DRAWING: Headwall Connections	ORGANIZATION: TRANCAS ASSOCIATES	
PLOT DATE: 12/8/2016	SCALE: N/A	
FILENAME: sitesketchCivil3Da.dwg	SHEET: 14 of 18	J–14



Anchor Rod Total Lengths from the Headwall		
Connection A	18 ft 3 in	Connect to Opposite Headwall
Connection B	11 ft 2 in	Connect to Crown Plate
Connection C	9 ft 10 in	Connect to Crown Plate
Connection D	18 ft 3 in	Connect to Opposite Headwall

Connection Plates	
Connection A	8 Each
Connection B	8 Each
Connection C	4 Each
Connection D	16 Each

Connection D

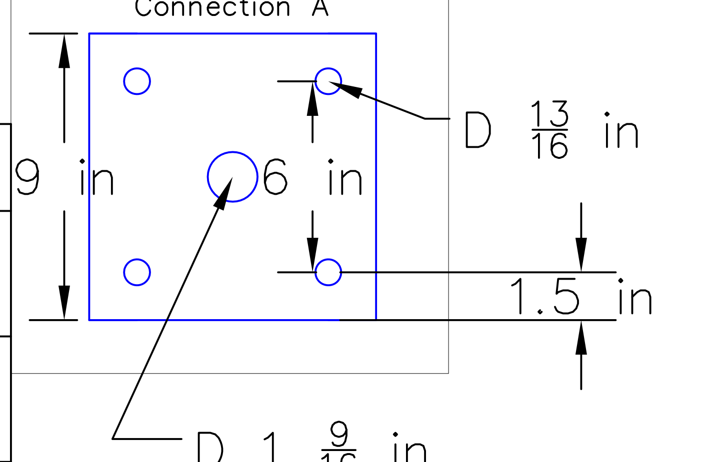


Steel Anchor Rods to be A992 Grade

Construction Items

Connecting Bolts (1.5" Long)	72 Each
Anchor Rods (1.5" Diameter)	150 Ft
Anchor Rods (1" Diameter)	420 Ft
Steel Plates	20.25 sft

