A Bridge to the Comarca:
Chucunaque River Footbridge

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This report, titled "A Bridge to the Comarca: Chucunaque River Footbridge", 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 FOOTBRIDGE UNLESS PLANS HAVE BEEN APPROVED BY A PROFESSIONAL ENGINEER.**

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1.0 Executive Summary

This report includes the final design, technical drawings, construction schedule, and cost estimate for a suspension bridge to be located at Puerto Limón, Panama. This port is one of the primary access points into the Embera-Wounaan Comarca in the Darién Province of eastern Panama. During the rainy season this Comarca is only accessible by motorized, dugout canoes. A temporary bridge is built during the dry season at this location; however, it only lasts a few weeks to three months before being washed away by high waters.

An assessment trip was taken in August 2013 to determine the need for a crossing over the Chucunaque River. Interviews were conducted with members of the community of Alto Playón, one of six communities in the Comarca, to understand the potential uses and benefits for a river crossing. It was determined that a suspension bridge would be the most feasible solution for this location. The design was completed according to Panamanian and International Building Codes. Technical drawings that highlight main components and correlating details of the design are included as the final component of this report.

The cost of constructing this bridge was estimated using Panamanian materials, equipment, and labor costs. The suspension bridge will take two dry seasons (mid-December to early March) to be constructed. The slopes and foundations will be completed during the first dry season, and the towers, cables, and decking will be completed during the second dry season.

Financial support for the project will need to come from organizations outside of the Embera-Wounaan Comarca. The non-profit organizations Bridges to Prosperity and Engineers Without Borders-Panama are two potential supporters for the project. The communities in the Comarca will also need to form a bridge committee that will be in charge of monitoring and repairing the suspension bridge.

2.0 Introduction

Del Puente Engineering presents the following report, which includes the analysis and design of a suspension footbridge across the Chucunaque River in Eastern Panama for the six communities of the Embera-Wounaan Comarca. A Comarca is an administrative region specifically for the indigenous people of Panama. These communities are discussed in greater detail in Section 3.0. A design team was sent on an assessment trip in August 2013 to discuss the existing river crossing and expectations for a permanent crossing with community members and leaders. It was decided that providing pedestrian and equestrian access at Puerto Limón would be the most beneficial option.

Currently, the only way to access this Comarca during nine to ten months of the year, Panama's rainy season, is by motorized dugout canoes. The limited capacity of the canoes and high cost of gasoline restrict the size of loads and frequency of trips out of the Comarca. Community members currently only use the dugout canoes during the day and members can become stranded at the port after dark. They must either wait until the morning or create their own rafts in order to float downstream. There are crocodiles in this portion of the Chucunaque River, which makes the trip even riskier. A permanent bridge would provide a much safer river crossing.

A bridge would also provide an opportunity for economic expansion. The members of this Comarca are primarily subsistence farmers. Many of the community members stated that they would produce a surplus crop if they could transport it to markets in neighboring towns and if transportation was more economical. Additionally, high school students could continue to live with their parents and either walk or ride a bike to the nearest school. This would make continued education much more affordable because the families would not have to pay to board their children at the school or for the gas required to travel back and forth by canoe.

Puerto Limón is the desired location for a bridge on the Chucunaque River as described in Section 4.0. Local politicians fund the construction of a simple vehicular bridge during the dry season at this port. Local trees are cut down by the community members and moved to span the river at the port. A packed gravel path is then put into place to serve as a driving surface for vehicle, pedestrian, equestrian, and bicycle traffic. This temporary bridge will last anywhere from a few weeks to three months before being washed downstream.

The travel team of Del Puente Engineering surveyed the proposed crossing location to complete an alternatives analysis for each potential river crossing option. These surveying methods are further discussed in Section 5.0 and the initial data is compiled in Appendix A. After it was determined that a suspension bridge would best meet the community's needs, the team developed an appropriate design following the procedures outlined in Section 6.0. Calculations are included in Appendix E to Appendix K, and the accompanying technical drawings are shown in Appendix L. After the final design was completed, a cost estimate and construction schedule were developed as shown in Section 6.2. Final recommendations can be found in Section 6.3.

3.0 Community Background

The Embera-Wounaan Comarca is located in the Darién Province of eastern Panama, and is composed of approximately 2,000 people living in six communities. Alto Playón, one of the six communities, is located on the Chucunaque River six miles northeast of the town of Metetí, approximately 150 miles east of Panama City on the Inter-American Highway. Alto Playón was founded in 1987 by three brothers, Florentino, Estereo, and Elpidio Chanapí, who rotate leadership roles as the heads of the community. They moved from one of the other communities with sixteen people because they needed more land for their farms. The community has since grown to 262 people, 117 of which are under the age of 15 years old. Alto Playón is still a growing community and is expecting ten new families to move in during the next dry season.

Figure 1: Team Members and Location of Alto Playón

Alto Playón is the closest community to Puerto Limón and is representative of the communities in the Comarca. It consists mainly of Embera families (Panamanian descent), with two Wounaan families (Columbian refugees). There are 57 houses in the community which are arranged in two circles - one surrounding the soccer field and the other surrounding the basketball court. Their houses are open-air houses that are elevated on stilts to protect from wild animals and flooding. These homes typically have slab floors and half-walls, both made out of wood, with a thatch or zinc corrugated steel roof. In Alto Playón there are two small stores, two cantinas, and a school that teaches through ninth grade. Students must travel to Metetí for further schooling.

Figure 2: Typical Home in Alto Playón

The people of Alto Playón are mainly subsistence farmers. They sell some of their crops outside the community in Metetí or Panama City, but only if they have the money to travel to these cities. Some of their farms are a two-hour hike away, while some are located directly behind their house. They grow crops such as potatoes, corn, rice, plantains, oranges, zapotes, and coffee.

Figure 3: Alto Playón and the Chucunaque River

The community members do not have running water, a water purification system, or a sanitation system. As a result, the Chucunaque River is used for drinking water, bathing, laundry, defecation, and trash disposal. Some homes have rain catchment containers for clean water, but the storage tanks are often open to contamination from the environment. If the rain catchment systems run out of water, community members will resort to drinking water from the Chucunaque River. There are four composting latrines within the community for sanitation purposes. The current Peace Corps volunteer, Amber Naylor, has been working on raising funds for building more. Alto Playón also has no electricity aside from a few generators for the stores and cantinas.

4.0 Project Location

Puerto Limón is a small port which serves as an entry point to the Embera-Wounaan Comarca. It is thirty minutes away from the town of Metetí by truck down a muddy, dirt road. The only way for the people of the six communities to leave during the rainy season is by motorized dugout canoe, which can often be a very long ride. One community has a four-hour boat ride, but with a bridge at the port, they could walk to their community in 45 minutes. The temporary bridge that is built during the dry season allows for travel into and out of the Comarca but does not always last very long - sometimes as little as three weeks. Since the members of this Comarca often have to travel into the city, their travel can become quite difficult during the rainy months after the temporary bridge has washed away. Del Puente Engineering plans to aide these communities with the development of a footbridge in order to provide safer, easier, cheaper, and faster travel year-round.

Figure 4: Puerto Limón

Puerto Limón is the site at which the design team proposes to develop a bridge. This site was chosen for a potential bridge because there is already infrastructure in place to accommodate a bridge. This is also the only feasible location to put a bridge because it is the only land in the area that is publicly owned and available for development. All other surrounding land is privately owned and would not be able to be acquired for construction.

5.0 Methods and Procedures

5.1 Surveying

The site survey at Puerto Limón was completed over a number of days while the design team was in Panama. Rough locations for each end of the bridge were first selected based on factors such as relative elevations within the appropriate property boundaries, signs of erosion, and relative span distance. An initial benchmark was then staked and the rough location obtained with a GPS unit. A compass was also used to obtain an initial bearing from this benchmark.

A topographic survey was conducted using a transit with stadia lines, Vernier's, and an optical plummet along with a level rod. Point data was gathered for approximately 160 points, which can be seen in Appendix A. This point data was gathered in a rough grid on both sides of the Chucunaque River around the bridge site, and additional data was gathered for points of interest, such as existing structures and steep embankments. Several pictures of the site were taken for later reference. This data was then entered into Carlson Civil Suite Software and used to create the site topographic map in Drawing 2 of Appendix L.

5.2 Soil Classification

The soil at Puerto Limón was classified by two methods. First, it was classified according to the Unified Soil Classification System (USCS) for geotechnical uses. The design team took a sample of the soil at the bridge site, performed a visual classification, and determined the relative strength of the soil. According to USCS, the soil where both abutments will be constructed can be classified as reddish-brown clay, with little fine sand, little small gravel, and slight organic matter, well graded, moist, CL (Holtz). Second, the design team classified the soil according to its hydrologic soil groups in order to estimate the runoff curve number for the region. The soil was determined to be in hydrologic soil groups C and D because the soil has slow to very slow infiltration rates and mainly consists of clay soils which impede the downward movement of water (Sorrell). The land use in the area is mainly woods or jungle and row crop for subsistence farming and the moisture condition is wet (AMC III). The runoff curve number was determined to be about 75 based on the analysis included in *Computing Flood Discharges for Small Ungaged Watersheds.* The design team is aware that this analysis is fairly inaccurate and only provides a rough estimate because the Chucunaque Watershed is very large.

5.3 River Flow Rate

The Chucunaque River is 134 miles in length, making it the longest river in Panama. The source of the Chucunaque River is near Cerro Grande, in the north of the Darién region. It is a tributary of the Tuira River, the largest river in the country, and together with the Tuira and Balsas Rivers, forms the Chucunaque Watershed. The Chucunaque Watershed is the largest in Panama at 4118 square miles [5].

The design team was not able to obtain adequate flow data while in Panama to aid in determining the 100-year flood line. However, the community members told the design team that the highest level they could remember the Chucunaque River reaching was at 112 ft elevation on the site survey, which occurred in 2010. Rainfall data from the flood in Panama in December was collected by *Servir Mesoamerica* [10] and verified that this flood was adequate to then model the 100-year flow and flood line. The precipitation map below is from December 2010 and is one example of the verifying data.

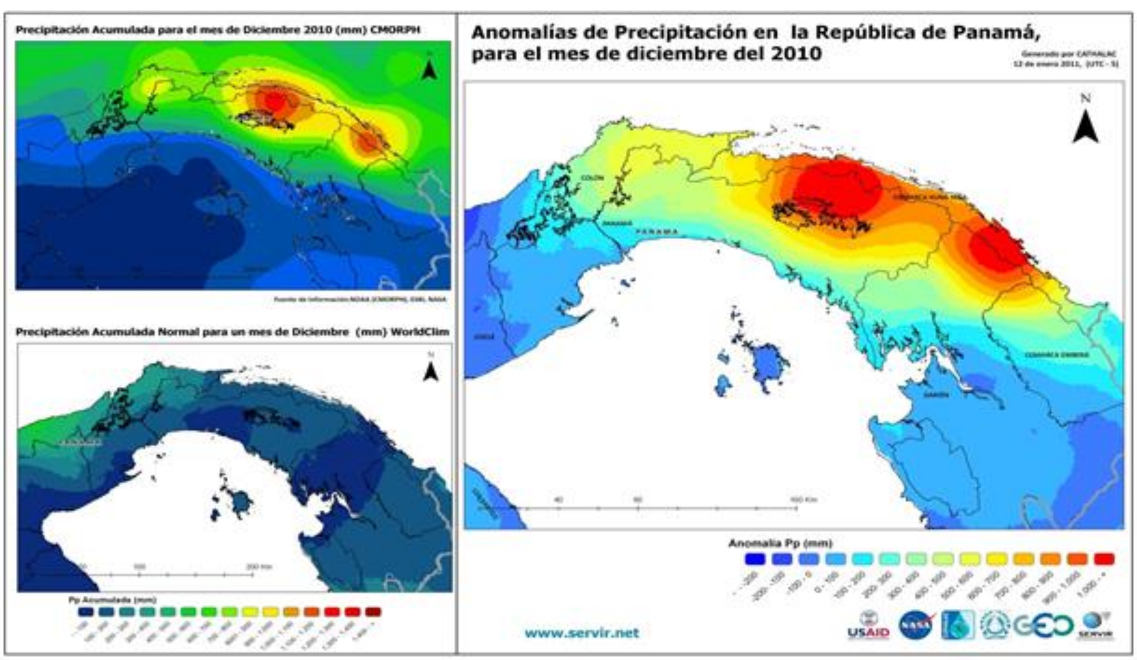

Figure 5: Anomalies of Precipitation in the Republic of Panama for December 2010

Flow data since 1982 for the Chucunaque Watershed was also obtained from ETESA, the Electric Transmission Company S.A. The maximum recorded flow rate, 14,588 cubic feet per second (cfs), occurred in December 2010, as expected. The 100-year flood flow was determined using the average maximum flood flow rates and was based on a normal distribution. Therefore, the 100-year flood flow rate was calculated to be 16,372 cfs. The full tables and calculations can be seen in Appendix B.

6.0 Final Recommendation

Determining what type of bridge would be appropriate was the first step of the final design process. Del Puente Engineering met with the community leaders and members of Alto Playón to discuss whether a pedestrian or vehicular bridge would be the most cost effective. Although a vehicular bridge would be the most beneficial to community members, the substantial increase in construction and maintenance costs made this option unfeasible. Once it was determined that a pedestrian bridge should be constructed, three design options were considered. The first option was to construct a full suspension bridge. The second option considered was to construct a pair of permanent abutments with a temporary bridge deck that would slide into and out of place as the water level changes. The third option was to design a floating barge system that would run on cables from one side of the port to the other. Options two and three were discarded because they would not provide adequate clearance over the drastically changing water level. High water and large debris floating down the river could damage the structures. Thus, Del Puente Engineering pursued the design of a suspension bridge.

6.1 Design Recommendations

6.1.1 Loadings

The suspension bridge was designed using the Allowable Strength Design (ASD) method. Unfactored loads were applied to the bridge model and individual members, and a factor of safety was then used to reduce the allowable capacity of the members. The appropriate pedestrian bridge loads were determined using *LRFD Guide Specifications for Design of Pedestrian Bridges*[13], and they remained unfactored throughout the analysis. Wind pressures were calculated using Chapter 26 of ASCE 7-10[18] for a base wind speed of 140 km/h (87 mph) as specified by the Structural Design Code for the Republic of Panama[18]. This resulted in a lateral load of 20 psf being applied to all members to account for the effects of wind loading. Seismic loading was also considered. A two dimensional model of the suspension bridge's profile was created in the educational version of SAP2000. The self-weight of members were incorporated into the model, and a modal analysis was completed. This analysis yielded a fundamental frequency of 0.73 seconds. This fundamental period was then used in coordination with the *Seismic Code Evaluation of Panama* [15]to calculate the appropriate loads. The remainder of the design was completed once these loads were determined. Loading calculations can be seen in Appendix D.

6.1.2 Cable design

A standard design for a suspension bridge was used with *Survey, Design and Construction of Trail Suspension Bridges for Remote Areas* [12] as a guideline for the layout and calculations. The general layout consists of several different parts that all work to support the specified loads on the bridge deck to cross the river as efficiently as possible. The largest structural components are the towers on each end of the span supported by concrete foundations that resist all vertical and lateral applied loads. A set of main cables are suspended from tower to tower, and each end is anchored into large concrete blocks. These concrete blocks are located farther away from the river bank and provide resistance to the main cable tension. The vertical suspender cables are first fastened to the main cables on each side of the bridge and cut at specific lengths to allow for the bridge deck camber, which are then attached to a set of spanning cables that run under the decking surface. The bridge deck is also connected to the many suspender cables and the spanning cables, which will be discussed in further detail in Section 6.1.7 of this report. The vertical loads are applied to the decking surface and then transferred to the spanning cables and crossbeams which are supported by the suspenders. The suspenders then transfer the load to the main cables and finally into the anchor blocks.

The cable geometry and tension were also calculated using the text *Survey, Design and Construction of Trail Suspension Bridges for Remote Areas* [12] for reference, which are found in Appendix E – Overall Bridge Calculations. The calculations are based on catenary cable equations to determine the cable sag and tensions in the main cables under various loading conditions. The bridge is designed to have a minimum of 15 ft of clearance under the bridge and an even larger clearance in the center due to a deck camber of about 8 ft. The overall bridge plan and profile can be seen in Drawing 4 of Appendix L, which defines the size of cables required with additional cable details and data shown in Drawing 3 of Appendix L.

The bridge span is 275 ft from tower to tower with a 4 ft wide deck crossing the Chucunaque River. The main cables consist of two 1 ½ in cables bundled together on each side of the bridge, each cable is capable of handling 36.8 kips of tension which resists against the maximum total main cable tension of 95.9 kips (under full dead and live loads). The calculated sag in the center of the bridge with a fully applied dead load is 1.84 ft. The hoisting load, which is the load from only the main cable self-weight during its placement, of 1.40 kips plus dead loads result in a total tension of 18.5 kips on the main cables. The spanning cables are pre-tensioned with 10.32 kips, which is the maximum loading for the 1 ½ in cables on each side of the bridge. As stated, these spanning cables are anchored into the tower foundations on each end and are discussed in section 6.1.4 of this report. The vertical suspenders are made of ½ in cables, which are cut to the specified lengths and clipped to the main cables. Each wire is capable of supporting 4.28 kips of tension which is sufficient to carry the dead and live loads of the bridge with a resultant tension of 1.1 kips applied to each suspender. The connection of the suspenders to the decking structure is discussed in section 6.1.7 of this report.

6.1.3 Tower Design

The towers support the main cables which provide enough height to allow the cables to sag under self-weight and loading conditions. The overall tower height is 50 ft from the top of the foundation to where the main cables are connected. The towers were designed in RAM Elements Software based on the applied loading conditions from dead loads, live loads, and wind loads transferred to the towers through the main cables as shown in Appendix F. The loads are primarily vertical under normal loading conditions because the main cables are allowed to slide freely over the top of the tower within a fabricated steel pipe section.

The towers themselves are composed of prefabricated steel box truss sections that will be fabricated offsite and assembled at the project site according to Drawing 6 of Appendix L. There are two 50 ft box truss sections tied together by 4”x4”x3/8” A36 angle iron cross-bracing spaced 12 ft apart. These sections provide the towers with lateral stiffness and allow for clearance between them to accommodate the walkway. Each truss section is made of vertical 4”x4”x3/8” A36 angle iron sections that make up the corners of the box section. These are tied together by 5/8” A36 solid round steel bars in a truss-bracing pattern to stiffen the section. The solid bars are welded to the inside corners of the angles in a shop setting to ensure proper strength as opposed to facing the many challenges of field welding, especially in a rural setting. Each section will then be hoisted vertically, placed on top of the previous section, and bolted together with 3/8” A36 steel plates on all four sides with 5/8” A307 bolts, which will sufficiently tie the sections together.

The bottom and most critical section, where the largest bending forces are located, utilize a more substantial section in order to resist the increased forces. To account for this, 4”x4”x3/8” A36 HSS sections were used instead to angle from the bottom of the second section to the baseplate where the structure is anchored to the foundation as shown in Drawing 6 of Appendix L. The baseplate itself is made of a 2” A36 steel plate welded to the base of the tower. The tower is then anchored to the foundation by 1 ¼” B7 anchor rods that are pre-installed in the concrete foundation. Once the tower is assembled and anchored to the foundation, the main cables can be strung and the rest of the bridge construction process can continue.

6.1.4 Foundation Design

The tower foundations were designed primarily to transfer the vertical loading from the towers to the soil below. The foundations were designed with concrete, and are 6 feet in height, with a bearing area of 12 ft by 20 ft. The foundation on both sides of the bridge will be the same size. The concrete is enclosed in a cage of reinforcing steel that will resist any tension load due to lateral loading from the towers and from the soil bearing force on the bottom of the foundation.

The spanning cables that give stability to the bridge deck are attached to cable anchorage hooks, which are embedded in the foundation and extend out the front face of the foundation. The towers sit on steel bearing plates that transfer the load to the concrete, and are held in place by anchor bolts embedded in the concrete.

Due to poor existing soil conditions at the site, the foundations will be placed on compacted engineered fill providing 3000 psf. A factor of safety of 2.5 was used in the design calculations, which can be found in Appendix I. Due to the large size of the foundation needed to meet the bearing capacity requirement, the foundations are able to resist overturning by self-weight without accounting for any lateral soil pressure.

6.1.5 Slope Stability

Gabions have been chosen as the primary form of slope stability and erosion protection on the project. They will be constructed using chain link fence as the walls of the individual units and filled with 4 in to 12 in diameter rocks. Since the site soil consists primarily of clay material, the gabions will prevent soil erosion when the water levels are high. Granular backfill material enclosed by the gabion system will provide adequate bearing pressure under the foundation, and the gabions will provide the primary support for the bridge approach. This method of slope protection successfully protected the foundation of the pump house for Metetí's water supply, also located at the port, during the flood of 2010. The full slope and foundation of this pump house was underwater during the flood and the structure remained in place. Also, this method will be familiar to the individuals performing the work since the same method has already been used in the area.

The gabions were designed to act as gravity walls per the *Modular Gabion Systems Manual*'s recommendations [14]. The full calculations can be seen in Appendix K. A one foot, vertical strip of the gabion wall was analyzed for slip and overturning failure. It was assumed that the north wall would provide a counter-moment for the south wall during the analysis. The gabions are stable without any additional guying, thus the gravity forces of the system will provide adequate support. The gabion will be constructed in five-foot-by-five-foot cubes. The cubes will then be stacked as shown in Drawing 5 of Appendix L.

6.1.6 Anchor Design

Anchor blocks are required as part of the tieback system of the suspension bridge. These blocks are designed to resist sliding, overturning, and bearing from the structure and surrounding soil. The anchorage design used in *Survey, Design and Construction of Trail Suspension Bridges for Remote Areas* (Section 8.51)[12] assumes that the anchor blocks are sitting on top of the soil, but in reality, these blocks will be buried. This design approach was used since the soil at Puerto Limón is mostly clay and therefore has minimal strength. The bridge site will also often be flooded, which will decrease the resisting pressure against the anchor block. Designing the blocks to rest on top of the soil is, as a result, a more conservative approach and provides greater safety to the bridge.

Each anchor block will have four cable anchorage hooks embedded into the concrete because there are four main cables bundled in pairs that span the bridge, that are then separated for anchorage. Each main cable will be connected to a turnbuckle by looping the cable through one end and clamping it back to itself. Attached to the other end of each turnbuckle is a cable anchorage hook. The turnbuckle is then tightened to its appropriate tension. The attached cable anchorage hooks are anchored approximately four feet deep at thirty-degree angles to mirror the angle at which the cables go over the towers. The width, length, and height of each anchor block required by the 36.8 kip tension force of each cable is 18 feet, 26 feet, and 8 feet, respectively, according to Table 65 of Krahenbuhl [12]. Details of the anchor block, cable anchorage hook, and turnbuckle can be found in Drawing 7 of Appendix L.

The total weight of the concrete anchor block was calculated to be about 560 kips. The design team also made the assumption that compacted granular soil will be used to backfill the area surrounding the anchor blocks in order to increase its stability. Therefore, the friction factor between concrete and dry gravel used in calculation was 0.50. In the design, the factors of safety for sliding, overturning, and bearing were 1.5, 2.0, and 2.5, respectively. Complete calculations and free body diagrams are found in Appendix J.

6.1.7 Suspender and Walkway Design

The suspenders will be made of ½ in diameter cables. These cables were designed to withstand the dead load from the walkway and the calculated bridge live loads. The calculations for the suspenders are shown in Appendix E. These suspenders were designed to fit between the main cables and the spanning cables. Detailed drawings for the suspenders are located in Drawing 4 of Appendix L. These suspenders will be bolted in place during construction – an attachment detail can also be found in Drawing 4 of Appendix L.

The bridge will have a wooden walkway made out of Alemendro, which can be found locally. The decking will be supported by a steel frame made out of double angles spaced at 3 ft. Calculations were performed on the timber and the steel to ensure it that it would withstand the worst-case loadings. The three limit states that were checked were bending, shear, and deflection. The timber passed is adequate for bending and shear, but not for deflection. These calculations can be seen in Appendix G and Appendix H. The design team deemed it acceptable to not meet the deflection requirement under full loading since deflection is a serviceability factor, not an ultimate failure state. The wood available locally is therefore adequate to use as the decking. A detailed drawing of the walkway and connections can be seen in Drawing 4 of Appendix L.

6.2 Construction and Estimating

To complete the project cost estimate, a quantity take-off was completed on the final design. Minor adjustments were made to the design for constructability purposes. These quantities were then used to determine what equipment would be necessary for construction. A front end loader will be the only major piece of machinery on site. Much of the work will be performed by hand to reduce costs. However, an experienced project manager in Panama will be needed to be the foreman on the project. Other small tools such as whacker compactors, shovels, torque wrenches, and wheel barrows were estimated as a five percent increase in the overall cost of the project. It was then verified that this equipment would be locally available or could be transported and delivered to the site.

Production rates for each item task were was then estimated. It was assumed that all unskilled labor will be completed by members of the benefiting communities. The typical daily wage for members of the community assisting others as farmhands and completing other similar work is ten dollars per day. All skilled labor will need to be completed by trained construction workers in their fields of expertise. These production rates were used to develop average daily outputs of each item (items/day.) These daily outputs were used in combination with the expected daily crew cost, daily equipment cost, and material costs to develop a unit cost ($/item) of each task. Finally, these unit costs where used as the base of the project estimate.

An additional five percent was added to the estimated costs above to account for the cost of delivering materials to site. Additional temporary structures will be needed to complete the construction of the suspension bridge. However, the costs of these structures have not been incorporated into the estimated costs. The final estimated cost of the project is $418,000. An estimate breakdown can be found in Appendix N.

Task durations were also calculated from the estimated quantities and production rates. These durations were then used to complete the project schedule. The final project duration is 194 days. This means the work will need to be completed over two and one half dry seasons. The pursuit of appropriate building codes, material ordering, and timber cutting can be completed during the dry seasons. The maximum available construction season aligns with Panama's dry season that runs from mid-December to the end of April. It is not guaranteed that the dry season will last this long so the construction schedule was extended over two dry seasons rather than being accelerated into one. Construction during the first dry season will consist of the development of each slope and approach, and will finish with the pouring of the bridge tower foundations. The equipment and external crew members will then be demobilized for the coming dry season. The full construction schedule can be seen in Appendix M.

6.2 Funding and Maintenance

In order to fund the project, a partnership will need to be established between the community and a non-profit organization. Possible organizations that can help provide funding are Bridges to Prosperity, the Peace Corps of Panama, and Engineers Without Borders-Panama. Bridges to Prosperity has already shown interest in this crossing location into the Embera-Wounaan Comarca. The design completed by Del Puente Engineering will need to be reviewed by a professional engineer in order to ensure that all aspects of the project and design are adequate. The sites soil conditions will need to be verified prior to bringing any large equipment off the gravel road into the port.

A bridge maintenance committee will need to be formed to increase the lifespan of the suspension bridge. Common repairs may include replacement of wood decking, ordering and replacing individual bolts, retying portions of the gabion system, or recompacting the bridge's approach slope. The knowledge of how to complete these repairs will need to be shared amongst the community members on this council. This committee will also need to have the ability to collect an annual tax from the members of the communities that utilize the bridge. After talking with the community leaders of Alto Playón, it has been determined that a small annual tax for the bridge would be a more efficient way to cover the expenses of need maintenance rather than applying a toll to all bridge crossings.

7.0 Conclusion

This report has outlined the completion of a final design, construction schedule, and cost estimate for a pedestrian bridge into the Embera-Wounaan. After considering multiple alternatives, a suspension bridge was determined to be the most appropriate solution at Puerto Limón. Collaboration of the Embera-Wounaan communities with an organization that can financially support the $418,000 bridge will be essential for the construction of the bridge. A maintenance committee with the ability to collect annual taxes will ensure that the bridge will not prematurely fall into disrepair.

8.0 Acknowledgements

International Senior Design Advisors:

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9.0 References

[1] *AASHTO LRFD Bridge Design Specifications*. 6th ed. Washington D.C.: American Association of State Highway and Transportation Officials, 2012. Print.

[2] "Aggregates, Sand, Gravel, Rock." *Anchorage Sand and Gravel*. N.p., n.d. Web. 27 Nov. 2013. <http://www.anchsand.com/default.aspx?tabid=131>.

[3] "Almendro." *Pacific Rim Hardwoods RSS*. N.p., n.d. Web. 27 Nov. 2013. <http://pacificrimhardwoods.com/exotic-hardwood-species/almendro/>.

