The Soloy Bridge Project

Los Agua Niños International Senior Design December 12, 2009

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Disclaimer

The following represents the efforts of undergraduate students in the Civil and Environmental Engineering Department of Michigan Technological University. While the students worked under the supervision and guidance of associated faculty members, the contents of this report should not be considered professional engineering.

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

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1.0 Executive Summary

Los Agua Niños was developed as part of the International Senior Design (ISD) program at Michigan Technological University. In August 2009 the team traveled to Panama. The following report represents the field work and background information gathered while in Panama. From this information, possible bridge designs were analyzed. This report will cover why the proposed bridge was the most feasible option for this project.

While in Soloy, Los Agua Niños gained a better understanding of the local people and their culture. We determined that the local people needed a way to safely access an island within the Rio Fonseca. The data needed to design such a bridge was collected. This data included a survey of the site, the velocity of Rio Fonseca, classification of the soil at the site and the prices of the materials needed for the proposed design.

The current design is a suspension bridge that spans Rio Fonseca to an island containing the best fútbol field in Soloy as well as other fields containing potential as the Ngäbe see fit. The current design consists of concrete masonry unit foundations filled with sand, concrete anchors, steel cables, and local hardwood decking and towers.

The next step for Los Agua Niños will be to submit this design to the Peace Corps volunteers who live in Soloy. These volunteers have agreed to organize the construction of the bridge, using volunteer labor from the people of Soloy, the 16 fútbol teams in particular.

In conclusion, Los Agua Niños is dedicating its efforts to accommodate the needs of the local community of Soloy. Los Agua Niños hopes to provide an affordable yet sustainable bridge design which can be constructed by the Soloy people.

2.0 Introduction

Los Agua Niños, comprised of Eryk Anderson, Nicholas Childs, Jonathan Dobrei, and Hampton Waring, is a student team formed as part of Michigan Technological University's International Senior Design class. The purpose of this class is to give students real-world experience collecting field data and designing structures intended to aid those living in developing countries. Each student in this group is majoring in civil engineering with an emphasis in structural design.

In August 2009, Los Agua Niños traveled to the country of Panama with the intent of designing a bridge for the Ngäbe village of Soloy. During the team's stay, Los Agua Niños collected data from the existing project site needed for a bridge design proposal. When the construction of the proposed design is complete, the Ngäbe villagers will once again be able to safely cross the Rio Fonseca to their recreation area.

During the week of data collection, the proposed bridge site was surveyed and soil, river velocity, and erosion information were obtained. This data, along with input from local people and Peace Corps volunteers stationed in Soloy, resulted in a proposed bridge plan. The bridge design began upon the return to campus in the United States. Major design challenges included protecting the proposed bridge from sizable flooding events and debris clutter while keeping material and construction costs at a minimum. The majority of construction in Panama follows United States design code. The bridge has been designed to comply with the following codes: ACI 318-08 for reinforced concrete design and NDS 2005 for wood design. The suspension bridge design presented in Survey, Design, Construction of Trail Suspension Bridges for Remote Areas by Grob, et al. was used to design the steel cables. It was also used as a starting point for tower design.

In the following report, the background of Soloy along with the methods and procedures used to collect data will be discussed. Furthermore this data was analyzed, yielding several feasible bridge design options. A concrete, steel, and wood suspension bridge is the most suitable option, and the details of its design, estimated cost, construction schedule, and constructability will be discussed. A final recommendation will be made based on this information.

3.0 Background

This section first describes the location, weather, and population of the country of Panama. It then focuses on the area surrounding the potential bridge site, discussing its location, population, and geography. Finally, the specific conditions impacting the project are discussed, including how the project idea arose, why it is important, who benefits from it, and how it will potentially be funded, designed, and implemented. Specific goals of the project will also be identified.

3.1 Location

Panama is located in the middle of the Americas and forms an isthmus, or land bridge, from the North American continent to the South American continent. The country has a total area of 30,420 square miles and is located east of Costa Rica and west of Columbia. The narrowest section of Panama is 80 kilometers, making it the ideal location for a passageway between the Pacific and Atlantic Oceans.

Panama is divided into nine provinces: Bocas del Toro, Chiriquí, Coclé, Darién, Herrera, Los Santos, Panamá, and Veraguas. Out of these provinces, three separate provincial-level comarcas have been set aside for the indigenous peoples of Panama: Emberá, Kuna Yala, and Ngöbe-Buglé (Figure 1).

Figure 1: Provinces and Comarcas of Panama
Source: http://en.wikipedia.org/wiki/Provinces_and_regions_of_Panama

3.2 Weather

The weather of Panama is tropical, with temperatures ranging from 80-90 degrees Fahrenheit. There are only two seasons in Panama: December to April is the dry season with bright and hot sun, while May to November is the rainy season. There is rain nearly every day during this time of the year and the air is very humid. Because Panama is located south of most hurricane paths, it is rarely affected by tropical storms.

3.3 Population

With a population of 3.2 million people, Panama does not contain a great amount of diversity. The majority of Panama is made up of Mestizos, a combination of Indian and Spanish, who make up 70 percent of the total population. Following Mestizos are West Indians who make up about 14 percent, then Caucasians who make up about 10 percent of the population. The Amerindian or indigenous groups, of which there are 7 different tribes, make up around 10 percent of the population and take great pride in their separate languages and cultures.

One such tribe is the Ngäbe, whose members reside in land set aside for them by the Panamanian government. This land is called the Comarca, which roughly translates to 'county.' There are several Ngäbe villages within the Comarca, one of which, Soloy, will be the focus of this report.

The team learned that the villagers receive a monthly 35 dollar stipend from the government, which is their primary source of income. This information became design criteria for the team's final bridge design. The government will not be sponsoring the construction costs so the total cost will be raised by the Ngäbe.

3.4 Soloy: Location, Project, Plan

Soloy is a small city/village located in the Comarca of Ngöbe-Buglé (Figure 2). Soloy is 40 kilometers due east of the city of David, where groceries, clothing, building supplies, hardware, electronics, and other goods may be obtained. It takes two hours to travel from Soloy to David by bus. Soloy is populated by approximately 6000 Ngäbe who live in small, light, wooden structures with metal roofing and no electricity or indoor plumbing. The common profession among the Ngäbe men is coffee farming and harvesting. The women stay at home with the children and sew, weave, wash clothes, cook, and perform other household tasks. The Ngäbe are fun-loving people who enjoy participating in games such as fútbol, volleyball, and baseball. According to Peace Corps volunteers, Soloy's educational system seems to be struggling due to the lack of properly trained teachers.