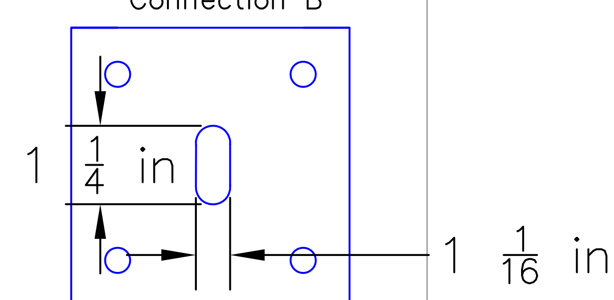
Connection A



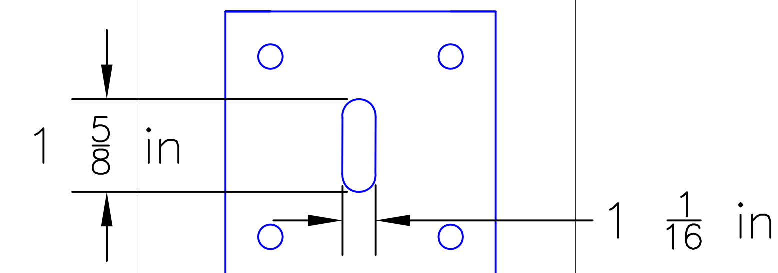
Typical Dimensions for all Plates

Steel Plates to be A36 Grade

Connection B

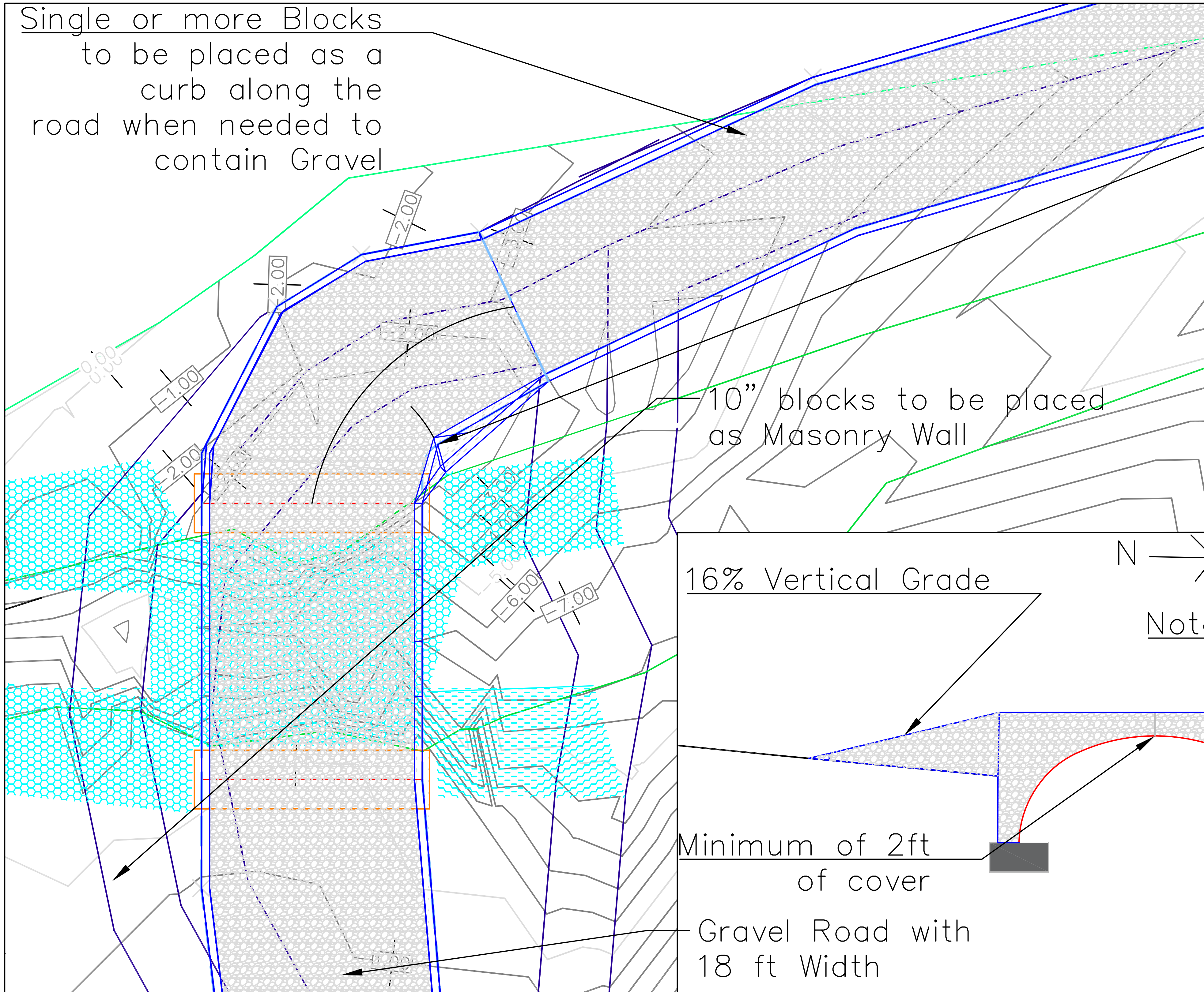


Connection C



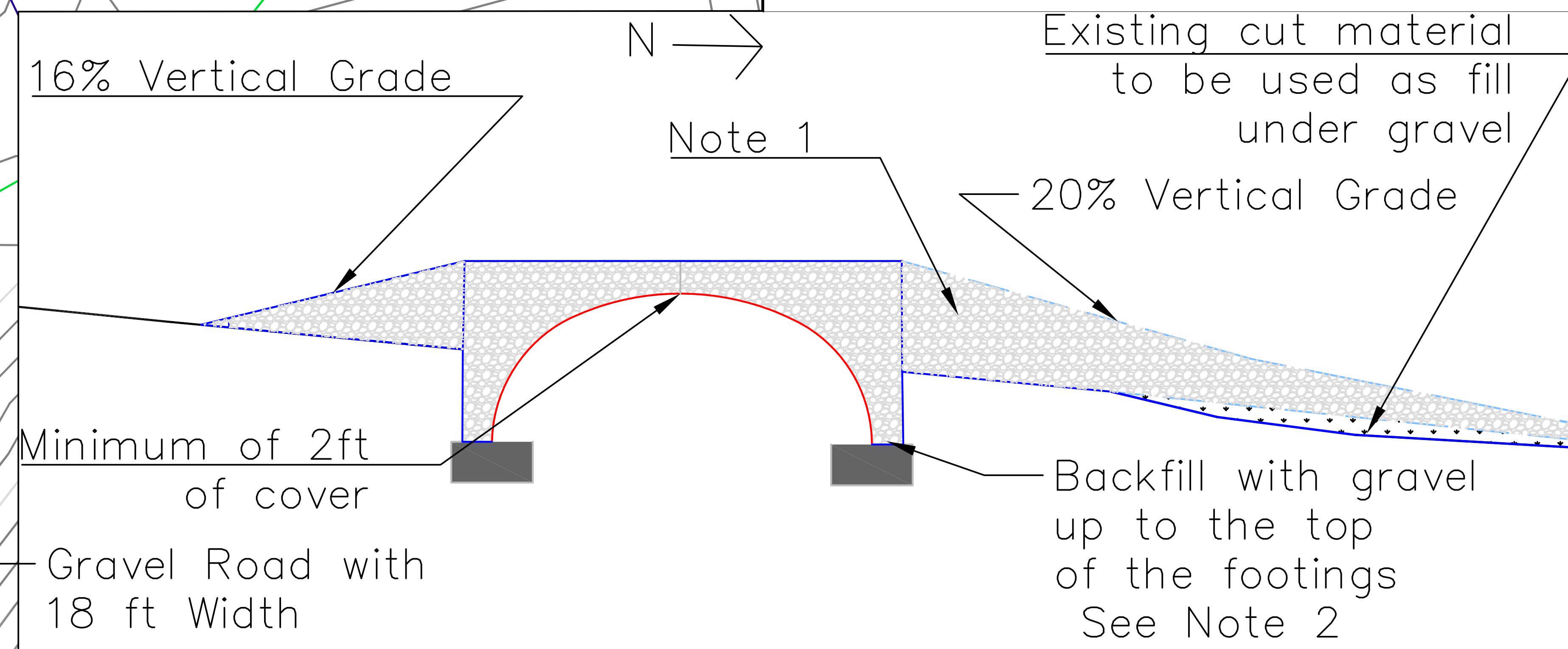
PROJECT: LAS TRANCAS STREAM CROSSING

DRAWING: Anchor Rods Dimensions	ORGANIZATION: TRANCAS ASSOCIATES	Trancas Associates
PLOT DATE: 12/8/2016	SCALE: N/A	
FILENAME:sitesketchCivil3Da.dwg	SHEET: 15 of 18	J–15



Note 1

1" crushed, angular gravel stone (AASHTO M145, A-1-a) to be placed on fill along the headwall and masonry wall. The fill is to be compacted at a minimum of 90% and in lifts of 6"-8"



Construction Items	
Masonry Wall	8 cyd
Motar	5 sacks
Gravel (GW)	14 ton

Note 2

The backfill of the gravel should coincide with the anchor rod and plate connections