[4] "Bright Wire Rope EIPS IWRC - 6x19 Class - 2"" *US Cargo Control*. N.p., n.d. Web. 27 Nov. 2013. <http://www.uscargocontrol.com/Rigging-Supplies-Hardware/Bright-Wire-Rope-6x19-EIPS-IWRC-6x26/Bright-Wire-Rope-EIPS-IWRC-6x19-Class-2-Lineal-Foot>.

[5] "Chucunaque and Tuira Rivers." *Panama*. Autoridad de Turismo Panama, n.d. Web. 26 Sept. 2013. <http://www.visitpanama.com/en/component/k2/1422/-1422.html>.

[6] "COEFFICIENT OF FRICTION." *Super Civil CD*. N.p., n.d. Web. 28 Oct. 2013. <http://www.supercivilcd.com/FRICTION.htm>.

[7] "Computer Modeling Examples." *Bridges to Prosperity*. N.p., n.d. Web. 27 Nov. 2013. <http://bridgestoprosperity.org/resources/technical-resources/>.

[8] "Cumaru (Almendro)." *Lumber Max Fine Wood Supplier*. N.p., n.d. Web. 27 Nov. 2013. <http://www.lumbermax.biz/species/almendro.php>.

[9] "ETESA - Inicio - Uniendo Panamá con Energía." *Empresa de Tramisión Eléctrica, S.A.* N.p., n.d. Web. 26 Sept. 2013. <http://www.etesa.com.pa/>.

[10] "Heavy rains and flooding in Panama, Dec 2010." *Servir Mesoamerica - The Regional Visualization and Monitoring System*. N.p., 12 Jan. 2011. Web. 26 Sept. 2013. <https://servirglobal.net/Mesoamerica/Articles/tabid/241/Article/1001/heavy-rains-and-flooding-in-panama-dec-2010.aspx>.

[11] Holtz, Robert D., William D. Kovacs, and Thomas C. Sheahan. *An Introduction to Geotechnical Engineering*. 2nd ed. Upper Saddle River: Pearson Education, 2011. Print.

[12] Krahenbuhl, J., Wagner, A. *Survey, Design and Construction of Trail Suspension Bridges for Remote Areas*. B. Gallen, Switzerland: SKAT, 1983. Print.

[13] LRFD Guide Specifications for Design of Pedestrian Bridges, American Association of State Highway and Transportation Officials, Washington, DC.

[14] "Mechanically Stabilized Earth (MSE) Gabion Wall [Reinforced Soil Wall." *Gabions*. Modular Gabion Systems, Nov. 2004. Web. 23 Oct. 2013. <http://www.gabions.net/downloads/Documents/MGS_Design_Guide.pdf>.

[15] Penzien, J. *Seismic Performance of Transportation Structures*. Berkley: International Civil Engineering Consultants, Inc, n.d. PDF.

[16] *Portunus Port Spares & Services*. N.p., n.d. Web. 27 Nov. 2013. <http://www.portunus.com/ropes.html>.

[17] Sorrell, P.E., Richard C. "DEQ Hydrology Redirect." *Computing Flood Discharges for Small Ungaged Watersheds*. N.p., 22 June 2010. Web. 26 Sept. 2013. <http://www.michigan.gov/hydrology>.

[18] *Structural Design Code for the Republic of Panama*. Panama City: MINISTERIO DE OBRAS PUBLICAS JUNTA TECNICA DE INGENIERIA Y ARQUITECTUR, 29 Sept. 2004. PDF. http://www.eird.org/cd/acs/English/CodeEval/SpaSpeak/Wind/PANAMwce.pdf

[19] "Wire Rope - Strength." *The Engineering Toolbox*. N.p., n.d. Web. 27 Nov. 2013. <http://www.engineeringtoolbox.com/wire-rope-strength-d_1518.html>.

[20] "Woods of Panama." *Tropical Woods of Panama*. N.p., n.d. Web. 27 Nov. 2013. <http://www.plantingempowerment.com/tropical-woods>.

[21] Wurbs, Ralph Allen, and Wesley P. James. *Water Resources Engineering*. Upper Saddle River, NJ: Prentice Hall, 2002. Print.

[22] Zhu, Tieyuan. "Some Useful Numbers on the Engineering Properties of Materials (Geologic and Otherwise)." *Stanford University*. Department of Geophysics, n.d. Web. 23 Oct. 2013. <http://www.stanford.edu/~tyzhu/Documents/Some%20Useful%20Numbers.pdf>.

10.0 Appendices

Appendix A:
Survey Point Data

Survey Point Data

1	51.6	12.3	113.2	SE corner foundation
2	1.6	31	113.9	SW fence foundation
3	-18.7	13.3	110.8	front corner of erosion protection
4	-27.9	17.6	109.4	edge of driveway
5	-25.7	3.8	109	turning point
6	-89.2	11.8	106.3	edge of turn
7	-168.3	47.8	109.6	road by hut (north side)
8	-218.6	74.6	110.5	road (north side)
9	-289.5	78.6	110.3	road (south side)
10	-172.7	21	107.6	road behind hut (south side)
11	-129	-2.8	104	road before hut
12	-71.7	-35.5	105.2	end of drive (south side of road)
13	-33.4	-41.2	107.1	road near lagoon (south side)
14	-13.1	-55.5	100.6	road edge of lagoon (ss)
15	3.6	-64.9	101.8	road in front of house (ss)
16	12.6	-83	101.3	road W edge of grass (ss)
17	22.7	-93.3	99.9	road E edge of grass
18	41	-81.2	98.9	muddy pit
19	28.7	-52.7	99.3	S side of tree
20	10.3	-42.8	102	road W of tree (ns)
21	1.6	-30	104.5	road (ns)
22	-31.5	-17.4	106.4	road base of drive (ns)
23	-16.1	-8	109.1	part way up drive
24	-43.1	25.3	105.8	back slope
25	-101.8	44.3	100.8	road, base of rocks (ns)
26	-91.7	62.5	100.7	base of telephone pole
27	-183.8	68.2	112	road, edge of rock (ns)
28	-183.2	82.6	100.9	parallell to 27, base of rock
29	-45.4	18.3	105.1	top of culvert
31	86.3	-57.3	98.5	top of culvert
32	123	23	90	base of water, 1st u.s.
33	113.5	6.6	90.2	base of water, far end at inlet
34	112.3	3.8	90	base of water, at inlet
35	98.7	-19.6	90.9	water
36	91.1	-54.8	90.5	water
37	90.1	-98.7	89.9	water
38	71.8	-135.1	89.3	water
39	273.9	-53.1	114.3	other side of river (ds)
41	259.9	-125.2	101.8	outside trees (ds)
42	248.9	-155.8	100.3	in shrubs (us)
43	209.2	-192.6	96.6	road (us)
44	236.8	-214.1	98.6	side road (us)
45	287.9	-237.1	99.5	back part of road (us)
46	165.6	-149.5	92.9	end of road before river (us)

47	56.8	-171.5	89.5	bank of river in front of latrine
48	0.3	-115.2	104.5	NE corner of house
49	-21.2	-109.8	104.5	NW corner of house
50	-22.5	-51.1	101.1	roadside of lagoon entrance
51	-22.3	-38.2	100	W bank toward house
52	-86.1	-77.8	100.3	W bank
53	-46.2	-94.2	99.9	E bank, parallel to 52
54	-20.1	-84.8	99.5	E bank, parallel to 51
55	-7.2	-1.2	110.8	Backsight B to A
56	61.5	-86.1	94.4	water
57	60	-118.4	93.9	water
58	53.6	-160.9	94.1	water
59	65.8	-45.3	94.4	water
60	88.6	0.5	94.3	water
61	99.1	16.6	94.3	water
63	102.5	47.8	94.8	up slope toward pump station
64	77.4	-15.4	98.2	up slope toward pump station
65	89.6	22.7	100.5	along N side of inlet
66	73.6	27.8	106.1	along N side of inlet
67	80.3	47.7	106.1	N of inlet
68	89.4	60.8	101.2	further NE of inlet
69	48.2	44	111.9	NE corner of pump station
70	57	38.1	113.4	top of inlet N side
71	75	-6	99	S of inlet, parallel to 65
72	61.6	-41.6	97.1	N of culvert
73	58.1	-48.9	95.5	S of culvert
74	45.9	-89.1	97.4	E of mud pit
75	27.9	-78.3	98.7	N of mud pit
76	41.3	-39.7	99.9	edge of S scour
77	47.6	-12.1	106	middle of S scour
78	58.2	2.5	106.9	middle of S scour
79	65.5	19.4	107.1	E downhill from SE corner
80	82.7	29.4	112.6	scour protection slope
81	47.1	-3	108.3	SE of SE corner
82	41.4	3.4	112.1	scour protection slope
83	55.7	11	113.3	N edge of slope
84	72.4	-1	111.1	scour protection slope
85	39.2	-13.5	106.8	scour protection slope
86	46.4	-12	107.5	scour protection slope
87	63.5	-16.1	108.6	scour protection slope
88	64.6	-14.3	107.9	edge of slope
89	64.6	-14.3	106.7	edge of slope
90	53.7	-23.1	105.1	edge of slope
91	54.5	-22.2	104.3	edge of slope
92	40.9	-34.6	101.3	edge of slope
93	62.3	-22	101.3	edge of slope
94	51.7	-31.9	102.1	edge of slope

95	29.6	-44.8	100.6	edge of slope
96	21.4	-38.8	101.5	edge of slope
97	13.8	-40.6	103	edge of slope
98	5.5	-45.5	102.6	edge of slope
99	5.4	-45.6	101.3	edge of slope
100	0.4	-45.6	101.2	edge of slope
101	23.4	-40.5	100.6	edge of slope
102	26.6	-49.1	100	edge of slope
103	15.3	-48.4	99.7	edge of slope
104	12.2	-34.8	103.7	SW tree
105	19.8	-25.9	102.4	Middle tree
106	28.7	-32.8	101.8	N tree
107	20.1	-121.4	101.5	base of tree roots
108	17.2	-122.3	103.5	top of tree roots
109	26.9	-154.3	100.2	near tree
110	36.1	-155.5	103.2	half way to latrine
111	43.6	-177	101.8	base of latrine
112	43.7	-178	105	latrine
113	42.4	-183.2	105.1	latrine
114	37	-175.5	105.1	latrine
115	37.1	-174.5	103	latrine
116	44.7	-193	103.7	fence corner
117	-7.9	-179.1	103.6	fence corner
118	6.7	-157.5	104.1	yard
119	12.2	-140.9	104.2	yard
120	271	-117	100.3	Backsight from B to C
121	221.1	-137.3	94.9	water, base of roots
122	226.8	-154.3	94.8	water, N side of road
123	199.1	-183	94.4	water, S side of road
124	205.7	-202.6	94.2	water
125	246.8	-110.8	94.8	water
126	237.2	-101.3	94.7	water
127	236.3	-86.7	94.8	water
128	250.1	-78.3	94.9	water
129	256.8	-48.3	94.7	water
130	258.1	-72.5	100.9	edge of partial eroded bank
131	212.8	-98.5	99.5	edge of partial eroded bank
132	274.4	-166.4	101.9	N edge of road
133	304.6	-162.4	100.1	further NE along road
134	128.6	-206.2	96.5	Control pt D
135	212.3	-77	102.5	jungle opening
136	201.4	-69.9	103.5	jungle line
137	188.6	-64	104.6	jungle line
138	173.3	-54.4	106.1	jungle line
139	218.1	-79.3	98.5	front jungle line base
140	199	-86.9	103.4	jungle, upper edge
141	180.8	-98.7	103.5	jungle, upper edge

142	158.5	-106.8	102	jungle, upper edge
143	158	-115.4	99.9	jungle, upper edge
144	170.8	-130	100.8	edge of bushes
145	173.2	-123	102.8	edge of bushes
146	185	-104	97.2	jungle edge base
147	210.6	-103.1	98.2	ground
149	246.2	-95.2	100.1	E of jungle hole
150	224.2	-137.4	99.4	line to the S
151	203.3	-176.3	99.9	line to the S
152	221.6	-197	99.8	line to E from pt 151
153	252.8	-143.3	100.4	coming back N from 152
154	296.3	-58.2	101.7	Tom's jungle hole, far E jungle hole
155	274.7	-70.1	101.4	Wes' mini jungle hole, W of Tom's
156	273.9	-96.2	100.5	SE of mini jungle hole
157	98.6	-214.5	93.2	water

Appendix B:
Water Flow Analysis

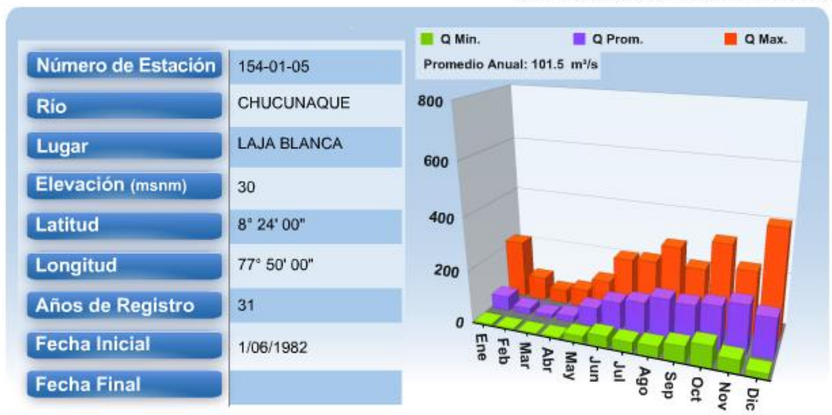
Chucunaque Watershed

Area: 4118 square miles
Length: 134 miles

Flow data observed since 1982:

Avg. Annual Flow = 3583 ft^3/s
Min. Flow = 170 ft^3/s
Max. Flow = 14,588 ft^3/s

Chucunaque River Flow		
Month	Avg Q (ft^3/s)	Max Q (ft^3/s)
Jan	1960	7918
Feb	791	3192
Mar	466	1826
Apr	819	2430
May	2507	4164
Jun	3863	7737
Jul	4428	7978
Aug	5460	10330
Sep	5227	7896
Oct	5682	11696
Nov	6431	8599
Dec	5361	14589



Assume Normal Distribution for Flow Data:

$$100\ year\ Flood\ Flow, Q_{100} = \bar{Q} + kS$$

$\bar{Q}$ = average max flow
k = 2.326 (Wurbs, p. 419, Table 7.3)
S = standard deviation

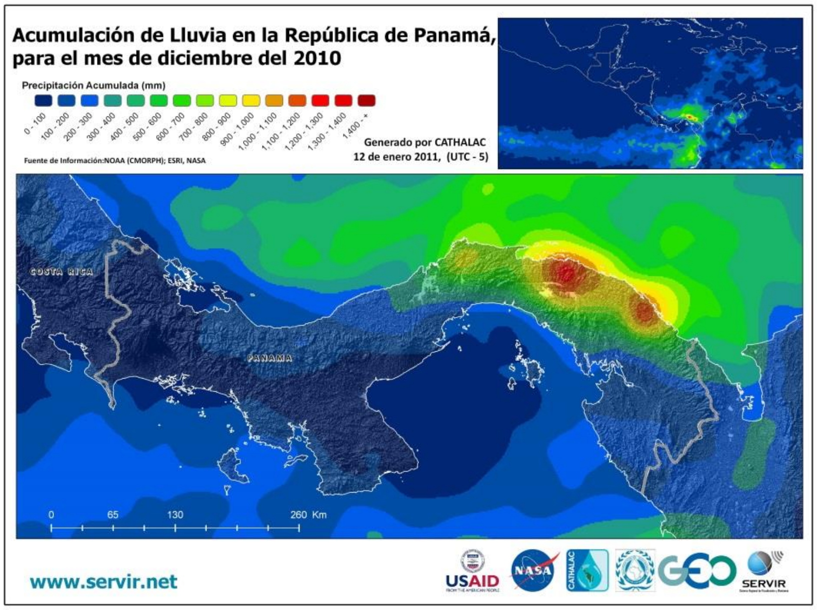
$$Q_{100} = 7363\frac{ft^3}{s} + 2.326 * 3874\frac{ft^3}{s} = 16372\frac{ft^3}{s}$$

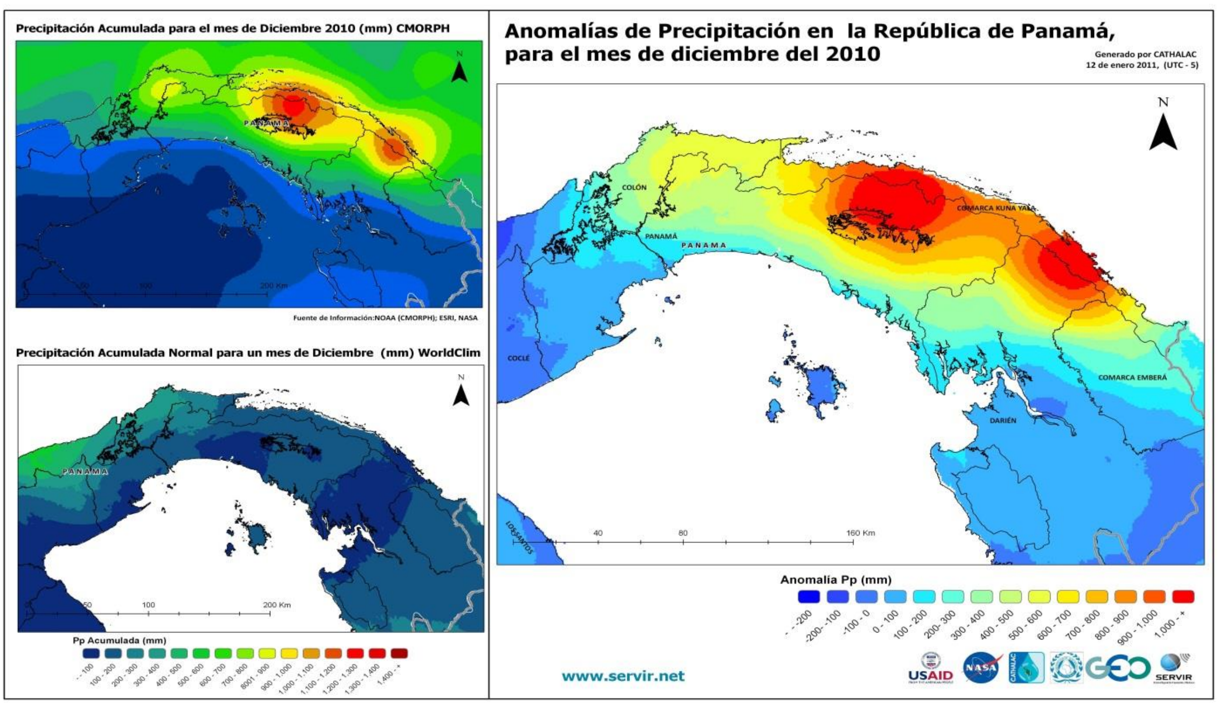
References:

"ETESA - Inicio - Uniendo Panamá con Energía." *Empresa de Tramisión Eléctrica, S.A.* N.p., n.d. Web. 26 Sept. 2013. <http://www.etesa.com.pa/>.

Wurbs, Ralph Allen, and Wesley P. James. *Water Resources Engineering*. Upper Saddle River, NJ: Prentice Hall, 2002. Print.

Rainfall Data for 2010 Panama Flood





Reference:

"Heavy rains and flooding in Panama, Dec 2010." *Servir Mesoamerica - The Regional Visualization and Monitoring System*. N.p., 12 Jan. 2011. Web. 26 Sept. 2013. <https://servirglobal.net/Mesoamerica/Articles/tabid/241/Article/1001/heavy-rains-and-flooding-in-panama-dec-2010.aspx>.

Appendix C:
Soil Testing Data

Soil Characteristics

Hydrologic Soil Group	% Total Drainage Area	Land Use	% Soil Group	RCN	Partial RCN
C	20	Row Crop	50	82	8.2
C	30	Woods	50	70	10.5
D	50	Woods	100	77	38.5
				Sum:	57.2

Hydrologic Soil Group Descriptions:
C – Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes the downward movement of water or soils with moderately fine to fine texture. These soils have a slow rate of water transmission (Sorrell).
D – Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission (Sorrell).

Land Use:
Woods – good condition
Row Crop – good condition, contoured fields

Runoff Curve Number (RCN):
Table 6.1
RCN for Row Crop Soil C: 82
RCN for Woods Soil C: 70
RCN for Woods Soil D: 77

Antecedent Moisture Condition (AMC): III (wet)

Final RCN Calculation to account for AMC:

$$RCN(III) = \frac{23 * RCN(II)}{10 + 0.13 * RCN(II)} = \frac{23 * 57.2}{10 + 0.13 * 57.2} = 75.45$$

Unified Soil Classification System
The soil where both abutments will be constructed can be classified as reddish-brown clay, with little fine sand, little small gravel, and slight organic matter, well-graded, moist, CL (Holtz).

References:
Holtz, Robert D., William D. Kovacs, and Thomas C. Sheahan. *An Introduction to Geotechnical Engineering*. 2nd ed. Upper Saddle River: Pearson Education, 2011. Print.
Sorrell, P.E., Richard C.. "DEQ Hydrology Redirect." *Computing Flood Discharges for Small Ungaged Watersheds*. N.p., 22 June 2010. Web. 26 Sept. 2013. <http://www.michigan.gov/hydrology>.

Appendix D:
Loadings

Basic Bridge Properties:

$L := 275\ ft$ — length of bridge span

$h := 50\ ft$ — height of tower

$width := 4.5\ ft$ — width of bridge

$Wt_{self} := 7540\ lbf$ — Self Weight from RAM

Load Deflection Restrictions:

$$vert_{deflect} := \frac{L}{360} = 9.167\ in$$

$$horiz_{deflect} := \frac{L}{360} = 9.167\ in$$

permittable deflections of entire structure

Pedestrian Live Load - AASHTO Ped. Bridge

$PL := 90\ psf$ — pedestrian loading

Vehicle Loading - small vehicle - AASHTO Ped. Bridge

$LL_f := 0.1 \cdot 2\ kip = 200\ lbf$

$LL_b := 0.1 \cdot 8\ kip = 800\ lbf$

factored for a motor cycle

vehicle live loads (small truck)

Axial/ Wheel Spacing 3.5ft

Equestrian Loading - AASHTO Ped. Bridge

$LL_{horse.w} := 1\ kip$ — weight of horse

*used for calculating shear

$Area_{horse} := 4\ in \cdot 4\ in = 16\ in^2$ — area of hoof

$$LL_{horse} := \frac{LL_{horse.w}}{Area_{horse}} = 9000\ psf$$

pressure under each hoof

Basic Bridge Properties:

$L := 275\ ft$ — length of bridge span

$h := 50\ ft$ — height of tower

$width := 4.5\ ft$ — width of bridge

$Wt_{self} := 7540\ lbf$ — Self Weight from RAM

Earthquake Load - Seismic Code Evaluation "Panama"

Seismic Zoning - Section 2.1

$A_a := 0.22$ — ground intensity

"El Real" closest city

$A_v := 0.27$ — effective peak acceleration; related to velocity

Site Classification - Section 2.5

Soil Profile E - Soft Soil

$v_s := 180\ \frac{m}{s}$ — shear wave velocity

$N_{ch} := 15$

$s_u := 50\ kPa$ — undrained shear strength

Seismic Performance Category C - 0.19<A<0.29 and "Non-essential"

Peak ground accelerations (vertical and horizontal) - Section 2.6

Guess Values

$F_a := 1.5$

Table 1: Section 2.6

Constraints

$$\left(\frac{(1.2-1.7)}{0.3-0.2}\right) = \left(\frac{(1.2-F_a)}{0.3-A_a}\right)$$

Solver

$F_a := \text{find}(F_a) = 1.6$

$F_a = 1.6$

Guess Values

$F_v := 3.0$

Constraints

$$\left(\frac{(2.8-3.2)}{0.3-0.2}\right) = \left(\frac{(2.8-F_v)}{0.3-A_a}\right)$$

Table 2: Section 2.6

Solver

$F_v := \mathbf{find}\,(F_v) = 3.12$

$F_v = 3.12$

Seismic Actions - Section 4

$C_a := A_a \cdot F_a = 0.352$ Section 4.1

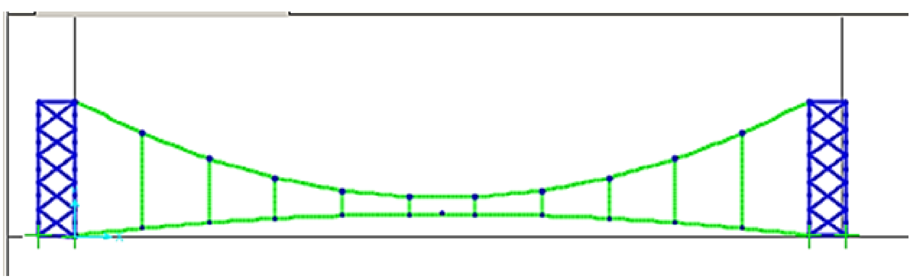
$C_v := F_v \cdot A_v = 0.842$ Section 4.1

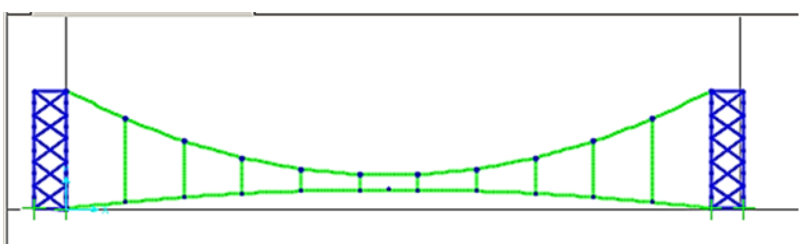
Period Determination

The theoretical period of the bridge was determined by modeling the bridge's cross-section with an educational license of SAP2000. The self weight of the bridge members were estimated in RAM and then applied to the SAP200 model.