Figure 2: Location of Soloy

The rivers that flow through Soloy are the Rios Gevay, Fonseca, and Soloy. These rivers have been troublesome to the Ngäbe and have altered the geography of the area. In November 2008, a massive flood took place, which covered much of the land in the lower elevations. Among the land that was enveloped in water was an island on which people lived, engaged in cultural activities, and spent leisure time. Among the damage from the flood was a wood and cable footbridge that connected the mainland of Soloy to this island. Without this bridge, the Ngäbe have to wade across the Rio Fonseca. This river is rated a Class II on the White Water Rafting

Scale, meaning the river is very dangerous. Since the bridge was destroyed, several villagers have drowned while trying to reach the island due to the strong current. Villagers have also been stranded on the island, as sudden rain events can drastically increase river depth in a short amount of time.

The Ngäbe have been in need of a new footbridge to cross the Rio Fonseca, but the local political figures are wary of constructing one. The mayor of the city and the district fear that if a new bridge is constructed, people would build new homes on the island, putting themselves in harm's way if another great flood were to occur. However, with elections nearing, these political figures are trying to fulfill some of the promises they made from their last election. They hope that if the bridge is distanced away from the school, people will use it specifically for access to recreational purposes instead of living quarters. Under these circumstances, the city mayor has pledged to budget for all of the wood required for the project.

The plan is to design a suspension footbridge using this donated wood. Donated steel cable may also be secured from the non-profit organization Bridges to Prosperity. Concrete will be used for abutments and anchors. These building materials were selected because the local people are familiar with them. The locals intend to construct the bridge themselves to help minimize cost. These preliminary plans were developed over a week of intense research. Local people, Peace Corps volunteers and local government officials were consulted. Data vital to the design of this project was also collected, as will now be described.

4.0 Methods and Procedures

Los Agua Niños spent a week in Soloy collecting data in order to design a bridge that is well suited to both the physical characteristics of the site and for the community of Soloy. The team performed a 150-point survey of the bridge site. Soil classification, river velocity measurement, and erosion inspection was also performed. These on-site tasks, along with a detailed description of how each task was done, will be examined in the following sections.

4.1 Surveying

The team was outfitted with the following survey equipment:

- Total station
- Tripod
- Prism rod
- Data collector
- GPS

The team's first task was to position the total station so that it could read most of the points needed to characterize the proposed site. This was necessary because traversing the total station would require time, which was limited due to early nightfall and short battery life of the equipment.

The team took the extra precaution of designating a note-taker to write the physical description for each of the points to help reference them later (Appendix 1: Point Descriptions).

The fourth member of the team was assigned the task of taking pictures with compass directions and getting the GPS coordinates of the benchmark the team had selected. The GPS locations are very critical to the project because they are universal and can be easily retraced for when the project begins its construction. Also, the pictures with the direction referenced would help later in the designing process to get an exact visual on the location and to help remember details of the proposed site.

The team's equipment also included a computer that was equipped with AutoCAD. This allowed the team to look at the data points collected for the day and plot the points in a three dimensional plot, where the points can make a topographic view of the site. Every night would consist of making sure that all the points collected for the day were correct and looked comparable to the pictures taken of the site.

Surveying was done on both sides of the river and included some points in the river. The island where the fútbol fields are located was a very flat terrain with very little differences in elevation, which reduced the amount of points needed for that particular area. However, the mainland had more diverse terrain with multiple high and low spots. This also included a large ditch that flowed into the river as drainage from the mainland. This meant more points had to be taken in order to obtain an accurate surface model of the proposed site.

4.2 Soil Classification

Soil boring equipment was not available, so simple visual soil classification was used to characterize the soils at the bridge site. Soil classification was performed according to the USCS method using the ASTM D 2488 – Standard Practice for Description and Identification of Soils. Please refer to Appendix 2: ASTM Standard D-2488 -Visual Classification of Soils for further information.

Three, twenty-centimeter diameter test pits were dug on each riverbank to depths of roughly one meter. Soil samples were taken at fifteen centimeters and one meter to be visually inspected. Different types of soil were found at these two depths, so each was classified.

4.3 Erosion Inspection

Erosion inspection included examining the riverbanks for any signs of erosion, including exposed tree roots and soil. Panoramic photographs of each riverbank were taken. Apart from the trees, there is little vegetation on the site and riverbanks, making the site especially prone to erosion. Other than visual inspection there was no equipment that could be used to calculate how much erosion had occurred.

4.4 River Velocity

The river velocity tests that were taken covered a stretch 15 meters perpendicular to the proposed bridge. The equipment for the test was low tech but it served the purpose. It included:

- Stopwatch
- Measuring tape
- Calculator
- Empty Bottle

The team started by having two of its members wade into the water and measure a length of 15 meters. One person was upstream while the other was downstream. The member upstream would set the empty bottle in the water and measure with the stopwatch how much time it took to travel 15 meters to the member downstream. This test was performed three times, at positions ¼, ½, and ¾ of the river width away from the riverbank. This test was done three times at each position and then the times were averaged for that section of the river.

4.5 Additional Information

Attempts were made to obtain topographic maps, aerial photographs, and rainfall/river flow data from agencies in Panama City. Topographic maps were purchased and shipped to the U.S. via a third party, but the scale on the maps proved to be too large, rendering them useless for watershed delineation. However, a delineated watershed for the entire Rio Fonseca was obtained from ETESA of Panama (an organization similar to the United States Geological Survey or USGS). Information from stream flow gages, including hydrographs, was also obtained from the ETESA Hidrometeorología website, but this data alone was not enough for a complete hydrologic analysis.

Prices for common construction materials were obtained at Materiales Karen, a hardware and building supply store in David. These materials include:

- Cement
- Sand
- Plywood
- Wire fence
- Wood nails
- Bolts
- Treated lumber
- Antifungal varnish

Heavier construction materials, such as structural steel, must be purchased from Panama City. It is unlikely that such materials will be used due to cost. The local government has pledged untreated wood for the bridge and, as previously mentioned, steel cables may be donated by Bridges to Prosperity. However, concrete, plywood for formwork, fasteners, and rebar will still be needed. There are no funding sources for these items as of yet. This issue will be further discussed in the following section. Figures and tables describing all collected data will also be presented and analyzed.

5.0 Analysis and Design Options

Existing conditions at the bridge site will now be discussed. All collected data—surveying, soil, erosion, and river velocity will be analyzed and sample hydrographs and material prices will be given. Finally, project concerns, significance, and feasibility will be addressed.

5.1 Existing Conditions

The proposed pedestrian bridge will replace a foot bridge that was destroyed in a major flood event in 2008. The bridge will connect the mainland of Soloy to an island bordered by two branches of the Rio Fonseca (Figure 3).

