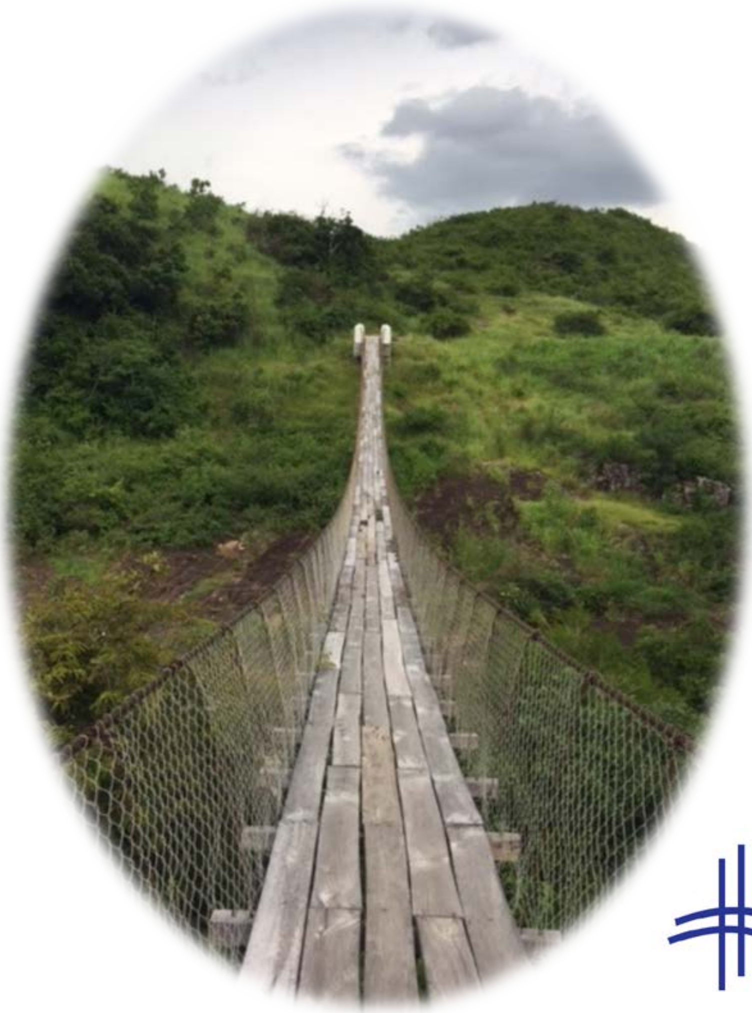
Rio Cabuya River Crossing

El Hatillo, Coclé Province
Panama



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iDesign Fall 2017

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To Julius Kellinghusen and all other relevant parties,

The Better Bridges design team has put together this report to fully outline all necessary information needed to understand and construct a bridge over the Rio Cabuya. This bridge will provide a safe and reliable way for pedestrians to cross over the river regardless of flooding conditions. We would like to thank Julius Kellinghusen, the residents of El Hatillo and Caimital, and anyone else who helped us collect information for the bridge design while in Panama.

Sincerely,

The Better Bridges Team

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DISCLAIMER:

This report, titled "Rio Cabuya River 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.**

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Executive Summary

Better Bridges' mission is to engineer sustainable bridges that provide efficient and reliable transportation to communities in need through surveying, design work, and construction. The Better Bridges multidisciplinary team traveled to El Hatillo, Panama in August of 2017 to collect the data necessary to design a low traffic foot bridge. This report contains the integrated framework for the project and outlines the problem, constraints, analyses, and construction schedule. Details of the analyses include the hydraulic and structural elements that influenced the final design.

The final design is a suspension style bridge that crosses the river just downstream of the current path. It will consist of 20 foot tall towers on each bank that the main cables will be draped from. The main cable will be attached to an underground anchor on either side of the bridge crossing. The deck spans 160 feet and will be supported by cross beams that are connected to suspension cables under the main cables.

The watershed analysis describes calculations that resulted in a 17 foot maximum flood depth, which proved that the designed bridge height 45 feet above the riverbed provided adequate freeboard. Thus the estimated maximum floodplain will not compromise the structural integrity of the bridge. The structural analysis describes the overall bridge calculations that influenced the dimensions of the major bridge components, including the required cable size, anchor sizes, and foundation size.

A construction schedule outlines the timeline for erection of the bridge and is expected to begin at the end of the wet season and continue into the dry season. The schedule includes Saturdays as work days and excludes major holidays, resulting in a duration of 35 calendar days. Required construction for this design has resulted in the labor, material, and equipment costs totaling $90,000 USD.

Stakeholders will include the Panamanian government and local community members of El Hatillo. Donations such as time, labor, and materials are expected to decrease the cost, while also providing the community with a reliable bridge that they can take ownership of.

The quoted cost includes all raw materials and labor necessary, based on erection of the proposed bridge within the United States. Labor costs may be reduced using volunteers from members of the El Hatillo and surrounding communities. Material costs may be reduced by utilizing the existing cable in the community. Better Bridges advises strength testing to be conducted on this cable and approved by a qualified engineer. All calculations, estimates, and the schedule can be found in the appendices of this report.

1 Introduction

El Hatillo is a rural community located in the Coclé Province of Panama and has requested design services from the Better Bridges Engineering Team for a low traffic foot bridge over the Rio Cabuya River. This river floods several times a year, making it impassable for the members of El Hatillo and surrounding communities. Residents must be able to cross this river for work, supplies, and education. There are two current bridges near the community, but they do not make crossing the Cabuya River convenient for the members of El Hatillo. They are poorly located and currently pose safety hazards, which raises concern by the community.

After traveling to El Hatillo and being immersed within the community, the Better Bridges team was able to gain exposure to the problem, identify the major constraints, and develop a solution customized to the needs of the inhabitants. Data was initially collected from four prospective sites, and the optimum location for the bridge was determined after analysis of major constraints. Two existing bridges were observed and referenced for comparing bridge styles. After a community meeting, it was determined that the favored style of bridge to local users was a suspension bridge for its sturdy walkway. The residents of El Hatillo preferred the location shown in Figure 1 for its proximity to the current river crossing.

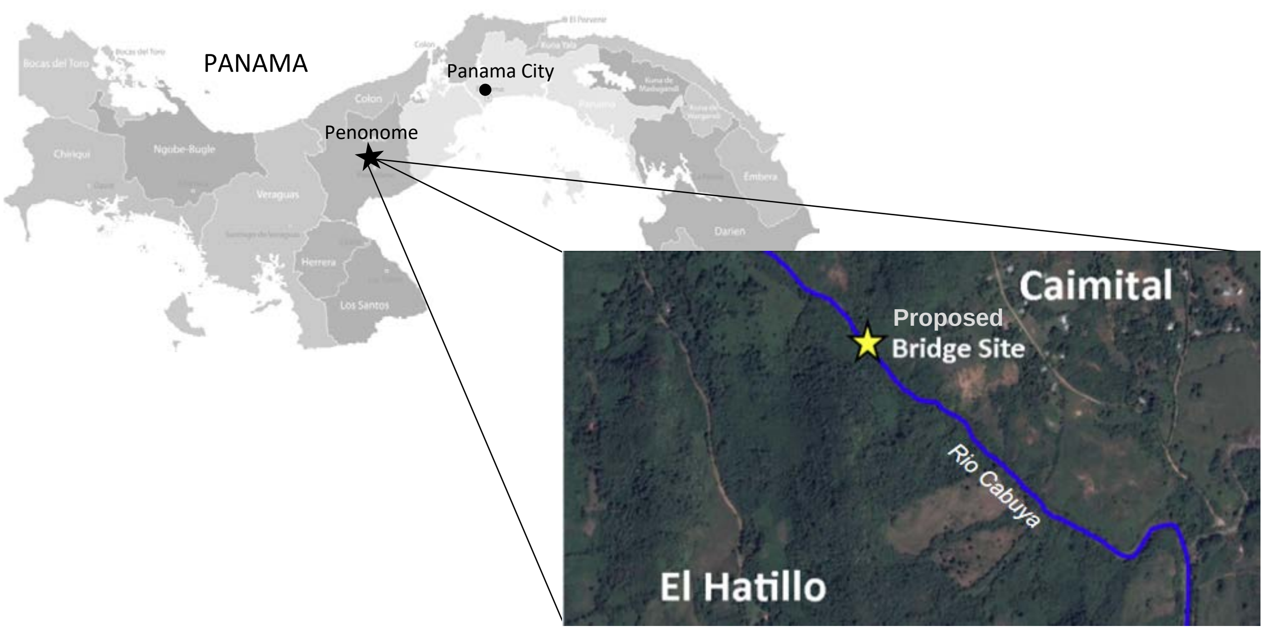

Figure 1: Location of El Hatillo and Bridge Location

Upon returning to Michigan Tech, Better Bridges analyzed the collected data to provide details for the proposed bridge. Data described by the drawings that are attached to this report reflect the constraints that were set from the beginning. The outlined design was tailored to the needs of the El Hatillo community, and the Better Bridges Design team has produced the attached drawings that integrate this framework. Reducing the cost was emphasized by finding the cheapest available prices for materials while still meeting design requirements of the bridge. The final design is a suspension style bridge that provides a safe and sustainable solution for the community. Design drawings can be found in Appendix G.

2 Community Background

Penonome, located in the Coclé province is the largest city surrounding El Hatillo, and is where many residents work and attend school. Caimital is the community nearest El Hatillo that provides reliable daily transportation to Penonome. While the Better Bridges team was in El Hatillo, the active Peace Corps volunteer, Julius Kellinghusen, worked closely with the team to survey possible sites and gather information on water levels during flooding as well as information on community opinions.

Locations of homes within El Hatillo are very spread out; therefore, the layout of the community makes outside connections challenging. There is no electricity, and most homes do not have running water. Its 27 residents rely heavily on connections between the neighboring communities, Caimital and Penonome, for these amenities. The household structure commonly consists of extended family members residing in the home. Men are often working as farmers or laborers, and women tend to stay home to cook, clean, and care for the home. Children attend a primary school that is located in Caimital and receive a middle and high school-equivalent education in Penonome. El Hatillo's small population is unable to provide the available opportunities that already exist in Caimital and Penonome, which is why residents commonly pursue work and education outside of the community.

Members of El Hatillo travel approximately 30 minutes to Caimital by foot, where motor transportation to Penonome is available. Currently, people must walk directly through the river and use large rocks as stepping stones to cross. During the rainy season, the river becomes impossible to cross by foot, as the river depth drastically increases. Isolation from Penonome and Caimital by the Cabuya River gives reason for the need of a reliable crossing to support and alleviate the current commute. The two existing bridges are the only route for crossing the river during flooding, but are not practical due to their locations, as shown in red at points (a) and (b) in Figure 2. Further observations of these bridges are discussed in Section 4, Data Collection.