PROJECT: LAS TRANCAS STREAM CROSSING

DRAWING: Gravel Road Placement

ORGANIZATION: TRANCAS ASSOCIATES

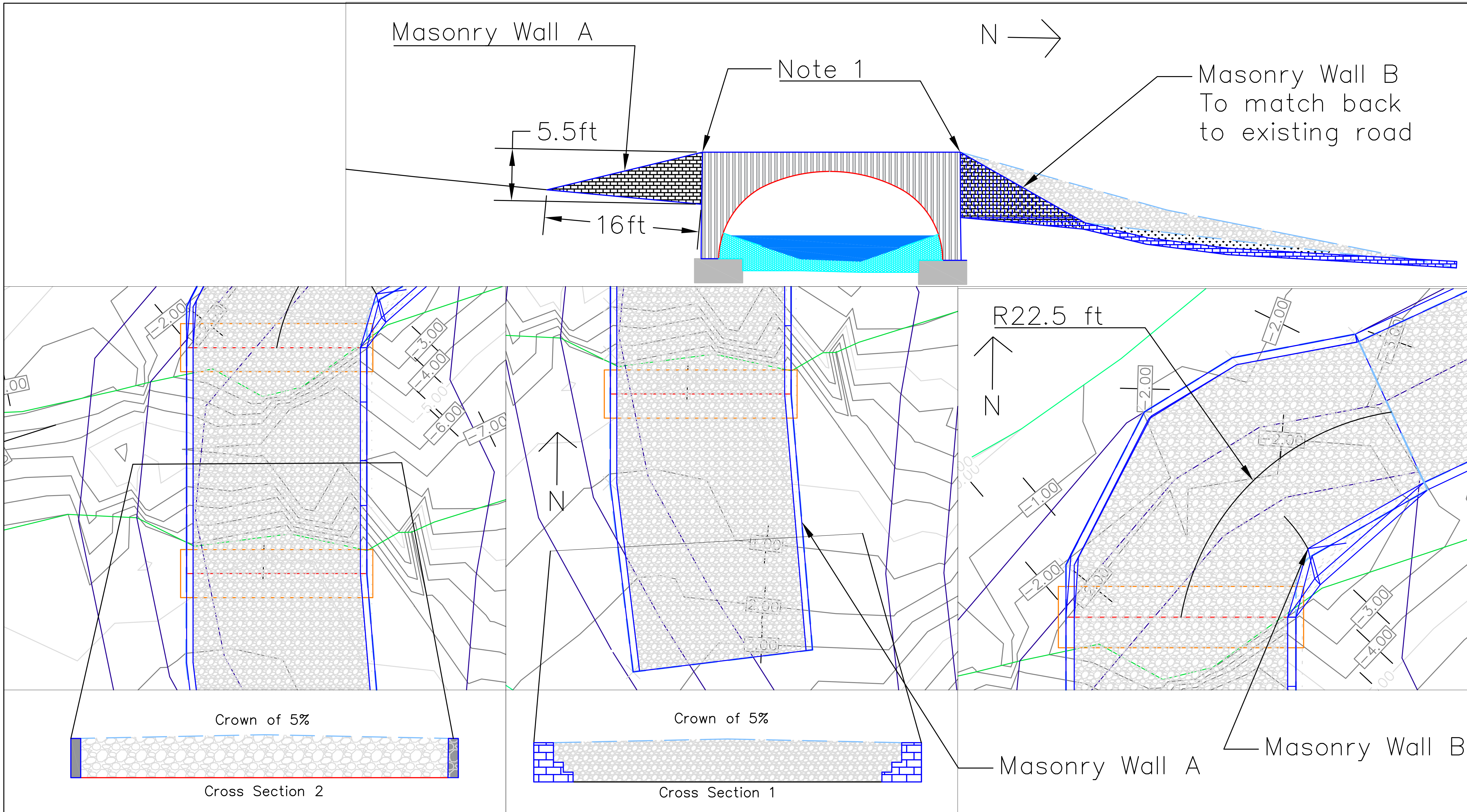
PLOT DATE: 12/8/2016

SCALE: 1" = 10'

FILENAME:sitesketchCivil3Da.dwg

SHEET: 16 of 18

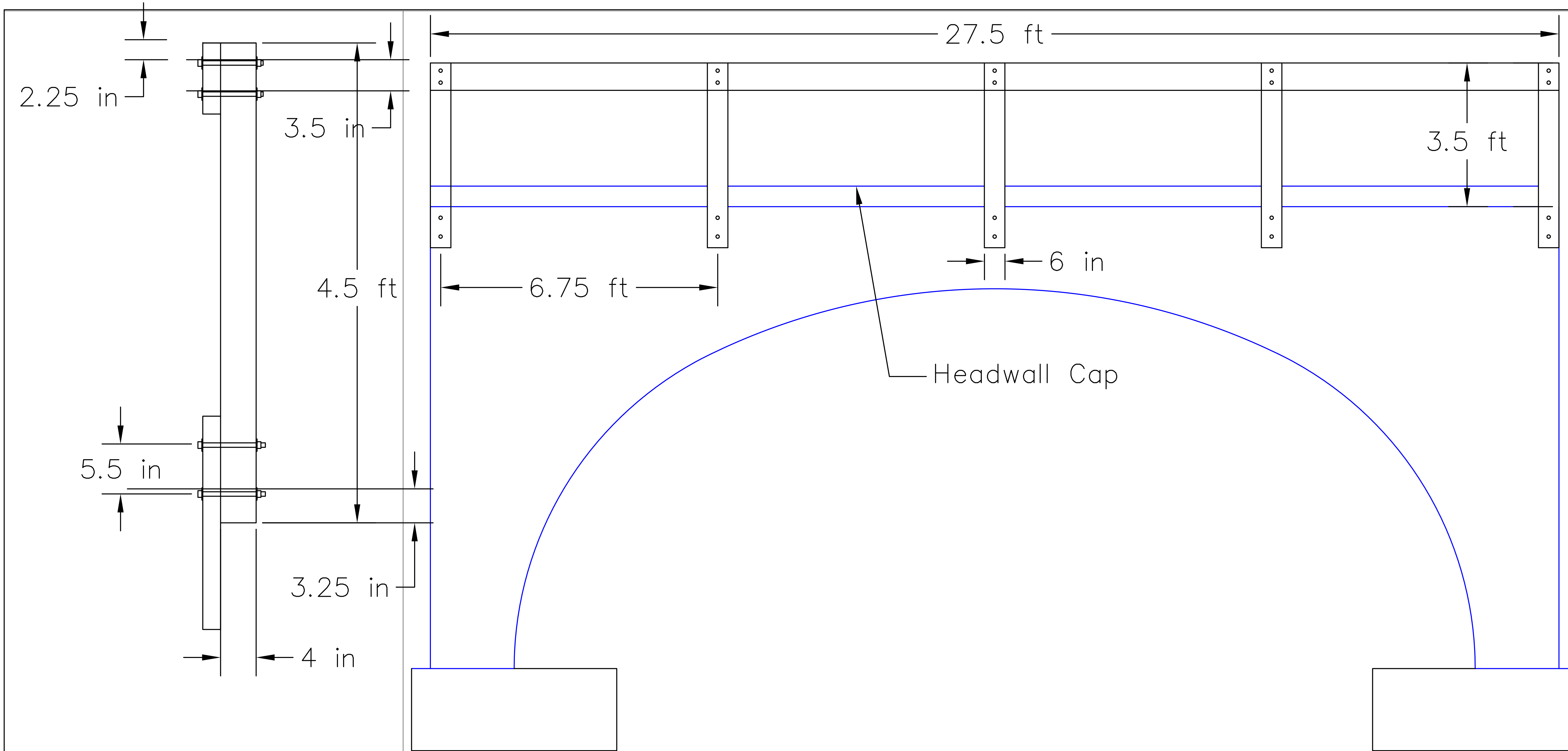
J-16



Note 1

Fiber expansion joints to be placed in between the masonry wall and steel headwall. To be mortared in place.

PROJECT: LAS TRANCAS STREAM CROSSING	
DRAWING: Bridge Cross Section	ORGANIZATION: TRANCAS ASSOCIATES
PLOT DATE: 12/8/2016	SCALE: 1" = 10'
FILENAME: sitesketchCivil3Da.dwg	SHEET: 17 of 18
	J-17



Construction Items

Connecting Bolts (7" Length)	40 Each
4" x 6" Green Treated Wood, SPF	45 ft
2" x 4" Green Treated Wood, SPF	55 ft