Figure 3: Rivers in Soloy

The island houses Soloy's best fútbol field, used by fifteen mens' teams and one womens' team. The field is also used for baseball and other cultural events, drawing crowds of Ngäbe from surrounding villages. Since the flood, the mens' teams have relocated to a field in the mountains that takes an hour long hike to reach. The womens' teams still use the field on the island, but the Fonseca can be dangerous to cross and is inaccessible at times. The water level can rise suddenly during the frequent rain events in Soloy, and the womens' team has at times been trapped on the island.

The proposed site is approximately one kilometer north of the original bridge site. The original site was unusable because of space requirements (road interferes with abutment placement) and a government decision. The Panamanian government does not want to rebuild a bridge in the original site because it would facilitate re-inhabitation of a flood-prone island. The original bridge linked an inhabitable part of the island to the Soloy school. The proposed bridge, however, is across from the fútbol field, so it does not directly link to an inhabitable part of the island. The proposed bridge is also significantly closer to the center of Soloy, making it more convenient to access the recreation area. While building the bridge away from an inhabitable part of the island may discourage re-inhabitation, it cannot entirely prevent it.

The west bank of the proposed bridge site is bordered by the Rio Fonseca on one side, and a road on the other (Figure 4). It is directly across from the recreation area where the villagers of Soloy play fútbol and baseball. It is also where citizens currently cross the Rio Fonseca to access the field. There is a home located on the site, though there is ample space to construct a foot bridge without disturbing the building. The homeowner stated that he wanted to donate a portion of his land for the bridge. He said that he wanted to donate the land for the good of the community. Outdoor recreation is very popular in Soloy, and safe access to the recreation area is important. He also is interested in opening a concession stand. One concern is that he will try to charge admission for the use of the bridge, though he assured us that he will not.

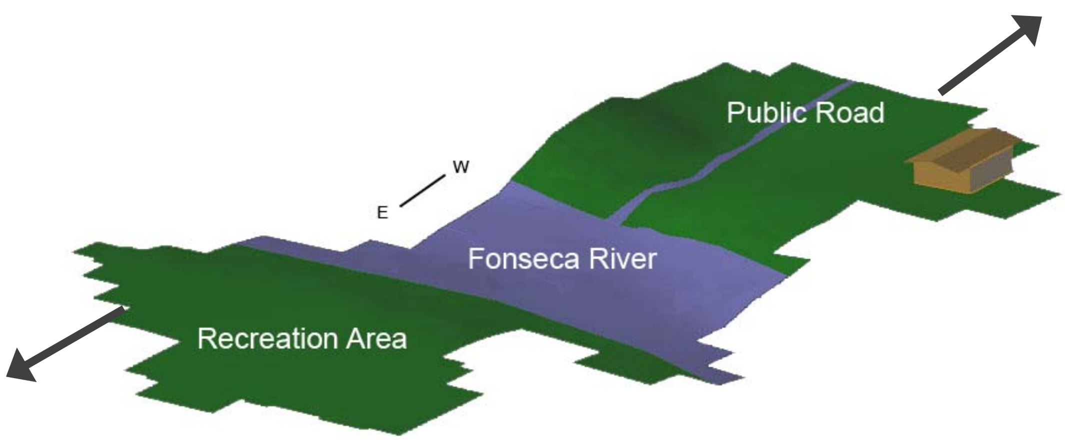

Figure 4: Aerial View of Site

In addition to the location (proximity to the recreation area) being ideal, the site is physically well-suited for building a bridge. There is ample space on both sides of the river to place abutments and anchors. Flooding and erosion is a concern, so the ability to place anchors farther back from the river is important. The elevation on both sides of the river is relatively equal, reducing the need for earthwork or unequal towers to compensate. Some trees will have to be removed, however.

5.2 Surveying

The methods described in Section 3 yielded surveying, soil, river velocity, and erosion data. A 250-point survey describes the topography of the river bottom and on both sides of the river (Figure 5). The backsight point (far right on below figure) marks an existing public road. The survey data also marks the high water point (represented by a blue line) of the flood in 2008 that destroyed the old bridge.

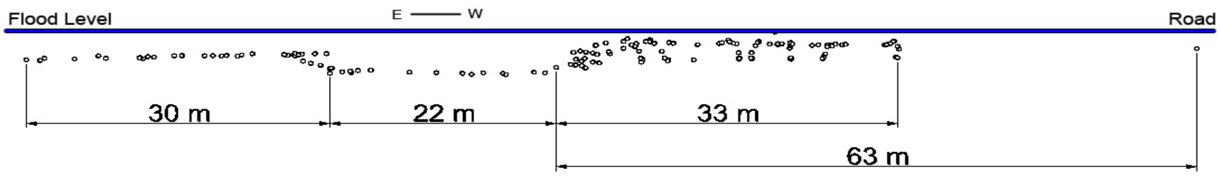

Figure 5: Surveying Cross-Section

5.3 Data Analysis: Soil Classification

Soil classification indentifies the type of soil the bridge will be built on. Soil type must be taken into consideration when designing bridge abutments and anchors. If the design is not well-suited to the soil it will be built on, the stability of the structure will be compromised.

Table 1: Summary of Visual Soil Classification

Summary of Visual Soil Classification		
Bank	*Depth (cm)*	*Classification*
East	15	Grey-brown silty fines, some fine sand, low plasticity, moist (ML)
	100	Medium brown sand, poorly graded, rounded, moist (SP)
West	15	Red-brown clay, trace fine sand, trace fine gravel, low plasticity, moist (CL)
	100	Medium brown sand, poorly graded, rounded, moist (SP)

Table 1 summarizes the data obtained from visual classification of soil at the bridge site. At a depth of one meter on both river banks, the soil is moist, poorly graded medium brown sand with rounded grains. At a depth of 15 centimeters on the west side of the river, the soil is a moist, low plasticity red-brown clay with trace fine sand and trace fine gravel. At a depth of 15 centimeters on the east bank near the recreation area, the soil is moist and low plasticity with grey-brown silty fines and some fine sand. The full classifications may be found in Appendix 3: Soil Classifications.

5.4 Erosion Inspection

Soil is eroded along both banks of the river (Figure 6). The roots of large (1-3.5 meters in circumference) higeuron trees are exposed. Vegetation is scarce on much of the bank, and it seems that these trees are primarily responsible in preventing further erosion of the riverbank. There are fewer trees on the east bank of the river, and it is apparent that this riverbank has eroded to a greater degree when compared to the west bank.

Figure 6: Erosion of Riverbanks

5.5 River Velocity

Table 2: River Velocity Before Rain Event

River Velocity	
Position	*Velocity (m/s)*
West (1/4)	0.9
Center (1/2)	1.1
East (3/4)	0.4

River velocity can also be useful when determining the effects of erosion on a riverbank. Velocity was found to be greatest in the center of the river, with slower velocities near the shore (Table 2). Both velocity and river depth were observed to increase after rain events. This not only hastens erosion, but has the potential to put villagers in a dangerous situation. The effect of rain on the river can be so sudden and dramatic that fútbol players who waded across the river and were on the island when a rain event occurred have been stranded there for several hours.