Figure 2: Locations of Existing Bridges
(a) Suspension and (b) Suspended

3 Design Constraints

The identification and consideration of major constraints helped to determine the final design of the bridge. It is essential to minimize costs and materials in order to develop the most practical design for this project. Therefore, the design is simple, efficient, and possible to repair by members of the El Hatillo community.

3.1 Design Alternatives

Better Bridges assumed the simple design of a *suspension* or a *suspended* style bridge would be the best fit for the El Hatillo community. Figure 4 in section 4.1 shows the differences in each style. The main constraint that was identified through comparison of these bridges was the cost of materials required for their respective structural elements. The greatest difference between the two styles is that the suspension bridge requires tower construction and has a level deck, while the suspended has no towers and a concave deck.

Overall, the suspended bridge is a simpler design because of the lack of towers. Comparatively, due to the large span of the river, the suspension style bridge is ideal in providing a steady walkway and maximum freeboard. Freeboard is the distance from the maximum floodplain to the base of the bridge deck. This is important for the safety and reliability of the bridge. After data was compiled, and further discussed in section 4 Data Collection, it was decided that the Rio Cabuya River Crossing would utilize the *suspension* style bridge as modeled in Figure 3.

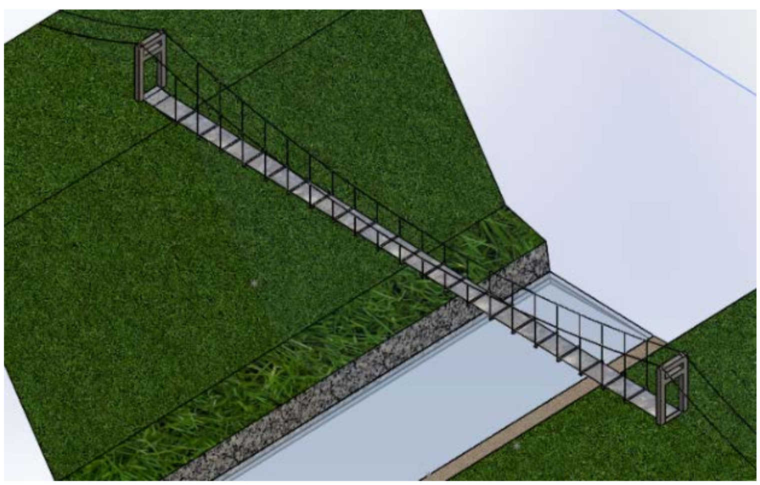

Figure 3: Initial Rendering of Suspension Bridge

4 Data Collection

During the site visit, the Better Bridges team collected data through observing existing bridges, conducting a meeting with the clients, surveying alternative locations, and taking soil samples for its classification. Major technical constraints that were identified from analyzing the surveyed sites included the floodplain elevation and the difference in bridge landing elevations on opposite sides of the river. This data was used to design a suspension bridge that will provide a personalized solution for efficient and reliable transportation between El Hatillo and Caimital.

4.1 Existing Bridges

Two existing bridges are pictured in Figure 4. The first bridge, Figure 4a, is a *suspension* style bridge that was built and financed by the Panamanian government in 2004. The main cable is anchored into the ground and rests on the towers. The steel diamond plate deck is then supported by crossbeams which hang from the suspender cables. This style creates a large tensile force on the anchors of the main cable, which is the main concern for stability of the foundation. The figure shows that there is a large gap between the walkway and the fencing, which is also a safety concern.

The second bridge, Figure 4b, is a *suspended* style bridge that was built by the Bridges to Prosperity Foundation in 2014. The main cable is attached to the pile foundation and sways freely from each side. The deck consists of wooden planks that are beginning to deteriorate after just three years.

(a) (b)

Figure 4: Existing Suspension (a) and Suspended (b) bridges

4.2 Client Consultation

After a community meeting, residents were able to express their opinions about what they liked and disliked about the two existing bridges. Safety issues were their main focus and included the stability, width of the deck, and the deterioration of the wood planks. It was concluded from this that the community favored the suspension style bridge exemplified in Figure 4a for its level walkway.

After the consultation, it was concluded that the community favored a location near the current river crossing. There was a concern regarding the ability to get equipment to the site during construction. Machinery such as a bulldozer for excavation would require clearing of brush that can be done by the community members. The Better Bridges team used this information to incorporate what the clients preferred when gathering the following data for the design proposal. Determination of a site location is discussed further in section 4.3.

4.3 Site Surveying

Prospective sites were chosen in the field based on their geometry and location relative to the current path. A short span and minimal difference in elevation were ideal geometric features that the team knew would produce a simple and cost effective design. Better Bridges utilized an Abney level and a digital rangefinder to record measurements in the field from an assigned reference point. Important features such as elevations, ridges and current river depths were recorded.

Four initial sites were surveyed and later compared to elect the optimal location that would satisfy constraints. The first site, called the Husky site herein, is located along the river just north of a tributary. This location would require another bridge to be constructed over a tributary at a site referred to as McNair. Since this option would require the construction of two bridges, it was ultimately found to be impractical due to cost constraints. The other two sites that were considered are just downstream of the confluence, called the Wadsworth and DHH sites. All sites were within a five minute walk from one another.

The data collected from surveying is illustrated in the cross sections for the Wadsworth and DHH sites, displayed in Figure 5 and Figure 6, respectively. Major dimensions are shown such as the width of the banks, the elevation from the riverbed, and the difference of elevations of each ridge. After determining that the elevation differential was the main difference between the two sites, it was decided that the optimal location for the designed footbridge was the DHH site.

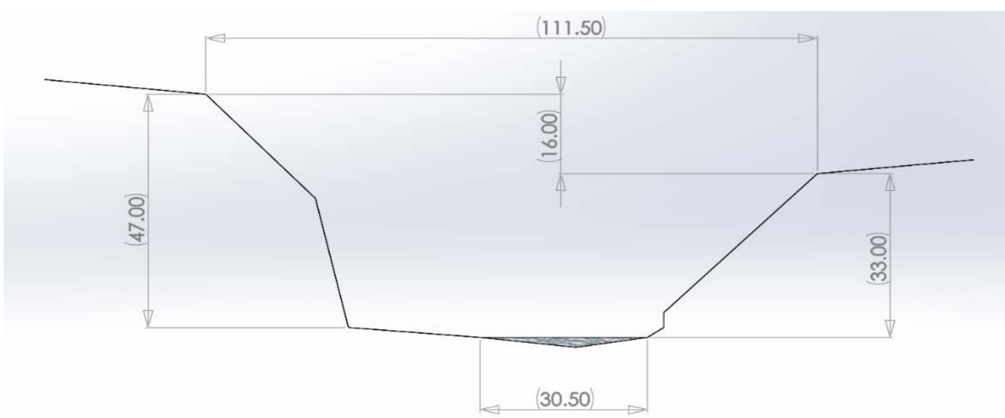

Figure 5: Wadsworth Site Cross Section (Dimensions in ft.)

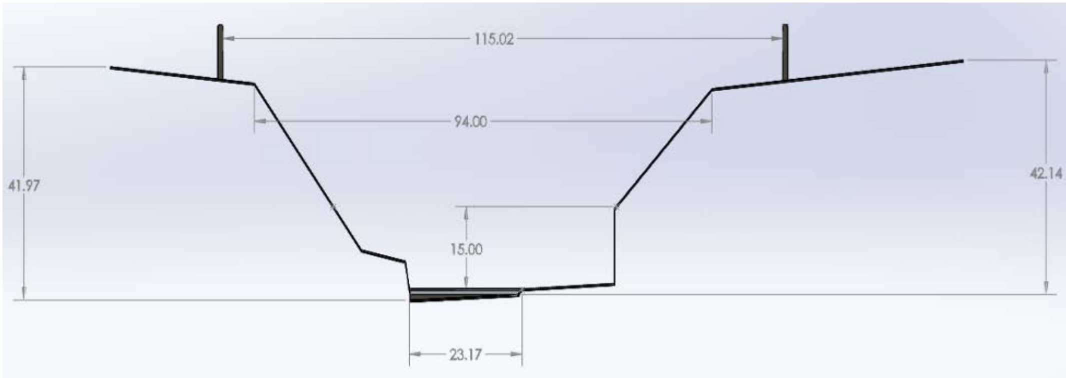

Figure 6: DHH Site Cross Section (Dimensions in ft.)

4.4 Soil Classification

During initial surveying, a soil sample was classified by its visual characteristics using the Unified Soil Classification System (USCS), shown in Table 1. The soil was collected at the approximate locations of the anchors and foundations for towers. The tannish-brown sediment sample consisted of mainly sand and fines, with some gravel and organic material. Due to the lack of testing equipment for determining an accurate grain size distribution of the soil, the Better Bridges team used their best judgement to estimate that the soil is classified as OL-Organic Silt, Organic Clay.

This classification provided the appropriate soil properties during the structural analysis. These properties include the angle of internal friction and soil density. Because the soil is the only resistive force for the suspension bridge, these properties allowed the team to perform calculations to determine the required dimensions for the tower foundations and cable anchors. These dimensions ensure failure will not occur in the soil when the predicted forces act on the bridge.

Table 1: United Soil Classification System from American Society for Testing Materials, 1985

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO.200 SIEVE	GRAVEL MORE THAN 50% OF COARSE FRACTION RETAINED ON NO.4 SIEVE	CLEAN GRAVEL	GW	WELL-GRADED GRAVEL, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND MORE THAN 50% OF COARSE FRACTION PASSES NO.4 SIEVE	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY-GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE GRAINED SOILS MORE THAN 50% PASSES NO.200 SIEVE	SILT AND CLAY LIQUID LIMIT LESS THAN 50	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILT AND CLAY LIQUID LIMIT 50 OR MORE	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		ORGANIC	OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT

From the USCS table, typical soil property values for OL-Organic Silt, Organic Clay classified materials were researched. Better Bridges assumed conservative values to account for uncertainty and ensure safety.

The internal friction angle is used to describe a soil's shear strength, which is the maximum force the soil can withstand before failure occurs. Typically ranging from 25°-35°, Better Bridges assumed an internal friction angle of 28° for the collected soil sample at the site. This data is important when determining the bearing capacity of the soil under the foundation so settlement and/or slope failure does not occur. Further explanation of soil properties in foundation calculations can be found in section 6.2.

An estimated soil density of 110 lb/ft^3, was used in calculating the weight of the soil above the buried anchor. This produced the required resisting force to prevent the uplift of the anchor from the tension in the main cables. Further explanation of the soil density in the anchor calculations can be found in section 6.3.