PROJECT: LAS TRANCAS STREAM CROSSING

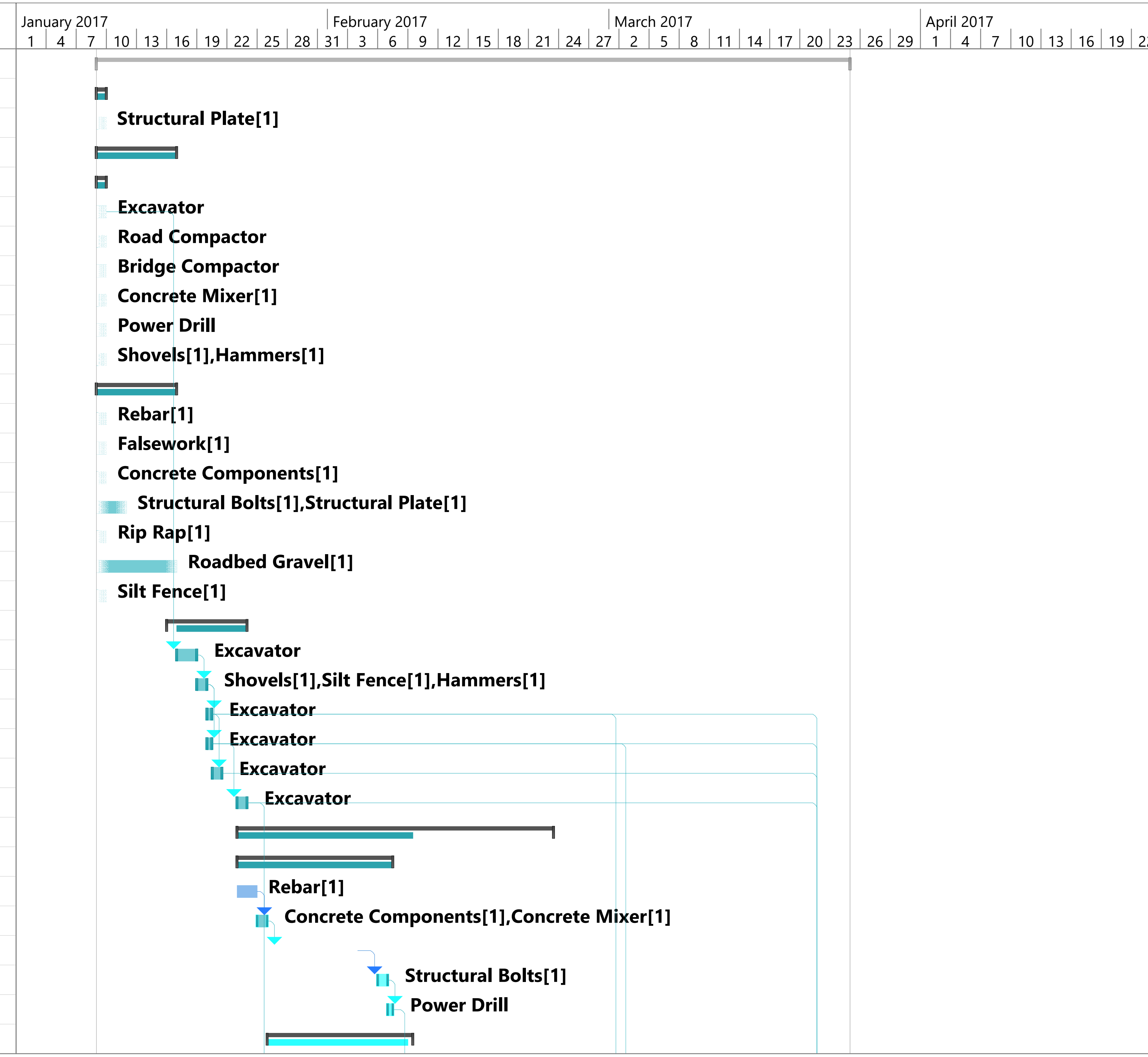
DRAWING: Guardrail	ORGANIZATION: TRANCAS ASSOCIATES	Trancas Associates
PLOT DATE: 12/8/2016	SCALE: N/A	
FILENAME:sitesketchCivil3Da.dwg	SHEET: 18 of 18	J–18

Appendix K:

Construction Schedule and Work Breakdown Structure

(Construction Schedule Attached)

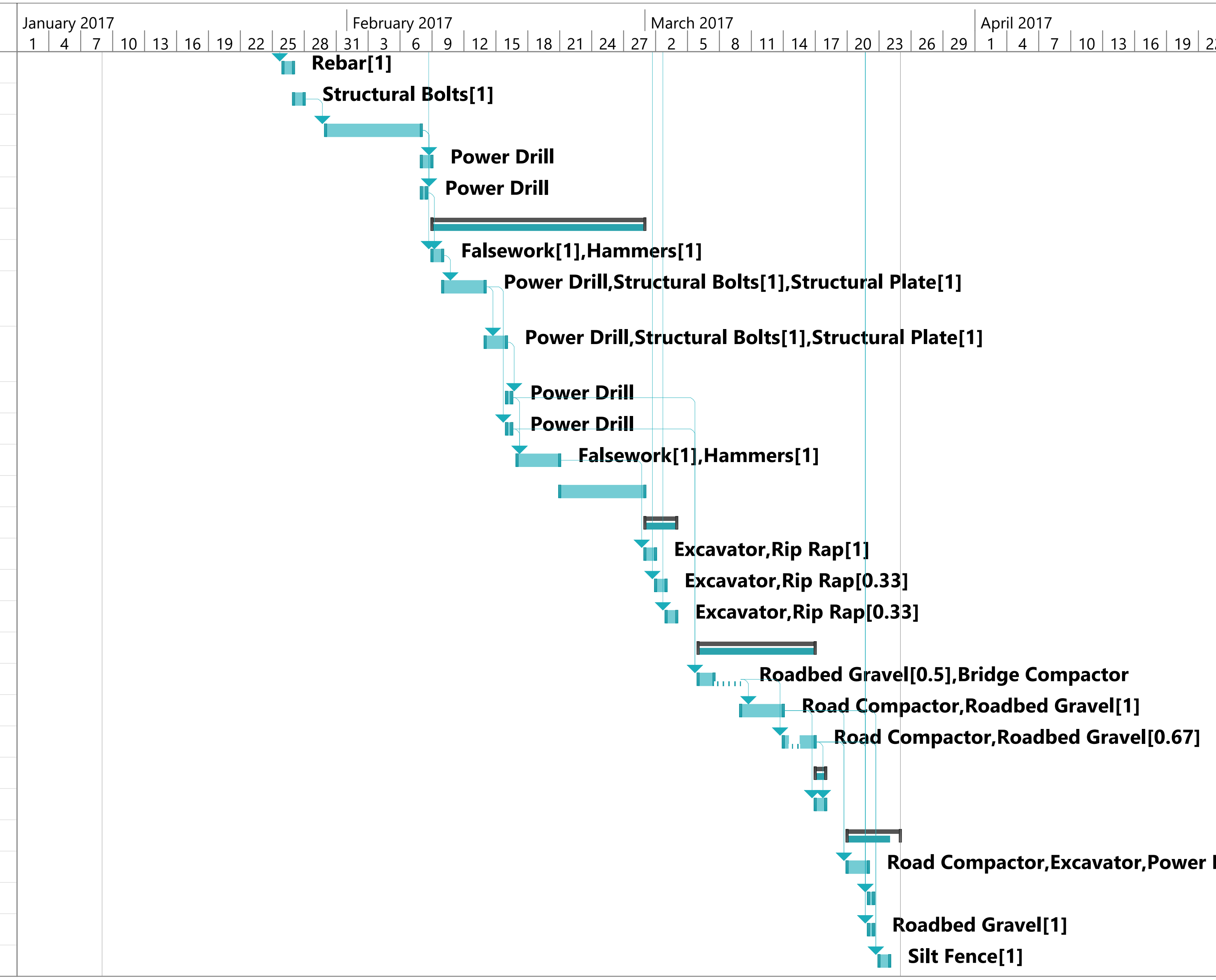
ID	Task Name	Duration	Start	Finish
0	**Project Schedule**	**55 days**	**Mon 1/9/17**	**Fri 3/24/17**
1	**Material Preparation**	**1 day**	**Mon 1/9/17**	**Mon 1/9/17**
2	Order structural plate	1 day		
3	**Mobilization**	**6 days**	**Mon 1/9/17**	**Mon 1/16/17**
4	**Equipment**	**1 day**	**Mon 1/9/17**	**Mon 1/9/17**
5	Bring in excavator	1 day		
6	Bring in road compactor	1 day		
7	Bring in bridge compactor	1 day		
8	Bring in concrete mixer	1 day		
9	Bring in power drill	1 day		
10	Bring in miscallaneous equiptment	1 day		
11	**Materials**	**6 days**	**Mon 1/9/17**	**Mon 1/16/17**
12	Bring in rebar	1 day		
13	Bring in falsework	1 day		
14	Bring in concrete components	1 day		
15	Bring in steel plate & bolts	3 days		
16	Bring in rip rap	1 day		
17	Bring in roadbed gravel	6 days		
18	Bring in silt fence	1 day		
19	**Site Preparation**	**6 days**	**Mon 1/16/17**	**Mon 1/23/17**
20	Remove old bridge components from channel	2 days	Tue 1/17/17	Wed 1/18/17
21	Place silt fences around river channel	1 day	Thu 1/19/17	Thu 1/19/17
22	Excavate and slope north bank walls	0.5 days	Fri 1/20/17	Fri 1/20/17
23	Excavate and slope south bank walls	0.5 days	Fri 1/20/17	Fri 1/20/17
24	Excavate north foundation base	1 day	Fri 1/20/17	Sat 1/21/17
25	Excavate south foundation base	1 day	Mon 1/23/17	Mon 1/23/17
26	**Footings**	**23.5 days**	**Mon 1/23/17**	**Thu 2/23/17**
27	**North Side**	**11.5 days**	**Mon 1/23/17**	**Tue 2/7/17**
28	Position rebar	2 days	Mon 1/23/17	Tue 1/24/17
29	Mix and pour concrete	1 day	Wed 1/25/17	Wed 1/25/17
30	Allow concrete to cure	7 days	Thu 1/26/17	Fri 2/3/17
31	Position anchor bolts for structure & epoxy	1 day	Mon 2/6/17	Mon 2/6/17
32	Attach steel plate connector panel	0.5 days	Tue 2/7/17	Tue 2/7/17
33	**South Side**	**10.5 days**	**Thu 1/26/17**	**Thu 2/9/17**