5.6 Additional Information

Because it was not possible to obtain sufficient rainfall and river flow data for the Rio Fonseca, a complete hydrologic model identifying the high water line of a 100-year flood could not be completed. Therefore, the observed high-water line (from the major flood that occurred in 2008) and historical knowledge will be used for design. It is known that the last major flood (similar in severity to the one in 2008) occurred in 1908. Because of this, these floods have been coined the "100-year floods" by locals, even though a proper analysis based on hard data has never been performed. Some data on the Rio Fonseca was obtained, though, and it does help characterize the behavior of the river even if it is not enough for a complete analysis. For example, the hydrograph (Figure 7) shows that the Rio Fonseca historically experiences its peak flow (m^3/s) in October. Therefore, the values measured during the assessment trip in August likely do not represent the maximum depth and velocity the Rio Fonseca, on average, experiences in one year. Such information is invaluable if the bridge is to be designed to withstand both average flow and potential flooding.

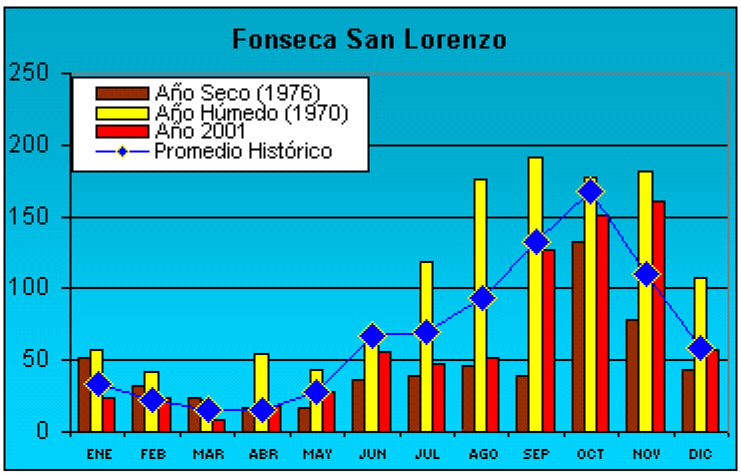

Figure 7: Hydrograph of Rio Fonseca
Source: www.hidromet.com.pa

Table 3: Material Prices at Materiales Karen (8-17-09)

Material Prices (8-17-09)			
Material		*Quantity*	*Price*
Cement		42.5 kg	\$7.80
Sand		1 m^3	\$13.90
Plywood	1/2"	4'x8'	\$24.65
	3/4"	4'x8'	\$32.40
Rebar	3/8"	30'	\$3.35
	1/2"	30'	\$5.60
Wire fence	4' tall	30 m	\$73.00
	5' tall	30 m	\$74.95
Wood Nails - Any size		1 lb	\$0.85
Bolts - Any size		1 lb	\$0.20
Treated Lumber	10'	1 - 2"x4"	\$0.90
	12'	1 - 2"x4"	\$0.90
Antifungal varnish		1 gallon	\$14.50

A list of prices for several construction materials was obtained at Materiales Karen in David (Table 3). Heavier construction items, such as structural steel, must be obtained from suppliers in Panama City.

5.7 Concerns

These results raise several concerns. The site is prone to flooding, and the soil is comprised mainly of clay and silt fines with medium sand. There is little vegetation on the site, and the high clay content of the soil further impedes water infiltration, leading to pooling of water on the site. Even small rain events observed on the site compromised the stability of the soil. As the soil was saturated with water, it became more like mud than a cohesive mass suitable for building upon (Figure 8). It must be noted that these photos represent an exaggerated case. The ground shown in these photographs was directly underneath the edge of a metal roof, so the amount of water pooling underneath it is greater than what would accumulate in this timeframe from rain alone. However, similar pools of water were beginning to collect in small depressions on the property after two hours of steady rain. These photos represent what can happen on the bridge site after an hour of rain.

Figure 8: Effect of Rain on Soil

The significant erosion present at the site demonstrates that this soil is not ideal for building; measures will have to be taken during design to mitigate the effects of erosion and soil instability. For example, the bridge abutments and anchors should be placed sufficiently far from the riverbanks to prevent soil from eroding underneath them. Placing riprap on the riverbanks to minimize erosion is another possible option.

Another issue affecting erosion is river depth and velocity. The river velocities from Table 2: River Velocity Before Rain Event were taken when the river was roughly one meter deep at its center. This river, Rio Fonseca, was observed to deepen even after small rain events. As it deepened, its velocity was observed to increase. Increasing river velocity and depth can hasten erosion of the riverbank.

Finally, there is the issue of cost. City Mayor Bejerano has pledged wood (likely a local untreated hardwood) for the bridge deck. Apart from that, there are presently no funds set aside to build a bridge to the island in Soloy. The people of Soloy have expressed great interest in re-using the steel cables of the destroyed bridge, which are currently tangled on the riverbank near the Soloy school. An assessment of these cables revealed them to be of varying quality, with some cables rusted, swollen, and fraying (see the top and bottom cables in Figure 9) and others in reasonable condition (middle cable).

Figure 9: Cables from Old Bridge

Table 4: Materials from Old Bridge

Materials from Old Bridge		
Material	*Approx. Length (m)*	*Quantity*
Steel Cable		
1/2" diam.	12	1
	70	2
1" diam.	12	1
	30	4
Steel Clamp		
2"	n/a	12

Table 4 gives approximate measurements of the length and type of cables and clamps available for re-use. The possible re-use of these cables is a major concern. Even if the bridge design specifies new steel cables, the people of Soloy will most likely try to re-use the old cables if new ones are not available. Some of these cables appear fit for re-use, but many are in poor condition. Additionally, the old bridge spanned a much shorter distance than the new bridge will need to, so the both the length and diameter of available cable is insufficient. For both practicality and safety, it is imperative that new cable (or donated used cable in acceptable condition) be obtained.

5.8 Significance

These findings are significant because they affect the bridge design. It is clear that methods of erosion prevention need to be incorporated into the bridge design. The bridge should also be able to withstand flooding and the resulting soil instability. These problems have been addressed in the bridge design.

5.9 Feasible Design Options

Once the team had a good understanding of the background of the area and the needs of the people, our main decisions were where to build the bridge, what general design to use for the bridge, and what materials to use to construct the bridge. The decisions of where to build the bridge and what style of bridge were quickly resolved.