5 Watershed Analysis

Better Bridges performed a watershed analysis on the upstream area leading to the proposed bridge site and developed a hydrologic simulation model. This aimed to determine a maximum design flow rate and depth through the channel during a flood event. This analysis was critical to properly design and locate a bridge that will withstand a major flood based on the maximum calculated depth. Better Bridges utilized ArcGIS, Google Earth Pro, and HEC-HMS to perform the watershed analysis and hydrologic calculations. A digital elevation model (DEM) was downloaded from the United States Geological Survey [6] and used to gather the data necessary to perform the analysis. This analysis was done based on the NRCS Curve Number Method outlined in Wurbs [3].

5.1 Watershed Characteristics via ArcMAP 10.4.1

A Digital Elevation model was imported into an ArcGIS map to begin the analysis. The watershed boundary was delineated via tools within ArcGIS. The resulting map can be seen below in Figure 7. Through utilization of ArcMAP tools, the Rio Cabuya Watershed upstream area was determined to be 6.99 mi^2 and the main channel length was found to be 4.52 mi. Based on the amount of trees and vegetation, it was estimated that the watershed is 67% forested, and 33% open. Detailed explanations of these calculations can be found in Appendix A.

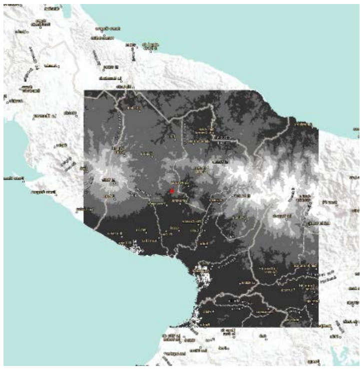

Figure 7: DEM File over Terrain Basemap

Based on initial site surveys, it was determined that the Rio Cabuya watershed belongs to the NRCS Hydraulic group C. Through utilization of Table 8.3 in Wurbs [3], the runoff curve number of the watershed is estimated to be 74.28.

5.2 Slope calculations via Google Earth Pro

To find the slope of the watershed Google Earth Pro was utilized. Following GPS coordinates from ArcMAP, high and low points on the watershed boundary were located. The points were connected to the bridge location and the path was used to create an elevation profile. Some elevation profiles were gradual, while others had larger hills and deeper valleys. Of all paths, the highest percent slope was 13% while the lowest was 2%. The estimated average slope of the entire watershed was found by taking the weighted average of the 5 paths, where the weight was proportional to the length of each path. This was done using the SUMPRODUCT function within excel. The weighted average of the entire watershed was found to be 5.5%. A more detailed explanation of slope calculations can be found in appendix A.

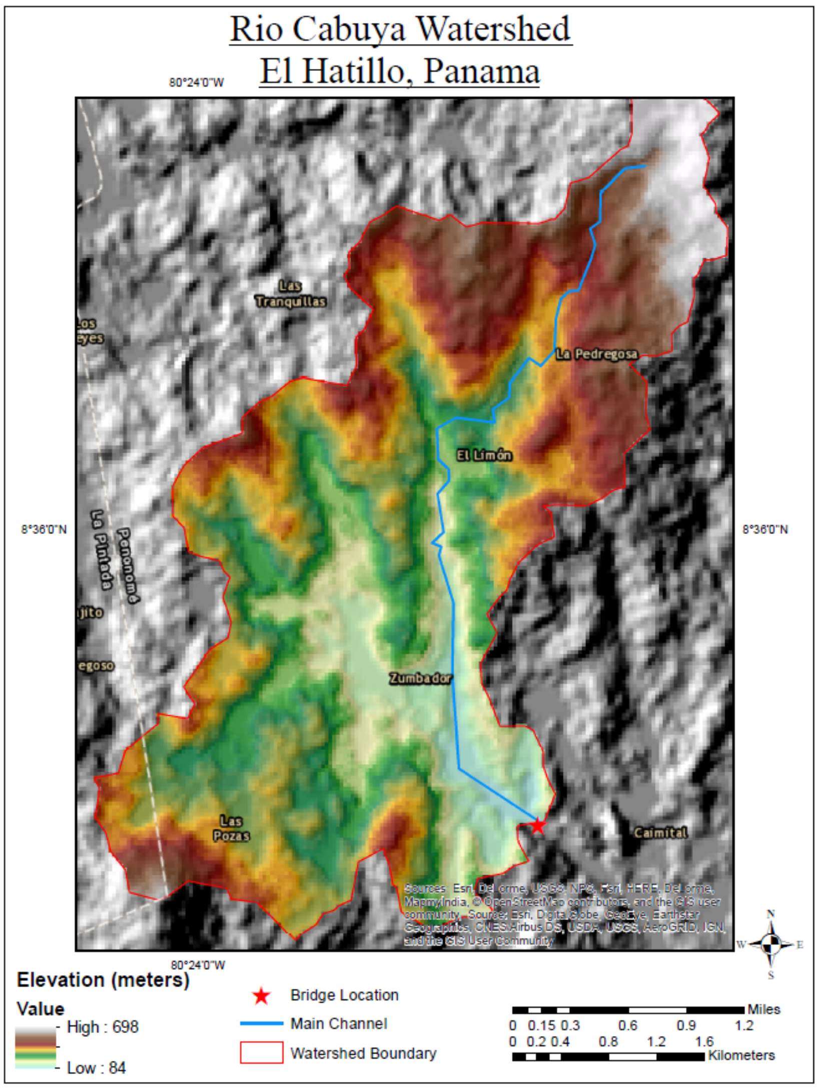

Figure 8: Rio Cabuya Watershed

5.3 Runoff & Discharge Determination via HEC HMS 4.0

A 100-year, 24-hour flood event was modeled after the December 7-8, 2010 extreme precipitation event explained in Shamir [2]. This flood event occurred at the Panama Canal watershed, rather than the project location, but was considered the most reliable data available. This 24-hour flood event accumulated 10.87 inches of precipitation. HEC-HMS, a hydrologic simulation model, was used to estimate the storm discharge rate at the bridge site. Total runoff depth was found to be 7.76 inches. A flood Hydrograph was developed to find the maximum discharge (Q) of approximately 7990 cubic feet per second (cfs), with a peak discharge occurring after 14 hours. Figure 8 below shows the summary table of hydrologic simulation model, while figure 9 illustrates the hydrograph summarized by these values.

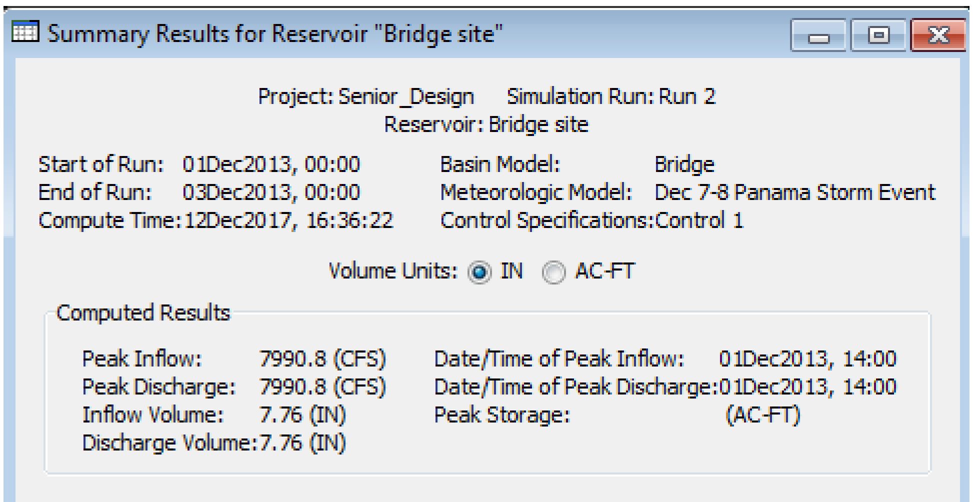

Figure 9: Summary of Hydrologic Simulation Model

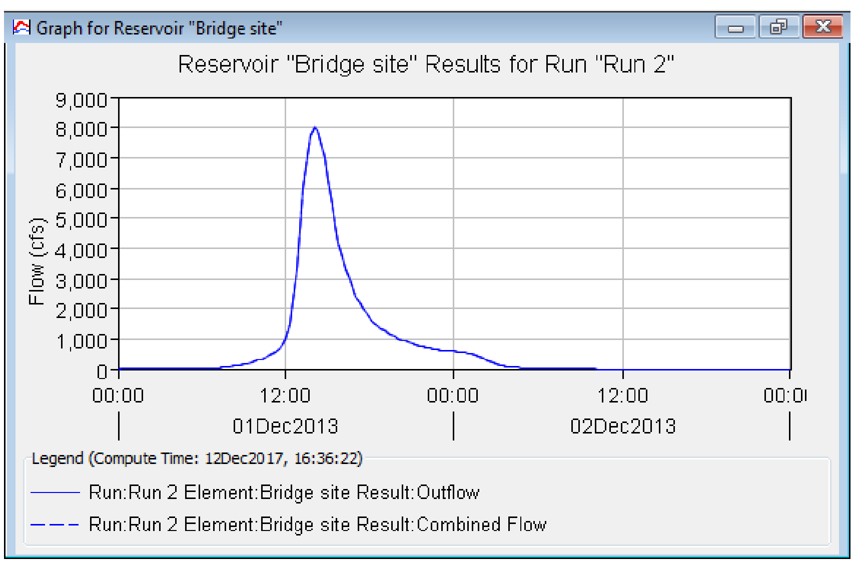

Figure 10: Hydrograph of Design Storm

5.4 Depth calculations via Manning's Equation

The maximum discharge was then applied to Manning's equation, below, which is outlined in Wurbs [3]. Through a trial and error method we were able to solve for the area (A). Since the area is a function of depth, in turn we were able to solve for a maximum depth of 17.3 ft.

$$Manings\ equation = \ Q = \frac{C_m}{n} * A * R^{\frac{2}{3}} * S^{\frac{1}{2}} = S^{\frac{1}{2}} * k$$

$$Manings\ equation = \ Q = \frac{1.49}{.045} * 772.4\ * 9.28^{\ \frac{2}{3}} * 0.005^{\ \frac{1}{2}} = 0.005^{\frac{1}{2}} * 96711 = 7990 \text{ cfs}$$

C_m = 1.49 for US Standard Units
A = Cross Sectional Area (ft^2)
R = hydraulic Radius = $\frac{A}{wp}$ (ft)
WP = Wetted Perimeter = Perimeter-width of channel (ft)
S = Slope = .005 ft/ft (or 0.5%)
n = Manning's Coefficient = 0.045 for 49% cobble, and 51% brush and vegetation
Q = Discharge =7990 $\frac{ft^3}{s}$

5.5 Design Considerations

The watershed analysis provided a calculated maximum depth for a major storm event. This was used in the final design proposal for confirmation that the bridge would not be affected by the flow. The calculations in Appendix A influenced the location of the tower foundations and the height of the bridge deck by confirming they will both withstand the calculated maximum flood level. The bridge design with this depth gives a freeboard of 28 feet. This was determined to be a sufficient freeboard for any debris that may pass under the bridge and for the placement of the foundation to not be affected by high velocity flow.