Las Trancas Bridge Project

Task	Inactive Task	Duration-only	Finish-only	Progress
Split	Inactive Milestone	Manual Summary Rollup	External Tasks	Manual Progress
Summary	Inactive Summary	Manual Summary	Critical	
Project Summary	Manual Task	Start-only	Critical Split	

ID	Task Name	Duration	Start	Finish
34	Position rebar	1 day	Thu 1/26/17	Thu 1/26/17
35	Mix and pour concrete	1 day	Fri 1/27/17	Fri 1/27/17
36	Allow concrete to cure	7 days	Mon 1/30/17	Tue 2/7/17
37	Position anchor bolts for structure & epoxy	1 day	Wed 2/8/17	Wed 2/8/17
38	Attach steel plate connector	0.5 days	Wed 2/8/17	Wed 2/8/17
39	**Steel Plate Assembly**	**14 days**	**Thu 2/9/17**	**Tue 2/28/17**
40	Install falsework for initial support	1 day	Thu 2/9/17	Thu 2/9/17
41	Use excavator to position bottom plates and connect to anchor bolts	2 days	Fri 2/10/17	Mon 2/13/17
42	Use excavator to position top plates and connect to bottom plates	2 days	Tue 2/14/17	Wed 2/15/17
43	Tighten connections between plates	0.5 days	Thu 2/16/17	Thu 2/16/17
44	Tighten connections to anchor bolts	0.5 days	Thu 2/16/17	Thu 2/16/17
45	Remove false work	2 days	Fri 2/17/17	Mon 2/20/17
46	Headwall Plate Assembly	6 days	Tue 2/21/17	Tue 2/28/17
47	**Rip Rap Backfilling**	**3 days**	**Wed 3/1/17**	**Fri 3/3/17**
48	Place rip rap over foundations	1 day	Wed 3/1/17	Wed 3/1/17
49	Place rip rap on north bank wall	1 day	Thu 3/2/17	Thu 3/2/17
50	Place rip rap on south bank wall	1 day	Fri 3/3/17	Fri 3/3/17
51	**Roadbed Creation**	**9 days**	**Mon 3/6/17**	**Thu 3/16/17**
52	Fill, grade, and compact roadbed over structure	1.5 days	Mon 3/6/17	Thu 3/9/17
53	Fill, grade, and compact south roadbed	2 days	Fri 3/10/17	Mon 3/13/17
54	Fill, grade, and compact north roadbed	2 days	Tue 3/14/17	Thu 3/16/17
55	**Site Repair**	**1 day**	**Fri 3/17/17**	**Fri 3/17/17**
56	Place grass seed down over new exposed areas	1 day	Fri 3/17/17	Fri 3/17/17
57	**Cleanup / Demobilization**	**5 days**	**Mon 3/20/17**	**Fri 3/24/17**
58	Remove equiptment from site	2 days	Mon 3/20/17	Tue 3/21/17
59	Disperse clay cut	0.5 days	Wed 3/22/17	Wed 3/22/17
60	Disperse any excess fill	0.5 days	Wed 3/22/17	Wed 3/22/17
61	Remove silt fence	1 day	Thu 3/23/17	Thu 3/23/17



Task		Inactive Task		Duration-only		Finish-only		Progress	
Split		Inactive Milestone		Manual Summary Rollup		External Tasks		Manual Progress	
Summary		Inactive Summary		Manual Summary		Critical			
Project Summary		Manual Task		Start-only		Critical Split			

Trancas Associates

Las Trancas Bridge Project

Table K1. Key Project Tasks

No.	Task
1	Material Preparation
2	Mobilization
3	Site Preparation
4	Footings
5	Steel Plate Assembly
6	Rip Rap Backfilling
7	Roadbed Creation
8	Site Repair
9	Cleanup / Demobilization

Tasks

1. Material Preparation

This task involves ordering the steel to be manufactured. As this steel requires specialized manufacturing, it was given its own task here. The contractor should order this plate early, prior to construction commencing.