Assumed:
-19x6 2in Cable Wire

Blue: L4x4x
Green: Cable





**Calculated using SAP2000 Educational Version*

$T = 0.715\ s$

Seismic Actions - Cont'd

$$R := 1.25$$

Section 3.2 reduction factor

$$C_s := min\left(\frac{2.5 \cdot C_a}{R}, \frac{1.2 \cdot C_v \cdot sec^{\frac{2}{3}}}{\left(R \cdot T^{\frac{2}{3}}\right)}\right) = 0.704$$

Section 4.2 - seismic coefficient

Static Method Procedure - Section 5.3

$$W_x := Wt_{self} = 7.54\ \boldsymbol{kip}$$

total structural weight

$$V_E := C_s \cdot W_x = 5.308\ \boldsymbol{kip}$$

total base shear force

$$\begin{aligned} k_E := &\text{if } T < 0.5\ \boldsymbol{s} \\ &\quad \| \ 1 \\ &\text{else if } T > 2.0\ \boldsymbol{s} \\ &\quad \| \ 2 \\ &\text{else} \\ &\quad \| \ \text{“}\mathit{Interpolate\ by\ hand}\text{”} \end{aligned}$$

$$k_E = \text{“}\mathit{Interpolate\ by\ hand}\text{”}$$

Therefore:

Guess Values

$$k_E := 1.1$$

Constraints

$$\frac{(0.5\ s - 2\ sec)}{(1-2)} = \frac{(0.5\ s - T)}{(1-k_E)}$$

Solver

$$k_E := \mathbf{find}\left(k_E\right) = 1.143$$

Load Distribution: Accounts for shear load to be split between two towers
Load applied to quarter point of tower

$$h_{x1} := \frac{50\ \boldsymbol{ft}}{4}$$

$$C_{vx1} := \frac{\frac{W_x}{4} \cdot {h_{x1}}^{k_E} \cdot \boldsymbol{ft}^{(-k_E)}}{W_x} = 4.49$$

Section 5.3

$$F_{x1} := \frac{\left(C_{vx1} \cdot V_E\right)}{2} = 11.916\ \boldsymbol{kip}$$

Load applied to half point of tower

$$h_{x2} := \frac{50\ ft}{2}$$

$$C_{vx2} := \frac{\frac{W_x}{4} \cdot {h_{x2}}^{k_E} \cdot ft^{(-k_E)}}{W_x} = 9.918$$ Section 5.3

$$F_{x2} := \frac{(C_{vx2} \cdot V_E)}{2} = 26.324\ kip$$

Load applied to three-quarter point of tower

$$h_{x3} := 3 \cdot \frac{50\ ft}{4}$$

$$C_{vx1} := \frac{\frac{W_x}{4} \cdot {h_{x3}}^{k_E} \cdot ft^{(-k_E)}}{W_x} = 15.769$$ Section 5.3

$$F_{x3} := \frac{(C_{vx1} \cdot V_E)}{2} = 41.851\ kip$$

Load applied to full height of tower

$$h_{x4} := 50\ ft$$

$$C_{vx2} := \frac{\frac{W_x}{4} \cdot {h_{x4}}^{k_E} \cdot ft^{(-k_E)}}{W_x} = 21.911$$ Section 5.3

$$F_{x4} := \frac{(C_{vx2} \cdot V_E)}{2} = 58.153\ kip$$

Total Applied Lateral Loads for Seismic Design

$$F_{max} := F_{x1} + F_{x1} + F_{x3} + F_{x4} = 123.8\ kip$$

Note: The designed towers are inadequate to bear these lateral loads. Due to the economical restraints, the bridge will not be built to sustain the loads seen in a large earthquake. Bridge users will be recommended to remove themselves from the bridge prior to its complete failure in the event of such an earthquake

Basic Bridge Properties:

$L := 275\ ft$ — length of bridge span

$h := 50\ ft$ — height of tower

$width := 4.5\ ft$ — width of bridge

$Wt_{self} := 7540\ lbf$ — Self Weight from RAM

Horizontal Wind Loading - Example in Ped. Bridge (pg19)

Use Signs 3.8 and 3.9

$K_z := 1.00$ — height and expos. factor

$G := 1.14$ — gust factor

$V := 140\ \frac{km}{hr} = 86.992\ mph$ — Panama Design Code

$I_r := 1.15$

$C_d := 2.00$

$$P_z := 0.00256 \cdot K_z \cdot G \cdot V^2 \cdot I_r \cdot C_d = 10.151\ Sv$$

Horizontal Wind Loading - based on Example ASCE 07 "Open Sign"

Exposure:

$K_{d.a} := 0.85$ — open sign - lattice work — gust factor ASCE 07 - sect. 26.6

$K_{z.a} := 1.13$ — Exposure C. z = 60ft — exposure coefficient ASCE 07 - table 29.3-1

$H_{esc} := 12\ ft$ — height of escarpment

$L_h := 2\ ft$ — length of escarpment

$$K_1 := 0.85 \cdot \left(\frac{H_{esc}}{L_h}\right) = 5.1$$

$\mu_{wind} := 1.5$

$x := 0\ ft$

$$K_2 := \left(1 - \left(\frac{x}{\mu_{wind} \cdot L_h}\right)\right) = 1$$

$\gamma_{wind} := 2.5$

$z_{wind} := 60\ ft$

$$K_3 := e^{\left(-\gamma_{wind} \cdot \frac{z_{wind}}{L_h}\right)} = 2.679 \cdot 10^{-33}$$

$$K_{zt.a} := \left(1 + K_1 \cdot K_2 \cdot K_3\right)^2 = 1$$

topo. coefficient
ASCE 07 - sect. 26.8.2

$$V = 86.992\ mph$$

$$q_{z.a} := 0.00256 \left(\frac{(psf)}{(mph)^2}\right) \cdot K_{z.a} \cdot K_{zt.a} \cdot K_{d.a} \cdot V^2 = 18.608\ psf$$

$$WL := \text{Ceil}\left(q_{z.a}, psf\right) = 19\ psf$$

Design for 19 psf

Uplift force of wind on bridge

$$WS_{up} := 0.02\ ksf$$

AASHTO Ped. Bridge 3.4

Applied at windward quarter point

Appendix E:
Overall Bridge Design Calculations

Span Length: $l := 275\ ft \qquad l = 83.82\ m$

Dead Load Camber: $c_d := .03 \cdot l = 2.515\ m$

Dead Load Sag: $f_d := .12 \cdot l = 10.058\ m$

Theoretical Tower Height: $h_T := f_d + c_d + 1.05\ m = 13.623\ m$

From table 45: $w_{walk} := 1.2\ m$

- tower height = 14.77m
- Tower number 7
- 4 main cables
- 32/36/40mm main cables
- 32mm spanning cables

effective dead load sag: $f_d := 14.77\ m - 1.05\ m - c_d = 11.205\ m$

Table 46 - full load gf: $g_f := .63 \cdot \frac{ton}{m}$

full load cable tension: $T_f := \frac{g_f \cdot l^2}{8.4 \cdot f_d} \cdot \sqrt{1 + 17.64 \cdot \left(\frac{f_d}{l}\right)^2} = 53.93\ ton$

Table 52 - determine:
main cable size and number

$T_{perm} := 61.2\ ton$

use (4) 32mm diam. main cables w/ thimbles and bulldog grips

$\text{if}\left(T_f \le T_{perm}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

preliminary main cable angle: $\beta := \frac{4.2 \cdot f_d}{l} = 32.17\ deg$

frontstay angle = backstay angle

Design values:

$h_T := 14.77\ m$
$n := 4$
$\phi_M := 32\ mm$
$f_d = 11.205\ m$
$\beta = 32.17\ deg$
$\phi_S := 32\ mm$
$\phi_W := 26\ mm$
$w := 2$
$E := 12\ \frac{ton}{mm^2}$
$D_R := 80\ ft$
$D_L := 80\ ft$

Dead Load Determination:

hoisting load: $g_h := .00038058 \cdot n \cdot \phi_M$

$$g_h := .0156 \cdot \frac{ton}{m} \qquad g_h = 9.51 \ \frac{lb}{ft}$$

walkway incl. planks: $w_{walk} := .088 \cdot \frac{ton}{m}$

rail and fixation cables: $w_{rail} := .003 \cdot \frac{ton}{m}$

wiremesh netting: $w_{mesh} := .006 \cdot \frac{ton}{m}$

suspenders (avg): $w_{suspend} := .017 \cdot \frac{ton}{m}$

windties (avg): $w_{ties} := .004 \cdot \frac{ton}{m}$

spannng cables: $w_{spaning} := .00038058 \cdot \left(2 \cdot {\phi_S}^2\right)$

$$w_{spanning} := .0078 \cdot \frac{ton}{m}$$

windguy cables: $w_{windguy} := .00038058 \cdot \left(w \cdot {\phi_W}^2\right)$

$$w_{windguy} := .0051 \cdot \frac{ton}{m}$$

total dead load: $g_d := g_h + w_{walk} + w_{rail} + w_{mesh} + w_{suspend} + w_{ties} + w_{spanning} + w_{windguy} = 0.147 \ \frac{ton}{m}$

$$g_{d\_ft} := g_d \cdot 9.81 \ \frac{m}{s^2} = 89.337 \ \frac{lbf}{ft} \qquad g_{d\_psf} := \frac{g_{d\_ft}}{4 \ ft} = 22.334 \ psf$$

Live Load Determination:

span less than 100m:
width = 1.2m

$$q_l := 90 \ psf$$

$$g_l := \frac{q_l \cdot 4 \ ft}{9.81 \ \frac{m}{s^2}} = 0.59 \ \frac{ton}{m}$$

Full Load Determination:

full load value factored: $g_f := 1.2 \cdot g_d + 1.6 \cdot g_l = 1.12 \ \frac{ton}{m}$

Pretension spanning cables:
(pretension with 10% of gd)

$$p_s := .42 \cdot g_d = 0.062 \ \frac{ton}{m}$$

$$P_s := p_s \cdot l = 5.157 \ ton$$

$$P_s := P_s \cdot 9.81 \ \frac{m}{s^2} = 10.318 \ kip$$

Determine Full and hoisting load displacements:

filling factor: (DIN 3060) $f := .5278$

Total cross-section area: main cables $A_{tot} := n \cdot \frac{\pi \cdot \phi_M^{\ 2}}{4} \cdot f = \left(1.698 \cdot 10^3\right)\ mm^2$

Length of DL cable: $L_d := l \cdot \left(1 + \frac{8}{3} \cdot \left(\frac{f_d}{l}\right)^2 - \frac{32}{5} \cdot \left(\frac{f_d}{l}\right)^4\right) = 87.643\ m$

Horizontal cable tension for DL: $H_d := \frac{g_d \cdot l^2}{8 \cdot f_d} = 11.482\ ton$

Horizontal cable tension for LL: $H_d := \frac{\left(g_d + p_s\right) \cdot l^2}{8 \cdot f_d} = 16.304\ ton$

dead load main cable tension: $T_d := \frac{\left(g_d + p_s\right) \cdot l^2}{8 \cdot f_d} \cdot \sqrt{1 + 16 \cdot \left(\frac{f_d}{l}\right)^2} = 18.489\ ton$

$a := 16 \cdot \left(\frac{f_d}{l}\right) \cdot \left(5 - 24 \cdot \left(\frac{f_d}{l}\right)^2\right) = 9.777$

$b := 15 - 8 \cdot \left(\frac{f_d}{l}\right)^2 \cdot \left(5 - 36 \cdot \left(\frac{f_d}{l}\right)^2\right) = 14.377$

$\beta_f := \beta = 32.17\ deg$

$f_h := .98 \cdot f_d = 10.981\ m$

Iteration 1:

initial full load sag guess: $f_f := 1.05 \cdot f_d = 11.766\ m$

Horizontal cable tension for LL: $H_1 := \frac{\left(g_f\right) \cdot l^2}{8 \cdot f_f} = 83.627\ ton$

$T_1 := H_1 \cdot \sqrt{1 + 16 \cdot \left(\frac{f_f}{l}\right)^2} = 95.907\ ton$

change in length due to LL: $\Delta L := \frac{\left(2 \cdot H_1 \cdot T_1\right) \cdot L_d}{3 \cdot E \cdot A_{tot}} \cdot \frac{\left(g_l - g_d - p_s\right)}{g_l} = \left(1.351 \cdot 10^4\right)\ kg \cdot m$

$\Delta L := .041505 \cdot m$

average tension in main cable: $T_{avg} := \frac{2 \cdot H_1 + T_1}{3} = 87.72\ ton$

max tension in all cables: full loading
$$T_{f\_max} := \frac{g_f \cdot l^2}{8 \cdot f_f} \cdot \sqrt{1 + 16 \cdot \left(\frac{f_f}{l}\right)^2} = 95.907\ \mathit{ton}$$

max hoisting load in all main: cables
$$T_h := \frac{g_h \cdot l^2}{8 \cdot (f_d)} \cdot \sqrt{1 + 16 \cdot \left(\frac{f_d}{l}\right)^2} = 1.386\ \mathit{ton}$$

Total Length of main cable: $L_{tot} := 572.23\ \mathit{ft}$

deflection due to LL: $\Delta f_f := f_f - f_d = 1.838\ \mathit{ft}$

deflection due to hoisting: $\Delta f_h := f_h - f_d = -0.735\ \mathit{ft}$

Tower Calculations:

Dead load: $q_l := 90\ \mathit{psf}$

Live Load: $g_{d\_ft} := g_d \cdot 9.81\ \frac{m}{s^2} = 89.337\ \frac{\mathit{lbf}}{\mathit{ft}}$

Live load on each tower: $P_{LL} := \frac{q_l \cdot 275\ \mathit{ft} \cdot 4\ \mathit{ft}}{2} = 49.5\ \mathit{kip}$

Dead load on each tower: $P_{DL} := \frac{g_{d\_ft} \cdot 275\ \mathit{ft}}{2} = 12.284\ \mathit{kip}$

horizontal load on top of tower: due to wind on span

Area of components: $A_{main} := 1.5\ \mathit{in} \cdot L_{tot} = 71.529\ \mathit{ft}^2$

$A_{suspend} := .5 \cdot \mathit{in} \cdot 25\ \mathit{ft} \cdot 88 = 91.667\ \mathit{ft}^2$

$A_{deck} := 42\ \mathit{in} \cdot 275\ \mathit{ft} = 962.5\ \mathit{ft}^2$

Total area: $A_{total} := A_{main} + A_{suspend} + A_{deck} = 1125.695\ \mathit{ft}^2$

Resultant load on each tower: $F_{wind} := \frac{A_{total} \cdot 35\ \mathit{psf}}{2} = 19.7\ \mathit{kip}$

- wind loading of 35psf applied on RAM model
- wind load of 35psf applied to span cables
- deflection of .948 in horizontally (WL+1.2DL)
- deflection of .082in vertically (1.2DL+1.6LL)
- factored forces at each pin:
  - Fy = 58.88k
  - Fz = 13.72k
  - M_wind = 60k-ft
  - Uplift due to wind on 1 tower = 69.12k
  - Downforce due to wind on 1 tower = 107.64k

- Tower materials
  vertical members = L4x4x5/8"
  webbing = 5/8" rounds
  bracing = L4x4x3/8"

Anchor Block Loads:

Max angled tension at anchor block: $T_{f\_max} := T_{f\_max} \cdot 9.81 \ \frac{m}{s^2} = 191.88 \ kip$

horizontal force component: $T_{max\_h} := T_{f\_max} \cdot \cos\left(\beta_f\right) = 162.421 \ kip$

vertical force component: $T_{max\_v} := T_{f\_max} \cdot \sin\left(\beta_f\right) = 102.163 \ kip$

unit weight of concrete: $\gamma_c := 150 \ pcf$

preliminary anchor length and width: $L_{anch} := 20 \ ft$ $\quad W_{anch} := 8 \ ft$

required thickness for uplift: $T_{anch\_up} := \frac{T_{max\_v} \cdot 1.5}{\gamma_c \cdot L_{anch} \cdot W_{anch}} = 6.385 \ ft$

required thickness for sliding: $T_{anch\_sl} := \frac{T_{max\_h} \cdot 1.5}{\gamma_c \cdot L_{anch} \cdot W_{anch}} = 10.151 \ ft$

Preliminary anchor block dimensions w/ FS = 1.5 for both:
L = 20 ft
W = 8 ft
T = 6 ft

Cable Checks:

Main Cable Properties (LRFD)			
Load Case	**Load (plf)**	**Tension (ton)**	**Sag (ft)**
Hoisting	9.5	1.39	-0.735
Dead Load	90	18.49	0
Live Load	360	NA	NA
Full Load	450	95.08	1.84

Main cables:

full load value unfactored: $g_f := g_d + g_l = 0.737 \ \frac{ton}{m}$

max tension in all cables:
full loading unfactored
$$T_{f\_max} := \frac{g_f \cdot l^2}{8 \cdot f_f} \cdot \sqrt{1 + 16 \cdot \left(\frac{f_f}{l}\right)^2} = 63.077 \ ton$$

Max tension in (4) main cables: $T_{f\_max} := T_{f\_max} \cdot 9.81 \ \frac{m}{s^2} = 126.197 \ kip$

Max tension per cable: $T_{f\_max\_one} := \frac{T_{f\_max}}{4} = 31.549 \ kip$

Safe Load for 1 1/2" IPS: $T_{safe} := 36.8 \ kip$

Conditional statement: $\text{if}\left(T_{safe} \geq T_{f\_max\_one}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

(4) 1 - 1/2" main cables

Spanning cables:

Pretension spanning cables:
(pretension with 10% of gd)

$p_s := .42 \cdot g_d = 0.062 \ \frac{ton}{m}$

$P_s := p_s \cdot l = 5.157 \ ton$

$P_s := P_s \cdot 9.81 \ \frac{m}{s^2} = 10.318 \ kip$

Max tension per cable: $P_{s\_one} := P_s = 10.318 \ kip$

Safe Load for 1 1/2" IPS: $T_{safe} := 36.8 \ kip$

Conditional statement: $\text{if}\left(T_{safe} \geq P_{s\_one}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

(2) 1 - 1/2" spanning cables

Suspender Cables:

Suspender spacing: $S_{susp} := 3.125\ ft$

Max load on each cable: $T_{susp\_one} := \frac{g_{d\_ft} \cdot S_{susp}}{2} = 139.589\ lbf$

Safe Load for 1/2" IPS: $T_{safe} := 4280\ lbf$

Conditional statement: $\text{if}\left(T_{safe} \geq T_{susp\_one}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

1/2" suspending cables @ 3.125' spacing

Appendix F:
Tower Calculations

Tower Calculations:

Load determination:

Dead load: $q_l := 90\ psf$

Live Load: $g_d := .147 \cdot \frac{ton}{m}$

$g_{d\_ft} := g_d \cdot 9.81\ \frac{m}{s^2} = 89.642\ \frac{lbf}{ft}$

Live load on each tower: $P_{LL} := \frac{q_l \cdot 275\ ft \cdot 4\ ft}{2} = 49.5\ kip$

Dead load on each tower: $P_{DL} := \frac{g_{d\_ft} \cdot 275\ ft}{2} = 12.326\ kip$

$P_{u\_tower} := 1.2 \cdot P_{DL} + 1.6 \cdot P_{LL} = 93.991\ kip$

Length of main cable: $L_{tot} := 572.23\ ft$

Horizontal load on top of tower due to wind on span:

Area of components: $A_{main} := 1.5\ in \cdot L_{tot} = 71.529\ ft^2$

$A_{suspend} := .5 \cdot in \cdot 25\ ft \cdot 88 = 91.667\ ft^2$

$A_{deck} := 42\ in \cdot 275\ ft = 962.5\ ft^2$

Total area: $A_{total} := A_{main} + A_{suspend} + A_{deck} = 1125.695\ ft^2$

Resultant load on each tower: $F_{wind} := \frac{A_{total} \cdot 35\ psf}{2} = 19.7\ kip$

- wind loading of 35psf applied on RAM model
- wind load of 35psf applied to span cables
- deflection of .948 in horizontally (WL+1.2DL)
- deflection of .082in vertically (1.2DL+1.6LL)
- factored forces at each pin:
  - Fy = 58.88k
  - Fz = 13.72k
  - M_wind = 60k-ft
  - Uplift due to wind on 1 tower = 69.12k
  - Downforce due to wind on 1 tower = 107.64k

- Tower materials
  vertical members = L4x4x5/8"
  webbing = 5/8" rounds
  bracing = L4x4x3/8"

Member Sizing Checks:

Webbing check:

Bottom 2 section webbing

Max axial force bottom section: $P_u := 6912\ lbf$ Wind loading RAM Model
1" bar required

Input Variables: $E := 29000\ ksi$ $F_y := 36\ ksi$ $d_{bar} := 1\ in$ $L := 2.25\ ft$

$$A_{bar} := \frac{\pi}{4} \cdot d_{bar}^{\ 2} = 0.785\ in^2$$

radius of gyration: $r := \frac{d_{bar}}{4} = 0.25\ in$

$K := 1.0$ pinned pinned

Elastic buckling stress: $F_e := \frac{\pi^2 \cdot E}{\left(\frac{K \cdot L}{r}\right)^2} = 24.539\ ksi$ E3-3

Critical stress: $\mathrm{F_{cr}} := \mathrm{if}\left(\frac{K \cdot L}{r} \le 4.71 \cdot \sqrt{\frac{E}{F_y}}, .658^{\frac{F_y}{F_e}} \cdot F_y, .877 \cdot F_e\right) = 19.482\ ksi$ E3-2/E3-3

$$\phi P_n := .9 \cdot \mathrm{F_{cr}} \cdot A_{bar} = 13.771\ kip$$

$$\mathrm{if}\left(\phi P_n \ge P_u, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

Use 1" A36 bar

Upper section webbing

Max axial force bottom section: $P_u := 777\ lbf$ Wind loading RAM Model
5/8" bar required

Input Variables: $E := 29000\ ksi$ $F_y := 36\ ksi$ $d_{bar} := \frac{5}{8}\ in$ $L := 2.25\ ft$

$$A_{bar} := \frac{\pi}{4} \cdot d_{bar}^{\ 2} = 0.307\ in^2$$

radius of gyration: $r := \frac{d_{bar}}{4} = 0.156\ in$

$K := 1.0$ pinned pinned

Elastic buckling stress: $F_e := \dfrac{\pi^2 \cdot E}{\left(\dfrac{K \cdot L}{r}\right)^2} = 9.585\ ksi$ E3-3

Critical stress: $\mathrm{F_{cr}} := \mathrm{if}\left(\dfrac{K \cdot L}{r} \le 4.71 \cdot \sqrt{\dfrac{E}{F_y}}, .658^{\frac{F_y}{F_e}} \cdot F_y, .877 \cdot F_e\right) = 8.406\ ksi$ E3-2/E3-3

$$\phi P_n := .9 \cdot \mathrm{F_{cr}} \cdot A_{bar} = 2.321\ kip$$

$$\mathrm{if}\left(\phi P_n \ge P_u, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

Use 5/8" A36 bar

Upper Angle Check:

Max axial force upper sections: $P_u := 30326\ lbf$ Wind loading RAM Model
L4x4x3/8" reqd'

Input Variables: $E := 29000\ ksi$ $F_y := 36\ ksi$ $L := 5\ ft$

$K := 1.0$ pinned pinned

Effective length: $K \cdot L = 5\ ft$

$\phi P_n := 40.1\ kip$ T4-12 pg. 4-194

$$\mathrm{if}\left(\phi P_n \ge P_u, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

Use L4x4x3/8" A36

Cross bracing Check:

Max axial force cross bracing: $P_u := 8667\ lbf$ Wind loading RAM Model
L4x4x3/8" reqd'

Input Variables: $K := 1.0$ pinned pinned $E := 29000\ ksi$ $F_y := 36\ ksi$ $L := 14.5\ ft$

Effective length: $K \cdot L = 14.5\ ft$

$\phi P_n := 12.6\ kip$ T4-12 pg. 4-194

$$\mathrm{if}\left(\phi P_n \ge P_u, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

Use L4x4x3/8" A36

Lower vertical check:

Max axial force lower sections: $P_u := 47652\ lbf$ Wind loading RAM Model
HSS4x4x3/8" reqd'

Input Variables: $E := 29000\ ksi$ $F_y := 36\ ksi$ $L := 5\ ft$

$K := 1.0$ pinned pinned

Effective length: $K \cdot L = 5\ ft$

$\phi P_n := 177\ kip$ T4-4 pg. 4-63

$\text{if}\left(\phi P_n \geq P_u, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

Use L4x4x3 / 8" A36

Connection Checks:

Cross-bracing:

Max axial force cross bracing: $P_u := 8667\ lbf$ Wind loading RAM Model
3 5/8" bolts

A307 5/8" bolts typ.: $\phi r_{n\_shear} := 6.23\ kip$ $\phi r_{n\_tens} := 10.4\ kip$ T7-2 pg. 7-22

Number of bolts: $n := 3$

$\text{if}\left(n \cdot \phi r_{n\_shear} \ge P_u, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

Check for tensile rupture/yield: $A_g := 2.86\ in^2$ $F_y := 36\ ksi$ $F_u := 58\ ksi$

tensile yielding: $\phi P_n := .9 \cdot F_y \cdot A_g = 92.664\ kip$

$\text{if}\left(\phi P_n \ge P_u, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

tensile rupture: $A_n := 7.5\ in \cdot \frac{3}{8} \cdot in - 2.5 \cdot \frac{1}{2} \cdot in \cdot \frac{3}{8} \cdot in + 1\ in \cdot \frac{3}{8} \cdot in = 2.719\ in^2$

$A_e := 1.0 \cdot A_n$

$\phi P_n := F_u \cdot A_e = 157.688\ kip$

$\text{if}\left(\phi P_n \ge P_u, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

Connection OK

Vertical section connection:

Max axial force upper verticals: $P_u := 30326\ lbf$ Wind loading RAM Model
2 5/8" bolts

A307 5/8" bolts typ.: $\phi r_{n\_shear} := 6.23\ kip$ $\phi r_{n\_tens} := 10.4\ kip$ T7-2 pg. 7-22

Number of bolts: $n := 6$ 3 bolts per face x 2 faces

$\text{if}\left(n \cdot \phi r_{n\_shear} \ge P_u, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

tensile yielding: $\phi P_n := .9 \cdot F_y \cdot A_g = 92.664\ kip$

$\text{if}\left(\phi P_n \ge P_u, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

tensile rupture: $A_n := 10\ in \cdot \frac{3}{8} \cdot in - 2.5 \cdot \frac{1}{2} \cdot in \cdot \frac{3}{8} \cdot in + 1\ in \cdot \frac{3}{8} \cdot in = 3.656\ in^2$

$A_e := 1.0 \cdot A_n$

$\phi P_n := F_u \cdot A_e = 212.063\ kip$

$\text{if}\left(\phi P_n \ge P_u, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

Connection OK

Base plate design:

$$P_u := \frac{P_{u\_tower}}{2} = 46.995\ kip$$

Base plate dimensions: $L := 36\ in$ $W := 8\ in$ $t := 2\ in$

$m := \frac{W}{2} = 0.333\ ft$ $n := \frac{L}{2} = 1.5\ ft$ AISC 14-5

$$n' := \frac{\sqrt{2\ in \cdot 24\ in}}{4} = 0.144\ ft$$

$l := \max\left(m, n, n'\right) = 1.5\ ft$

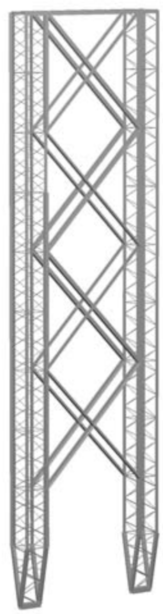
min. plate thickness: $t_{min} := l \cdot \sqrt{\frac{2 \cdot P_u}{.9 \cdot F_y \cdot L \cdot W}} = 1.807\ in$ 14-7a

$\text{if}\left(t \ge t_{min}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

Concrete bearing: $\phi P_p := .65 \cdot .85 \cdot 2500\ psi \cdot L \cdot W = 397.8\ kip$

$\text{if}\left(\phi P_p \ge P_u, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