There were a total of three proposed sites for potential bridges in Soloy. The first was at a flooded roadway that led up into the mountains where more Ngäbe lived and farmed. It was being safely crossed by horse and truck, but at higher water levels it would be nearly impossible to cross. Since it is part of the road system, a vehicle bridge would be required. The layout of the land and the ground surrounding the area would require a multi-span bridge. The advantages to this location include safer travel during the wet season and the convenience of being close to the center of the town. The disadvantage is the magnitude of the project: a multi-span vehicle bridge would be too large of a project to complete in the given amount of time. The final decision was to find a different location because this project was not feasible.

The second possible project was to replace the bridge to the island in the middle of the Rio de Fonseca that was destroyed during a major flood event last year. The previous bridge was a suspension bridge constructed of wood planks and steel wire rope. The steel ropes were attached to trees on one side of the river and to posts on the other. During the large flood event last year, the bridge was washed out and transportation to the island slowed to a halt. The island is the location of one of the best fútbol fields in the area and the locals are currently wading across the Rio de Fonseca to get to the island. Building a bridge in this location would provide a familiar and safer mode of transportation to get to the island. On the downside, the road runs very close to the river and anchoring a bridge would require placing cables across the road. Another concern was the opposition of the local government to building another bridge there. The Ngäbe people were building houses on the island to be close to the school. Once the island flooded and the bridge washed out the government deemed it unsafe to live there and fear they will inhabit the island again with the construction of a bridge. Due to the complexity of the anchoring system and the opposition by the government, the team decided not to design for this location.

The third and final site was the closest crossing to the fútbol fields on the island. Many of the fútbol players wade through the river at this point because it is the closest and the river is relatively shallow and narrow. This site was close to the residence of one of the locals. He stated that he would be more than happy to donate his land for the construction of a pedestrian bridge. This location would provide easy access to the field while being far enough from the school to discourage the construction of housing. It is also far enough from the road where designing an anchor system would not be inhibited by road. Both sides of the riverbank are roughly the same elevation to provide easier designing and construction. Unfortunately, since it will be on an individual's property, a path will have to be cleared for foot traffic. This site was the best location for the footbridge because of its location and interest by the locals.

After the location was decided, the next step was to decide what style of bridge to design. The team quickly decided on a suspension bridge design. This is the style used for several of the other pedestrian bridges in the area so it will be familiar to the locals. It is generally a lighter design option so it creates an easier construction for the Ngäbe people. The materials that would be used for the suspension bridge would be easier to obtain than materials for a heavier bridge style like a truss bridge. From these simple details it was an easy to decide that a suspension bridge was the best option.

The final decision to be made was what materials to use for this suspension bridge. The first option was rope and wood planks that could be anchored to posts or trees. These materials would be very light and easy to obtain. Since the materials would be very light, it would make construction and maintenance very easy. The downside of this design comes with the limited durability. The rope would be subject to snapping or falling apart due to weather and other forces of nature. The durability issue of the rope was the deciding factor. The team decided that this was not one of the final options.

The second option was to create a design similar to some of the existing pedestrian bridges. These bridges were steel cable suspension with concrete abutments and towers anchored into concrete blocks in the ground. The decking was steel plates on top of c-beams. This design has proved to be very durable for the conditions in Soloy, even though maintenance is not regularly performed. All of the bridges constructed this way survived the large flood event. Although the steel is very durable, it is also a heavy design. The steel needed for this would be difficult for the locals to obtain and an outside contractor would be needed for construction. This would make for an expensive project that the community cannot afford. The team has not yet decided on whether or not these materials will be the final design materials.

The third and final option is a combination of the first two. It would be a steel cable bridge with the same reinforced concrete abutments and anchors as the steel bridge but the decking and towers would be replaced with local hardwood. This would also allow the Ngäbe people to construct the bridge using familiar materials, some of them obtained locally (hardwood, water and possibly gravel for concrete). Wood has been pledged for this project by city Mayor Bejerano, steel cables may be obtained from Bridges to Prosperity, and cement, rebar, and fasteners can be brought in from the city of David. The disadvantage to this design is that the wood deck and towers may not last as long as steel but the team anticipates that the Ngäbe would be more likely to fix and maintain wood than they would steel.

Based on the advantages and disadvantages of each option the team has decided that the bridge will be a suspension bridge with steel cables, reinforced concrete abutments and anchors, wood decking and timber towers. It will be located at the crossing point nearest to the fútbol field. The design details of this bridge will now be presented and a final recommendation will be made.

6.0 Final Recommendation and Conclusion

Upon completion of the initial investigation, Los Agua Niños has produced a final recommendation. The following will discuss the design option chosen to benefit the people of Soloy. This section will discuss the final design, who will benefit from it and how it will benefit them, the difference in benefit to cost, the impacts the project would have on the community, and will also discuss an implementation plan.

6.1 Design Recommendations

Team Los Agua Niños recommends that the best design for the proposed site would be a suspension footbridge with steel cables, concrete masonry unit foundations, timber towers, a wooden plank walkway, and concrete anchors. Shown below (Figure 10) is a surface model that displays the proposed final design of the Soloy Bridge Project. The following paragraphs will explain this design in more detail. The benefits to the local people and cost for the bridge are both major design criteria. Both criteria have been taken into account and will be discussed in the following sections. Detailed design drawings will be heavily referenced throughout design recommendations and are located in Appendix D: Construction Drawings.

Figure 10: Bridge Model

6.1.1 Foundations

The towers require a strong foundation able to bear the forces of the tower and the bridge. The foundation must allow these forces to be spread evenly and thinly enough across its base to keep the bearing pressure lower than the bearing strength of the soil. The foundation must be able to resist flood forces and also raise the tower base above the flood line. The design of the foundation consists of two main parts. In order to ensure that the soil will support of the bridge, the foundation base slab was the first part to be designed. The second part of the design was the column to be placed on top of the base slab.

The concrete slab at the base of the foundation was calculated using concepts and examples from Reinforced Concrete Mechanics & Design by James K. Wight and James G. MacGregor. There were several factors involved in the calculation of the base slab. The first design criterion to check was the minimum area that would ensure the soil could hold the weight of the bridge. Next, the slab had to be checked to make sure it could handle the two-way and one way shear. After the shear checks, the flexure reinforcement followed in design. At this point, the development, or bond length, of the steel had to be checked. Lastly, the steel for a column-footing joint needed to be designed. For this design, the bearing strength of the slab was large enough that steel was not required. Refer to Appendix E: Foundation Slab Design, for full calculations of the slab.

The purpose of the column is to raise the timber towers up to the required freeboard. Originally, the column was going to be constructed using concrete. The size of the column would make the construction of such a large concrete structure expensive and very difficult. The solution was to build a hollow column out of concrete masonry units (CMU) and fill it with soil. This would allow easier construction as the blocks could be stacked instead of having to pour all of the concrete needed to make a solid concrete column. The first step in building the masonry column was to determine the course layouts of the masonry. The course layouts were finalized as essentially two separate columns being connected with steel ties. See Appendix D: Construction Drawings Sheet 7 for drawing of course layout. Next the masonry was designed with the steel reinforcement required to prevent the soil in the filled center from blowing the bottom CMU's out. It also had to be checked to ensure that the weight of the column would resist the force produced by the flood. See Appendix F: Foundation Column Design for the full calculations for the column.