2. Mobilization

This tasks involves mobilizing the equipment and material to the project site. The contractor should be responsible for bringing their own equipment, or renting out equipment in advance. Additionally, the contractor should arrange for the material to be ordered and brought to the site in prior to the date it is required for construction to continue.

3. Site Preparation

This task involves preparing the river channel for the installation of a new structure. The past bridge components must be removed, the north and south bank walls must be cut for placing rip rap, and the north and south foundation sites must be excavated. This work is to be done with the mini excavator.

4. Footings

On either side of the channel, footings must be placed. Placing forms will be skipped, and instead holes will be excavated and the surrounding soil will act as forms. Rebar must be placed as specified, concrete will be mixed on site and poured, the concrete will be allowed to cure, anchor bolt holes will be drilled, anchor bolts will be epoxied in, and a steel connector will be attached to the top of the footing.

5. Steel Plate Assembly

To place the steel plates, some falsework is required. The bottom plates on either side will be bolted onto the anchor bolts as specified. The mini excavator will be used to hoist the plates into place and then be supported by the falsework. Next, the top plates will be positioned to the bottom plate and connected. After everything has been connected, the contractor shall check and tighten down all the bolting across the entire structure to ensure proper connections. Next the contractor shall assemble the headwall on both sides of the buried bridge. This includes the specified connections needed and anchor rods installation. After the steel headwall is assembled, the masonry wall will be constructed. This includes placing concrete blocks with motar to act as the wingwall. A piece of 10 foot joint filler will be placed in between the headwall plate and the masonry wing walls. After this is all completed, the falsework will be removed.

6. Rip Rap Backfilling

The contractor will place rip rap over the foundations on the inside of the channel using the mini-excavator to the specified slope. The contractor will also place rip rap on the north and south bank walls at the specified slopes.

7. Roadbed Creation

The contractor would first hand-grade the roadbed gravel using the mini-excavator over the structural steel plate arch. They would follow the specified grades and keep a 3.5 ft minimum depth between the plate and the top of the road. They would place 6” - 8” lifts for the gravel and use the hand compactor to compact it. The contractor will also create the south roadbed with the same compaction method and following the specified grades, except compaction will be done by the large compactor. The north roadbed would be done in this same manner as well.

8. Site Repair

The contractor will place down grass seed on the disturbed soil and above the rip rap to initiate the site recovery.

9. Cleanup / Demobilization

The contractor will disperse any leftover cut clay and excess fill, remove the silt fence, and remove the equipment off the job site.

Appendix L:

Construction Cost Estimate

Table L1. Overall project estimate breakdown

Totals		
Equipment	$14,219.60	21.38%
Labor	$10,472.00	15.74%
Material	$37,959.25	57.06%
Hand Tools	$3,870.00	5.82%
Final Cost	**$66,520.85**	

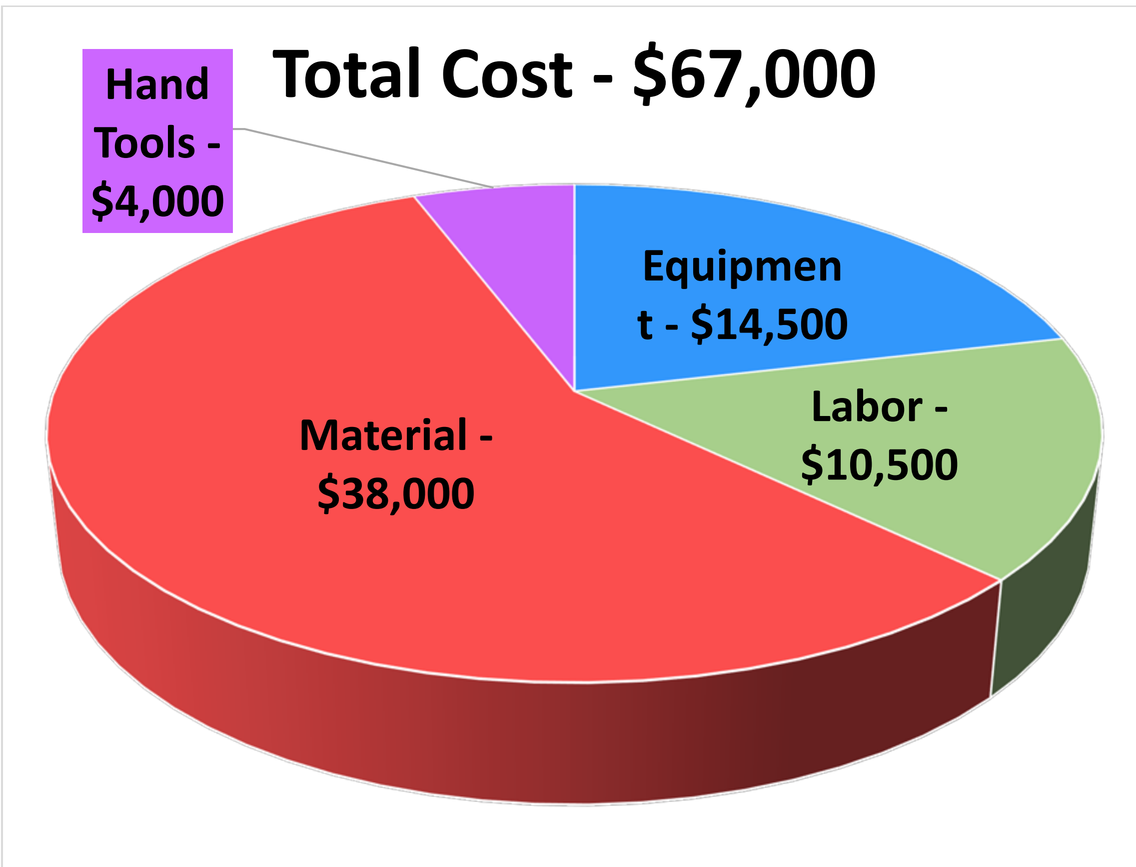


Figure L1. Pie chart breakdown of project estimate (Costs rounded up)