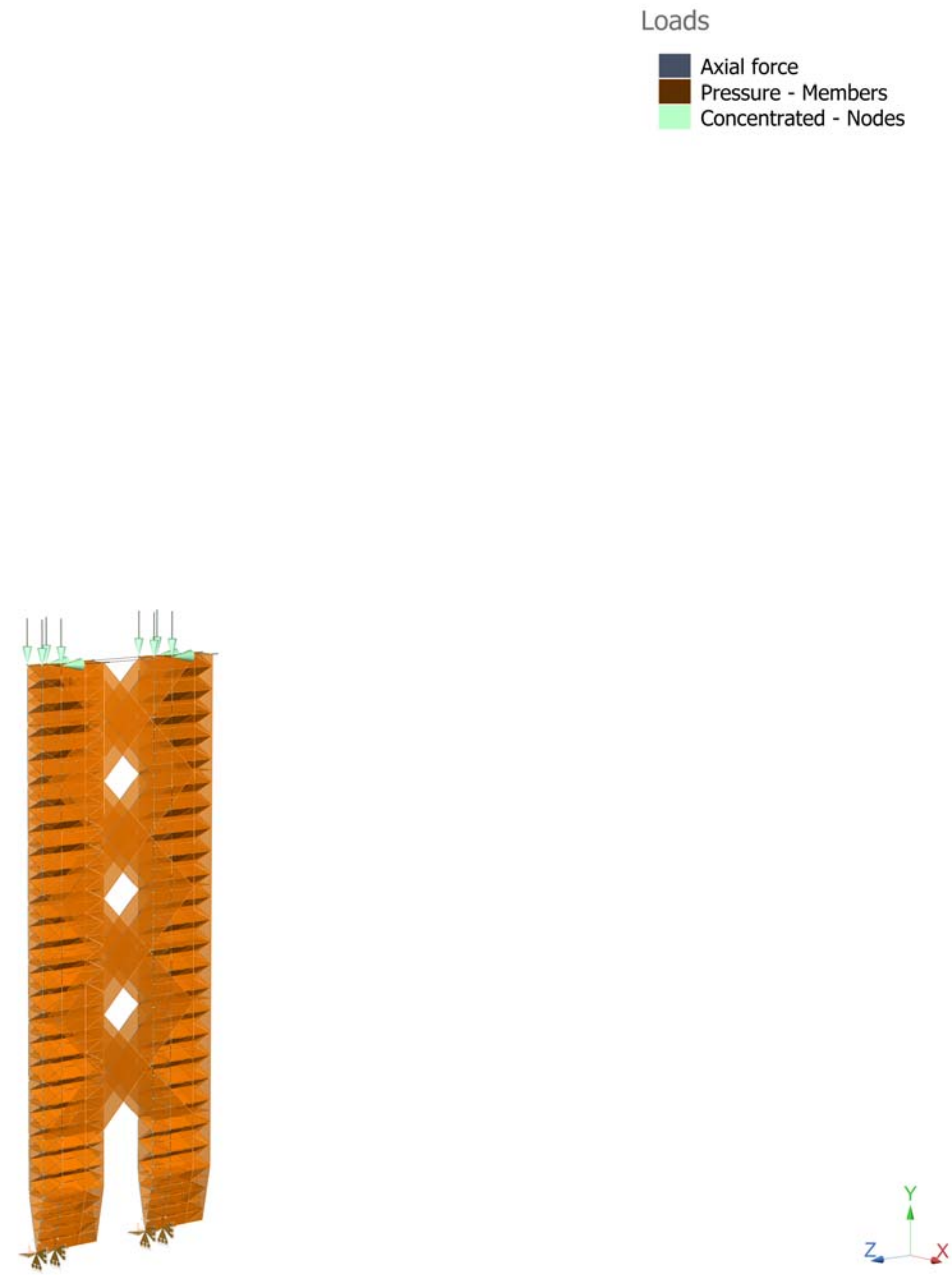
Base plate OK

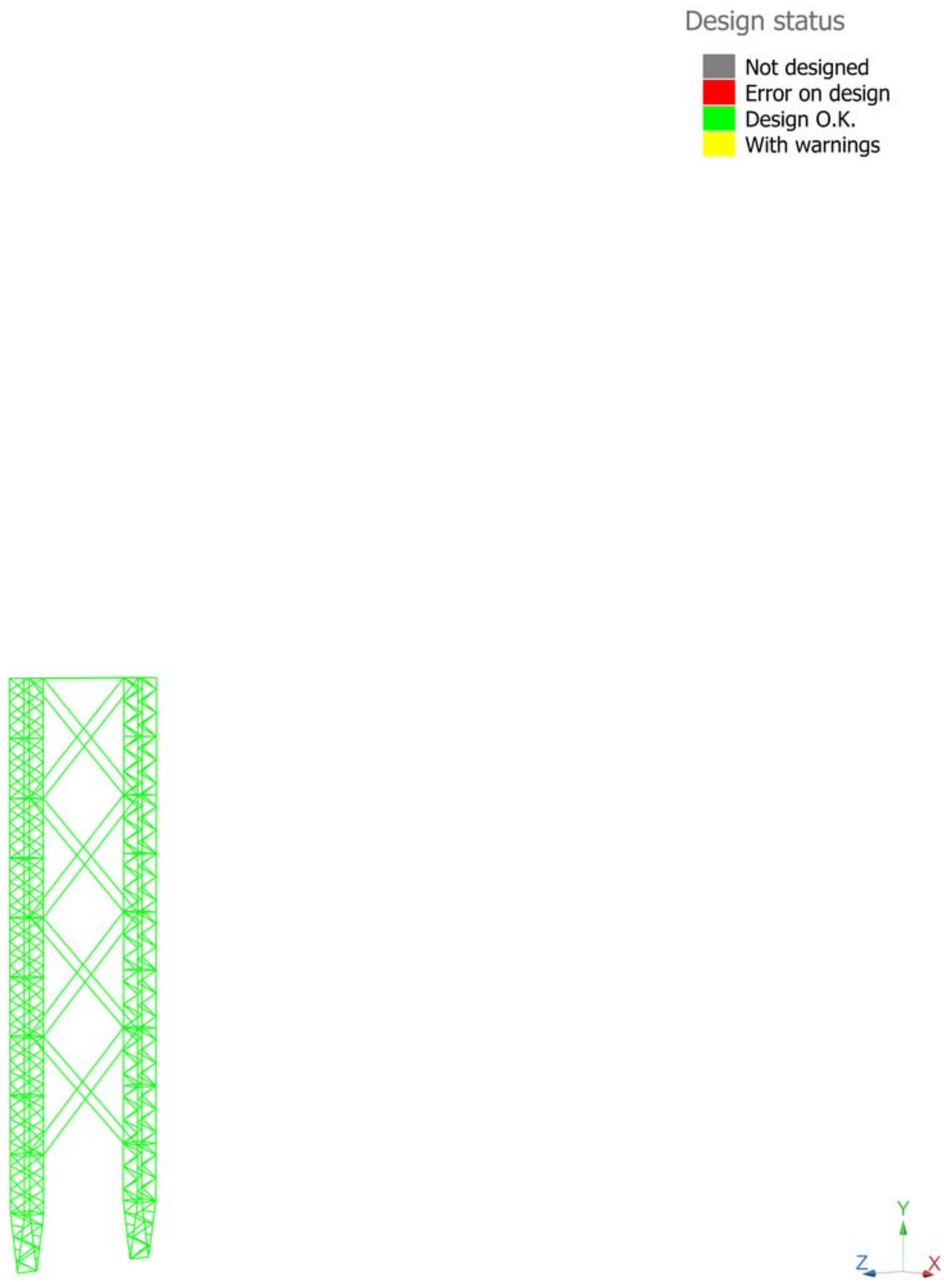



Y
Z
X

Bentley MTU

Current Date: 10/28/2013 1:39 PM
Units system: English
File name: \\mtucifs1.iso.mtu.edu\home\Desktop\iDesign\Analysis\TOWERS 5.etz\
Load condition: wind=1.2DL+WL




Design status
Not designed
Error on design
Design O.K.
With warnings
Y
Z
X

Bentley MTU

Current Date: 10/28/2013 1:42 PM
Units system: English
File name: \\mtucifs1.iso.mtu.edu\home\Desktop\iDesign\Analysis\TOWERS 5.etz\

List of materials

Note.- Only the graphically selected members and shells are listed

Members:

Profile	Material	Uweight [Lb/ft]	Length [ft]	Weight [Lb]
HSS_SQR 4X4X3_8	A36	1.63E+01	40.792	664.514
HSS_SQR 6X6X3_8	A36	2.58E+01	4.000	103.331
L 4X4X3_8	A36	9.75E+00	614.274	5987.257
RNDBAR 1	A36	2.67E+00	105.621	282.465
RNDBAR 5_8	A36	1.04E+00	920.263	961.358
Total weight [Lb]				7998.924

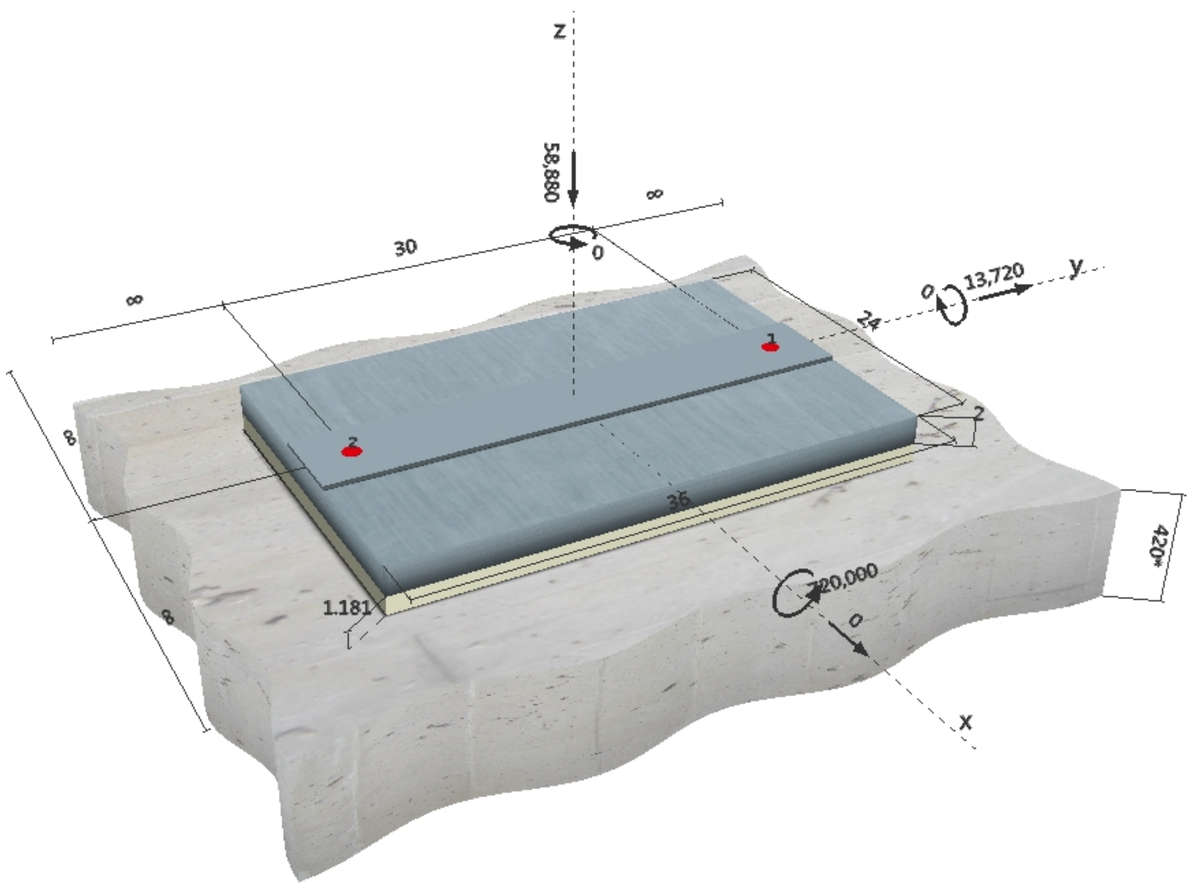
Company:	del Puente Engineering	Page:	1
Specifier:	Wes Karras	Project:	Chucanaque Bridge
Address:		Sub-Project I Pos. No.:	
Phone I Fax:	|	Date:	10/30/2013
E-Mail:			

Specifier's comments:

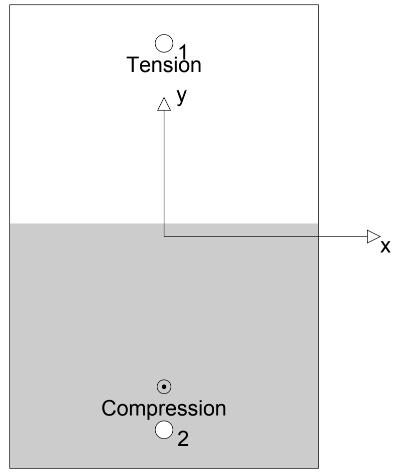
1 Input data

Anchor type and diameter:	**HIT-HY 200 + HAS B7 1 1/4**
Effective embedment depth:	$h_{ef,opti}$ = 5.000 in. ($h_{ef,limit}$ = 25.000 in.)
Material:	ASTM A 193 Grade B7
Evaluation Service Report:	ESR-3187
Issued I Valid:	4/1/2013 | 3/1/2014
Proof:	design method ACI 318 / AC308
Stand-off installation:	without clamping (anchor); restraint level (anchor plate): 2.0; e_b = 1.181 in.; t = 2.000 in.
	Hilti Grout: CB-G EG, epoxy, $f_{c,Grout}$ = 14939 psi
Anchor plate:	l_x x l_y x t = 24.000 in. x 36.000 in. x 2.000 in.; (Recommended plate thickness: not calculated)
Profile:	Rectangular plates and bars (AISC); (L x W x T) = 6.000 in. x 36.000 in. x 0.000 in.
Base material:	cracked concrete, 2500, f_c' = 2500 psi; h = 420.000 in., Temp. short/long: 32/32 °F
Installation:	hammer drilled hole, installation condition: dry
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	no

Geometry [in.] & Loading [lb, in.lb]



2 Load case/Resulting anchor forces



Load case: Design loads

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	1252	6860	0	6860
2	0	6860	0	6860

max. concrete compressive strain: 0.06 [‰]
max. concrete compressive stress: 264 [psi]
resulting tension force in (x/y)=(0.000/15.000): 1252 [lb]
resulting compression force in (x/y)=(0.000/-11.661): 60132 [lb]

3 Tension load

	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	1252	90851	2	OK
Bond Strength**	1252	11618	11	OK
Concrete Breakout Strength**	1252	6177	21	OK

* anchor having the highest loading **anchor group (anchors in tension)

3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-3187
$\phi N_{steel} \geq N_{ua}$ ACI 318-08 Eq. (D-1)

Variables

n	$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
1	0.97	125000

Calculations

N_{sa} [lb]
121135

Results

N_{sa} [lb]	ϕ_{steel}	ϕN_{sa} [lb]	N_{ua} [lb]
121135	0.750	90851	1252

3.2 Bond Strength

$N_{ag} = \left(\frac{A_{Na}}{A_{Na0}}\right) \psi_{ed,Na}\ \psi_{g,Na}\ \psi_{ec,Na}\ \psi_{p,Na}\ N_{a0}$ ICC-ES AC308 Eq. (D-16b)

$\phi\ N_{ag} \geq N_{ua}$ ACI 318-08 Eq. (D-1)

A_{Na} = see ICC-ES AC308, Part D.5.3.7

$A_{Na0} = s^2_{cr,Na}$ ICC-ES AC308 Eq. (D-16c)

$s_{cr,Na} = 20d \sqrt{\frac{\tau_{k,uncr}}{1450}} \leq 3\ h_{ef}$ ICC-ES AC308 Eq. (D-16d)

$c_{cr,Na} = \frac{s_{cr,Na}}{2}$ ICC-ES AC308 Eq. (D-16e)

$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{cr,Na}}\right) \leq 1.0$ ICC-ES AC308 Eq. (D-16m)

$\psi_{g,Na} = \psi_{g,Na0} + \left[\left(\frac{s_{avg}}{s_{cr,Na}}\right)^{0.5} \cdot (1 - \psi_{g,Na0})\right] \geq 1.0$ ICC-ES AC308 Eq. (D-16g)

$\psi_{g,Na0} = \sqrt{n} - \left[(\sqrt{n} - 1) \cdot \left(\frac{\tau_{k,c}}{\tau_{k,max,c}}\right)^{1.5}\right] \geq 1.0$ ICC-ES AC308 Eq. (D-16h)

$\tau_{k,max,c} = \frac{k_c}{\pi \cdot d} \sqrt{h_{ef} \cdot f'_c}$ ICC-ES AC308 Eq. (D-16i)

$\psi_{ec,Na} = \left(\frac{1}{1 + \frac{2e''_N}{s_{cr,Na}}}\right) \leq 1.0$ ICC-ES AC308 Eq. (D-16j)

$\psi_{p,Na} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{c_{cr,Na}}{c_{ac}}\right) \leq 1.0$ ICC-ES AC308 Eq. (D-16p)

$N_{a0} = \tau_{k,c} \cdot \kappa_{bond} \cdot \pi \cdot d \cdot h_{ef}$ ICC-ES AC308 Eq. (D-16f)

Variables

$\tau_{k,c,uncr}$ [psi]	d_{anchor} [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	s_{avg} [in.]	n	$\tau_{k,c}$ [psi]
1880	1.250	5.000	∞	30.000	1	910

k_c	f'_c [psi]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	c_{ac} [in.]	κ_{bond}
17	2500	0.000	0.000	5.664	1.00

Calculations

$s_{cr,Na}$ [in.]	$c_{cr,Na}$ [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$	$\tau_{k,max}$ [psi]
15.000	7.500	225.00	225.00	1.000	484

$\psi_{g,Na0}$	$\psi_{g,Na}$	$\psi_{ec1,Na}$	$\psi_{ec2,Na}$	$\psi_{p,Na}$	N_{a0} [lb]
1.000	1.000	1.000	1.000	1.000	17874

Results

N_{ag} [lb]	ϕ_{bond}	$\phi\ N_{ag}$ [lb]	N_{ua} [lb]
17874	0.650	11618	1252

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E-Mail:			

3.3 Concrete Breakout Strength

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ ACI 318-08 Eq. (D-5)

$\phi\ N_{cbg} \geq N_{ua}$ ACI 318-08 Eq. (D-1)

A_{Nc} see ACI 318-08, Part D.5.2.1, Fig. RD.5.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-08 Eq. (D-6)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e_N'}{3\ h_{ef}}}\right) \leq 1.0$ ACI 318-08 Eq. (D-9)

$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-08 Eq. (D-11)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-08 Eq. (D-13)

$N_b = k_c\ \lambda\ \sqrt{f_c'}\ h_{ef}^{1,5}$ ACI 318-08 Eq. (D-7)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
5.000	0.000	0.000	∞	1.000

c_{ac} [in.]	k_c	λ	f_c' [psi]
5.664	17	1	2500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
225.00	225.00	1.000	1.000	1.000	1.000	9503

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
9503	0.650	6177	1252

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	6860	37794	19	OK
Steel failure (with lever arm)*	6860	9223	75	OK
Pryout Strength (Concrete Breakout Strength controls)**	13720	26609	52	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (relevant anchors)

4.1 Steel Strength

V_{sa} = ESR value refer to ICC-ES ESR-3187
$\phi\, V_{steel} \geq V_{ua}$ ACI 318-08 Eq. (D-2)

Variables

n	$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
1	0.97	125000

Calculations

V_{sa} [lb]
72680

Results

V_{sa} [lb]	ϕ_{steel}	ϕ_{eb}	$\phi\, V_{sa}$ [lb]	V_{ua} [lb]
72680	0.650	0.800	37794	6860

4.2 Steel failure (with lever arm)

$V_s^M = \frac{\alpha_M \cdot M_s}{L_b}$ bending equation for stand-off

$M_s = M_s^0\left(1 - \frac{N_{ua}}{\phi N_{sa}}\right)$ resultant flexural resistance of anchor

$M_s^0 = (1.2)\,(S)\,(f_{u,min})$ characteristic flexural resistance of anchor

$\left(1 - \frac{N_{ua}}{\phi N_{sa}}\right)$ reduction for tensile force acting simultaneously with a shear force on the anchor

$S = \frac{\pi(d)^3}{32}$ elastic section modulus of anchor bolt at concrete surface

$L_b = z + (n)(d_0)$ internal lever arm adjusted for spalling of the surface concrete

$\phi V_s^M \geq V_{ua}$ ACI 318-08 Eq. (D-2)

Variables

α_M	$f_{u,min}$ [psi]	N_{ua} [lb]	ϕN_{sa} [lb]	z [in.]	n	d_0 [in.]
2.00	125000	1252	90851	2.181	0.500	1.250

Calculations

M_s^0 [in.lb]	$\left(1 - \frac{N_{ua}}{\phi N_{sa}}\right)$	M_s [in.lb]	L_b [in.]
20186.542	0.986	19908.375	2.806

Results

V_s^M [lb]	ϕ_{steel}	ϕV_s^M [lb]	V_{ua} [lb]
14190	0.650	9223	6860

4.3 Pryout Strength (Concrete Breakout Strength controls)

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b \right]$ ACI 318-08 Eq. (D-31)

$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-08 Eq. (D-2)

A_{Nc} see ACI 318-08, Part D.5.2.1, Fig. RD.5.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-08 Eq. (D-6)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e_N'}{3\ h_{ef}}} \right) \leq 1.0$ ACI 318-08 Eq. (D-9)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}} \right) \leq 1.0$ ACI 318-08 Eq. (D-11)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-08 Eq. (D-13)

$N_b = k_c\ \lambda\ \sqrt{f_c'}\ h_{ef}^{1.5}$ ACI 318-08 Eq. (D-7)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	5.000	0.000	0.000	∞

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ	f_c' [psi]
1.000	5.664	17	1	2500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
450.00	225.00	1.000	1.000	1.000	1.000	9503

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
38013	0.700	26609	13720

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.203	0.744	5/3	69	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

6 Warnings

- To avoid failure of the anchor plate the required thickness can be calculated in PROFIS Anchor. Load re-distributions on the anchors due to elastic deformations of the anchor plate are not considered. The anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the loading!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- ACI 318 does not specifically address anchor bending when a stand-off condition exists. PROFIS Anchor calculates a shear load corresponding to anchor bending when stand-off exists and includes the results as a shear Design Strength!
- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions
- The present version of the software does not account for adhesive anchor special design provisions corresponding to overhead applications. Refer to the ICC-ES Evaluation Service Report (e.g. section 4.1.1 of the ICC-ESR 2322) for details.
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!

Fastening meets the design criteria!

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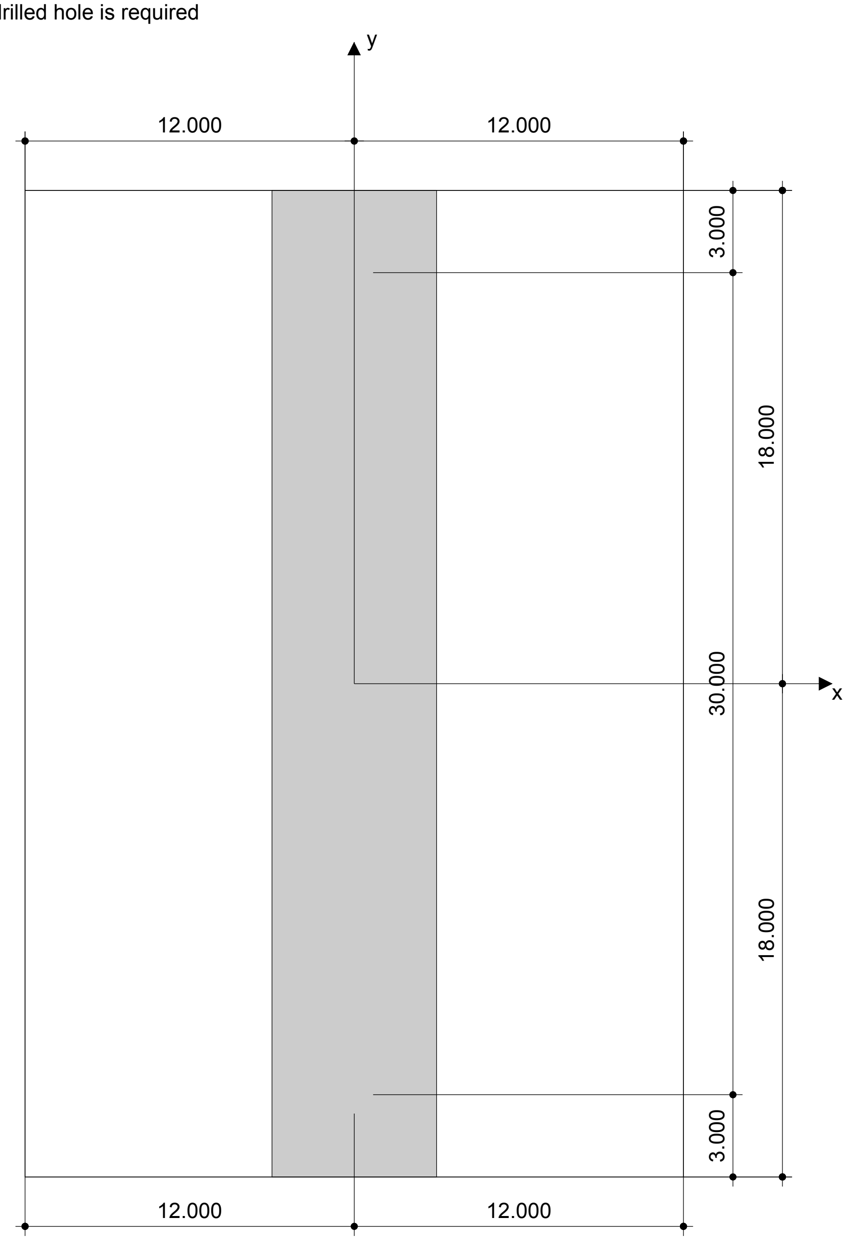
Phone I Fax:	|	Date:	10/30/2013
E-Mail:			

7 Installation data

Anchor plate, steel: -
Profile: Rectangular plates and bars (AISC); 6.000 x 36.000 x 0.000 in.
Hole diameter in the fixture: d_f = 1.375 in.
Plate thickness (input): 2.000 in.
Recommended plate thickness: not calculated
Cleaning: Premium cleaning of the drilled hole is required

Anchor type and diameter: HIT-HY 200 + HAS B7, 1 1/4
Installation torque: 2400.000 in.lb
Hole diameter in the base material: 1.375 in.
Hole depth in the base material: 5.000 in.
Minimum thickness of the base material: 7.750 in.



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	15.000	-	-	-	-
2	0.000	-15.000	-	-	-	-

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8 Remarks; Your Cooperation Duties

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Appendix G:
Walkway Calculations: Wood

This calculates the capacity of a single timber member in strong or weak axis bending.

$$check(var) := \left\| \begin{array}{l} \text{if } var = 1 \\ \quad \left\| \text{"True"} \right. \\ \text{else} \\ \quad \left\| \text{"False"} \right. \end{array} \right.$$

Geometric Properties: Determined from Table 1B (pg 14) in the NDS Supplement.

Inputs $b := 6\ in$ $\quad d := 2\ in$ $\quad L := 3.125\ ft$

$A := b \cdot d = 12\ in^2$

$I_x := \dfrac{(b \cdot d^3)}{12} = 4\ in^4$ $\quad S_x := \dfrac{(2 \cdot I_x)}{d} = 4\ in^3$ $\quad r_x := \sqrt[2]{\dfrac{I_x}{A}} = 0.577\ in$

$I_y := \dfrac{(d \cdot b^3)}{12} = 36\ in^4$ $\quad S_y := \dfrac{(2 \cdot I_y)}{b} = 12\ in^3$ $\quad r_y := \sqrt[2]{\dfrac{I_y}{A}} = 1.732\ in$

$t_A := 3.125\ ft$ tributary area (vertical cable spacing)

Species Properties: Determined from [http://www.lumbermax.biz/species/almendro.php]

$F_b := 13576\ psi$ ←Green Properties bending strength

$F_v := 2441\ psi$ shear strength

$E := 2690\ psi$ modulus of elasticity

Maximum stresses: (from models)

$w_p := 60\ psf$ pedestrian load

$w := w_p \cdot 6\ in$

$P := 1\ kip$ equestrian load

$f_b := F_b$

$f_v := F_v$

Bending Allowable: Determined from NDS Specifications 3.3 (pg 13) and Table 4.3.1

$C_D := 0.9$ Estimated from Figure B1 NDS Specification (p. 147) - greater than a 10 year Load Duration

$C_M := 1.0$ (equal to 1.0) by Table 10.3.3 in NDS (p. 59) - Assume threaded nail

$C_t := 1.0$ (equal to 1.0 if temperature < 100F) by 10.3.4 in NDS Specification (p. 59)

$C_F := 1.3$ NDS Supplement p. 30

$C_r := 1.15$ Increase for repetive members by 3.9 NDS Suppliment (p. 30)

$C_i := 1.0$ 4.3.8 NDS Specification (p.27) when at described measurements

$C_f := 1.0$ (equal to 1.0 if not a circular or diamond section) by 3.3.4 in NDS Spec. (p. 15)

$C_T := 1.0$ Increase for combined truss member by NDS Specification (p. 28)

$C_{fu} := 1.0$ Flat Factor Use NDS 4.3.7

Calculate Effective Length :

$\frac{L}{d} = 18.75$ $L_e := 1.63 \cdot L + 3 \cdot d = 5.594\ ft$

Uniformly distributed load simple support Table M3.3.3

Calculate Slenderness Ratio:

$$R_B := \sqrt[2]{\frac{(L_e \cdot d)}{b^2}} = 1.931$$

NDS 3.3.3.6 pg 15

$check\ (R_B < 50) = \text{"True"}$

$k_{bE} := 1.20$

Euler buckling coefficient for beams

Calculate critical buckling design value:

$E'_b := E \cdot C_M \cdot C_t \cdot C_i \cdot C_T = 2690\ psi$

Adjusted modulus of elasticity Table 4.

$$F_{bE} := \frac{(k_{bE} \cdot E'_b)}{{R_B}^2} = 865.6\ psi$$

$F_{b.star} := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_f \cdot C_F \cdot C_i \cdot C_r = 18267\ psi$

$$C_L := min\left(\frac{\left(1 + \frac{F_{bE}}{F_{b.star}}\right)}{1.9} - \sqrt[2]{\left(\frac{\left(1 + \frac{F_{bE}}{F_{b.star}}\right)}{1.9}\right)^2 - \left(\frac{\left(\frac{F_{bE}}{F_{b.star}}\right)}{0.95}\right)}, 1.0\right) = 0.047$$

NDS 3.3.3.8

$C_{fu} = 1.0$

$F'_b := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_f \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 863.467\ psi$

Table 4.3.1 NDS pg 27

$F_{b2} := F'_b \cdot b \cdot d = (1.036 \cdot 10^4)\ lbf$

$$f_b := \frac{\left(\frac{(w \cdot L^2)}{8}\right)}{L} = 11.719\ lbf$$

$$I_{flexure} := \frac{f_b}{F_{b2}} = 0.001$$

Unity flexure

$check\ (I_{flexure} < 1) = \text{"True"}$

Shear Capacity Check: determined from NDS Specification 3.4 (pg 17)

$F'_v := F_v \cdot C_D \cdot C_M \cdot C_t \cdot C_i = \left(2.197 \cdot 10^3\right)\ psi$

$f_v := w \cdot \frac{L}{2} = 46.875\ lbf$

$F_{v2} := F'_v \cdot b \cdot d = \left(2.636 \cdot 10^4\right)\ lbf$

$I_{shear} := \frac{f_v}{F_{v2}} = 0.002$

Unity shear

$check\left(I_{shear} < 1\right) = \text{“True”}$

Deflection: Determined from NDS Specification 3.5 (pg 20) does not consider long-term loading

$\Delta_{max} := \frac{L}{180} = 0.208\ in$

deflection limit AASHTO Ped. Sect. 5

Actual Deflections:

$\Delta_1 := \frac{\left(5 \cdot w \cdot L^4\right)}{384 \cdot E \cdot I_y} = 0.665\ in$

Pedestrian and Dead load only

$check\left(\frac{\Delta_1}{\Delta_{max}} < 1.0\right) = \text{“False”}$

*Note: deflection okay under maximum live load

$\Delta_2 := \frac{\left(P \cdot L^3\right)}{48 \cdot ft^3 \cdot E \cdot L} = 0.006\ in$

Horse load only

$check\left(\frac{\Delta_2}{\Delta_{max}} < 1.0\right) = \text{“True”}$

Appendix H:
Walkway Calculations: Steel Angles

Limit States: Shear, flexure, and deflection

$$check(var) := \begin{array}{l} \text{if } var = 1 \\ \quad \text{"True"} \\ \text{else} \\ \quad \text{"False"} \end{array}$$

Geometric Properties: Determined from Table 1B (pg 14) in the NDS Supplement.