6.1.2 Anchorage Design

The bridge requires an anchor to hold the tensioned main cables in place. The anchor must resist the force of the cable both horizontally and vertically and also the force of buoyancy produced during a flood. The anchor is designed to be a concrete deadman anchor placed half way into the ground. The cables are to be run through tubing that has been wrapped around a steel cage and cast in the concrete. See Appendix D: Construction Drawings Sheet 9. The calculations in the anchor design were based upon soil properties and simple statics. The forces considered in the vertical direction are the tension in the cable, the buoyancy of a flood, and the weight of the anchor. The horizontal forces used in the design include the tension in the cable, the lateral bearing pressure of the soil on the side of the anchor, and the lateral sliding resistance of the soil on the base of the anchor. Refer to Appendix G: Anchor Design for full calculations considered.

6.1.3 Timber Towers

As previously mentioned, cost is a major factor for the proposed bridge design. In an effort to reduce cost, the towers and walkway were designed to be constructed mostly out of hardwood timber. Besides the fact that timber is much cheaper than steel, it was the chosen material because it will be donated by the mayor of Soloy.

Timber has many benefits, one of which is it helps ease the construction for the local community. This is a material that the Soloy people are familiar with since many of the Ngabe structures are made of wooden materials. Also, the location of Soloy makes the ability to obtain this material convenient.

The following is a brief description of the methods used to dimension each part of the timber design. A complete detailed drawing of the timber towers is located within Appendix D: Construction Drawings Sheet 5.

The tower design starts where the previously mentioned top slab connects to the timber base beam. Six anchor bolts are placed through the base beam to a depth of 12 inches into the top slab. These anchor bolts were designed to withstand the overturning moment created by the wind loads acting on the entire timber tower from the east or west direction. The design checks for the anchor bolts' capacities and geometry are located in Appendix H: Timber Tower Design.

The base beam is then connected to the timber columns by 3/8 inch thick gusset plates. The connections used in the base gusset plates were also designed to withstand the overturning moment caused by the wind loads acting on the towers. The design constraints on the connections consisted of capacity and shear checks, both of which can be found in Appendix H: Timber Tower Design. The design requirements called for six (1" dia. UNC) gusset plate bolts but eight will be used for a factor of safety.

To design the timber columns, several different design checks had to be considered. These design checks consisted of compression, bearing, bending, and combined bending and axial compression. The column design checks are located in Appendix H: Timber Tower Design. To meet the previously stated design constraints, both timber columns required a 12”x12” cross section. To make the tower design as well as tower construction as simple as possible, this cross section was used throughout the timber tower design. Also, since the type of wood that will be used in this project has yet to be clarified, White Oak was used to replicate dense hardwood design properties.

The timber towers are then connected to the top timber beam by another set of gusset plates. The top gusset plates are almost identical to the base gusset plates except they also are attached to a frictionless saddle which sits above the top beam. The saddle is made from shaped steel and a bent half pipe which cradles the main cable. The shaped steel is required for the pipe to keep its bent structure at the desired design angle. It is very important that the cable and saddle are heavily greased in order to prevent friction between them. If any friction is created, the towers will start to overturn which would lead to complete bridge failure.

6.1.4 Cable Design

Note: measurements given in tons are metric tons (1 ton = 1000 kg)

All cables are steel. Steel cable was selected because the durability of steel is substantially greater than the alternative non-metal rope. Though steel will last much longer in comparison to rope, the cost difference is also substantial. Fortunately Avery Bang, the Director of Operations for Bridges to Prosperity (BtP, an NGO that funds bridge projects in developing countries), informed Los Agua Ninos that BtP may donate cable if the project is approved.

The main, spanning, and suspender cables were designed using the methods presented in Survey, Design, and Construction of Trail Suspension Bridges for Remote Areas by A. Grob, J. Krahenbuhl, and A. Wagner. Main span cable diameters of 32mm were selected and were found to satisfy design requirements. The cable that BtP typically donates is also this diameter. The following parameters were calculated: main cable sag under dead, full, and hoisting loads, main cable length, spanning cable camber and length, and suspender cable lengths.

The calculations can be found in the attached Appendix I: Cable Design

6.1.5 Walkway Design

The walkway will be constructed with 6”x6” nominal (5.5”x5.5” actual) hanger beams with 2”x4” nominal (1.5”x3.5” actual) planking placed in a staggered pattern parallel to the cables. Hanger beams will be attached at the bottom of each suspender. See Appendix D: Construction Drawings Sheet 4 for suspender connection details and Appendix J: Suspender Design for suspender design checks. Planking will be nailed as well as bolted to the hanger beams as shown. See Appendix K: Walkway Design for design checks.

6.2 Implementation

The implementation plan will follow the construction schedule located in Appendix L: Construction Schedule. This schedule takes into account the curing time for concrete and has also been factored for the unskilled labor. Also, the implementation of this project relies heavily on a project foreman with a great amount of experience in these sorts of situations. Several different forms will have to be created out of limited materials and will have to be placed in difficult positions throughout construction. The timber towers weigh in excess of 2900 pounds, so the knowledge in both pulleys and scaffolding to hoist the structures is extremely important.

The project will start December 12th, early in Panama's dry season when Rio Fonseca's water elevation is low. This will allow for safer crossing when transporting materials to the island side tower.

Initial preparations will have to take place before the actual construction takes place. Due to the moderately heavy vegetation, clearing the site will have to take place before any other arrangements are made. Following preparations include mobilizing the equipment, earthwork required to access the project site, transporting materials to the project site, and a site survey/point layout.

Upon completion of the initial preparations, the actual construction of the Soloy Project can go underway. To avoid any initial construction obstacles, the west-side tower (island side) will be the first to be constructed.

The first step to creating the towers is to excavate an area for the foundation base slab. The excavation will have to be slightly larger than dimensions shown in Appendix D: Construction Drawings Sheet 8. The forms will have to be constructed within the excavation. When the forms are in place, a layer of concrete will be placed thick enough to create the cover needed for the placement of the steel reinforcement grid shown in Appendix D: Construction Drawings Sheet 8. The vertical steel for the masonry reinforcement must also be placed according to specifications shown in Appendix D: Construction Drawings Sheet 6. Mix and place the rest of the required concrete and let the slab cure for one week. After curing, strip and clean the forms for reuse.