6 Structural Analysis

Better Bridges conducted a structural analysis to determine the dimensions for elements of the bridge. Figure 11 illustrates the main forces and reactions that gave dimensions for the cable design, and ultimately, for the design of the foundations and anchors. The horizontal tension in the cable are approximately equal on both sides of the towers, making the moment in the foundation negligible. Wind loads are calculated and shown in Appendix B, but are also assumed to be negligible in creating a moment. This is an acceptable assumption, because the location of the bridge is surrounded by relatively thick brush, trees, and vegetation, which would dissipate the wind affects.

$$Working\ Load = DL + LL = 56{,}393lbs$$

$$Horizontal\ Tension\ (T_H) = \frac{56{,}393lbs \cdot L}{8 \cdot S} = 77{,}570lbs$$

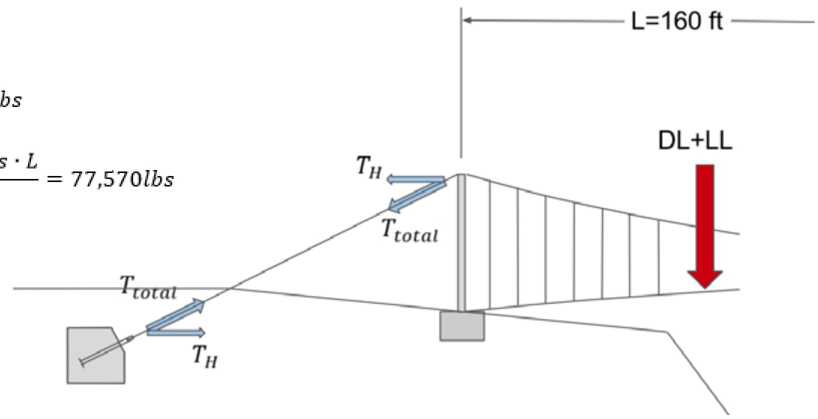

Figure 11: Tension in Cable from Loads

Outlined in the overall calculations found below are the loadings as well as cable, tower, and walkway designs. The results of this analysis gave the necessary values during the structural analysis of the foundation and anchor dimensions. Steps for the overall, foundation, and anchor calculations are attached in Appendix B, C, and D, respectively.

6.1 Overall Bridge Calculations

These values allowed for the selection of both main span cable diameter as well as the suspender cable diameter in design. Anchor size and cable attachment were determined using the backstay tension from cable to anchor on either side. Tower design was done using the Bridges to Prosperity manual and by summing moment forces from the cable tensions and angles [4]. Each calculation is shown in detail in Appendix B.

6.1.1 Loadings

Loads considered in the design of the bridge elements were the dead load, live load, and wind load. Dead load comes from the weight of all materials being supported by the bridge without having any pedestrians or other foreign load on the structure. The dead load is the weight of the bridge itself. The live load was calculated using a value of 90 pounds per square foot (psf) for the entire area of the walking surface of the deck [4]. These two forces are shown in Figure 11. Wind load also used a pounds per square foot assumption at 20 psf. The area of the profile of the bridge was calculated using AutoCAD software and the distributed load was applied to find a total load. These three loads were then applied using a safety factor of 5.0 to calculate the required component dimensions. These loads were then used to calculate cable tension and load per support beam.

6.1.2 Cable Design

The maximum tension for the loaded cable design illustrated in Figure 11 determined the required cable size to support the loading. The maximum tension in the cable was calculated to be 87,000 lbs. Two main cables will require a galvanized 1 5/8” size to accommodate the tension safely at a length of 280 linear feet. The 3/8” galvanized suspender cables support the deck by connecting to the main cable and cross braces using drop forged galvanized clips of corresponding size to the cable. Cable design calculations are shown in detail in Appendix B.

6.1.3 Tower Design

The towers were designed using the Bridges to Prosperity manual which included detailed tables on tower height with respect to span and loading. The main vertical tower members are made of 6” SCH40 circular steel tube. Cross members consist of different length 3’’x3’’x1/2’’ steel L sections that are bolted together. The towers are anchored to the foundation using 5/8’’x10’’ anchor bolts set through holes in a 2’x2’x1/2’’ baseplate. Cable saddles made of 2” schedule 80 steel pipe are on top of each vertical tower member to provide a safe and even place for the cables to rest free of unnecessary friction.

6.1.4 Walkway Design

The walking surface and deck supports were designed in a way to be easily repairable with minimal tools and construction knowledge. The 8’x2”x8”deck boards will be replaceable by simply removing the screws holding them to the nailers and replacing the board. The actual deck supports are two back to back 4”x4”x1/2” steel angles and a 1/4” steel plate all assembled using bolts to allow for removal and replacement if necessary. A design component that simplifies repairs uses two angles. This allows one to be removed and replaced while leaving the other to briefly support the deck. Each support has two eyebolts to connect to the suspender cables on either side.

6.2 Foundation Calculations

Foundation size was calculated by comparing the pressure exerted from the bridge to the bearing capacity of the soil below the foundation. Analysis of this comparison was conducted using the forces shown in the figure below. Forces from the tension in the cables at the towers, the weight of the towers themselves, and the weight of concrete foundation were added together to find a total vertical force on the soil. This point force reacts with the soil at the base of the foundation over an area. The total force over the area is the pressure on the soil. To find the factor of safety, the bearing capacity from the Bridges to Prosperity Handbook [4], of 3500 psf was divided by the pressure.

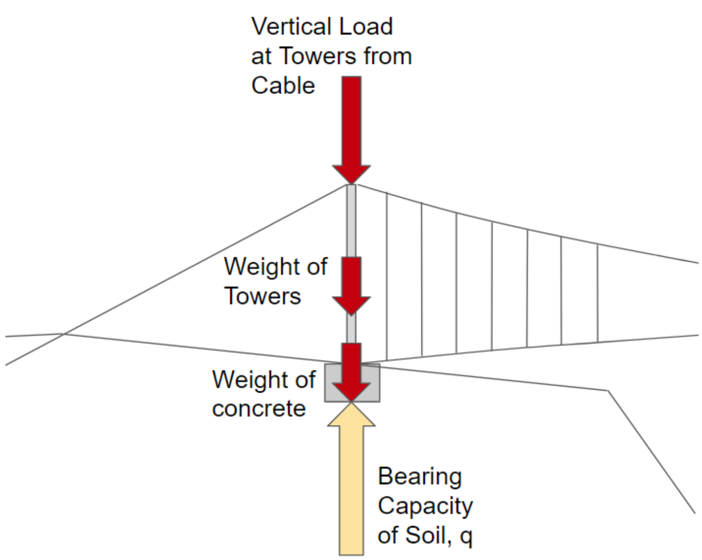

Figure 12: Forces Acting on Foundation

This method indicated that a 6.5' x 6.5' square foundation at a depth of 4' was adequate to satisfy a safety factor of 1.5. Rebar was added to reinforce the concrete foundation from tensile forces that arise when the concrete flexes from vertical compression loading. Appendix B contains the detailed calculations.

6.3 Anchor Calculations

Soil properties for the resistive forces on the anchor were calculated using the Principles of Foundation Engineering text [5]. Dimensions for the anchors on both sides of the bridge were determined to be 10' long x 8' wide x 8' tall with a corner chamfer facing the towards the bridge. The calculations were separated into horizontal and vertical components to determine when the anchor would slide or be uplifted in the soil due to the tension from the main cable. The actual factor of safety for the dimensions of the anchors was calculated to verify that they were greater than our projected factor of safety in the horizontal and vertical directions, respectively. The active and passive forces that were used in verifying the stability of the anchors are shown in Figure 13. Due to the importance of the anchors supporting the cable, the targeted factor of safety used was a minimum of 2.0 for both sliding and uplifting forces. These calculations can be found in Appendix D.

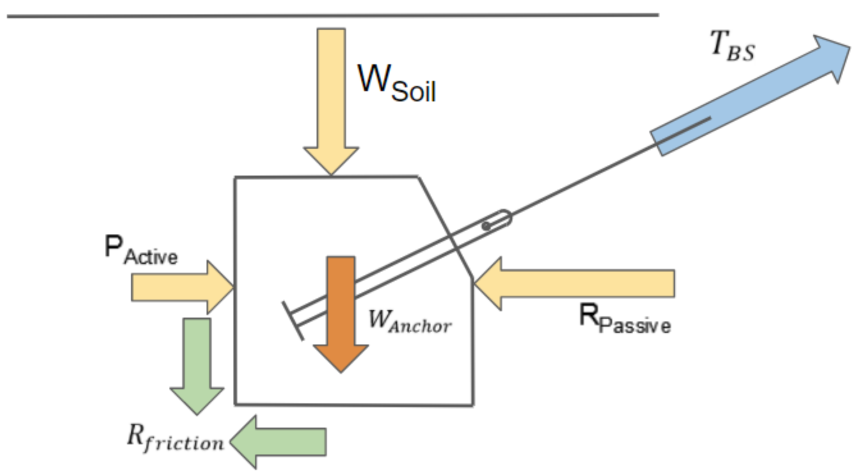

Figure 13: Forces acting on anchors

7 Cost Analysis

After conducting structural analysis and determining what materials would be required in the construction of a suspension bridge, the Better Bridges team was able to put together a detailed cost estimate. This estimate includes all the material components necessary to build the bridge from start to finish. Size and quantity of materials are based on calculations from structural analysis and templates from the 2016 Bridges to Prosperity Bridge Builder Manual, 5th Edition. The majority of material costs were found using rates from suppliers in the continental United States. The labor components are based on MDOT's construction labor rates for 2016 and RSMeans. The cost of labor is about the same as the overall materials cost of the bridge, using said rates. This means that if much of the labor was donated by members of the community, the total cost of the project could be reduced substantially. Some labor, such as a jobsite superintendent, will have to be paid for in order to ensure a professional level of construction. Material donations or reuse of an existing cable that is at the Bridges to Prosperity site in Caimital would also help lower cost to make the bridge more financially achievable.

The cost estimate was divided into material costs and labor costs. The total material cost was an estimated $47,000, total labor cost at an estimated $39,000, and equipment costs at roughly $4000. These figures result in the final total cost needed for construction of the bridge to be an estimated $90,000. Details for these estimates are outlined in Appendix E. Figure 14 illustrates a breakdown of the cost estimate.