$A := 4.22\ in^2$ $\quad L := 4\ ft$ $\quad E := 29000\ psi$

$I_x := 1.75\ in^4$ $\quad S_x := 0.825\ in^3$ $\quad r_x := \sqrt[2]{\frac{I_x}{A}} = 0.644\ in$

$I_y := 1.75\ in^4$ $\quad S_y := 0.825\ in^3$ $\quad r_y := \sqrt[2]{\frac{I_y}{A}} = 0.644\ in$

$F_y := 50\ ksi$

Maximum stresses: (from models)

$w_p := 90\ psf$ pedestrian load

$w := w_p \cdot 6\ in = 0.045\ \frac{kip}{ft}$

$P := 1\ kip$ equestrian load

Calculate Effective Length :

$k := 1.0$ $\quad L_e := k \cdot L = 4\ ft$

Calculate Slenderness Ratio:

$$\frac{(K \cdot L)}{r_x} = 74.538\ K$$

$$check(74.538 < 300) = \text{"True"}$$

Calculate Critical Buckling Design Value:

$$F_e := \frac{(\pi^2\ E)}{\left(\frac{L^2}{{r_x}^2}\right)} = 0.052\ ksi$$

$$check(F_e < 36\ ksi) = \text{"True"}$$

$P_{cr} := F_e \cdot A = 0.217\ kip$ Critical Buckling Load

Shear Capacity Check: determined from AISC Chapter G

$\phi_v := 0.9$ $\qquad C_v := 1$

$$V_{max} := \frac{w \cdot L}{2} = 90\ lbf$$

$$V_n := 0.6\ F_y \cdot A \cdot C_v = \left(1.266 \cdot 10^5\right)\ lbf$$

$$check\left(\phi_v \cdot V_n > V_{max}\right) = \text{“True”}$$

Deflection: determined from AISC Part 3

$$\Delta_{max} := \frac{L}{180} = 0.267\ in$$

deflection limit

Actual Deflections:

$$\Delta_1 := \frac{\left(5 \cdot w \cdot L^4\right)}{384 \cdot E \cdot I_y} = 5.107\ in$$

pedestrian and dead load only

$$check\left(\frac{\Delta_1}{\Delta_{max}} < 1.0\right) = \text{“False”}$$

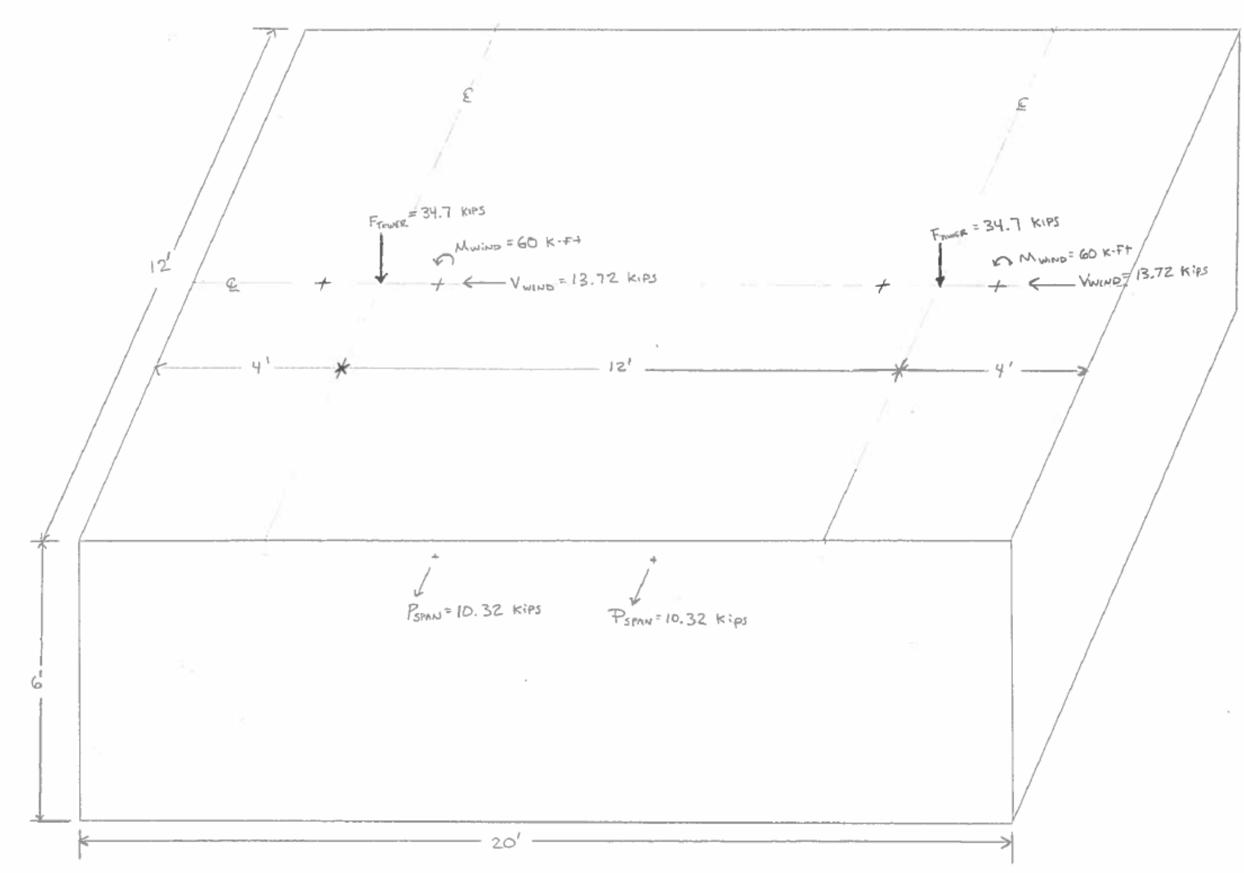
*Note: deflection okay under maximum live load

$$\Delta_2 := \frac{\left(P \cdot L^3\right)}{48 \cdot ft^3 \cdot E \cdot L} = \left(9.579 \cdot 10^{-4}\right)\ in$$

pedestrian and horse loads

$$check\left(\frac{\Delta_2}{\Delta_{max}} < 1.0\right) = \text{“True”}$$

Appendix I:
Tower Foundation Design Calculations



Tower Foundation Calculations:

Dimensions:

$L := 20\ ft$	N-S dimension, parallel to river
$W := 12\ ft$	E-W dimension, perpendicular to river
$H := 6\ ft$	Height of foundation
$Volume := L \cdot W \cdot H = 1440\ ft^3$	Volume of concrete
$Volume = 53.333\ yd^3$	

Force due to concrete weight:

$Wt_{concrete} := 150\ pcf$	Assumed wt. of concrete
$F_{concrete} := Wt_{concrete} \cdot Volume = 216\ kip$	Total force of concrete

Force of tower down:

$F_{tower} := 34.7\ kip$	Total force from tower

Total force down:

$$F_{down} := F_{concrete} + 2\ F_{tower} = 285.4\ kip$$

Soil bearing capacity:

$B := 3000\ psf$ — Assumed bearing capacity of fill

$F_{soil} := B \cdot L \cdot W = 720\ kip$ — Force of soil on bottom of foundation

Factored force of soil up:

$\Omega := 2.5$ — Factor of safety

$\frac{F_{soil}}{\Omega} = 288\ kip$ > $F_{down} = 285.4\ kip$ OK

Check overturning:

Tipping to the north (perpendicular to bridge):

$M_{wind} := 60\ kip \cdot ft$ — moment from tower due to wind

$V_{wind} := 13.72\ kip$ — shear from tower due to wind

$M_{overturning} := 2\ M_{wind} + 2 \cdot 6\ ft \cdot V_{wind} = 284.64\ kip \cdot ft$

$M_{resisting} := F_{concrete} \cdot 10\ ft = 2160\ kip \cdot ft$

Tipping to east/west (towards the bridge deck):

$P_{span} := 10.32\ kip$ — spanning cable force

$M_{overturning} := 2\ P_{span} \cdot 6\ ft = 123.84\ kip \cdot ft$

$M_{resisting} := F_{concrete} \cdot 6\ ft = 1296\ kip \cdot ft$

Appendix J:
Anchor Block Calculations

ANCHOR BLOCKS

Section 8.51 Anchorage in Soil

Table 65: Limits of Dimensions for Main Cable Anchorages

$T_f := 112 \cdot tonne$ Capacity

$T_f := T_f \cdot g = 246.918\ kip$

$ActualT_f := 126.16 \cdot kip$

$B := 18 \cdot ft = 216\ in$ Anchor block width

$L := 26\ ft = 312\ in$ Anchor block depth

$H := 8\ ft = 96\ in$ Anchor block height

$Volume := B \cdot H \cdot L = \left(3.744 \cdot 10^3\right)\ ft^3$

$\gamma_c := 150\ \dfrac{lbf}{ft^3}$

$W_c := Volume \cdot \gamma_c = 561.6\ kip$

Sliding

Friction Factor between concrete and dry gravel = 0.50
http://www.supercivilcd.com/FRICTION.htm

$F := ActualT_f = 126.16\ kip$

$\theta := 30 \cdot °$

$F_x := F \cdot \cos(\theta) = 109.258\ kip$

$F_y := F \cdot \sin(\theta) = 63.08\ kip$

$\mu := 0.50$

$F_{slide} := \mu \cdot \left(W_c - F_y\right) = 249.26\ kip$

$FS_{slide} := \dfrac{F_{slide}}{F_x} = 2.281$

$FS_{slide} \geq 1.5$ Section 8.56 Safety factor against sliding

OVERTURNING

$FS_{overturn} \geq 2.0$

$$FS_{overturn} := \left(W_c - F_y\right) \cdot \frac{\frac{B}{2}}{\left(F_x \cdot H\right)} = 5.133$$

BEARING

$FS_{bearing} \geq 2.5$

$$FS_{bearing} := \frac{3000 \cdot \frac{lbf}{ft^2}}{\frac{W_c - F_y}{(B \cdot L)}} = 2.816$$

Appendix K:
Gabion Calculations

Gabion Design calculations follow the procedure outlined in "Gabion Walls Design."

$$check(var) := \begin{array}{l} \text{if } var = 1 \\ \quad \text{"True"} \\ \text{else} \\ \quad \text{"False"} \end{array}$$

***Assume calculations for a one-foot wide, vertical, section of gabions

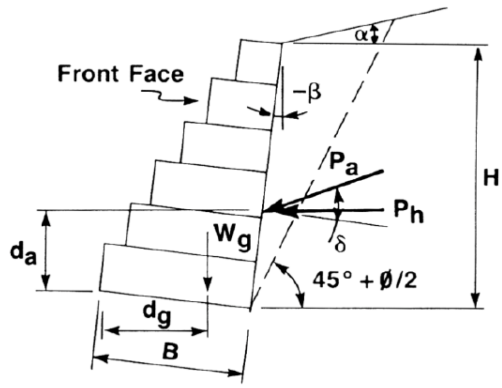
Free Standing without moment resistance for "side" gabion walls.
Conservative assumption

Geometry of Gabion Slope

$\alpha := 0\ deg$ Slop Angle of Backfill

$\beta := 58\ deg$ Acute Angle of Back Face

$\delta := 0\ deg$ Angle of Wall Friction



$\phi := 20\ deg$ Angle of Internal Friction of Soil

↑ *From "Some Useful Numbers"*

$H := 32\ ft$ Height of Gabion Wall

$$B := \frac{55\ ft}{2} = 27.5\ ft$$

A. Stepped Front Face

$$w_s := 1900\ \frac{kg}{m^3} \cdot 9.81\ \frac{m}{s^2} = 118.654\ pcf$$

← *From "Some Useful Numbers"* Soil Density

$F_{foundation} := 60\ kip$ Downward Force in Foundation

$$A_{foundation} := \frac{(12\ ft + 55\ ft)}{2} \cdot \frac{(20\ ft + 40\ ft)}{2} = 1005\ ft^2$$

Cross-sectional Area of Foundation

$$q := \frac{F_{foundation}}{A_{foundation}} = 59.701\ psf$$

Active Soil Pressure of Backfill

$$K_a := \frac{(\cos(\phi - \beta))^2}{(\cos(\beta))^2 \cdot \cos(\delta + \beta) \cdot \left(1 + \sqrt[2]{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \alpha)}{\cos(\delta + \beta) \cdot \cos(\alpha - \beta)}}\right)^2} = 1.541$$

Pressure Coefficient Equation2

$$P_a := K_a \cdot \left(\frac{w_s \cdot H^2}{2} + q \cdot H\right) = 96.578\ \frac{kip}{ft}$$

Total Active Force Equation 1A

Overturning Moment

$$d_a := \frac{H \cdot \left(H + \frac{3 \cdot q}{w_s}\right)}{3 \cdot \left(H + \frac{2 \cdot q}{w_s}\right)} + B \cdot \sin(\beta) = 34.151\ ft$$

Distance from Base of Wall to Center of Applied Load

$$P_h := P_a \cdot H = 3091\ kip$$

Horizontal Force for Soil Pressure for a 1ft wide Section of Wall

$$M_o := d_a \cdot P_h = 105543\ kip \cdot ft$$

Overturning Moment

River Side - Wall Weight Resistance

$$d_g := B + \frac{12\ ft}{2} = 33.5\ ft$$

Distance of Centroid to Toe

$$V_{cube} := 5\ ft \cdot 5\ ft \cdot 1\ ft = 25\ ft^3$$

Volume of 1ft wide Gabion Unit

$$V_{tot} := V_{cube} \cdot 11 = 275\ ft^3$$

Volume of 1ft wide Full Gabion Wall

$$UW_g := \frac{1.7 \cdot ton \cdot 32.2\ \frac{ft}{s^2}}{yd^3} = 3.403\ \frac{kip}{yd^3}$$

Unit Weight of Gravel

$$W_g := V_{tot} \cdot UW_g = 34658\ lbf$$

Self-Weight of a 1ft wide Section of Wall

$$M_{r.river} := d_g \cdot W_g = 1161\ kip \cdot ft$$

Resisting Moment

Non River Side - Wall Weight Resistance

$$d_g := \frac{30 \cdot ft}{2} + 12\ ft + 55\ ft = 82\ ft$$

Distance of Centroid to Toe

$$V_{tot} := V_{cube} \cdot 6 = 150\ ft^3$$

Volume of 1ft wide Full Gabion Wall

$$UW_g = 3.403\ \frac{kip}{yd^3}$$

Unit Weight of Gravel

$$W_g := V_{tot} \cdot UW_g = 18904\ lbf$$

Self-Weight of a 1ft wide Section of Wall

$$M_{r.non} := d_g \cdot W_g = 1550\ kip \cdot ft$$

Resisting Moment

Non River Side - Countering "Overturning" Moment

$H := 23\ ft$ Height of Wall Opposite of the River

$B := \frac{25\ ft}{2} + 12\ ft + \frac{55\ ft}{2} = 52\ ft$ Distance from Centroid Wall to Its Toe

$$d_a := \frac{H \cdot \left(H + \frac{3 \cdot q}{w_s}\right)}{3 \cdot \left(H + \frac{2 \cdot q}{w_s}\right)} + B \cdot \sin(\beta) = 51.926\ ft$$

Distance from Base of Wall to Center of Applied Load

$P_h := P_a \cdot H = 2221\ kip$ Horizontal Force for Soil Pressure for a 1ft wide Section of Wall

$M_{r.non.ot} := d_a \cdot P_h = 115343\ kip \cdot ft$ Overturning Moment

$M_r := M_{r.river} + M_{r.non} + M_{r.non.ot} = 118054\ kip \cdot ft$ $M_o = 105543\ kip \cdot ft$

Check Utilization of Wall

$check\left(M_r > M_o\right) = \text{“True”}$ Therefore, Design is Adequate

$Util := \frac{M_o}{M_r} = 0.894$ Utilization of Capacity

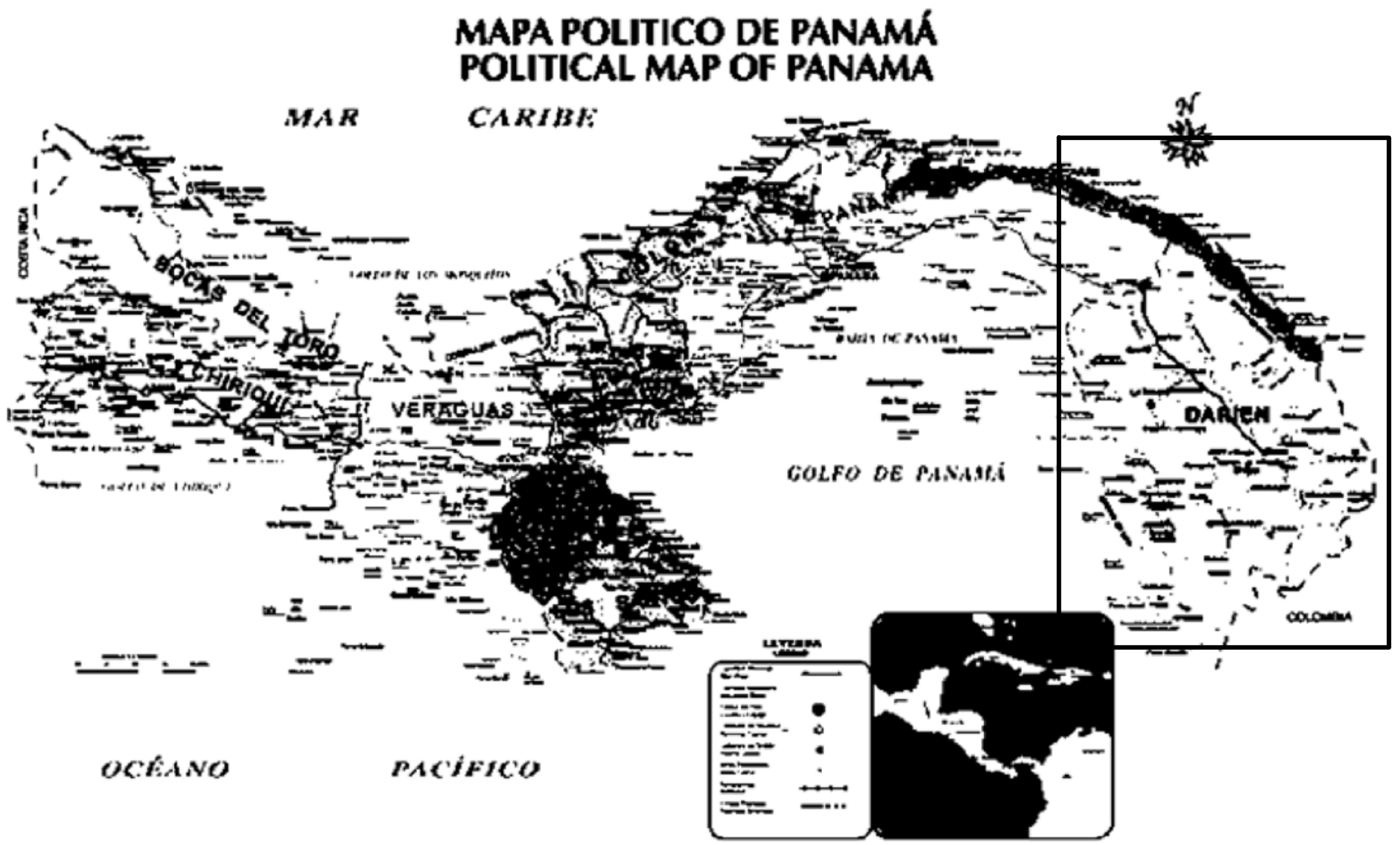
*All Dimensions From AutoCAD Drawing of Gabion Layout - Northwest Bank

"Mechanically Stabilized Eart (MSE) Gabion Wall [Reinforced Soil Wall." Gabions. Modular Gabion Systems, Nov. 2004. Web. 23 Oct. 2013. <http://www.gabions.net/downloads/Documents/MGS_Design_Guide.pdf>.

Appendix L:
Design Drawings

A BRIDGE TO THE COMARCA

EMBERA—WOUNAN COMARCA
PUERTO LIMON, DARIEN, PANAMA



INDEX OF SHEETS

DEL PUENTE ENGINEERING

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EMBERA—WOUNAN COMARCA
ALTO PLAYON

CHUCUNAQUE SUSPENSION BRIDGE
PUERTO LIMON

TITLE SHEET

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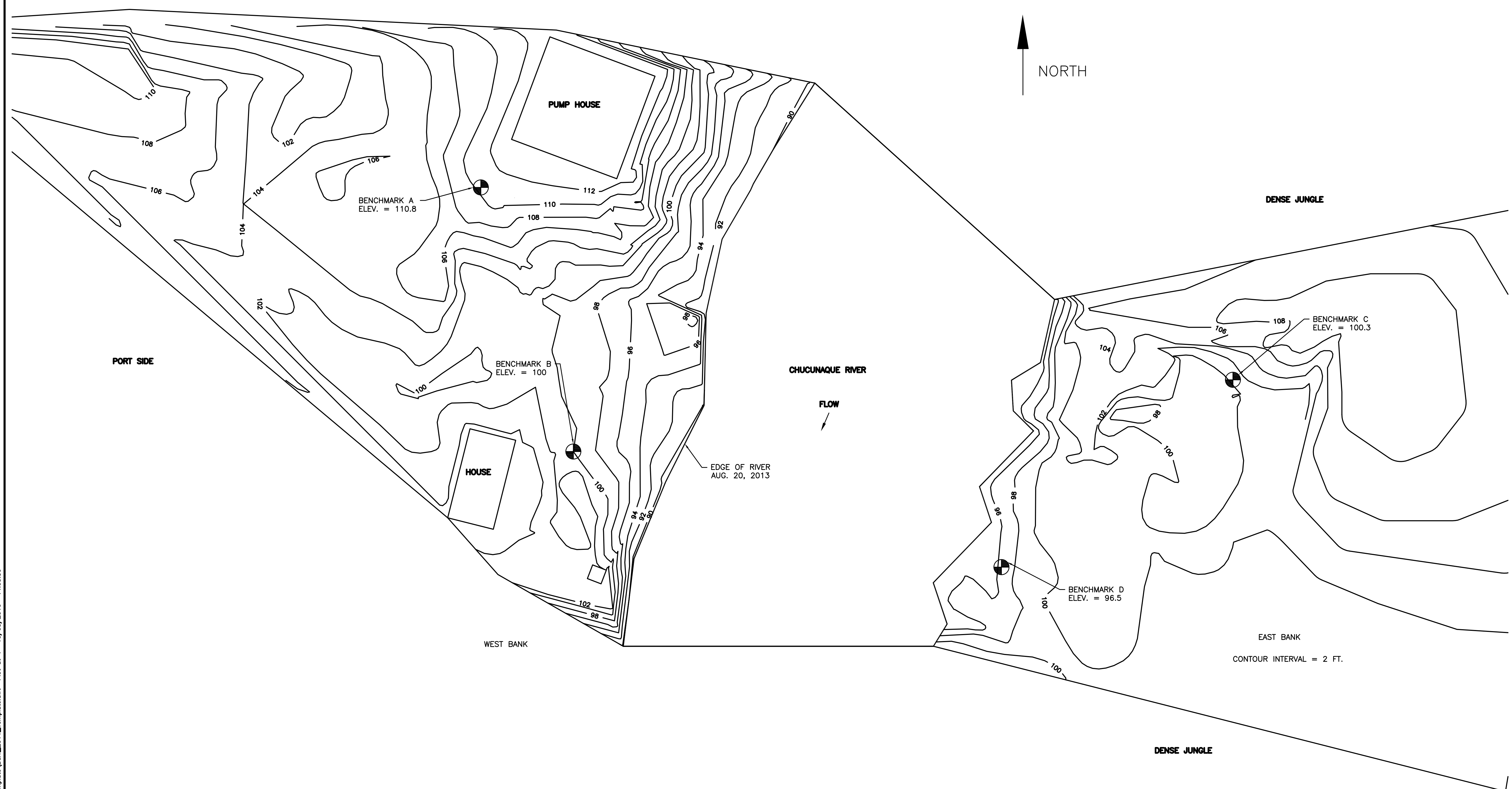
CLIENT PROJECT No.

DWG. No.	REV. No.
C—01	A



① BRIDGE SITE SURVEY

SCALE: 1/2" = 1'-0"

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EMBERA–WOUNAN COMARCA
ALTO PLAYON

CHUCUNAQUE SUSPENSION BRIDGE
PUERTO LIMON

SUSPENSION BRIDGE
SITE SURVEY

BARR PROJECT No.

CLIENT PROJECT No.