The foundation column will be formed upon the base slab. The column will start by using the two full grout masonry layout plans found in Appendix D: Construction Drawings Sheet 7. The CMU's will have to be threaded through the vertical steel already placed within the base slab. When the final full grout course has been laid, grout will be placed into the cores that do not have steel designed through the entire height of the foundation column. Also after the last full grout course has been laid, the partial grout masonry layout plans, found in Appendix D: Construction Drawings Sheet 7, will be utilized. When the final course has been laid, steel and grout will be placed throughout the entire height of the foundation column in the cores that were not grouted in the full grout courses. When the foundation columns are complete, they will be completely filled with clean soil. This soil will be constantly compacted by foot during the filling process.

With the foundation column complete, the foundation top slab can be constructed to the specifications in Appendix D: Construction Drawings Sheet 6 and Sheet 8. This part of the tower construction will require a very clever project foreman to position the forms. Also, several steps will have to be taken before the top slab can be poured. The first step will be stuffing the open cores in the foundation column's CMU's with fiberglass batt insulation to prevent the top slab concrete from falling into them. Also, bolts will have to be placed 3 courses down into the foundation column for the forms to rest upon. When the forms have been sized accordingly, they will be placed upon the previously mentioned bolts and wrapped tightly with a steel cable. 2"x4" lumber beams will be used to reinforce the forms. Wire ties placed through the 2"x4" beams will also help keep the forms in place. When the forms are positioned a layer of concrete will be placed thick enough to create the cover needed for the placement of the steel reinforcement grid shown in Appendix D: Construction Drawings Sheet 8. The vertical steel from the masonry reinforcement must be tied into the slab reinforcement as shown in Appendix D: Construction Drawings Sheet 6. The timber base beam to the timber towers must be cut to size and drilled for anchor bolt placement as shown in Appendix D: Construction Drawings Sheet 5. After the anchor bolts have been placed, correctly position the base beam set up upon the top slab forms. Mix and place the rest of the required concrete and let the slab cure for five days. After curing, strip and clean the forms for reuse.

When the foundation top slab has cured, the timber tower assembly can take place. The complete timber tower assembly can be viewed in Appendix D: Construction Drawings Sheet 5. With the timber base beam already in place, the rest of the column will have to be connected before it can be positioned upon the base beam. This includes the top gusset plates (with saddle) being bolted to the timber columns. The base of the timber columns will have to be pre-drilled (1 1/8 inch diameter holes) for the base gusset plate bolts. A combination of pulleys, rope, scaffolding, and manpower will be required to hoist the tower assembly into position. After the assembly has been positioned, the base gusset plate will be bolted into place while teams using pulleys and ropes directly connected to the tower along with people on the scaffolds hold the entire assembly in the correct position.

To place the anchors, an area will have to be excavated slightly larger than dimensions shown in Appendix D: Construction Drawings Sheet 9. The forms will have to be constructed and placed within the excavation. Before pouring the concrete, construct the steel cage shown in Appendix D: Construction Drawings Sheet 9. Tubing will be wrapped around the steel cage for the main cable to be threaded through connecting it to the anchor. After the steel cage set-up is complete, a layer of concrete will be placed thick enough to create the cover needed for the placement of the steel reinforcement cage shown in Appendix D: Construction Drawings Sheet 9. Mix and place the rest of the required concrete and let the slab cure for one week. After curing, strip and clean the forms for reuse.

Once the foundations, towers, and anchors have been constructed and the concrete has cured, the main cables must be hoisted into their saddles on top of the towers. Steel cables with a diameter of 32 mm (1.25 in) will be used for both the main and spanning cables. Main cables are strung from one anchor, span saddle-to-saddle between the towers, and end at the other anchor. There are two main cables with the layout shown in Appendix D: Construction Drawings Sheet 3. Total length for one main cable is 85.7 m (281.1 ft). The hoisting tension for each main cable is

0.425 metric tons. One end of each main cable should be attached to the anchor on the island. Each main cable should be hoisted onto its respective towers and the ends should be brought to the anchor on the mainland. Each cable must now be tensioned to 0.425 metric tons and attached to the mainland anchor. Cables should be fastened to the mainland anchor last because tensioning requires heavy equipment that would be difficult to transport to the island. A truck can be used to tighten the cable to the proper dead load sag of 4.2 m (13.8 ft) as shown in Appendix D: Construction Drawings Sheet 2, but the tensioning will not be exact and some design values such as the cable angle and length will be changed. If this is the case, an engineer should re-check the anchors to ensure that they are sufficient to resist the full tension load at this new angle.

Once both main cables have been hoisted, tensioned, and anchored, the suspenders must be hung from each main cable. See Appendix D: Construction Drawings Sheet 3 for spacing, length, and connection details of suspenders.

The two main span cables are strung between the foundations as shown in Appendix D: Construction Drawings Sheet 2 and are attached to each suspender. See Appendix D: Construction Drawings Sheet 3 for spanning cable connection details. The length for each main span cable is 40.7 m (133.5 ft).

The walkway will be constructed with 6”x6” nominal (5.5”x5.5” actual) hanger beams with 2”x4” nominal (1.5”x3.5” actual) planking placed in a staggered pattern parallel to the cables as seen in Appendix D: Construction Drawings Sheet 4. Attach hanger beams at the bottom of each suspender. See Appendix D: Construction Drawings Sheet 4 for walkway connection details. Nail planking to the hanger beams as shown.

See Appendix M: Selected Bridges to Prosperity Suspension Manual Pages for further information on hoisting cable.

6.3 Benefits and Cost

Benefits to the community of Soloy make this project very unique. It evolves around the lifestyle and passion of the community. Visiting the location gave Los Agua Ninos an edge in being able to help the community along with supplying them with a design for what they need. As previously mentioned in the introduction, the community has a strong passion for sports. Los Agua Ninos noticed this passion during their visit and realized that their goal would be to help these people by creating a safer means of travel to an athletic field. Also the possibilities of having families with young children watch the matches would bring the community together and share this experience and passion. Although the proposed bridge creates many benefits, it has be constructed on a very low budget. The following table (Table 5: Overall Cost Estimate) is a cost estimate for the construction of the proposed bridge. This estimate was done to illustrate the maximum cost along with the actual cost for bridge construction. The table has two columns, one of which represents the total cost without donations and while the other includes the donations. The actual cost of the proposed bridge came out to be a cost of $23,500. This estimate includes what the community would have to pay regardless of the local labor and other donations provided to the project. It is understood that this amount of money is nowhere near affordable for the community, however efforts are being made to find donations from various sponsors located in Panama and the United States. The detailed cost estimate is attached in Appendix N: Cost Estimate and includes details to how the estimate was formulated.