Table L2. Construction Cost Breakdown

Item	Quantity	Unit	Production Rate	Unit Rate	Hours	Labor Hour	Equipment Hour	Equipment	Labor	Material	Bare Cost
Concrete Sacks	83	each								$869.01	$869.01
Aggregate	540	cft								$928.80	$928.80
Sand	20	cyd								$340.00	$340.00
#5 Stirrups Rebar	200	lb								$126.00	$126.00
#6 Reinforced Rebar	202	lb								$113.12	$113.12
Falsework	189	ft								$71.82	$71.82
Screws	5	lb								$26.95	$26.95
Silt Fence	73	ft	100	ft/hr	1	1L+1F			$36.00	$28.47	$64.47
Channel Excavation	77	cyd	15	cyd/hr	16	2*16L+16O+16F	16E	$206.25	$768.00		$974.25
Foundation Excavation	60	cyd	10	cyd/hr	16	2*16L+16O+16F	16E	$550.00	$768.00		$1,318.00
Bank Excavation	27	cyd	15	cyd/hr	2	2*2L+2O+2F	2E	$68.75	$96.00		$164.75
Channel Sloping	152	syd	40	syd/hr	4	2*4L+4O+4F	4E	$137.50	$192.00		$329.50
Steel Rebar Placing Labor	725.5	LSUM			16	2*16L+16F+4O	4E	$137.50	$588.00		$725.50
Steel Rebar Wiring Roll	1	Each								$4.50	$4.50
4 ksi Concrete Footing Labor	15	cyd	3	cyd/hr	8	2*8L+8O+8F	8M	$89.20	$384.00		$384.00
Steel Base Unbalanced Channel 5" X 7"	38	ft								$836.00	$836.00
Anchor Bolts	27	Each								$79.38	$79.38
Anchor bolt epoxy	43	oz								$119.97	$119.97
Footing Plate Assembly	544	lb			16	2*16L+16O+16F	16E	$550.00	$768.00	$510.00	$1,828.00
Crown Plate Assembly	7075	lb			24	2*24L+24O+24F	24E	$825.00	$1,152.00	$10,612.50	$12,589.50
Connecting Bolts (2" Long)	202	Each								$375.72	$375.72
Connecting Bolts (1.5" Long)	514	Each								$848.10	$848.10
Headwall Plate Assembly	1632	lb			16	2*16L+16O+16F	16E	$550.00	$768.00	$1,632.00	$2,400.00
Expansion Joints	20	ft								$20.00	$20.00
Masonry Wall	8	cyd			16	2*16L+16O+16F	16M	$178.40	$768.00	$440.00	$1,208.00
Anchor Rod (1")	420	ft			8	2*8L+8F			$288.00	$11,760.00	$12,048.00
Anchor Rod (1.5")	150	ft			4	2*4L+4F			$144.00	$4,800.00	$4,944.00
Steel Plates	20.25	sft								$415.13	$415.13
Bevel Washers	12	Each								$6.72	$6.72
Angle Steel Plates	25.5	sft								$1,032.75	$1,032.75
Headwall Cap	25	ft								$75.00	$75.00
Guardrail	276	LSUM			8	2*8L+8F			$288.00	$276.90	$564.90
Rip Rap Placing	175	syd	25	syd/hr	24	2*24L+24O+24F	16E	$275.00	$1,152.00	$1,485.41	$2,912.41
Gravel (GW) 1" Crushed Gravel Stone	12	ton	62.5	syd/hr	48	48*2L+48O+48F	48E+48C	$1,650.00	$2,304.00	$120.60	$4,074.60
Site Restoration	45	syd			1	1L			$8.00	$4.40	$12.40
							Totals	$5,217.60	$10,472.00	$37,959.25	$52,831.25

Table L3. Material Unit Prices

Material Prices		
Gravel	$10.05	ton
Sand (Cl II)	$17.00	cyd
Aggregate	$1.72	cft
Concrete Sacks	$10.47	Each
Rip Rap	$16.69	ton
Silt Fence	$0.39	ft
#5 Stirrups Rebar	$0.63	lb
#6 Reinforced Rebar	$0.56	lb
2" Bolts+Nuts+Washer	$2.64	Each
1.5" Bolts+Nuts+Washer	$2.43	Each
Expoxy	$2.79	oz
Steel Rebar Wiring	$4.50	Each
Anchor bolts (3/4"d, 6"in length)	$2.94	Each
Falsework	$0.38	ft
Screws	$5.39	lb
Clover (Site Restoration)	$4.40	lb
Steel Base Unbalanced Channel	$22.00	ft
Masonry Block	$55.00	cyd
15.5 in x 6 in Corrugated Metal (galvinize	$1.50	lb
6 in x 2 in Corrugated Metal (galvinized)	$1.00	lb
Anchor Rods (1in)	$28.00	ft
Anchor Rods (1.5 in)	$32.00	ft
3/8" Thick Steel Plate	$20.50	sft
3/4" Beveled Washer	$0.56	Each
3/4 " Thick Steel Plate	$40.50	sft

Table L4. Labor / Equipment Rates

Labor/Equipment Rates Used		
Operator Wage	Hour Rate	$12.00
Labor Wage	Hour Rate	$8.00
Foreman Rate	Hour Rate	$20.00
Truck Driver	Hour Rate	$17.00
Dump Truck	Hour Rate	$43.75
Mini-Excavator (303CR C)	Hour Rate	$34.38
Concrete mixer, 16 C. F., 25 HP, gas	Hour Rate	$11.15
3 ton Capacity Trailer and Pickup	Hour Rate	$22.33
Pickup Truck	Hour Rate	$19.50

Table L5. Hand Tool Costs

Hand/General Tools	Price		Quantity	Bare Costs
Shovels	$15.00	Each	2	$30.00
Power Drills	$100.00	Each	1	$100.00
Hammer	$20.00	Each	2	$40.00
Rebar Pliers	$25.00	Each	1	$40.00
Compactor	$1,000.00	Each	1	$1,000.00
Generator	$1,000.00	Each	1	$1,000.00
Ladder	$100.00	Each	1	$1,000.00
Chain with Clamps for Lifting	$500.00	Each	1	$500.00
Wheelbarrow	$80.00	Each	2	$160.00
			Total	**$3,870.00**

Table L6. Material Transportation Costs

Material Transportation	Number of Truckloads	Type of Truck	Material Location		Truck Hours	Labor	Equipment	Bare Costs
Gravel	15	Dump	6	hr Pit	90	$630.00	$3,937.50	$4,567.50
Sand	2	Dump	6	hr Pit	12	$84.00	$525.00	$609.00
Rip Rap	4	Dump	6	hr Pit	24	$168.00	$1,050.00	$1,218.00
Aggregate	2	Pickup	6	hr Pit	12	$84.00	$525.00	$609.00
Plates + Bolts	2	Pickup	10	hr Panama City	20	$140.00	$390.00	$530.00
Masonry Block	1	Dump	6	hr Pit	6	$42.00	$262.50	$304.50
Rebar	2	Pickup	6	hr Pit	12	$84.00	$234.00	$318.00
Falsework	1	Pickup	6	hr Pit	6	$42.00	$117.00	$159.00
Generator+Compactor +Silt Fence+Tools	1	Pickup	6	hr Pit	6	$42.00	$117.00	$159.00
Cemet Mixer	1	3 ton Cap	9	hr Rough Terrain	9	$63.00	$201.00	$264.00
Mini Excavator	1	3 ton Cap	9	hr	9	$63.00	$201.00	$264.00
							Total	**$9,002.00**

Appendix M:

Construction Manual

Construction Manual for Las Trancas Stream Crossing

This is a construction manual for the flexible buried steel bridge designed for the Las Trancas Stream Crossing. The following provides a basic guide of constructing these types of bridges. The manual is to coincide with the design sheets provided in this report.