DWG. No.	REV. No.
C–02	A

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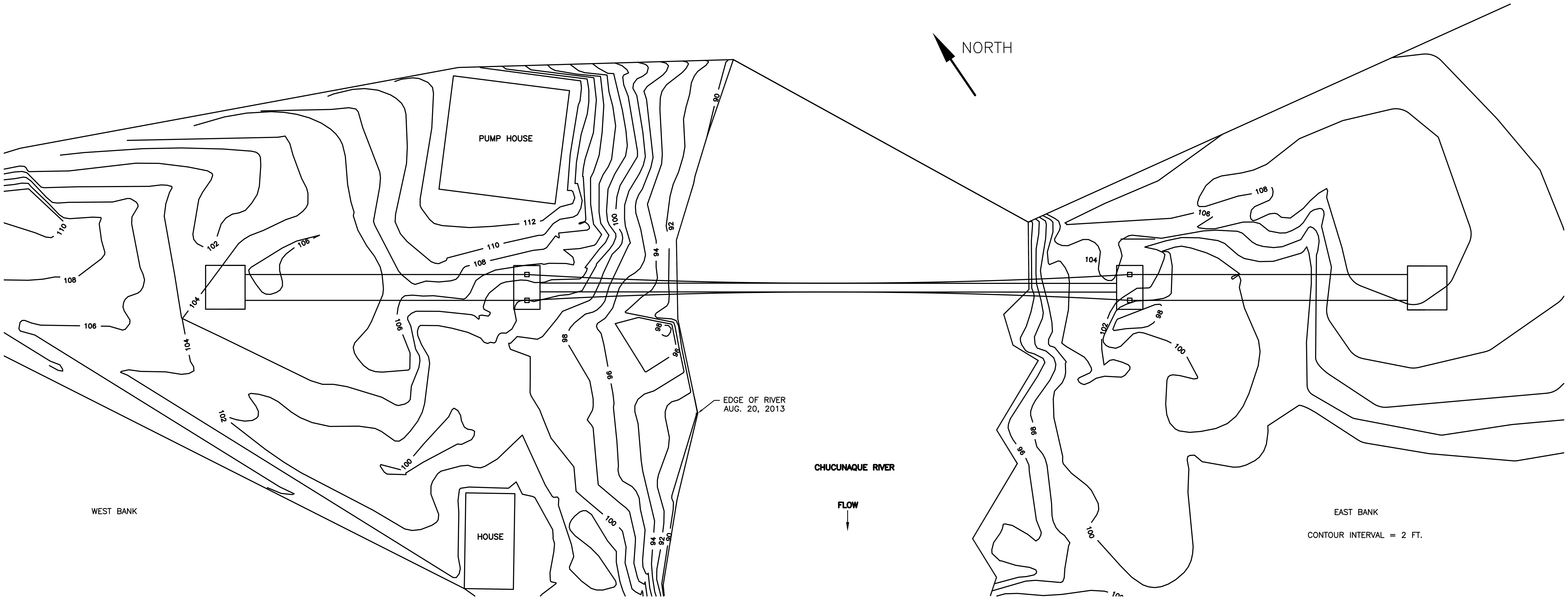
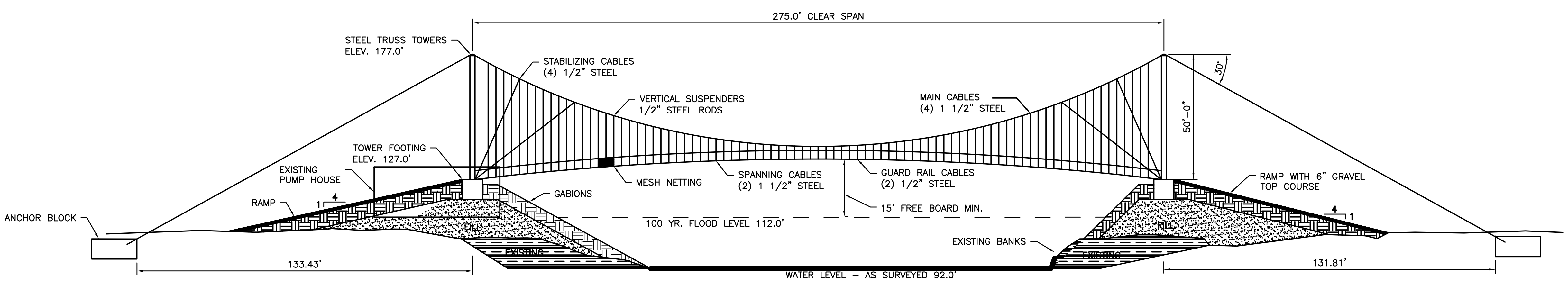


1 **PLAN: BRIDGE LAYOUT**
SCALE: 1/32" = 1'-0"



2 **ELEVATION: SUSPENSION BRIDGE**
SCALE: 1/32" = 1'-0"

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EMBERA–WOUNAN COMARCA
ALTO PLAYON

CHUCUNAQUE SUSPENSION BRIDGE
PUERTO LIMON

SUSPENSION BRIDGE
PLAN AND PROFILE

BARR PROJECT No.
CLIENT PROJECT No.
DWG. No. C–03
REV. No. A

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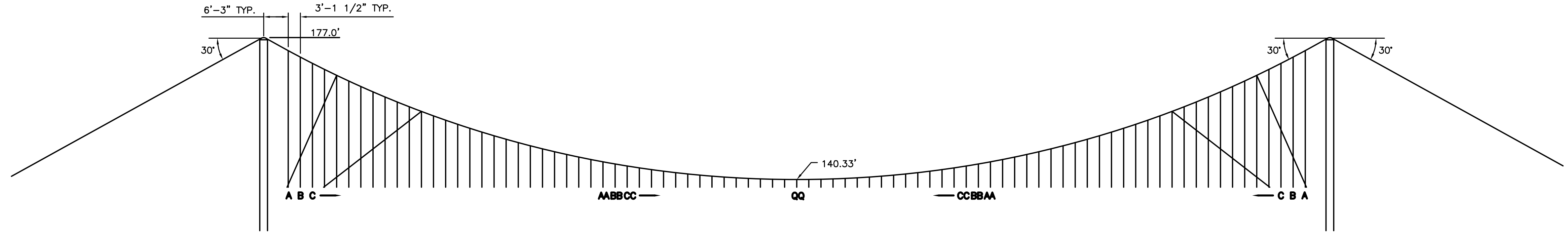
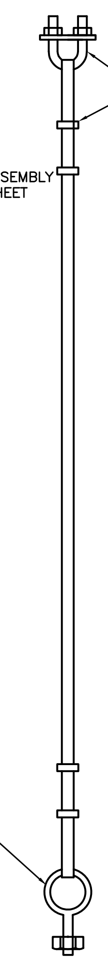
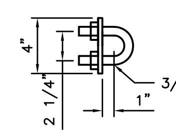


1 ELEVATION: CABLE LAYOUT
SCALE: 1/32" = 1'-0"

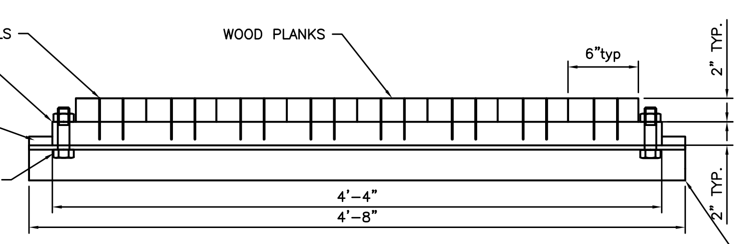
NOTE:
SEE HANGER CABLE ASSEMBLY LENGTH CHART THIS SHEET FOR LENGTHS



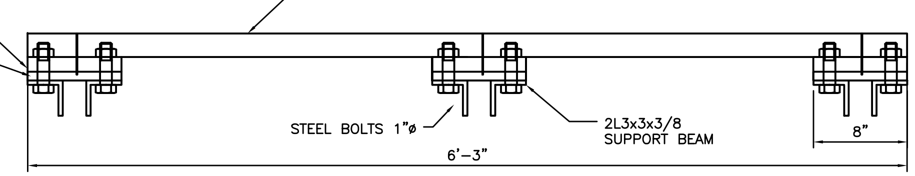
2 HANGER CABLE ASSEMBLY
SCALE: 3" = 1'-0"



3 HANGER/CABLE CLAMP DETAIL
SCALE: 3" = 1'-0"



4 BRIDGE DECKING – SECTION VIEW
SCALE: 1-1/2" = 1'-0"



5 BRIDGE DECKING – PROFILE
SCALE: 1-1/2" = 1'-0"

CABLE ASSEMBLY PROPERTIES		
LOCATION & DIAM.	LENGTH	NO. REQUIRED
MAIN – 1 1/2"	572.2300	4
SPAN – 1 1/2"	273.6500	2
RAIL – 1/2"	273.6500	2
STABILIZING – 1/2"	43.7800	4
STABILIZING – 1/2"	50.1400	4
SUSPENDERS – 1/2"	REFER TO TABLE	REFER TO TABLE

DESIGN DATA

DESIGN LOADS:
LIVE LOAD: 90 PSF (48" WIDE DECK)
DEAD LOAD: 22 PSF (48" WIDE DECK)
WIND LOAD: 22 PSF (APPLIED TO CABLES & ENTIRE RAIL)

MATERIALS/STRESS: (ult = ULTIMATE, d = DESIGN)
SOIL: Qult = 1.5 KSF Qd = .6 KSF
FILL: Qult = 3 KSF Qd = 1.2 KSF
CONCRETE: f'c = 3 KSI (28 DAY)
REINF. STEEL: fy = 60 KSI

MAIN/SPANNING CABLES (1 1/2") – 6X19 IPS:
Fult = 184KIP Fd = 36.8KIP

RAIL/STABILIZING/SUSPENDERS (1/2") – 6X19 IPS:
Fult = 21.4KIP Fd = 4.28KIP

STEEL PLATE – ASTM A36: Fult = 58KSI Fd = 36KSI

STEEL ANGLES – ASTM A36: Fult = 58KSI Fd = 36KSI

STEEL BOLTS – ASTM A307: Fult = 58KSI Fd = 36KSI

STEEL ANCHOR RODS – B7: Fult = 125KSI Fd = 105KSI

MAIN CABLE PROPERTIES			
LOAD CASE	LOAD (PLF)	TENSION (TON)	SAG (FT)
HOISTING	9.5000	1.3900	-0.7350
DEAD LOAD	90.0000	18.4900	0
LIVE LOAD	360.0000	NA	NA
FULL LOAD	450.0000	95.9100	1.8400

NOTE:

CABLE LENGTHS SHOWN ARE THE THEORETICAL LENGTHS UNDER NO LOAD CONDITIONS. CABLE LENGTHS SHOULD BE FIELD VERIFIED WITH ANCHOR LOCATIONS AND ELEVATIONS BEFORE ASSEMBLING. LENGTHS OF CABLE CONNECTIONS MUST BE SUBTRACTED FROM LENGTHS GIVEN.

HANGER CABLE ASSEMBLY LENGTH		
LOCATION	LENGTH	NO. REQUIRED
A	45.7700	4
B	43.7500	4
C	41.7900	4
D	39.8900	4
E	38.0400	4
F	36.2600	4
G	34.5200	4
H	32.8500	4
I	31.2200	4
J	29.6500	4
K	28.1400	4
L	26.6700	4
M	25.2600	4
N	23.9000	4
O	22.5900	4
P	21.3300	4
Q	20.1200	4
R	18.9600	4
S	17.8500	4
T	16.7900	4
U	15.7700	4
V	14.8000	4
W	13.8800	4
X	13.0000	4
Y	12.1800	4
Z	11.4000	4
AA	10.6600	4
BB	9.9700	4
CC	9.3300	4
DD	8.7300	4
EE	8.1700	4
FF	7.6700	4
GG	7.2000	4
HH	6.7800	4
II	6.4100	4
JJ	6.0800	4
KK	5.7900	4
LL	5.5500	4
MM	5.3500	4
NN	5.2000	4
OO	5.0900	4
PP	5.0200	4
QQ	5	2

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EMBERA–WOUNAN COMARCA
ALTO PLAYON

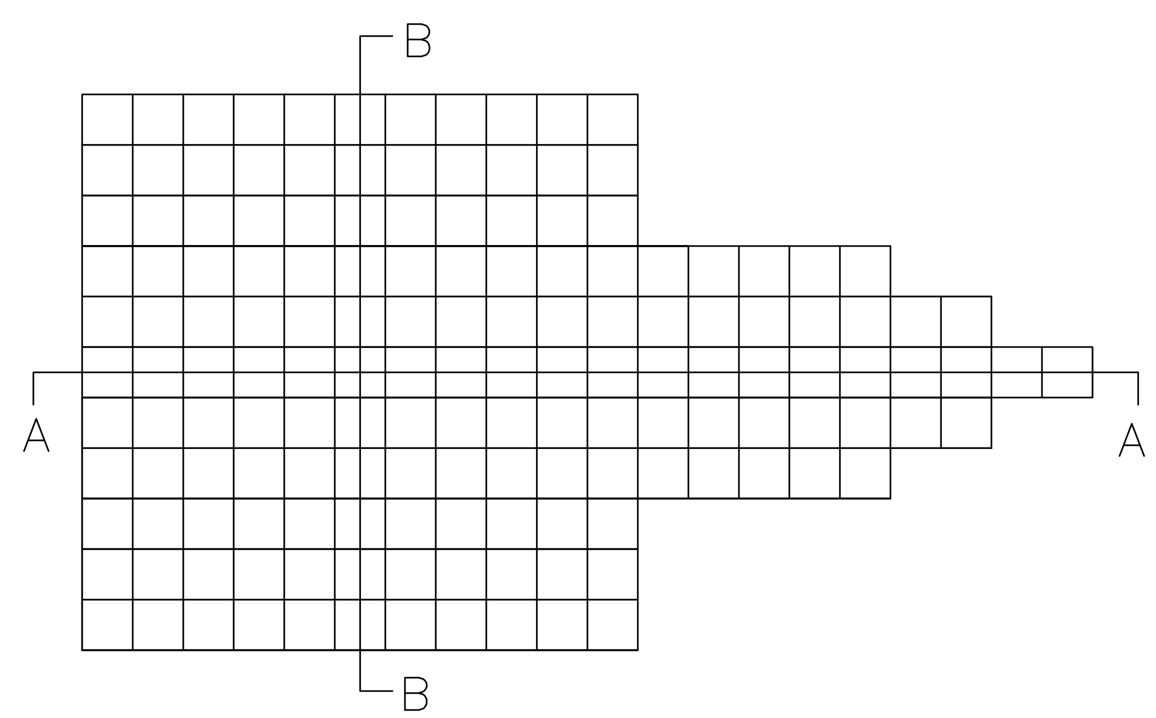
CHUCUNAQUE SUSPENSION BRIDGE
PUERTO LIMON

SUSPENSION BRIDGE
CABLE DETAILS

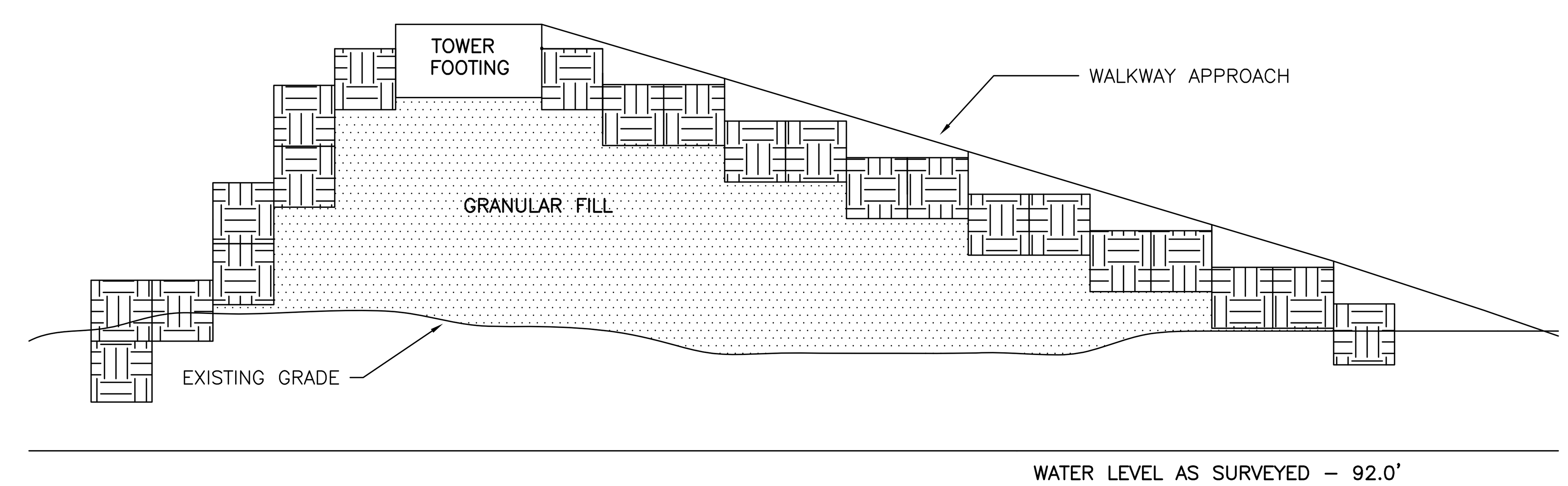
BARR PROJECT No.
CLIENT PROJECT No.
DWG. No. C-04
REV. No. A

NOTES:

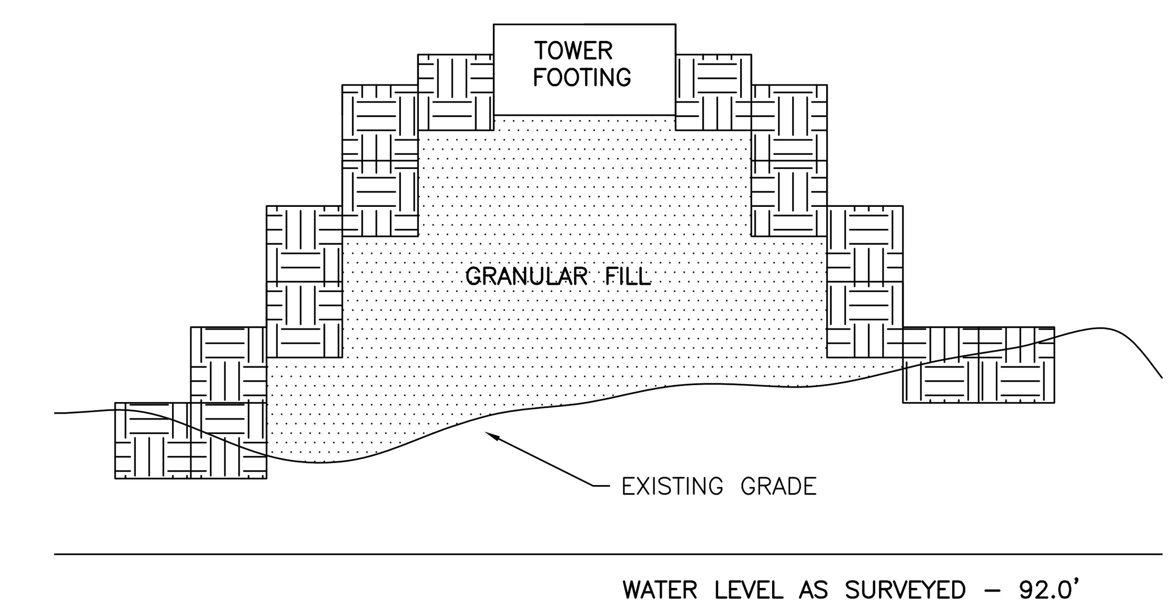
1. BACKFILL MATERIAL BEHIND GABIONS AND UNDER TOWER FOUNDATION WILL BE GRANULAR MATERIAL.
2. GABIONS WILL BE 5'X5'X5' AND CONSTRUCTED OF CHAIN LINK FENCING SECTIONS.
3. GABIONS SHALL BE FILLED WITH 4"–12" ROCK MATERIAL.
4. OVER–LAPING GABIONS WILL HAVE A 2' OVERLAP UNLESS SHOWN OTHERWISE.
5. APPROACH MATERIAL SHALL CONSIST OF COMPACTED GRAVEL BASE.



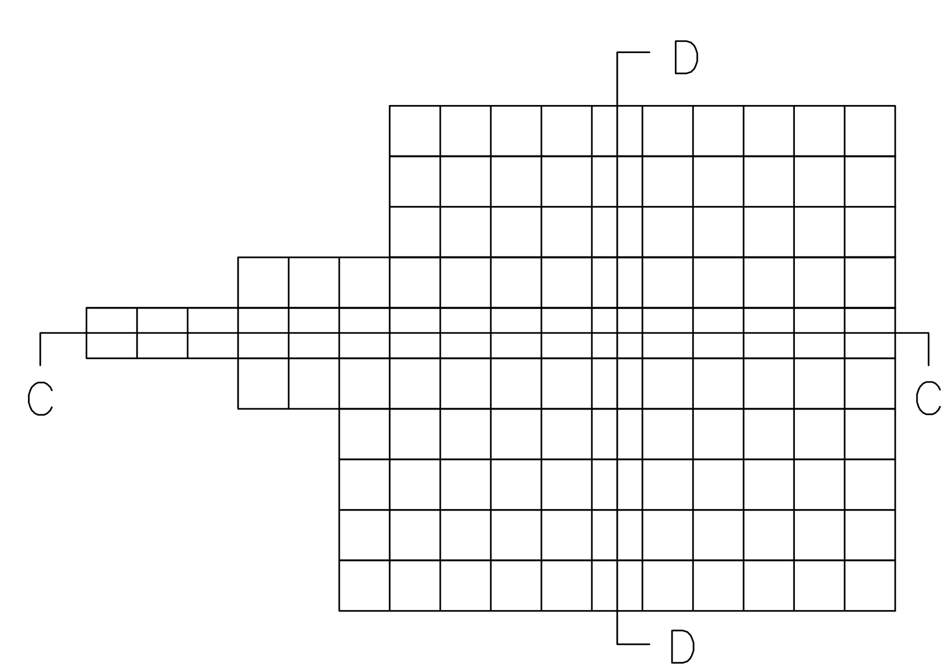
1 SECTION: PLAN VIEW EAST BANK
SCALE: 1/16" = 1'–0"



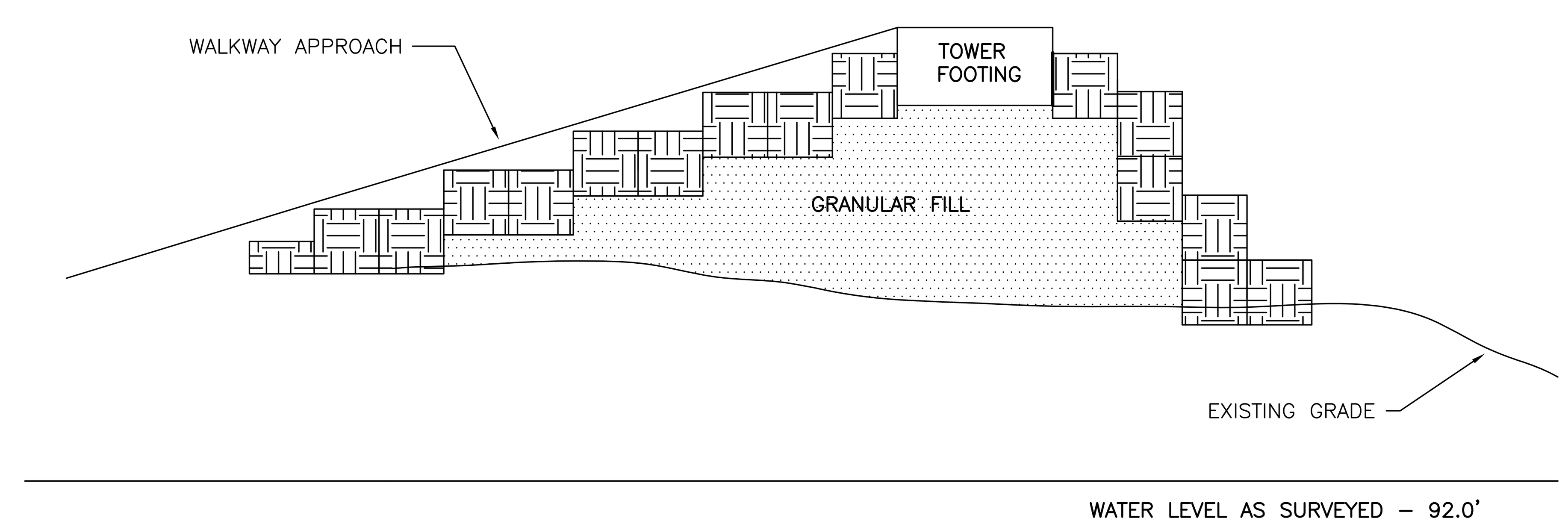
2 SECTION: PROFILE VIEW A–A
SCALE: 3/32" = 1'–0"



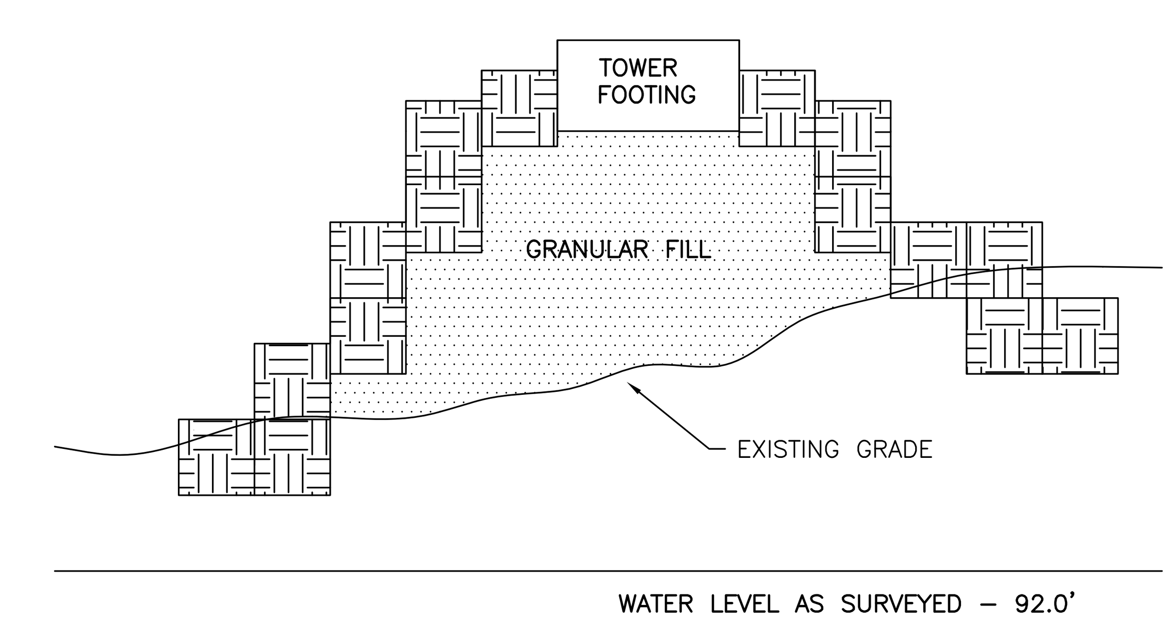
3 SECTION: PROFILE VIEW B–B
SCALE: 3/32" = 1'–0"



4 SECTION: PLAN VIEW WEST BANK
SCALE: 1/16" = 1'–0"



5 SECTION: PROFILE SECTION C–C
SCALE: 3/32" = 1'–0"



6 SECTION: PROFILE SECTION D–D
SCALE: 3/32" = 1'–0"

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CONSTRUCTION							
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EMBERA–WOUNAN COMARCA
ALTO PLAYON

CHUCUNAQUE SUSPENSION BRIDGE
PUERTO LIMON

APPROACH DETAIL AND SLOPE PROTECTION

BARR PROJECT No.

CLIENT PROJECT No.

DWG. No.	REV. No.
C–05	A

NOTES:

A. GENERAL

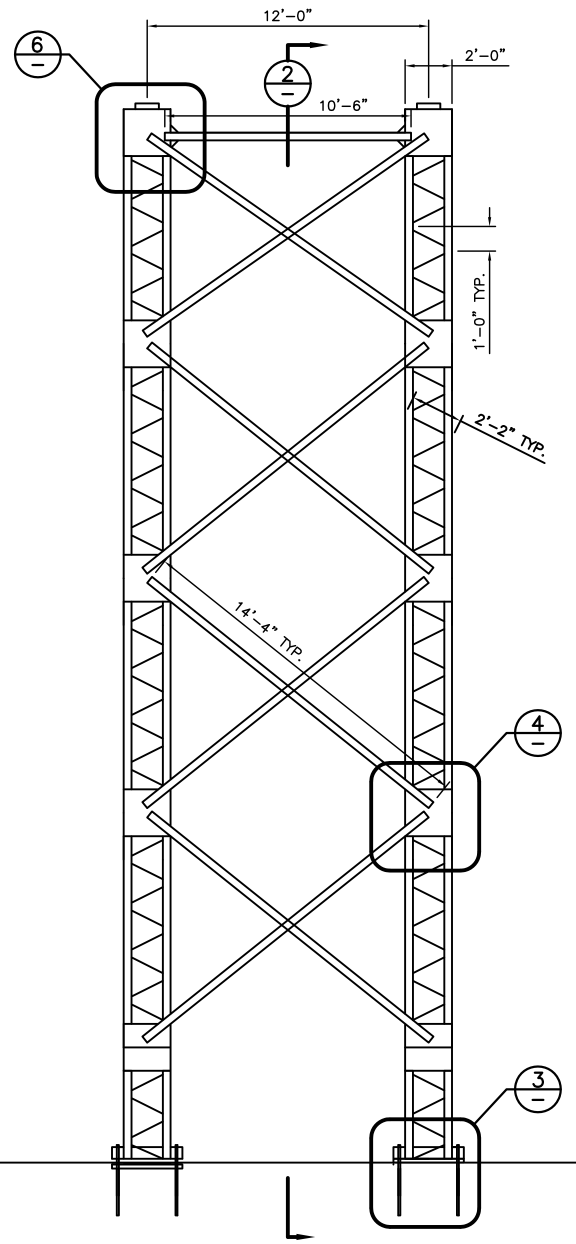
1. STRUCTURAL STEEL: AMERICAN INSTITUTE OF STEEL CONSTRUCTION (AISC) 13TH EDITION, STEEL CONSTRUCTION MANUAL.
2. STEEL FABRICATION: AMERICAN INSTITUTE OF STEEL CONSTRUCTION (AISC) 13TH EDITION, STANDARD PRACTICE FOR STEEL BUILDING AND BRIDGES APPLICABLE SECTION OF CHAPTER M FABRICATION, ERECTION AND QUALITY CONTROL.
3. WELDS: AMERICAN WELDING SOCIETY (AWS), STRUCTURAL WELDING CODE. AWS D1.1

B. PRODUCTS

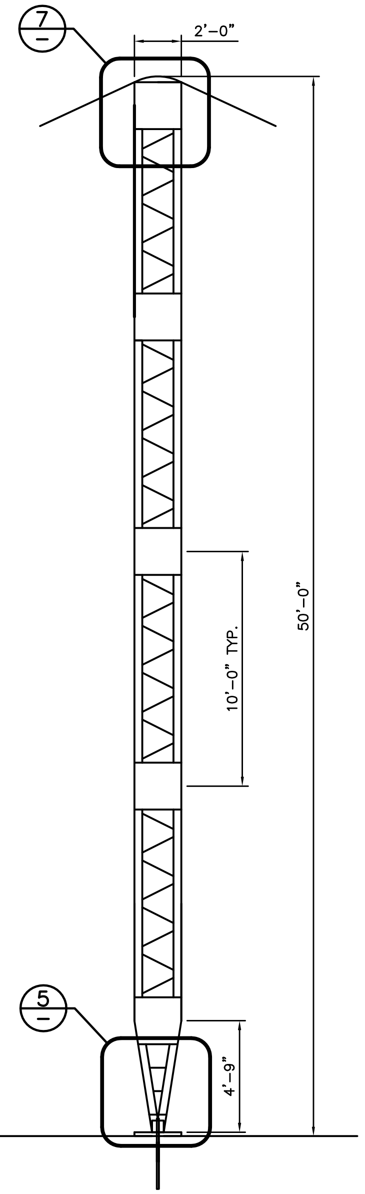
1. SQUARE HSS, RODS AND ANGLES: MUST MEET AISC STRUCTURAL STEEL SPECIFICATIONS.
2. BOLTS: 5/8"ø A307 BOLTS
3. ANCHORS: HILTI HAS BT 1-1/4" INSTALLED AHESIVE ANCHORS SECURED WITH HILTI HIT-HY 200 EPOXY ADHESIVE.
4. WELDS: E70XX ELECTRODES AND SHALL BE PERFORMED BY AN AWS CERTIFIED WELDER.
5. PAINT: ANY PART OF THE TOWER THAT WILL STAY IN PLACE SHALL BE PAINTED A COLOR OF CHOICE.