Cost Breakdown

Walkway	$8,000
Anchors	$10,000
Cables	$20,000
Towers	$9,000
Labor	$39,000
Equipment	$4,000
Total	**$90,000**

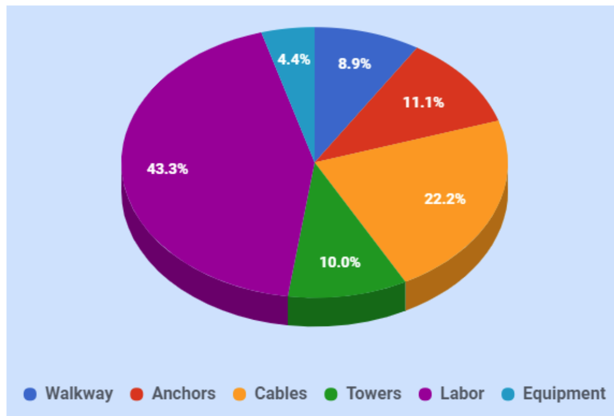

Figure 14: Cost Estimate Summary

8 Discussion

The Better Bridges team identified and compared constraints that influence both the site and style for the final bridge design. A critical factor for choosing the final site was the approach elevations on each side of the river. DHH site has more even approach levels and is therefore favorable because fewer materials and a simpler design are needed for a level walkway than at the Wadsworth site. A second reason for favoring the DHH site is that the bridge span is shorter. In order for the span of the Wadsworth site to be similar to DHH, a large foundation would need to be built to create a level surface, thus further increasing the cost. Finally, the clearance between the bottom of the bridge and the water level is greater at the DHH site than at the Wadsworth site. After all of these factors were considered, it was determined that the DHH site was most favorable.

After comparing variables to decide on the style of bridge to design, the Better Bridges design team favored the suspension style bridge. The biggest factor for this choice was the stability and strength of the bridge. Although the design for a suspension bridge requires more materials and is more expensive, the team concluded the design benefits outweighed the costs. Maximizing the clearance between the deck and the water level allows any debris or trees to pass under safely.

9 Conclusion & Recommendations

9.1 Conclusion

Compromises such as choosing a wooden deck over a steel deck, as well as larger anchors than first envisioned, had to be made once final calculations were completed. Recommendations for maintenance and reducing costs further are explained in section 9.2, below. The suspension bridge is located just down river of the current path which will provide easier access for construction. Shovels and machetes will be the main source for any clearing to the sites, but a bulldozer is assumed to be donated by the Panamanian government for any major clearing that may be needed. Members of El Hatillo should be the main source of labor with a certified supervisor to instruct construction methods. Resources such as lumber, cable, and connection assembly pieces should be available at the local hardware store in the city of Penonome.

9.2 Recommendations

Maintenance will be an important factor for maximizing the lifespan of the finalized bridge. Suspender cables and connections should be inspected monthly for any and all excessive and visual wear or corrosion. Main cables should also be inspected on the same monthly basis; checking the cable saddle points on the towers and the turn-back sections for wear or movement should be the priority. Connection points from the towers to the foundation should be monitored monthly for large cracks or signs of concrete pullout. Deck boards should be replaced once major cracking or wood deterioration occurs, and can be reported by bridge users. All parts, such as nuts and bolts, should be visually inspected twice a year for corrosion or displacement to keep the bridge in good working order.

Recommendations for reducing costs include utilizing materials such as the existing cable in the community. Better Bridges recommends that testing be done to ensure the strength properties can withstand the forces in the design of the bridge. It is also recommended that residents donate time towards the construction of the bridge, as this will significantly decrease the costs of the labor.

10 Acknowledgements

Better Bridges wants to acknowledge the efforts by those who guided the team throughout the data collection phase and design phase of the Rio Cabuya River Crossing Project.

During the data collection phase, Dr. David Watkins, PhD., P.E., helped Better Bridges in collecting appropriate survey data while in Panama. The team also wants to thank the active Peace Corp Volunteer, Julius Kellinghusen, for coordinating living arrangements, translating data gathered from the community, and for ensuring an overall positive experience for the Better Bridges team during our visit to El Hatillo.

During the design phase at Michigan Technological University, collaboration between International Senior Design Advisors, Mr. Michael T. Drewyor (P.E., P.S.) and Dr. David Watkins (PhD, P.E.) guided development for this proposal. Consultation is also acknowledged from Foundation Engineering Professor at Michigan Technological University, Zhen Liu (PhD, P.E.), on input and verification of assumptions within the anchor and foundation calculations. Special thanks to Environmental Engineering Masters Student and Water Resource TA, Ashley Hendricks for additional guidance navigating ArcMap and HEC-HMS.

11 References

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Appendix

Appendix A: Watershed Analysis

**Watershed analysis was performed using ArcMAP 10.4.1, HEC HMS 4.0, Google Earth Pro, and Excel 2013.

ArcMap 10.4.1:

A Digital Elevation Model (DEM) was downloaded using the earth explorer feature on the United States Geological Survey (USGS) Website [6] and is shown in *Figure 15*. The DEM file was then used to visualize hydrologic data and perform spatial analysis. Multiple hydraulic spatial analysis tools were used to find characteristics of the Rio Cabuya Watershed. Fill was used to fill in any missing data within the DEM. Flow Direction was used to determine which direction water flows within each cell. Flow accumulation was used to visualize the areas where runoff collects (rivers, streams, lakes, etc.). The Watershed feature was used to delineate the watershed, and find the area and perimeter of the watershed. A line feature was used to find the length of the main channel of the Rio Cabuya. The map created and all characteristics found using ArcMAP 4.0 are shown below in *Figure 16*.

Area: 6.99 mi^2
Max elevation: 2,290 ft
L = Length of main channel = 23861 ft
67% forested
33% open

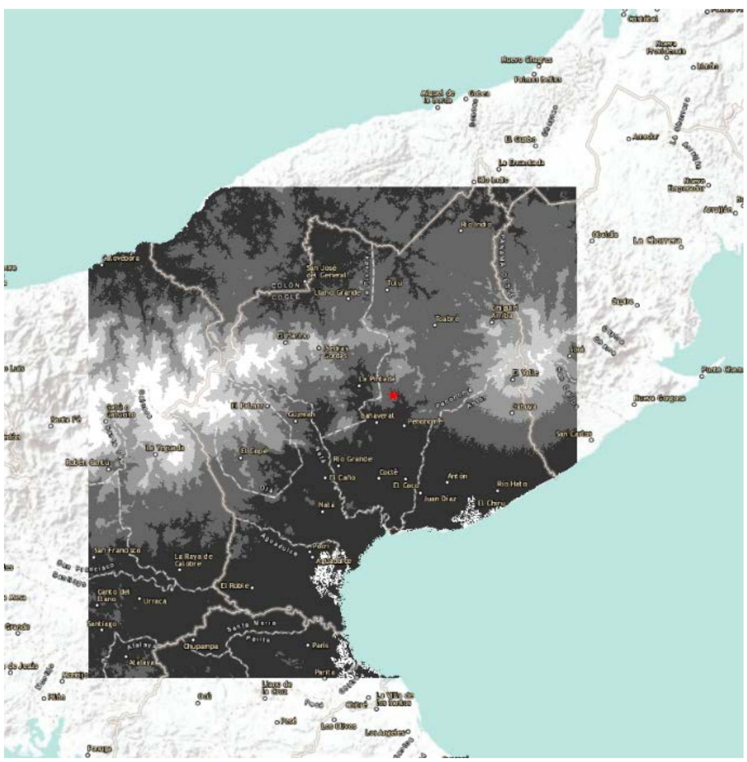


Figure 15 DEM file over a terrain baseman. Red Star indicates bridge location.

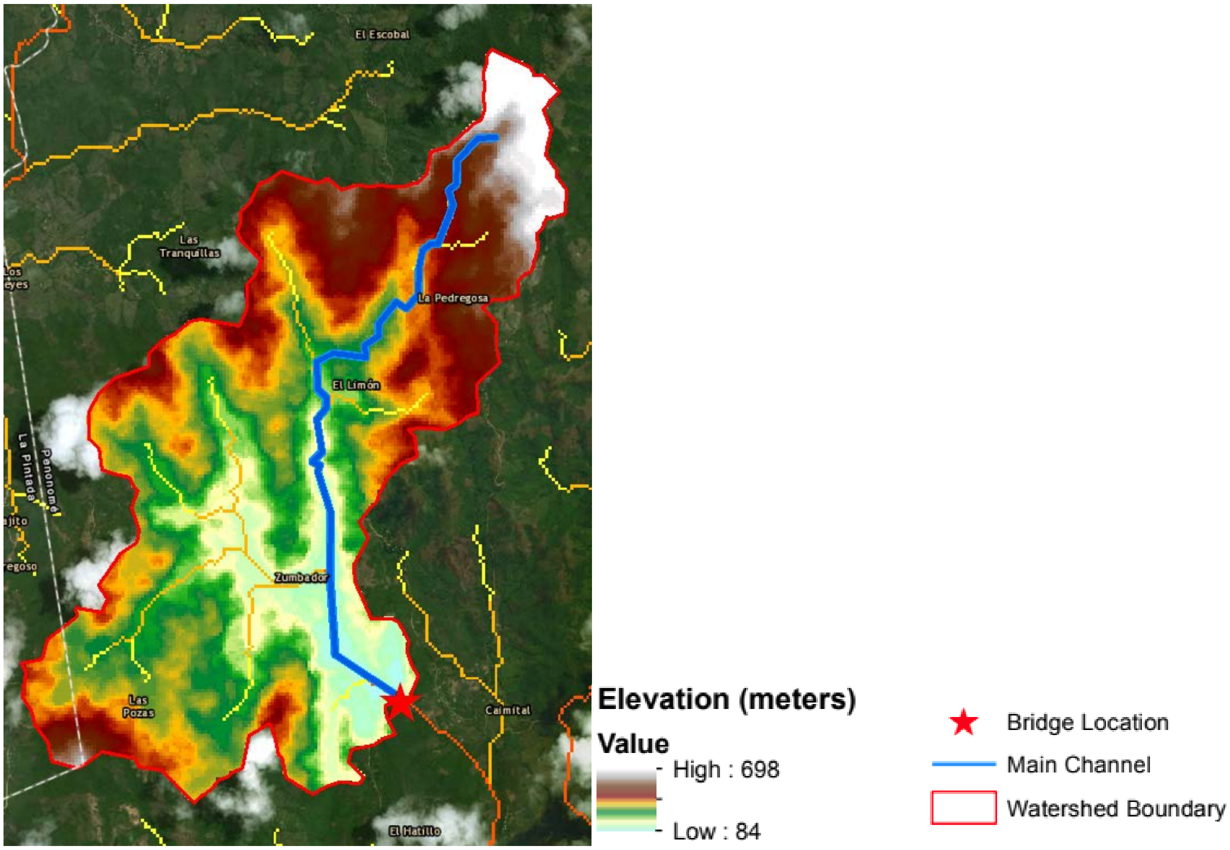


Figure 16: Rio Cabuya upstream watershed

Google Earth Pro

To find the slope of the watershed Google Earth Pro was utilized. Following GPS coordinates from ArcMAP, high and low points on the watershed boundary were located. The points were connected to the bridge location and the path was used to create an elevation profile. The Google Map and an example elevation profile can be seen in *figure 17 and figure 18*. Some elevation profiles were gradual as shown in *figure 17*, while others had larger hills and deeper valleys. Of all paths, the highest percent slope was 13% while the lowest was 2%