Excavation Cut Sheet

A silt fence is to be placed before the excavation in order to contain loose sediment that could erode during a large rainstorm. Trancas Associates recommend that construction be done in the dry season. The river channel excavation should be done first in order to clear all debris and make room for the equipment for other construction work.

The excavation is to be done with an appropriately sized mini-excavator with a 1 cyd – 1.5 cyd bucket. The river bank walls are to be cut to a 3.5:1 (Horizontal: Vertical) slope for the riprap placement.

The footing excavation should be done in the approximate location given on page J-3 and J-6 of the Final Design Drawings and Detailing. The depth of the footings are to be about 7.5 ft but should be at equal elevation. The backfill should be the road gravel after the bridge plates are assembled.

Footing Details

The rebar is to be placed in the excavated hole in the manner shown on page J-4 and to be tied together using steel rebar wiring. There is an option to place wood forms for the concrete footings but with the clay material on the project site, it is anticipated that the clay will be able to stay coherent to act as forms.

The concrete is to be mixed on site using a 3:2:1 mix. Meaning 3 parts aggregate, 2 parts sand and 1 part cement by mass. The footings should be poured either directly from the mixer or into wheelbarrows and then poured. The concrete mix should be poured as quickly as possible in order to avoid it setting in between mixes, thus reducing strength.

Footing Connection Details

After the concrete footings are set, the corrugated 6 in x 2 in steel footing should be placed and the 5 in x 7 in base channel on top. The anchor bolts should be epoxied into the concrete according to the spacing provided on page J-5.

Crown Plate Placement

The footings should be aligned so that the bridge is placed perpendicular to the river centerline. There are to be two different type of plates that will be connected together to make the crown plate. There will be end sections on each side and a middle section. The bridge will be 5 plates wide. The plates are to be lifted using a mini-excavator with a chain which can be clamped onto the plate. The biggest plate weight is about 700 lb – 800 lb and the mini-excavator will be able to handle this weight.

The crown plate width is to be 18.75 ft. The span length of the crown plate is to be 23 ft 4 in. And the center of the crown plate is to be 11 ft, 8.5 in from the end sections. The rise of the crown plate is to be 9 ft 3 in from the top of the footings.

Crown Plate

The crown plate is to be made of 10 end sections and 5 middle sections of the specified dimensions and radii in the design drawings. The plates have a 1.5 in lip in order to connect the plates together. The plates are to have a 15 in x 5.5 in corrugation.

Crown Plate Connections

The plates are to be connected using ¾ in diameter, 2 in and 1.5 in long bolts. The 2 in long bolts are to be used when 3 plates are to be connected. These laps would be in the middle sections of the crown plate.

The first step of placing the crown plate would be to connect the plates to one of the end sections. This would include lifting the plate and matching the bolt holes to the bolt holes in the base channel and then bolting them together. The best method would be to move along the width of the bridge, connecting the end sections to the footing and then placing the middle plate on top before proceeding to the adjoining plate. Falsework should be used to hold the different sections up during the connection phase. Page J-8 provides the amount of torque required for the bolting.

Riprap Placement

Riprap is to be placed on the bank walls of the river after they are prepared by the mini-excavator. The riprap is to have a mean diameter of 14 in, meaning 50% of the riprap is to be less than 14 in and 50% is to be greater than 14 in. The best method would to use the mini excavator and the wheelbarrows to spread the riprap and slope it at 3:1.

Headwall Lengths

The headwall plate is to fabricated to meet the arc of the crown plate. Each headwall is to be made of 7 plates to be connected together.

Headwall Bolt Holes

The bolt holes for the connecting bolts and for the anchor rods are to be punched during the fabrication process. The connecting plate holes are to be 13/16 in holes. The spacing for the holes are given on page J-11.

Connections Spacing

The connections of the plate connected to the footings should be done first. The plate should be connected to the footing with the anchor bolts to be epoxied in. The 2 in length bolts should be used to connect the footing connection plates to the headwall.

The connections of the headwall to the crown are to be placed following the dimensions found on page J-12. The 2 in length bolts are for the connection from the plates to the headwall. The 1.5 in length bolts are for the connection to the crown plate.

Angle Connections

The angle plates are to be placed at the bolt holes where they are called for on page J-13.

The headwall should be constructed by first connecting the angle plates to the crown plate. Next by constructing the anchor rods so that the plates can stand up straight when connecting the next plate.

Backfilling with the gravel after each angle plate and anchor rod is constructed can help to keep the headwall in place. It is important to make sure that the backfilling does not cover up a connection that needs to be made. It is even more important that connections that will interfere with backfilling later on are not made before backfilling.

Headwall Connections

The connections made for the headwall connections to the crown plate shall conform to the dimensions shown on page J-14.

Anchor Rod Dimensions

The 9 in by 9 in plates are to be connected to the anchor rod, before the rods are connected to the headwall. The lengths given are total lengths of the rods. Connections B and C are to be placed at an angle to then be connected to the crown plate by an angle plate.

Connections D should be backfilled and then the anchor rods can be connected since these connect to the opposite headwall. This should be a step by step process of backfilling and then placing the anchor rods systematically. The backfilling should be done in equal amounts on both sides of the crown plate so that the plate retains its shape and doesn't topple over on one side.

Gravel Road Placement

Masonry walls are to be constructed to contain the gravel when the roadway is vertically approaching the crown plate. The first wall is to be built to match the south end approach and should allow for a 16% vertical grade to the landing on the crown plate. The gravel roadway should be a width of 18 ft. The blocks are to be bonded together with mortar.

The second wall is to be built on the north end approach and should allow for a 20% vertical grade down to the existing road. The wall will have to be on a turn radius given on page J-17 and should bring the roadway back to the existing roadway. There is the possibility to use leftover existing cut material as fill under the gravel road in order to grade it back to the existing road. Also single or more blocks may be needed to be placed as a curb in order to contain the gravel.

The gravel is to be backfilled and compacted in 6" – 8" lifts and a compaction minimum of 90%. The backfilling is to be monitored to make sure that each side of the crown plate is backfilled equally to avoid toppling of the plate. The minimum cover of gravel over the plate is 2 ft and is located at the center of the crown plate.

The gravel is specified to be 1" crushed, angular gravel stone. The backfill should follow ASSHTO M145, group A-1-a specifications.

Bridge Cross Section

Fiber expansion joints should be placed in between the headwall and masonry wall and be mortared in place. The gravel roadway should have a crown of 5% in order to account for drainage.

Guardrail

A guardrail should be placed once the headwall is fully connected and backfilled before the last foot of gravel is compacted. A headwall cap should also be added on top of the headwall in order to protect it.

The plate should be inspected and monitored daily to make sure all connections are done correctly. The bridge should be tested thoroughly before public vehicles are allowed to cross it.