C. EXECUTION

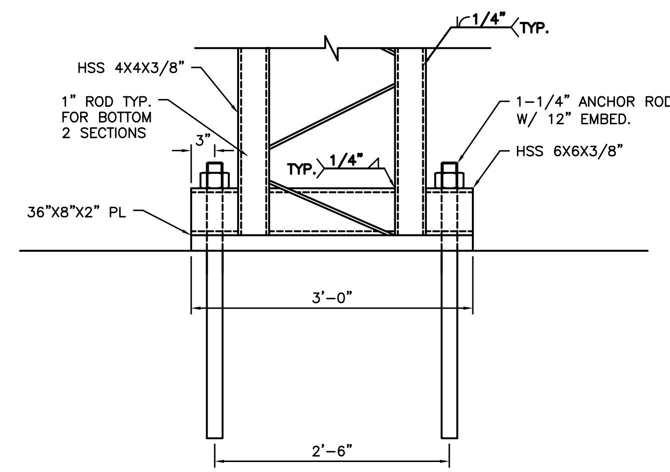
1. ALL STEEL COMPONENTS SHALL BE SHOP FABRICATED. ALL COMPONENTS SHALL BE PLUMB, HORIZONTAL AND SQUARE. TAKE MEASURES TO PREVENT WARPING.
2. CARE SHALL BE UTILIZED THROUGHOUT DELIVERY, STORAGE, AND HANDLING SO AS TO NOT SCRATCH, BEND, DENT OR OTHERWISE DAMAGE THE COMPONENTS.
3. ALL WELDS SHALL BE PERFORMED BY AN AWS CERTIFIED WELDER.
4. POST INSTALLED ADHESIVE ANCHORS SHALL BE EMDEDDED AND INSTALLED PER MANUFACTURERS SPECIFICATIONS.



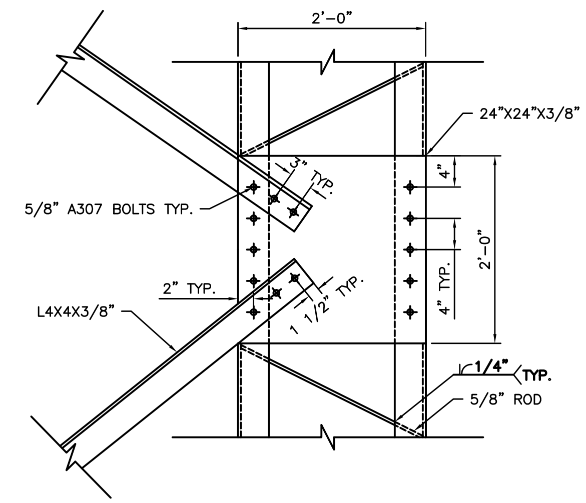
1 ELEVATION: TOWER
SCALE: 1/4" = 1'-0"



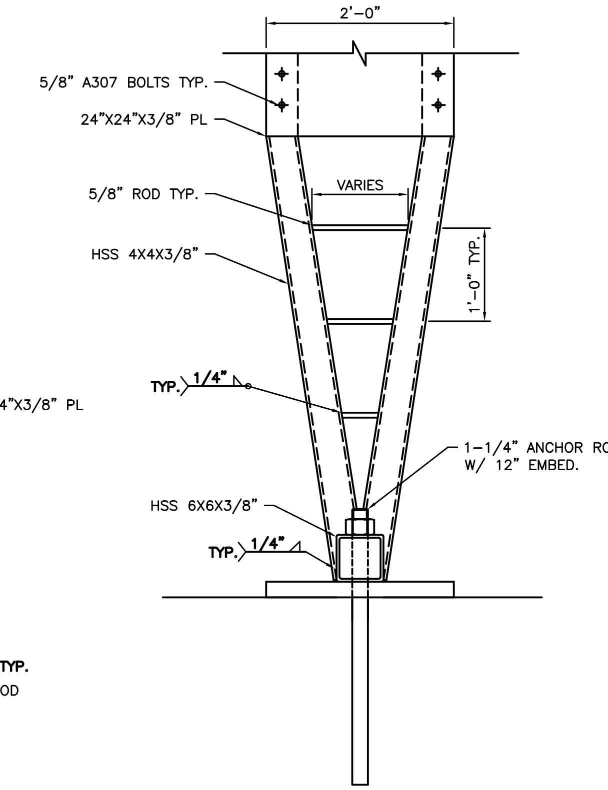
2 SECTION: TOWER
SCALE: 1/4" = 1'-0"



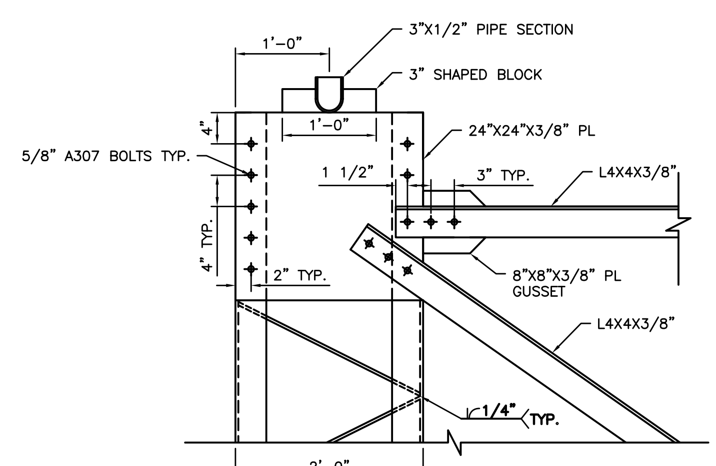
3 DETAIL: TOWER ANCHORAGE
SCALE: 1" = 1'-0"



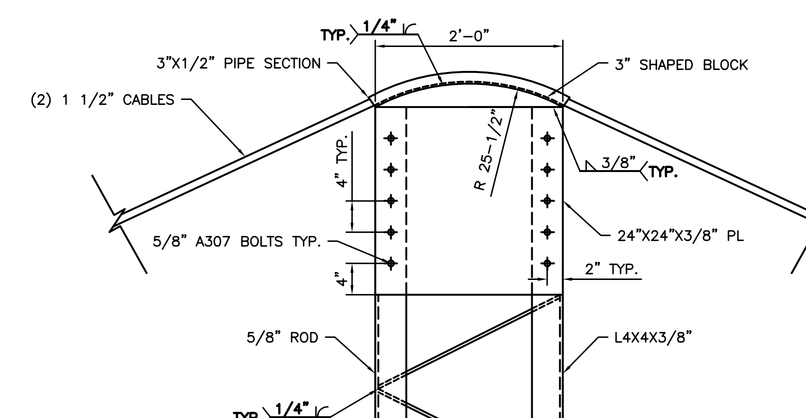
4 DETAIL: BRACING CONNECTION
SCALE: 1" = 1'-0"



5 DETAIL: TOWER ANCHORAGE
SCALE: 1" = 1'-0"



6 DETAIL: TOP CONNECTION/SADDLE
SCALE: 1" = 1'-0"



7 DETAIL: TOP CONNECTION/SADDLE
SCALE: 1" = 1'-0"

CADD USER: Cmbeyer FILE: \\MTUCIFS1.ISO.MTU.EDU\HOME\DESIGN\FINAL DESIGN\TOWER DETAILS.DWG PLOT SCALE: 1:1 PLOT DATE: 11/4/2013 10:22 AM

BAR M:\AutoCAD 2011\AutoCAD 2011 Support\enu\Template\Barr_2011_Template.dwt Plot at 1 10/05/2010 14:03:50

NO.	BY	CHK.	APP.	DATE	REVISION DESCRIPTION

I HEREBY CERTIFY THAT THIS PLAN, SPECIFICATION, OR REPORT WAS PREPARED BY ME OR UNDER MY DIRECT SUPERVISION AND THAT I AM A DULY LICENSED PROFESSIONAL ENGINEER UNDER THE LAWS OF THE STATE OF MINNESOTA.

SIGNATURE ____

PRINTED NAME ____

DATE ____ REG. NO. ____

RELEASED TO/FOR	A	B	C	0	1	2	3
CLIENT							
BID							
CONSTRUCTION							
	DATE RELEASED						

DEL PUENTE ENGINEERING
1400 TOWNSEND DRIVE
HOUGHTON, MI 49931

Scale	AS SHOWN
Date	9/29/2013
Drawn	WES K.
Checked	
Designed	WES K.
Approved	

EMBERA-WOUNAN COMARCA
ALTO PLAYON

CHUCUNAQUE SUSPENSION BRIDGE
PUERTO LIMON

TOWER DETAILS

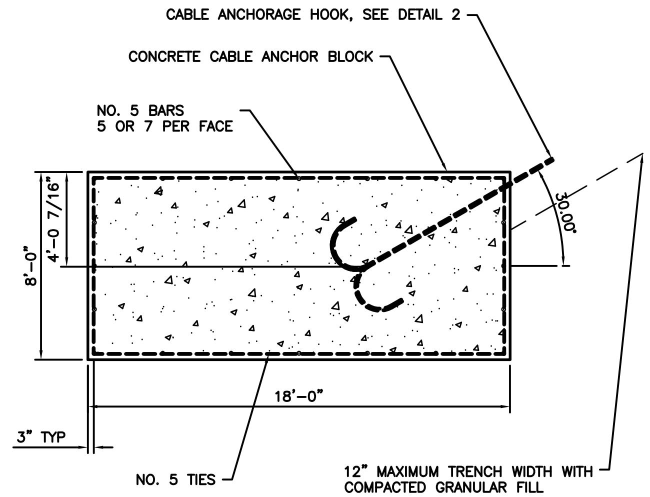
BARR PROJECT No.

CLIENT PROJECT No.

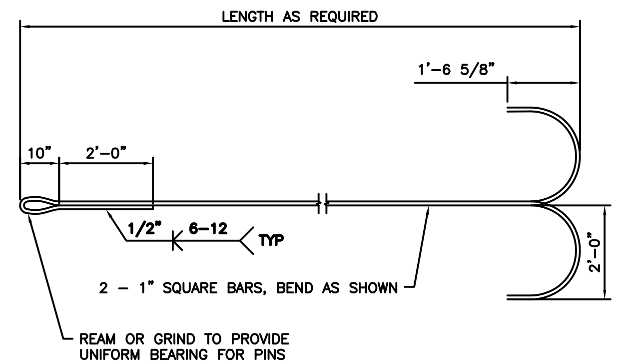
DWG. No. C-06

REV. No. A

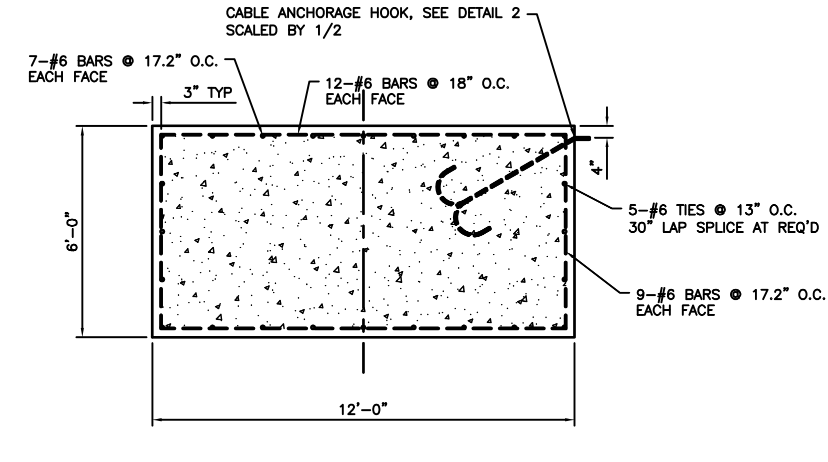
NOTE:
CLAY SOIL BENEATH ANCHOR BLOCKS MUST BE REPLACED WITH A DEPTH OF 3 FT OF GRANULAR MATERIAL.



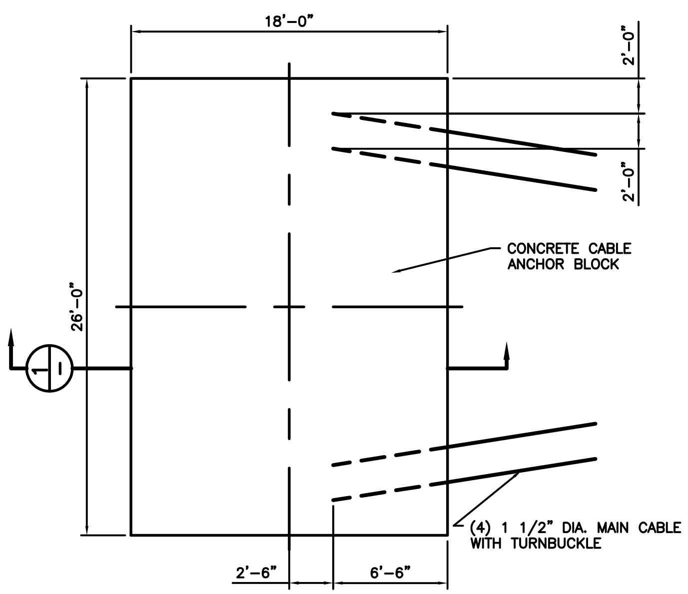
1 SECTION: CABLE ANCHORAGE
SCALE: 1/4" = 1'-0"



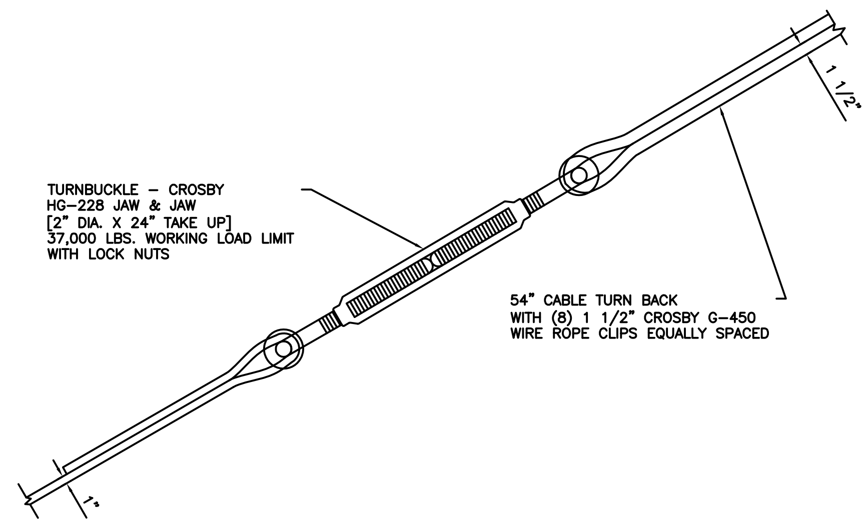
2 DETAIL: CABLE ANCHORAGE HOOK
SCALE: 1/2" = 1'-0"



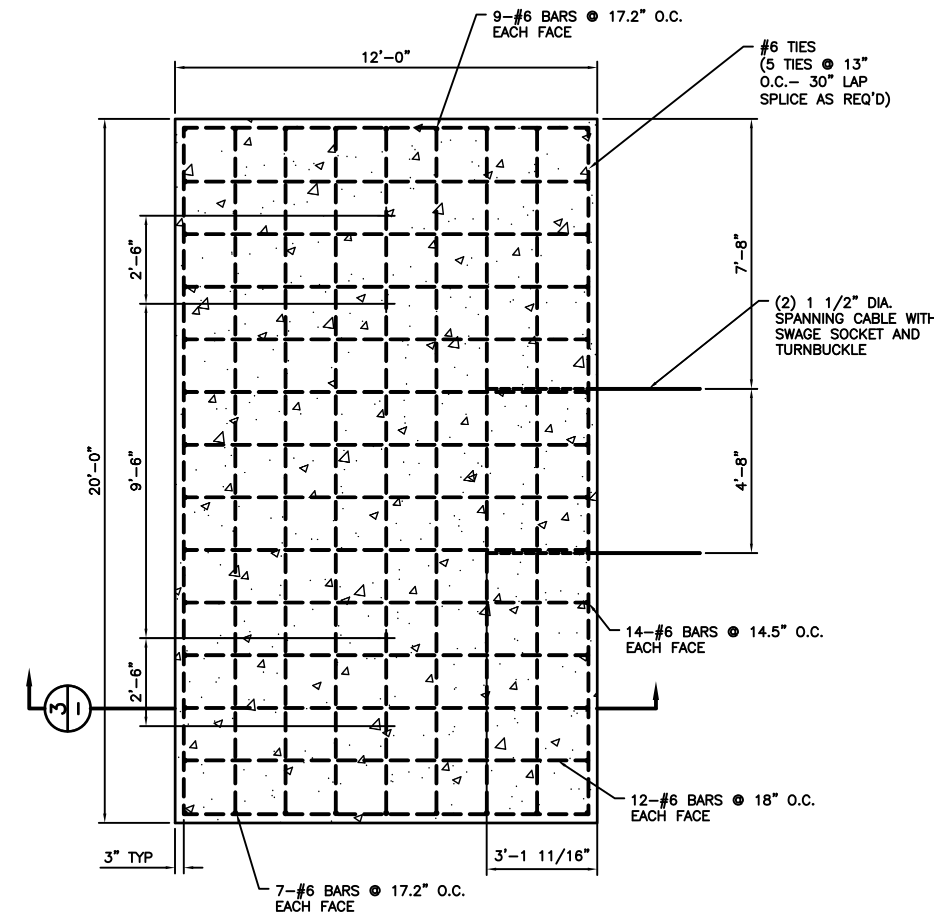
3 SECTION: TOWER FOUNDATION
SCALE: 3/8" = 1'-0"



4 PLAN: ANCHORAGE BLOCK
SCALE: 3/16" = 1'-0"



5 DETAIL: TURNBUCKLE ASSEMBLY
SCALE: 1" = 1'-0"



6 PLAN: TOWER FOUNDATION
SCALE: 3/8" = 1'-0"

CADD USER: Cmbeyer FILE: H:\DESIGN\FINAL DESIGN\FOOTING AND CABLE ANCHORAGE DETAILS FINAL.DWG PLOT SCALE: 1:1 PLOT DATE: 12/2/2013 5:12 PM
BAR M:\AutoCAD 2011\AutoCAD 2011 Support\emu\Template\Barr_2011_Template.dwt Plot at 1 10/05/2010 14:03:50

NO.	BY	CHK.	APP.	DATE	REVISION DESCRIPTION

I HEREBY CERTIFY THAT THIS PLAN, SPECIFICATION, OR REPORT WAS PREPARED BY ME OR UNDER MY DIRECT SUPERVISION AND THAT I AM A DULY LICENSED PROFESSIONAL ENGINEER UNDER THE LAWS OF THE STATE OF MINNESOTA.
SIGNATURE
PRINTED NAME
DATE REG. NO.

CLIENT							
BID							
CONSTRUCTION							
RELEASED TO/FOR	A	B	C	0	1	2	3
	DATE RELEASED						

DEL PUENTE ENGINEERING
1400 TOWNSEND DRIVE
HOUGHTON, MI 49931

Scale	AS SHOWN
Date	10/31/13
Drawn	CMB/TJS
Checked	
Designed	CMB/TJS
Approved	

EMBERA-WOUNAN COMARCA
ALTO PLAYON

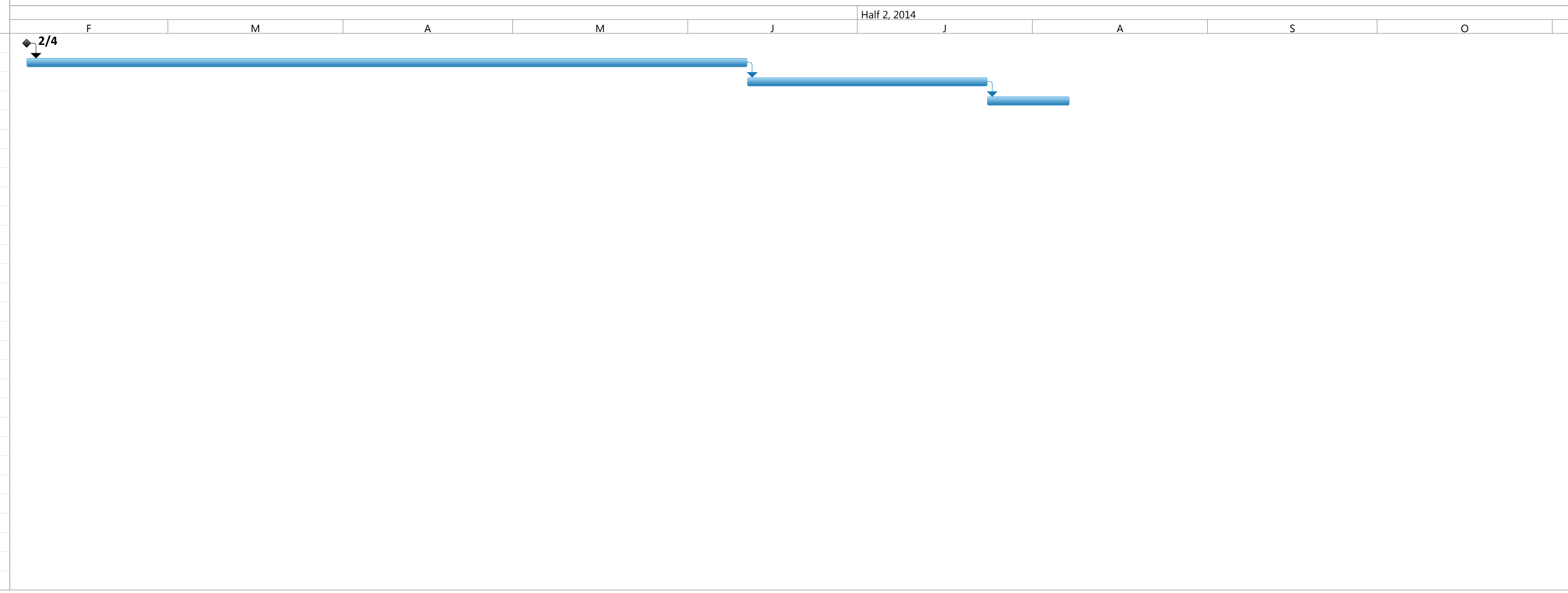
CHUCUNAQUE SUSPENSION BRIDGE
PUERTO LIMON
FOOTING AND CABLE ANCHORAGE DETAILS

BARR PROJECT No.
CLIENT PROJECT No.
DWG. No. C-07
REV. No. A

Appendix M:
Construction Schedule

Preliminary Phase of Construction - Completed Prior to Dry Season #1

ID	Task Mode	Task Name	Duration	Predecessors
1		Start Project	0 days	
2		Permitting	132 days	1
3		Order Materials	44 days	2
4		Cutting Timber	15 days	3
5		Beginning of Dry Season 2014	0 days	
6		Mobilization	5 days	5
7		Port Slope Construction	15 days	6
8		Jungle Slope Construction	15 days	7
9		Port Approach	1 day?	7
10		Port Foundation	3 days	8,9
11		Jungle Approach	1 day?	8,9
12		Jungle Foundation	3 days	10
13		Demobilization #1	5 days	12
14		End Dry Season #1	0 days	
15		Beginning of Dry Season 2015	0 days	
16		Mobilization #2	5 days	15
17		Port Anchor Excavation	3 days	16
18		Jungle anchor Excavation	3 days	17
19		Port Anchor Block	5 days	17
20		Jungle Anchor Block	5 days	18,19
21		Port Tower	19 days	20
22		Jungle Tower	19 days	21
23		Main Cable	6 days	22
24		Vertical Hangers	4 days	23
25		Decking Steel	5 days	24
26		Decking Wood	14 days	25
27		Demobilization #2	5 days	26
28		End Project	0 days	27
29		End Dry Season #2	0 days	



Project: Preliminary_Design_Sche
Date: Wed 12/4/13

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

Phase 2 of Construction - Completed During Dry Season #1

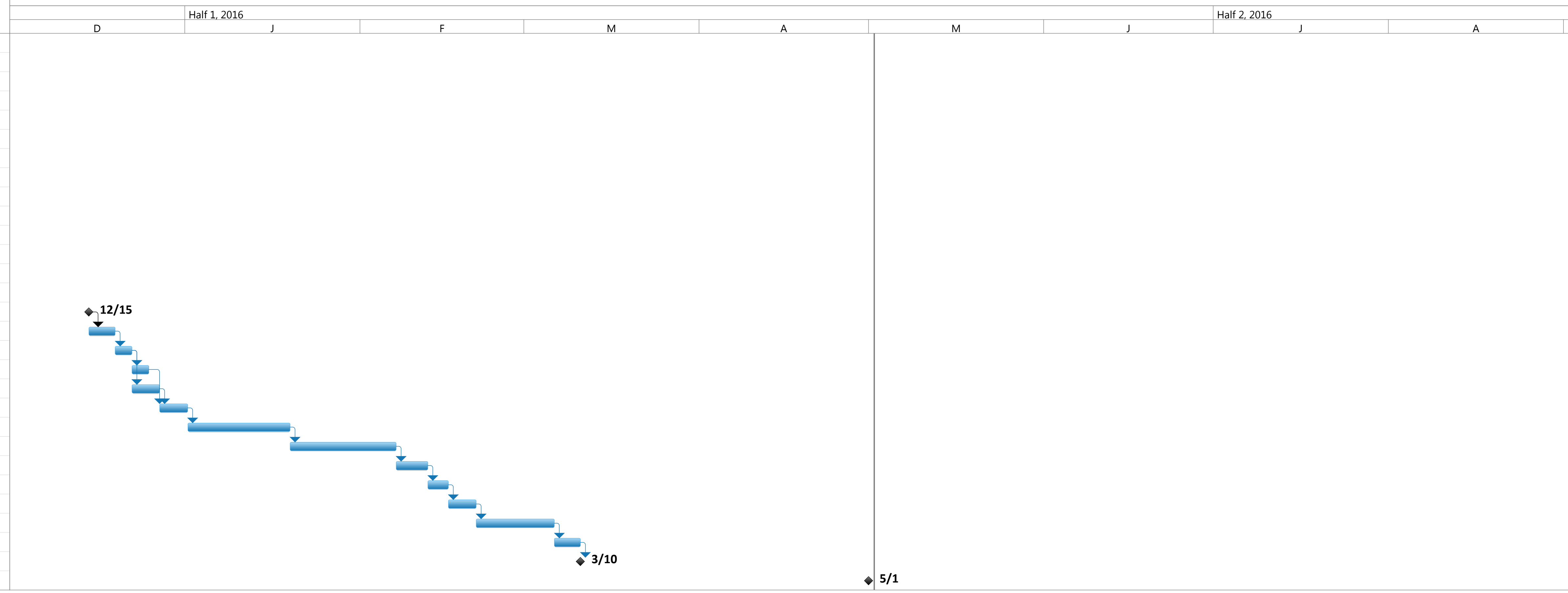
ID	Task Mode	Task Name	Duration	Predecessors
1		Start Project	0 days	
2		Permitting	132 days	1
3		Order Materials	44 days	2
4		Cutting Timber	15 days	3
5		Beginning of Dry Season 2014	0 days	
6		Mobilization	5 days	5
7		Port Slope Construction	15 days	6
8		Jungle Slope Construction	15 days	7
9		Port Approach	1 day?	7
10		Port Foundation	3 days	8,9
11		Jungle Approach	1 day?	8,9
12		Jungle Foundation	3 days	10
13		Demobilization #1	5 days	12
14		End Dry Season #1	0 days	
15		Beginning of Dry Season 2015	0 days	
16		Mobilization #2	5 days	15
17		Port Anchor Excavation	3 days	16
18		Jungle anchor Excavation	3 days	17
19		Port Anchor Block	5 days	17
20		Jungle Anchor Block	5 days	18,19
21		Port Tower	19 days	20
22		Jungle Tower	19 days	21
23		Main Cable	6 days	22
24		Vertical Hangers	4 days	23
25		Decking Steel	5 days	24
26		Decking Wood	14 days	25
27		Demobilization #2	5 days	26
28		End Project	0 days	27
29		End Dry Season #2	0 days	