Figure 17: Elevation profile of path from Rio Cabuya Bridge site to Point 1 on Google Map

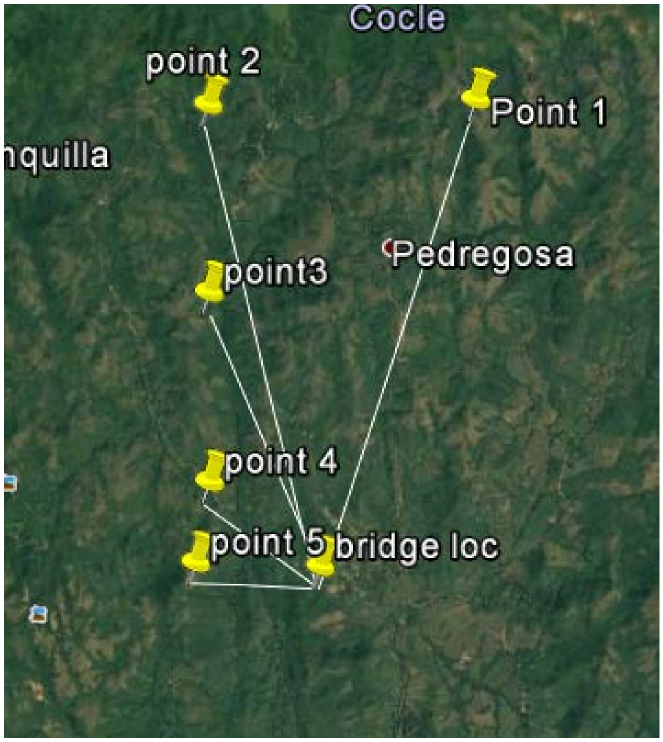


Figure 18: Google Earth Map of 5 points and their connecting paths

The slope of each elevation profile was found using the equation shown below. The estimated average slope of the entire watershed was found by taking the weighted average of the 5 paths, where the weight was proportional to the length of each segment. This was done using the SUMPRODUCT function within excel. The weighted average of the entire watershed was found to be 5.5%. Excel calculations can be found in table 2.

$$Percent\ slope = \frac{Height_2 - Height_1}{Length\ of\ segment} * 100$$

Table 2: Excel Slope Calculation

Cross section	Elevation at bridge	Elevation at peak	Distance from bridge	Point 2-Point 1	Slope	Average	Weighted Average
1	358	1520	15889	1162	7%	6%	5.5%
2	358	1010	15206	652	4%		
3	358	553	9293	195	2%		
4	358	510	4540	152	3%		
5	358	901	4118	543	13%		

HEC-HMS 4.0:

Hydrologic Engineering Center – Hydrologic Modeling System (HEC-HMS 4.0) was used to simulate a design storm based off of the Dec 7-8, 2010 major flood event in Panama City, Panama [2]. The total rainfall for the event was 10.87 inches. Before a model is made the Initial Abstraction (I_A), Max Soil Retention (S), and the lag time (L_t) must be calculated using known equations from Wurbs [3]. The equations and values are shown below.

$$I_A = 0.2 * Soil\ Retention$$

$$S = \frac{1000}{CN} - 10$$

$$L_t = L^{.08} \frac{(1000 - 9 * CN)^{0.7}}{1900 * CN^{.07} * \sqrt{Y}}$$

I_A = Initial Abstraction

L_t= Lag time

S = Max Soil Retention

Initial Abstraction (I_A) = 0.66 inches
Max Soil Retention (S) = 3.28 inches
Lag time (L_t) = 113 minutes
L = Length of main channel = 23861 ft

The Runoff curve number (RCN) is estimated based on soil type and land cover. Our watershed was determined to be type C soil, or soil with sandy clays, some fines, and low infiltration. The land cover was determined to be 33% open (pasture/range) with low ground cover, and woodland with high ground cover. The RCN values for these conditions are listed in table 8.3 of Wurbs [3]. The total RCN value for the whole watershed was found using the equation below.

$$RCN = (0.33) * (86) = (0.67) * (70)$$

RCN value for group C soil with pasture/range land use = 86
RCN value for group C soil with pasture/range land use =70

The components added to the model were a sub basin, a reservoir, and a reach. The sub basin represents the watershed, the reach represents the river channel, and the reservoir represents the bridge location. After the three elements of the model are created the watershed characterizes were applied to the sub basin. The characteristics included the area, Initial Abstraction, the Lag time, and the % impervious service. The % impervious surface was decided to be 1%, as this is a remote region with no developed roads or parking lots. The only cement found is for the foundation of the homes within the watershed. The loss method selected is the SCS method (the same as NRCS method), and the transform method selected is an SCS Unit Hydrograph.

Next a Meteorological (Precipitation) Model is built. The model chosen is an SCS type II event and is designed as a 100-Year Design storm, 24 hour rainfall event. The amount of precipitation is inputted as 10.87 inches. The start date was set for December 1st, simulating a similar time frame as the December 2010 major storm event in Panama. An image of the model is shown in *figure 18*. The model was run, and the results are shown in *figure 19* and *figure 20*. Peak Discharge at the bridge site was found to be 7990 cubic feet per second, with peak flow occurring after 14 hours.

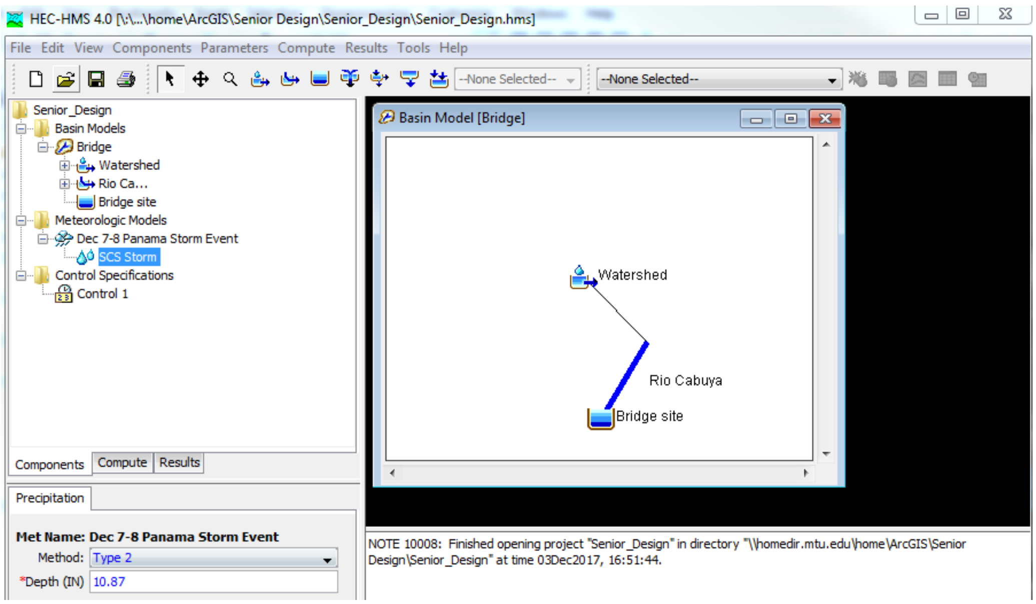


Figure 18: Design Storm HEC-HMS model

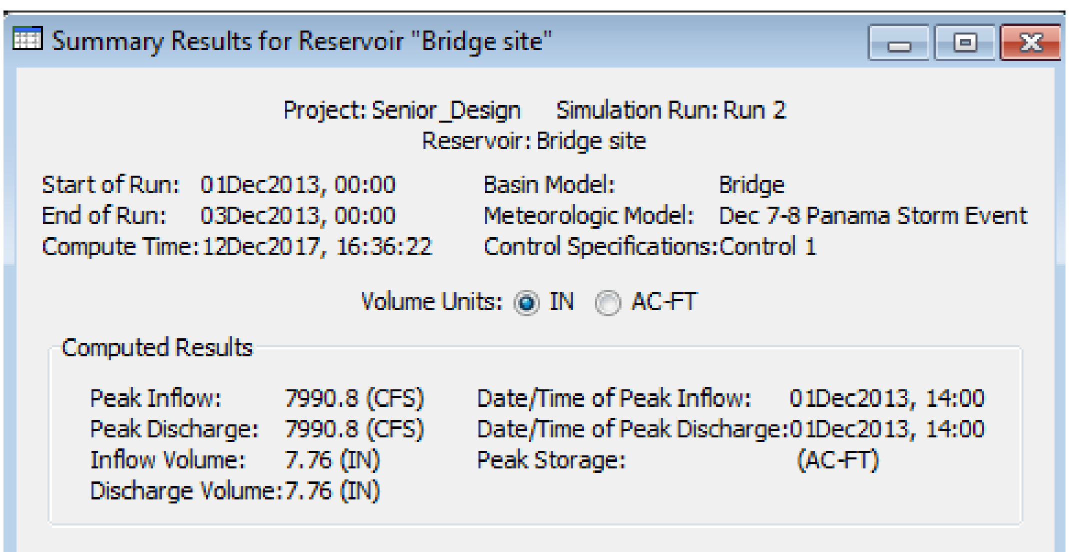


Figure 19: Summary table of design storm

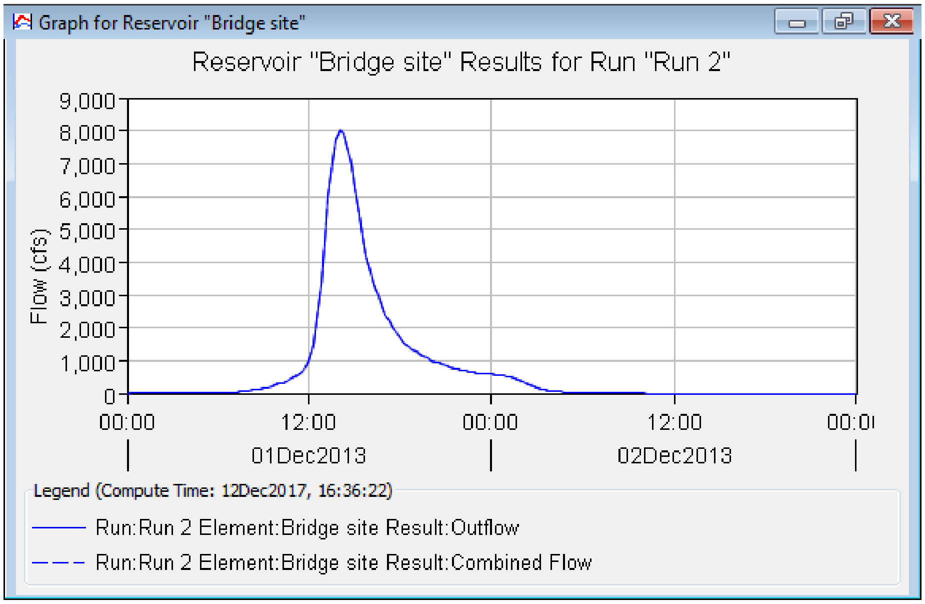


Figure 20: Unit Hydrograph of design storm

Depth Calculations:

Max water depth was found trough a trial and error method. Manning's Equation was applied to different depths over our cross-section until the discharge rate was equal to our max discharge found using the HEC-HMS model. The depth found that satisfied the equation was determined to be our max water level during a major storm event. Excel calculations can be found in table 3.

$$Manings\ equation = Q = \frac{C_m}{n} * A * R^{\frac{2}{3}} * S^{\frac{1}{2}} = S^{\frac{1}{2}} * k$$

$$Manings\ equation = Q = \frac{1.49}{.045} * 772.4 * 9.28^{\frac{2}{3}} * 0.005^{\frac{1}{2}} = 0.005^{\frac{1}{2}} * 96711 = 7990 \text{ cfs}$$

C_m = 1.49 for US Standard Units

A = Cross Sectional Area (ft^2)

R = hydraulic Radius = $\frac{A}{wp}$ (ft)

WP = Wetted Perimeter = Perimeter-width of channel (ft)

S = Slope = .005 ft/ft (or 0.5%)

n = Manning's Coefficient = 0.045 for 49% cobble, and 51% brush and vegetation

Q = Discharge =7990 $\frac{ft^3}{s}$

Table 3: Excel Depth Calculations

Trial	Depth	Area	Perimeter	Width	Wetted perimeter	hydraulic radius (R)= (A/WP)	k	Q
	(ft)	(ft^2)	(ft)	(ft)	(ft)	(ft)		(ft^3/s)
1.00	12.00	479.47	124.51	53.47	71.05	6.75	56693.88	4008.86
2.00	10.00	373.00	118.59	52.00	66.59	5.60	38953.74	2754.45
3.00	11.00	425.37	121.56	52.70	68.86	6.18	47417.70	3352.94
4.00	15.00	642.17	133.46	55.67	77.79	8.26	86850.28	6141.24
5.00	17.00	755.00	140.30	57.60	82.70	9.13	109198.08	7721.47
6.00	17.25	769.50	141.12	58.01	83.11	9.26	112348.58	7944.24
7.00	17.30	772.44	141.33	58.10	83.23	9.28	112952.30	7986.93

Max Depth = 17.3 ft

Appendix B: Overall Bridge Calculations

Main Span Load Calculations

$Span\ (L) = 160ft$

$Camber\ (c_d) = 0.03 \cdot L = 4.8ft$

$Design\ Sag\ (S) = \frac{L}{11} = 14.54ft$

$Angle\ to\ Horizontal\ (\theta) = \tan^{-1}\left(4 \cdot \frac{S}{L}\right) = 20°$

$Deck\ Width\ (w_d) = 3ft$

$Suspended\ Support\ Width\ (w_s) = 4ft$

Table 4: Material List and weights

Part	Weight per unit	Unit	Quantity	Total Weight (lbs)
L 4"x4"x1/2" - 4ft long	51.2	lb	78	3993.6
2"x8"x4' Wood deck (3ft wide)	4	lb/ft	640	2560
Connector plate 10"x4'x1/2"	68	lb	39	2652
Nailer	5.2	lb/ft	156	811.2
Main Cable 1 5/8"	1153.632	ea.	2	2307.264
Suspender Cable 3/8"	.26 lb/ft	ea.	346.8 ft	90.168
Bolts	1.45	lb	156	226.2
Eye bolts	0.92	lb	78	71.76
Fence	1.5	lb/ft	320	480
TOTAL				13192.2

The materials and their weights were added and used to calculate the dead load of the bridge are shown in table 4, above.

$DL = 13{,}193lbs$

$LL = 90psf \cdot 3ft \cdot 160ft = 43{,}200lbs$

$Working\ Load = DL + LL = 56{,}393lbs$

$Highest\ Ultimate\ Load\ Case = 1.2(DL) + 1.6(LL) = 84{,}951.6lbs$

$Load\ per\ Support = \frac{56{,}393lbs}{39} = 1.45kips \approx 1.5kips$

$Load\ per\ Suspender\ Cable = \frac{1.5kips}{2} = 0.75kips$

$Horizontal\ Tension\ (T_H) = \frac{56{,}393lbs \cdot L}{8 \cdot S} = 77{,}570lbs$

$Total\ Cable\ Tension\ (T_{total}) = \frac{T_H}{\cos(\theta)} = 82{,}548lbs$

$Vertical\ Cable\ Tension\ (T_v) = T_{total} \cdot \sin(\theta) = 28{,}233lbs$

$1\frac{5}{8}"\ Cable\ Ultimate\ Strength = 132\ tons = 264{,}000lbs$

$Cable\ Factor\ of\ Safety\ (FS_c) = 5.0$

$$Number\ of\ 1\frac{5}{8}"\ Cables\ (N_c) = \frac{T_{total} \cdot 5.0}{264{,}000lbs} = 1.56 \rightarrow 2\ cables\ total$$

$$\frac{3}{8}"\ Suspender\ Cable\ Breaking\ Strength = 7.55\ tons = 15{,}100lbs$$

$$Suspender\ Cable\ Factor\ of\ Safety = \frac{15{,}100lbs}{750lbs} = 20.1 > 5.0$$

Backstay and Tower Loads

$$Cable\ Angle\ to\ Horizontal\ (\alpha) = 26.56° \left(slope\ of\ \frac{2}{1}\right)$$

$$Total\ Backstay\ Tension\ \left(T_{BS_{total}}\right) = \frac{T_H}{\cos(\alpha)} = 86{,}722lbs$$

$$Vertical\ Backstay\ Tension\ \left(T_{BS_v}\right) = T_{BS_{total}} * \sin(26.56°) = 38{,}777lbs$$

$$Anchor\ Factor\ of\ Safety\ (FS_a) = 2.0$$

$$Concrete\ Weight = 4000\frac{lbs}{CY}$$

$$Concrete\ Needed\ to\ Resist\ T_{BS_v} = \frac{58{,}426lbs}{4000lbs} = 14.6CY \cdot 2.0 = 19.4CY\ of\ concrete$$

$$Total\ Vertical\ Reaction\ at\ Towers\ (F_{wv}) = T_{BS_v} + T_v = 67{,}010lbs$$

$$6"\ SCH40\ Pipe\ Area\ of\ Steel = 5.58in^2 \qquad Yield\ Strength = 36{,}000psi$$

$$Compression\ Strength = 36{,}000psi \cdot 5.58in^2 = 201{,}000lbs$$

$$FS = \frac{201{,}000lbs \cdot (2\ towers)}{67{,}010lbs} = 6.0 > 3.0 \quad \checkmark\ O.K.$$

Wind Loads

$$Deck\ Wind\ Load\ (WL_d) = \frac{1.22ft^2}{4ft\ section} \cdot 39\ sections = 47.7ft^2 \cdot 20\frac{lbs}{ft^2} = 953.3lbs$$

$$Main\ Cable\ Wind\ Load\ (WL_{mc}) = 236ft \cdot \left(\frac{1.625in}{12}\right) = 32.0ft^2 \cdot 20\frac{lbs}{ft^2} = 640lbs$$

$$Suspender\ Cable\ Wind\ Load\ (WL_{sc}) = 347ft \cdot \left(\frac{0.375in}{12}\right) = 10.84ft^2 \cdot 20\frac{lbs}{ft^2} = 216.8lbs$$

$$Total\ Wind\ Load\ (WL_{total}) = WL_d + WL_{mc} + WL_{sc} = 1810.1lbs$$

Cable to Anchor Attachment Piece

$$Steel\ Area\ Needed = \frac{T_{BS_{total}}}{36ksi} = 3.7in^2 \cdot 2.0 = 7.4in^2 \qquad 6in \cdot (3 \cdot .5in) = 9in^2 > 7.4in^2$$

Appendix C: Foundation Calculations

Compare areas needed for calculated force exerted on the soil:

$$Working\ Vertical\ Load\ at\ Towers\ (F_{wv}) = T_{BS_v} + T_v = 38{,}777lbs + 28{,}233lbs = 67{,}000\ lbs$$

$$Weight\ of\ Towers\ (W_t) = 1630lbs$$

$$Weight\ of\ Concrete\ Foundation\ (W_f) = 4ft \cdot 6.5ft \cdot 6.5ft \cdot 150\frac{lbs}{ft^3} = 25{,}350lbs$$

$$Forces\ on\ Soil\ (F_s) = W_f + W_t + F_{wv} = 93{,}990lbs$$

$$Assume\ working\ bearing\ capacity\ of\ soil\ (q)\ is\ the\ pressure\ on\ the\ soil\ (P)\ and\ is\ equal\ to\ 3500psf$$

$$A = \frac{F}{q} \qquad A = \frac{94{,}000\ lbs}{3{,}500\frac{lb}{ft^2}} \qquad A = \mathbf{27\ ft^2}\ \text{minimum}$$

Solving for A, we need 26.9 ft^2 of concrete to be safe. The design contains 6.5 ft·6.5 ft=**42 ft^2** of concrete, **the design dimensions are acceptable.**

OR:

Compare bearing capacity of soil to the pressure exerted by the foundation onto the soil:

$$Pressure\ on\ the\ soil, P = \frac{F}{A} = \frac{94{,}000\ lbs}{42\ ft^2} = 2{,}225\frac{lbs}{ft^2}$$