Project: Preliminary_Design_Sche
Date: Wed 12/4/13

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

Phase 3 of Construction - Completed During Dry Season #2

ID	Task Mode	Task Name	Duration	Predecessors
1		Start Project	0 days	
2		Permitting	132 days	1
3		Order Materials	44 days	2
4		Cutting Timber	15 days	3
5		Beginning of Dry Season 2014	0 days	
6		Mobilization	5 days	5
7		Port Slope Construction	15 days	6
8		Jungle Slope Construction	15 days	7
9		Port Approach	1 day?	7
10		Port Foundation	3 days	8,9
11		Jungle Approach	1 day?	8,9
12		Jungle Foundation	3 days	10
13		Demobilization #1	5 days	12
14		End Dry Season #1	0 days	
15		Beginning of Dry Season 2015	0 days	
16		Mobilization #2	5 days	15
17		Port Anchor Excavation	3 days	16
18		Jungle anchor Excavation	3 days	17
19		Port Anchor Block	5 days	17
20		Jungle Anchor Block	5 days	18,19
21		Port Tower	19 days	20
22		Jungle Tower	19 days	21
23		Main Cable	6 days	22
24		Vertical Hangers	4 days	23
25		Decking Steel	5 days	24
26		Decking Wood	14 days	25
27		Demobilization #2	5 days	26
28		End Project	0 days	27
29		End Dry Season #2	0 days	



Project: Preliminary_Design_Sche
Date: Wed 12/4/13

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

Appendix N:
Cost Estimate

Labor and Equipment Pricing

Equipment List

Equipment	Hourly Rate	Ownership Rate	Total Cost/DAY
Loader			$360.00
Concrete Truck			$82.19

Equipment Rate - Sources

Loader	http://www.encuentra24.com/panama-es/anuncios-casificados-construccion-y-mantenimiento-equipo-pesado-maquinaria/se-alquila-se-vende-cargadores-cat-950g-y-966g/1627221
Conc. Trk	http://www.encuentra24.com/panama-es/anuncios-casificados-construccion-y-mantenimiento-equipo-pesado-maquinaria/vendo-mixer-o-camion-revolvedor-de-concreto-precio-negociable/3035228

Labor Crew List

		Citation of Pricing
Labor Postion	Daily Rate	
Unskilled Labor	$10.00	Community Members of Alto Playón
Supervisor	$19.50	http://tuxtlagutierrez.olx.com.mx/sobrestante-obra-civil-iid-433957291

SUMMARY SHEET

Cost Estimate: Bridge Construction	Project Cost $ 418,000.00

Item	Category	Item Description	Quantity	Units	Unit Rate	Total Cost
000	Gabion	Gabion - Equipment	1	LS	$ 1,183.58	$ 1,184
001	Gabion	5' Chainlink Fence	10980	LF	$ 2.25	$ 24,747
002	Gabion	Rock/Gravel 4" - 12"	2372	TON	$ 49.27	$ 116,872
003	Gabion	Tie-wire 16"	25620	EACH	$ 0.06	$ 1,409
004	Gabion	Granular Backfill	1175	LCY	$ 22.00	$ 25,850
005	Approach	Gravel	390	LCY	$ 0.48	$ 186
006	Towers	L4x4x3/8" x 10'	640	LF	$ 9.42	$ 6,026
007	Towers	L4x4x3/8" x 5'	80	LF	$ 9.42	$ 753
008	Towers	L4x4x3/8"x14.2'	454.4	LF	$ 9.42	$ 4,279
009	Towers	HSS4x4x3/8" x 5.1ft	81.6	LF	$ 28.49	$ 2,325
010	Towers	5/8" Round Bar	1600	LF	$ 1.12	$ 1,795
011	Towers	5/8"x24"x24" Plate	80	EACH	$ 106.92	$ 8,554
012	Towers	5/8" A307 Plate Bolts	640	EACH	$ 0.42	$ 268
013	Towers	5/8" A307 Brac. Bolts	192	EACH	$ 0.42	$ 80
014	Towers	3"x3' Pipe Section	4	EACH	$ 26.99	$ 108
015	Towers	HSS6x6x1/2" x 3'	12	LF	$ 31.46	$ 378
016	Towers	1-1/4" B7 Anchor Rods	8	EACH	$ 38.78	$ 310
017	Towers	2"x2'x3' Plate	4	EACH	$ 636.90	$ 2,548
018	Towers	Steel Erection	1	LS	$ 4,780.00	$ 4,780
019	Cables	1-1/2" Main - 573' ea.	2292	LF	$ 27.90	$ 63,938
020	Cables	1-1/2" Span - 274' ea	548	LF	$ 27.90	$ 15,287
021	Cables	1/2" Rail - 274' ea	548	LF	$ 4.74	$ 2,598
022	Cables	1/2" Stabilizing - vary	190	LF	$ 4.74	$ 901
023	Cables	1/2" Suspenders - vary	884.75	LF	$ 4.74	$ 4,195
024	Cables	1-1/2" Cable Assembly	2840	LF	$ 0.26	$ 732
025	Cables	1/2" Cable Assembly	1623	LF	$ 0.27	$ 437
024	Anchor Block	Turnbuckle - Crosby	8	EACH	$ 8.92	$ 71
025	Anchor Block	Concrete	277.33	CY	$ 164.45	$ 45,606
026	Anchor Block	5/8" A36 Steel Round Bar	0.54236	TON	$ 817.14	$ 443
027	Anchor Block	5/8" A36 Steel Ties	0.650832	TON	$ 803.79	$ 523
028	Anchor Block	Crosby 1-1/2" G-450 Wire Rope Clips	64	EACH	$ 49.13	$ 3,144
029	Anchor Block	1" x 1" Square Bar Anchorage Hook	240	LF	$ 9.53	$ 2,288
030	Anchor Block	Anchor Block Excavation	139	CY	$ 4.56	$ 634
031	Tower Foundation	Concrete (Tower Foundation)	107	CY	$ 164.45	$ 17,596
032	Tower Foundation	Anchorage Hooks	60	LF	$ 5.14	$ 308
033	Tower Foundation	No. 6 Rebar	2.02	TON	$ 770.00	$ 1,555
035	Walkway	Wood - Alemendro	150	SY	$ 10.24	$ 1,535
036	Walkway	Steel Beams - LL3x3x3/8 (galvanized)	194	LF	$ 40.95	$ 7,945
037	Walkway	Cable Clips	850	EACH	$ 13.89	$ 11,809
038	Walkway	Steel Bolts - 1" Dia.	340	EACH	$ 3.39	$ 1,152
039	Walkway	Nut - 1" Dia.	680	EACH	$ 0.58	$ 396
040	Walkway	Nails - 3.5"	1360	EACH	$ 1.35	$ 1,829
041	Walkway	Steel Plate - 4"x2"x1/2"	1360	SF	$ 17.17	$ 23,353
042	Walkway	Eye-Bolt - 3/4" Dia.	170	EACH	$ 13.88	$ 2,360
043	Walkway	Nut - 3/4" Dia.	340	EACH	$ 0.58	$ 198
045	Concrete	Concrete Trucks	43	EACH	$ 90.41	$ 3,888
046	Towers	5/8" NUTS	832	EACH	$ 0.58	$ 485

Unit Price Breakdown

Item Number	Category	Item Description	Quantity	Units	Material Unit Rate	Equipment Unit Rate	Labor Unit Rate	Unit Cost	Unit Cost with Sm. Tools and delivery	Total Cost	Comments
000	Gabion	Gabion - Equipment	1	LS	$ -	$ 1,075.98	$ -	$ 1,075.98	$ 1,183.58	$ 1,183.58	
001	Gabion	5' Chainlink Fence	10980	LF	$ 1.80		$ 0.25	$ 2.05	$ 2.25	$ 24,747.32	
002	Gabion	Rock/Gravel 4" - 12"	2372	TON	$ 43.64		$ 1.15	$ 44.79	$ 49.27	$ 116,871.88	
003	Gabion	Tie-wire 16"	25620	EACH	$ 0.05		$ -	$ 0.05	$ 0.06	$ 1,409.10	Labor in chainlink fence
004	Gabion	Granular Backfill	1175	LCY	$ 20.00		$ -	$ 20.00	$ 22.00	$ 25,850.00	Equipment in "Gabion Equip" Labor in 5' chainlink fence
005	Approach	Gravel	390	LCY		$ 0.38	$ 0.06	$ 0.43	$ 0.48	$ 185.72	
006	Towers	L4x4x3/8" x 10'	640	LF	$ 8.56		$ -	$ 8.56	$ 9.42	$ 6,026.24	
007	Towers	L4x4x3/8" x 5'	80	LF	$ 8.56		$ -	$ 8.56	$ 9.42	$ 753.28	
008	Towers	L4x4x3/8"x14.2'	454.4	LF	$ 8.56		$ -	$ 8.56	$ 9.42	$ 4,278.63	
009	Towers	HSS4x4x3/8" x 5.1ft	81.6	LF	$ 25.90		$ -	$ 25.90	$ 28.49	$ 2,324.78	
010	Towers	5/8" Round Bar	1600	LF	$ 1.02		$ -	$ 1.02	$ 1.12	$ 1,795.20	
011	Towers	5/8"x24"x24" Plate	80	EACH	$ 97.20		$ -	$ 97.20	$ 106.92	$ 8,553.60	
012	Towers	5/8" A307 Plate Bolts	640	EACH	$ 0.38		$ -	$ 0.38	$ 0.42	$ 267.52	
013	Towers	5/8" A307 Brac. Bolts	192	EACH	$ 0.38		$ -	$ 0.38	$ 0.42	$ 80.26	
014	Towers	3"x3' Pipe Section	4	EACH	$ 24.54		$ -	$ 24.54	$ 26.99	$ 107.98	
015	Towers	HSS6x6x1/2" x 3'	12	LF	$ 28.60		$ -	$ 28.60	$ 31.46	$ 377.52	
016	Towers	1-1/4" B7 Anchor Rods	8	EACH	$ 35.25		$ -	$ 35.25	$ 38.78	$ 310.20	
017	Towers	2"x2'x3' Plate	4	EACH	$ 579.00		$ -	$ 579.00	$ 636.90	$ 2,547.60	
018	Towers	Steel Erection	1	LS	$ -		$ 4,345.45	$ 4,345.45	$ 4,780.00	$ 4,780.00	Total Erection
019	Cables	1-1/2" Main - 573' ea.	2292	LF	$ 25.36		$ -	$ 25.36	$ 27.90	$ 63,937.63	
020	Cables	1-1/2" Span - 274' ea	548	LF	$ 25.36		$ -	$ 25.36	$ 27.90	$ 15,287.01	
021	Cables	1/2" Rail - 274' ea	548	LF	$ 4.31		$ -	$ 4.31	$ 4.74	$ 2,598.07	
022	Cables	1/2" Stabilizing - vary	190	LF	$ 4.31		$ -	$ 4.31	$ 4.74	$ 900.79	
023	Cables	1/2" Suspenders - vary	884.75	LF	$ 4.31		$ -	$ 4.31	$ 4.74	$ 4,194.60	
024	Cables	1-1/2" Cable Assembly	2840	LF	$ -		$ 0.23	$ 0.23	$ 0.26	$ 732.00	
025	Cables	1/2" Cable Assembly	1623	LF	$ -		$ 0.24	$ 0.24	$ 0.27	$ 437.18	
026	Anchor Block	Turnbuckle - Crosby HG-228 Jaw & Jaw 2" x 24"	8	EACH	$ -		$ 8.11	$ 8.11	$ 8.92	$ 71.36	4 per block
027	Anchor Block	Concrete	277.33	CY	$ 149.00		$ 0.50	$ 149.50	$ 164.45	$ 45,605.65	138.67 per block
028	Anchor Block	5/8" A36 Steel Round Bar	0.54	TON	$ 702.38		$ 40.48	$ 742.86	$ 817.14	$ 443.19	40 EACH (20 per block)
029	Anchor Block	5/8" A36 Steel Ties	0.65	TON	$ 702.38		$ 28.33	$ 730.71	$ 803.79	$ 523.13	13 per block
030	Anchor Block	Crosby 1-1/2" G-450 Wire Rope Clips	64	EACH	$ 44.05		$ 0.61	$ 44.66	$ 49.13	$ 3,144.39	8 per cable
031	Anchor Block	1" x 1" Square Bar Anchorage Hook	240	LF	$ 8.00		$ 0.67	$ 8.67	$ 9.53	$ 2,288.00	4 per block (30ft per hook)
032	Anchor Block	Anchor Block Excavation	139	CY	$ -		$ 4.15	$ 4.15	$ 4.56	$ 633.90	about 70 cy per block
033	Tower Foundation	Concrete (Tower Foundation)	107	CY	$ 149.00		$ 0.50	$ 149.50	$ 164.45	$ 17,596.15	
034	Tower Foundation	Anchorage Hooks	60	LF	$ 4.67		$ -	$ 4.67	$ 5.14	$ 308.22	Labor included in Concrete
035	Tower Foundation	No. 6 Rebar	2.02	TON	$ 700.00		$ -	$ 700.00	$ 770.00	$ 1,555.40	
036	Walkway	Wood - Alemendro	150	SY	$ -		$ 9.31	$ 9.31	$ 10.24	$ 1,535.33	
037	Walkway	Steel Beams - LL3x3x3/8 (galvanized)	194	LF	$ 34.74		$ 2.49	$ 37.23	$ 40.95	$ 7,944.79	
038	Walkway	Cable Clips	850	EACH	$ 12.28		$ 0.35	$ 12.63	$ 13.89	$ 11,809.05	
039	Walkway	Steel Bolts - 1" Dia.	340	EACH	$ 3.08		$ -	$ 3.08	$ 3.39	$ 1,151.92	
040	Walkway	Nut - 1" Dia.	680	EACH	$ 0.53		$ -	$ 0.53	$ 0.58	$ 396.44	
041	Walkway	Nails - 3.5"	1360	EACH	$ 1.10		$ 0.12	$ 1.22	$ 1.35	$ 1,829.48	
042	Walkway	Steel Plate - 4"x2"x1/2"	1360	SF	$ 15.61		$ -	$ 15.61	$ 17.17	$ 23,352.56	
043	Walkway	Eye-Bolt - 3/4" Dia.	170	EACH	$ 12.62		$ -	$ 12.62	$ 13.88	$ 2,359.94	
044	Walkway	Nut - 3/4" Dia.	340	EACH	$ 0.53		$ -	$ 0.53	$ 0.58	$ 198.22	
045	Concrete	Concrete Trucks	43	EACH		$ 82.19	$ -	$ 82.19	$ 90.41	$ 3,887.67	
046	Towers	5/8" NUTS	832	EACH	$ 0.53		$ -	$ 0.53	$ 0.58	$ 485.06	
								Sum:		**$ 417,657.52**	

Item Number	000		Unit Price	$ -	LF
Item Description	Gabion Equipment				
Quantity	2372	TON	(from gabion gravel)		
Gravel	1694	LCY			
Granular	1175	LCY			

Labor	Unit Rate:	0 / Day		/LCY
Postition	**Wage ($/day)**	**Quantity**	**Total**	
Supervisor	19.5		0	
Unskilled Laborer	10		0	
***Labor Included in Actual Item Numbers				

Equipment	Unit Rate:	*UNIT*		
Loader (unloading gravel)	$ 360.00	/DAY	$ 0.38	/LCY
Capacity	3	LCY		
Production Rate	120	LCY/HR		
	960	LCY/DAY	2	Days

Item Number	001	Unit Price	$ 0.25 LF
Item Description	5' Chainlink Fence		
Quantity	10980 LF		

Labor	Unit Rate:	119.5 / Day	
Postition	**Wage ($/day)**	**Quantity**	**Total**
Supervisor	19.5	1	19.5
Unskilled Laborer	10	10	100
			0

$ 0.25 /LCY

Known Production
2 hr/unit

Production Rate	Unit Rate:	*UNIT*
Capacity		
Production Rate	60	LF/HR
	480	LF/DAY
Duration	23	DAYS

http://www.panamacompra.gob.pa/Adquisicion/CuadroComparativo/cuadro_comparativo.aspx?idlc=66720
2&idorgc=24822&tipo=2

Item Number	002	Unit Price	$ 1.15 TON
Item Description	Rock/Gravel 4" - 12"		
Quantity	2372 TON		

Labor	Unit Rate:	119.5 / Day	
Postition	**Wage ($/day)**	**Quantity**	**Total**
Supervisor	19.5	1	19.5
Unskilled Laborer	10	10	100
			0

2 units/hr
12.964 Tons/unit

Production Rate	Unit Rate:	*UNIT*

Production Rate	12.964	ton/hr
	103.712	ton/day
Duration	23	days

http://www.devale.cl/catalogo/816-PIEDRAS_PARA_AFILAR.html

Item Number	005	Unit Price	LF
Item Description	Approach		
Quantity	390 LCY		

Labor	Unit Rate:	69.5 / Day			
Postition	**Wage ($/day)**	**Quantity**	**Total**		
Supervisor	19.5	1	19.5	$ 0.06	$/LCY
Unskilled Laborer	10	5	50		
			0		

Production Rate	Unit Rate:	*UNIT*
Capacity	3	LCY
Production Rate	150	LCY/HR
	1200	LCY/DAY

Equipment	Unit Rate: $ 360.00	*UNIT*	$ 0.38	$/LCY
Loader	$ 360.00	/DAY		
Capacity	3	LCY		
Production Rate	120	LCY/HR		
	960	LCY/DAY		

Item Number	018	Unit Price	4345.45 EA
Item Description	Steel Erection Total		
Quantity	1 EA		

Labor	Unit Rate:	119.5 / Day	
Postition	**Wage ($/day)**	**Quantity**	**Total**
Supervisor	19.5	1	19.5
Unskilled Laborer	10	10	100
			0

Production Rate	Unit Rate:	*UNIT*	
Capacity			
Production Rate	440	lb/day	
Duration	36.36	days	***16000lb per 440lb/day
			RS Means=05120-0400

http://unionrope.com/Resource_/PageResource/General%20Purpose-GP-313.pdf

Item Number	024	Unit Price	$ 0.23 LF
Item Description	1-1/2" Cable Assembly		
Quantity	2840 LF		

Labor	Unit Rate:	119.5 / Day		$ 0.23 /LF
Postition	**Wage ($/day)**	**Quantity**	**Total**	
Supervisor	19.5	1	19.5	
Unskilled Laborer	10	10	100	
			0	

Production Rate	Unit Rate:	*UNIT*
Capacity		
Production Rate	510	LF/day
Duration	5.6	Days

http://unionrope.com/Resource_/PageResource/General%20Purpose-GP-313.pdf

Item Number	025	Unit Price	$	0.24 LF
Item Description	1/2" Cable Assembly			
Quantity	1623 LF			

Labor	Unit Rate:	119.5 / Day		$ 0.24 /LF
Postition	**Wage ($/day)**	**Quantity**	**Total**	
Supervisor	19.5	1	19.5	
Unskilled Laborer	10	10	100	
			0	

Production Rate	Unit Rate:	*UNIT*		
Capacity				
Production Rate	488	LF/day	Means-05150-0890	Account for difficult access to main cable
Duration	3.33	Days		

http://unionrope.com/Resource_/PageResource/General%20Purpose-GP-313.pdf

Item Number	026	Unit Price	EACH
Item Description	Turnbuckle - Crosby HG-228 Jaw & Jaw 2" x 24"		
Quantity	8 EACH		

Labor	Unit Rate:	259.5 / Day		$ 8.11 /Turnbuckle
Postition	**Wage ($/day)**	**Quantity**	**Total**	
Supervisor	19.5	1	19.5	
Unskilled Laborer	10	24	240	3 per turnbuckle
			0	

Production Rate	Unit Rate:	*UNIT*	
Capacity			
Production Rate	4	PER HR	15 min per turnbuckle
	32	units/DAY	
Duration	0.25	DAYS	

http://www.shamrocksupply.com/itemDetailFilterPH.action?RFP=IFD&IDP=Y&codeId=10754809&mfr=The%20Crosby%20Group%20IncThe%20Crosby%20Group%20Inc&mpn=1033054

Item Number 027 Unit Price 120.00 CY
Item Description Concrete
Quantity 277.33 CY

Labor	Unit Rate:	59.5 / Day	
Postition	**Wage ($/day)**	**Quantity**	**Total**
Supervisor	19.5	1	19.5
Unskilled Laborer	10	4	40
Operator		0	0

0.496 $/CY

Production Rate	Unit Rate:	*UNIT*	
Capacity	9	CY	Concrete Truck
Duration	30.8		
Production Rate	120	CY/D	
Duration	2.3	DAYS	
Labor Unit Cost:	0.50	$/CY	

Item Number	028	Unit Price	702.38 TON
Item Description	5/8" A36 Steel Round Bar		
Quantity	0.54236 TON		

Labor	Unit Rate:	59.5 / Day	
Postition	**Wage ($/day)**	**Quantity**	**Total**
Supervisor	19.5	1	19.5
Unskilled Laborer	10	4	40
			0

$ 40.48 /DAY

Production Rate	Unit Rate:	*UNIT*
Capacity		
Production Rate	1.47	TON/DAY
Duration	0.369	DAYS
Labor Unit Cost:	40.48	$/TON

http://www.metalsdepot.com/catalog_cart_view.php?msg=
http://www.hormigonexpress.com/precios.php

Item Number	029	Unit Price	$	702.38 TON
Item Description	5/8" A36 Steel Round Ties			
Quantity	0.65 TON			

Labor	Unit Rate:	59.5	/ Day
Postition	**Wage ($/day)**	**Quantity**	**Total**
Supervisor	19.5	1	19.5
Unskilled Laborer	10	4	40
			0

Production Rate	Unit Rate:	*UNIT*
Capacity		
Production Rate	2.1	TON/DAY
Duration	0.30992	DAYS
Labor Unit Cost:	28.33	$/TON

http://www.metalsdepot.com/catalog_cart_view.php?msg=

Item Number 030 Unit Price 44.05 EACH
Item Description Crosby 1-1/2" G-450 Wire Rope Clips
Quantity 64 EACH

Labor	Unit Rate:	29.5	/ Day
Postition	**Wage ($/day)**	**Quantity**	**Total**
Supervisor	19.5	1	19.5
Unskilled Laborer	10	1	10
			0

1 man / clip

Production Rate	Unit Rate:	*UNIT*	
Capacity			
Production Rate	6	PER HR	10 min / clip
	48	/day	
Duration	1.33	DAYS	
Labor Unit Cost:	$ 0.61	$/EACH	

http://www.fdlake.com/wrclips.html
http://www.westechrigging.com/wire-rope-wire-rope-fittings-clips-crosby-g-450-clips.html

Item Number 031 Unit Price 8 LF
Item Description 1" x 1" Square Steel Anchorage Hook
Quantity 240 LF

Labor	Unit Rate:	160	/ Day
Postition	**Wage ($/day)**	**Quantity**	**Total**
Supervisor	19.5	0	0
Unskilled Laborer	10	16	160
			0

Production Rate	Unit Rate:	*UNIT*
Capacity		
Production Rate	240	LF/DAY
Duration	1	DAYS
Labor Unit Cost:	0.67	$/DAY

http://www.metalsdepot.com/catalog_cart_view.php?msg=

Item Number	032	Unit Price	0 CY
Item Description	Anchor Block Excavation		
Quantity	139 CY		

Labor	Unit Rate:	99.5 / Day	
Postition	**Wage ($/day)**	**Quantity**	**Total**
Supervisor	19.5	1	19.5
Unskilled Laborer	10	8	80
			0

Production Rate	Unit Rate:	*UNIT*
Capacity		
Production Rate	24	CY/day
Duration	5.79	DAYS
Labor Unit Cost:	4.15	$/CY

Item Number	033	Unit Price	120 CY
Item Description	Concrete (Tower Foundation)		
Quantity	107 CY		

Labor	Unit Rate:	59.5 / Day		2.48 $/CY
Postition	**Wage ($/day)**	**Quantity**	**Total**	
Supervisor	19.5	1	19.5	
Unskilled Laborer	10	4	40	
			0	

Production Rate	Unit Rate:	*UNIT*
Capacity		
Concrete Pour		
Production Rate	120	CY/day
Duration	0.89	day
Concrete Formwork		
Production Rate	30	CY/day
Duration	3.57	day

http://www.hormigonexpress.com/precios.php

Item Number	035		Unit Price		700 Ton
Item Description	No. 6 Rebar				
Quantity		2.02 Ton			

Labor	Unit Rate:	59.5 / Day		$	28.33 /TON
Postition	**Wage ($/day)**	**Quantity**	**Total**		
Supervisor	19.5	1	19.5		
Unskilled Laborer	10	4	40		
			0		

Production Rate	Unit Rate:	*UNIT*
Capacity		
Production Rate	2.1	Ton/day
Duration	0.961904762	day

Item Number	036	Unit Price	$	9.31 SY
Item Description	Wood - Alemendro			
Quantity	150 SY			

Placement of Decking

Labor	Unit Rate:	59.5 / Day		$ 5.36 /SY
Postition	**Wage ($/day)**	**Quantity**	**Total**	
Supervisor	19.5	1	19.5	
Unskilled Laborer	10	4	40	
			0	

Production Rate	Unit Rate:	*UNIT*

Capacity

Placement

Production Rate	100	SF/day
	11.11	SY/day
Duration	13.50	Days

Cutting the Timer

Labor	Unit Rate:	39.5 / Day		$ 3.95 /SY
Postition	**Wage ($/day)**	**Quantity**	**Total**	
Supervisor	19.5	1	19.5	
Unskilled Laborer	10	2	20	
			0	

Production Rate	0.003273	Trees/day
	3055.00	SY/tree
	10.00	SY/day
Duration	15.00	Days

Item Number	037	Unit Price	34.74 LF
Item Description	Steel Beams - LL3x3x3/8 (galvanized)		
Quantity	194 LF		

Labor	Unit Rate:	119.5 / Day		$ 2.49 /LF
Postition	**Wage ($/day)**	**Quantity**	**Total**	
Supervisor	19.5	1	19.5	
Unskilled Laborer	10	10	100	
			0	

Production Rate	Unit Rate:	*UNIT*	
Capacity			
Production Rate	192	SF/day	
	48	LF/day	
Duration	4.04	Days	Use 5 days

http://www.metalsdepot.com/catalog_cart_view.php?msg=

Item Number 038 Unit Price 12.28 Each
Item Description Cable Clips
Quantity 850 Each

Labor	Unit Rate:	59.5 / Day	
Postition	**Wage ($/day)**	**Quantity**	**Total**
Supervisor	19.5	1	19.5
Unskilled Laborer	10	4	40
			0

$ 0.35 /EACH

Production Rate	Unit Rate:	*UNIT*
Capacity		
Production Rate	170	EACH/day
Duration	5.00	Days

http://www.westechrigging.com/wire-rope-wire-rope-fittings-clips-crosby-g-450-clips.html

Item Number	041	Unit Price	1.23 Each
Item Description	Nails - 3.5"		
Quantity	1360 Each		

Labor	Unit Rate:	29.5 / Day		$ 0.12 /EACH
Postition	**Wage ($/day)**	**Quantity**	**Total**	
Supervisor	19.5	1	19.5	
Unskilled Laborer	10	1	10	
			0	

Production Rate	Unit Rate:	*UNIT*	
Capacity			
Production Rate	240	Nails/day	2min/nail
Duration	5.67	Days	

http://www.novey.com.pa/category.php?id_category=1173

Item Number	045	Unit Price	LF
Item Description	Concrete (Trucks)		
Quantity	384.33 LCY	Labor included in ITEM 000	

Labor	Unit Rate:	0 / Day	
Postition	**Wage ($/day)**	**Quantity**	**Total**
Supervisor	0	1	0
Unskilled Laborer	0	5	0
			0

Production Rate		Unit Rate:	*UNIT*
Cost per Day	$	30,000.00	/ YEAR
	$	82.19	/ DAY