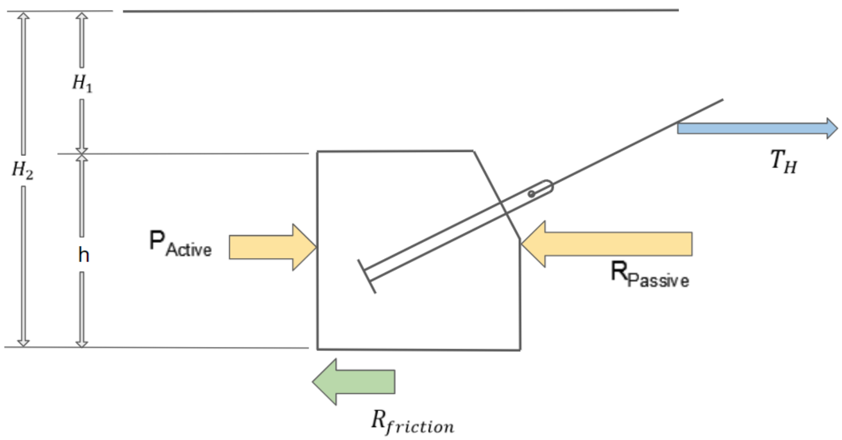
$$SF = \frac{q}{P} \qquad SF = \frac{3{,}500\frac{lbs}{ft^2}}{2{,}225\frac{lb}{ft^2}} = \mathbf{1.57} \therefore \boldsymbol{Acceptable}$$

Appendix D: Anchor Calculations

Anchor Beam Sliding Check

Horizontal Forces must satisfy that the resisting forces will be greater than the driving forces with the relationship:

$$R_S < \frac{R_R}{FS}$$



Where:

R_S = horizontal driving force

R_n = horizontal resisting force

FS = factor of safety = **2**

Figure 21: Free Body Diagram of Anchor with horizontal forces

Horizontal Driving Forces:

$$R_S = P_{Active} + T_H$$

$$K_a = \frac{1 - \sin(28°)}{1 + \sin(28°)} = 0.361$$

$$P_{Active} = \frac{1}{2}(K_a)(\gamma)(h^2)(w) = \frac{1}{2}(0.361)(115\frac{lb}{ft^3})(8\,ft)^2(10\,ft)$$

$$= 13{,}284.8 \text{ lbs} = 13.3 \text{ kips}$$

$$R_S = 13.3\,lbs + 77.6\,kips = \mathbf{91\ kips}$$

Where:

K_a= active earth pressure coefficient

γ = soil density = 115 lb/ft^3

h = anchor height = 8 ft

w = anchor width = 10 ft

Ø = angle of internal friction = 28 deg

Horizontal Resisting Force: $R_R = R_{friction} + R_{Passive}$

$$W_{soil} = \gamma * Volume = \left(115\frac{lb}{ft^3}\right) * 147\,ft^3 = 16{,}905 lb = 16.9\,kips$$

$$K_p = \frac{1 + \sin(28°)}{1 - \sin(28°)} = 2.77$$

$$R_{friction} = \mu * (T_H + W_{soil})$$

$$= \left[\tan\left(\frac{3}{4}\emptyset\right)\right] * (77.6\ kips + 16.9\ kips) = 36.3\ kips$$

$$R_{Passive} = \frac{1}{2} K_p \gamma \left(\frac{3}{2}h\right)^2 w(H_2 - H_1)$$

$$= \frac{1}{2}(2.77)\left(115\frac{lb}{ft^3}\right)\left(\frac{3}{4} * 8\ \ ft\right)^2 [10\ ft(13.65\ ft - 1.64\ ft)] = 230\ kips$$

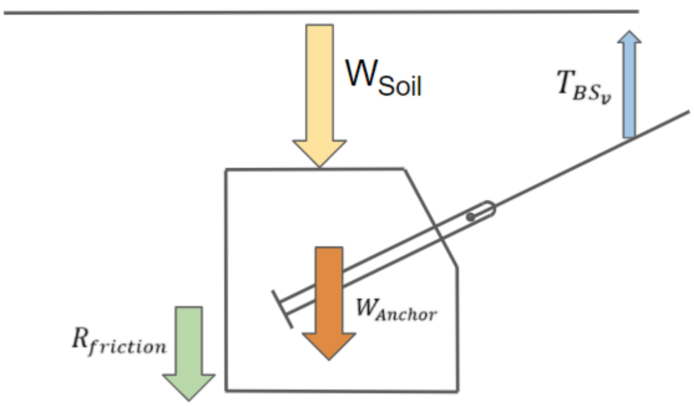
$$R_R = 36.3\ kips + 230\ kips = \mathbf{266.3\ kips}$$

Must Satisfy $R_S < \frac{R_R}{FS}$

$$FS = \frac{R_R}{R_S} = \frac{\mathbf{266.3\ kips}}{\mathbf{91\ kips}} = \mathbf{2.9} \ \therefore \boldsymbol{Acceptable}$$

Anchor Beam Uplift Check

Vertical Forces must satisfy that the resisting forces will be greater than the driving forces with the relationship:



$$T_{BS_v} < \frac{V_R}{FS}$$

Where

$T_{BS_v} = Vertical\ Driving\ Force = 38.8\ kips$

And, $V_R = Vertical\ Resistant\ Force$

Figure 22: Free Body Diagram of Anchor with Vertical forces

$$W_{Anchor} = 150\frac{lb}{ft^3} * \ 34.2\ ft^2 * \ 10\ ft = 51.3\ kips$$

$$V_R = R_{friction} + W_{Anchor} + W_{soil} = 36.3\ kips + 51.3\ kips + 17\ kips = \mathbf{104.6\ kips}$$

$$FS = \frac{V_R}{T_{BS_v}} = \frac{\mathbf{104.6\ kips}}{\mathbf{38.8\ kips}} = \mathbf{2.7} \therefore \boldsymbol{Acceptable}$$

Appendix E: Cost Estimate

Table 5: Cost Estimate

Item	Material Cost	Labor Cost	Equipment	Total
Clear and Grub		$2,391.37	$800.00	$3,191.37
Excavation		$1,888.70		$1,888.70
Anchors	$9,401.16	$12,569.77		$21,970.93
Tower Foundations	$1,290.12	$4,189.92		$5,480.04
Towers	$7,508.02	$1,224.15		$8,732.17
Cables	$19,578.98	$597.66	$200.00	$20,376.64
Walkway	$7,488.74	$4,317.02		$11,805.76
Erosion Control	$1,206.00	$647.55		$1,853.55
SuperIntendant		$9,000.00		$9,000.00
Misc Tools/Operations			$2,800.00	$2,800.00
Totals	**$46,473.02**	**$36,826.14**	**$3,800.00**	**$87,099.16**
				$88,000

Appendix F: Construction Schedule

	Task Mode	Task Name	Duration	Start	Finish	Predecessors
1		Mobilization	3 days	Sat 12/30/17	Tue 1/2/18	
2		Clear and Grub East Bank	2 days	Wed 1/3/18	Thu 1/4/18	1
3		Clear and Grub West Bank	2 days	Fri 1/5/18	Sat 1/6/18	2
4		Excavate East Foundation	2 days	Fri 1/5/18	Sat 1/6/18	2
5		Excavate West Foundation	2 days	Fri 1/12/18	Sat 1/13/18	6,3
6		Excavate East Cable Anchor	3 days	Mon 1/8/18	Thu 1/11/18	4
7		Excavate West Cable Anchor	3 days	Mon 1/15/18	Wed 1/17/18	5
8		Form and Pour East Foundation	3 days	Mon 1/8/18	Thu 1/11/18	4
9		Form and Pour West Foundation	3 days	Mon 1/15/18	Wed 1/17/18	5
10		Form and Pour East Anchor	4 days	Sat 1/13/18	Wed 1/17/18	6
11		Form and Pour West Anchor	4 days	Fri 1/19/18	Tue 1/23/18	7
12		Assemble and Errect East Tower	3 days	Sat 1/13/18	Tue 1/16/18	8
13		Assemble and Errect West Tower	3 days	Thu 1/18/18	Sat 1/20/18	9
14		Set Cables	3 days	Wed 1/24/18	Fri 1/26/18	10,11,12,13
15		Construct Deck	5 days	Sat 1/27/18	Thu 2/1/18	14
16		Install Fencing	2 days	Fri 2/2/18	Sat 2/3/18	15
17		Demobilize	6 days	Mon 2/5/18	Sat 2/10/18	16

Figure 23: Detail Schedule Tasks and Durations

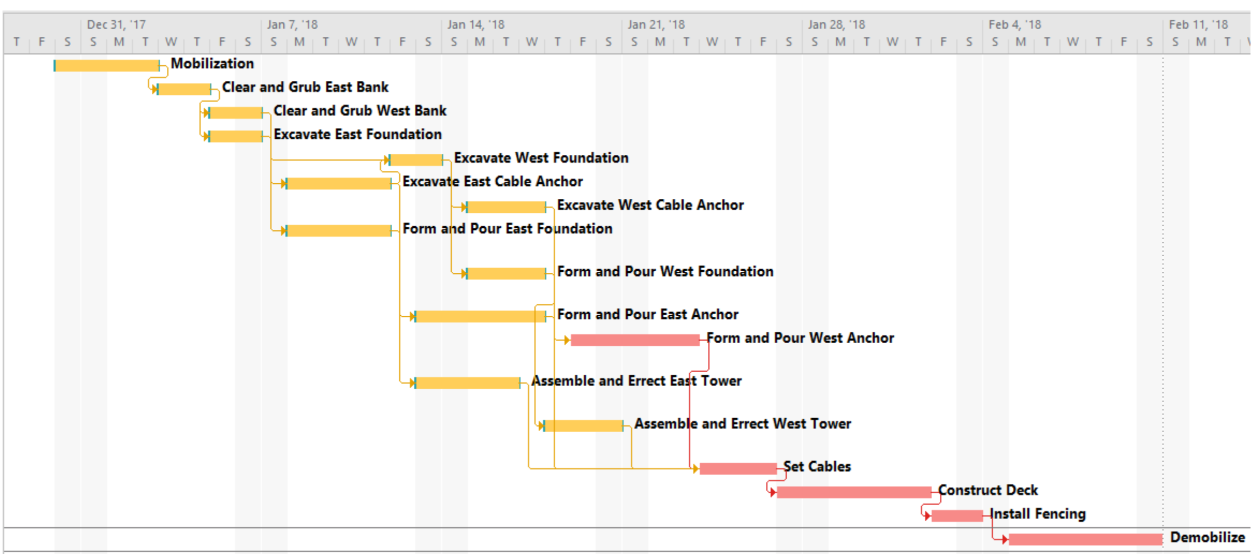


Figure 24: Roll-Up Schedule of Tasks to show critical tasks

Appendix G: Drawings