Table 5: Overall Cost Estimate

Overall Estimate		
Item	Total Cost	Actual Cost (Donations Subtracted)
Materials	$32,200	$16,800
Labor	$19,900	$5,800
Equipment	$2,000	$900
Total	**$54,100**	**$23,500**

6.4 Future Recommendation

Los Agua Ninos has seen that the need for a footbridge would greatly help this community. The team will put all its effort into not only submitting a design to the community, but helping to get funding for them as well. The team believes that the design submitted will benefit the whole community not only the fútbol players and want to reach beyond the scope and assignment of the class. The team will make a great effort to lower the cost to a reasonable amount that the community can handle. This amount would have to be below $1,000 because as mentioned in the introduction the community is comprised of subsidence farmers with a small government stipend.
The team has compiled a construction schedule that entails details from start to finish of construction. This schedule is attached in Appendix L: Construction Schedule.

The design implementation poses the biggest question to be answered. The team would recommend that the project be passed on to an experienced organization to help manage project. A few of these potential organizations include Bridges to Prosperity and Engineers Without Borders. Also the teams will keep its communication with the Peace Corps volunteers that are currently located on-site. The team believes this is the best option because the volunteers could manage the project construction to see the bridge to its completion.

6.5 Conclusion

Los Agua Ninos acknowledges the importance of this project within Soloy as well its importance for the parties affected by crossing the dangerous river to reach the island. Los Agua Ninos has considered many factors in its selection and design of a suspension bridge that meets the needs of the Soloy soccer teams. Some such factors include constructability, social, economic, and environmental considerations.

To summarize the benefits to the community we look first to the social aspect. The bridge will increase unity of the community with the construction of the bridge. The bridge would provide ease of access to the recreational area, increasing general happiness throughout the community. As for the cultural aspect recreation and sport is a big part of the community and culture. Enhancing recreational availability allows for continuation of this cultural aspect in a safer manner.

The team discovered a political impact while visiting Soloy. The local government doesn't want a bridge rebuilt to the island due to property damage of the flood. The proposed bridge will be further from the school and thus inhabitation of the island will be much less likely.

Environmentally, the construction of the bridge will be mostly by hand and thus environmental impacts of construction will be minimal. The construction of the bridge may require the removal of one or more large trees and thus create a greater potential of erosion at and around the project site. Erosion measures will have to be considered during construction, rip-rap, rock vanes, or other measures may need to be employed.

Economically, the cost of a project of this magnitude would be very expensive for the community. However, the steel cables will be donated, the wood is planned to be donated, and the labor will be volunteered. Also the team is looking for possible donations that would help lower the overall cost of the bridge.

The consideration of constructability for this project is important. Since the bridge is large in magnitude, it will be difficult for an unskilled community to build. It would require some management experienced in construction. However the team has designed with this in mind and made the construction materials and process familiar to the local community. All construction can be done without large equipment. The hoisting and tensioning can be done creatively.

Sustainability was also taken into account in the design of the bridge. The approach and walkway would be constructed with wooden planks and could easily be replaced if necessary. With proper education, the soccer teams could maintain the bridge. The team's design allows for the construction to be fairly easy and use little to no heavy equipment, which would increase the communities ability to construct.

Based on this study and design, Los Agua Ninos concludes these design recommendations to be an improvement to its benefactors and a much safer alternative to wading through the river. However this is a proposal completed by students not by licensed engineers. Design work was completed to the best of the students' abilities and should be checked by a licensed professional engineer (P.E). before being implemented.

7.0 Acknowledgements:

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Dr. Theresa Ahlborn

Dr. Stanley J. Vitton

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Appendix A

Point Descriptions

100	Hole "middle os site" top od ditch left side facing the river.
101	BS center of ditch closest to river 310 degrees (NW)
102	mouth or ditch entering the river (water edge) northside of river
103	Top of mound with the tree trunk behind (water edge) northside of river
104	Water edge northside of river East of mouth entrance
105	Water edge northside of river East of mouth entrance
106	Water edge northside of river East of mouth entrance
107	Water edge northside of river East of mouth entrance
108	Water edge northside of river East of mouth entrance
109	Water edge northside of river East of mouth entrance
110	Top of crest north of river top edge of drop off before the river
111	Top of crest north of river top edge of drop off before the river
112	Top of crest north of river top edge of drop off before the river
113	Top of crest north of river top edge of drop off before the river
114	Top of crest north of river top edge of drop off before the river
115	Top of crest north of river top edge of drop off before the river Bottom of Tree
116	Edge of Ditch west side of ditch
117	Center of Ditch leading to mouth which enters the river
118	Edge of Ditch east side
119	Edge of Ditch east side
120	Edge of Ditch east side
121	Edge of Ditch east side
122	Corner of shed closest to the river (SE corner of wooden shack)
123	Tree north of wooden shack
124	SW of tree toward the ditch (East of Ditch)
125	SW of tree toward the ditch (East of Ditch)
126	SW of tree toward the ditch (East of Ditch)
127	SW of tree toward the ditch (East of Ditch)
128	East Edge of ditch
129	Center of Ditch
130	West Edge of Ditch
131	Center of Ditch
132	West Edge of Ditch
133	Center of Ditch
134	East Edge of ditch
135	NE of Ditch
136	NE of Ditch
137	NE of Ditch
138	NE of Ditch Top of small slope
139	NE of Ditch Bottom of small slope
140	NE of Ditch
141	NE of Ditch peak of small slope
142	NE of Ditch, high top edge of ditch on east side front of tree
143	NE of Ditch (Top Edge)
144	NE of Ditch (Bottom Edge)
145	Center of ditch
146	SW end of ditch (Bottom)
147	SW end of ditch (Bottom)
148	Center of Ditch (Bottom)
149	NE of Ditch (Bottom Edge)

150	NE of Ditch (Top Edge)
151	NE of Ditch
152	NE of Ditch
153	NE of Ditch
154	SW Bottom Edge of Ditch mouth of river
155	Center of ditch edge northside of river
156	Top edge of ditch on northside of river
157	SW corner of ditch top part in mouth of ditch (westside)
158	SW Top edge of Ditch
159	SW of Ditch
160	SW of Ditch
161	SW Top edge of Ditch
162	SW od Ditch top elevation
163	SW Top edge of Ditch
164	SW of Ditch
165	SW Top edge of Ditch
166	SW of Ditch
167	N or river
168	N 1/4 in river
169	center of river
170	S 3/4 in river
171	S of river
172	S bank
173	S bank 1/2 way up bank slope
174	S bank slope top
175	S bank slope top
176	S bank path
177	S bank path
178	S bank slope top
179	S bank 1/2 way up bank slope
180	S bank water level
181	S bank water edge
182	S water edge in water
183	S 3/4 in river
184	Centerline of river
185	N 1/4 in river
186	N of river in water
187	N bank upstream
188	N in water
189	SE centerline of water
190	SE 3/4 in river
191	SE side of the short
192	SE Bank of water
193	S corner of House
194	E Corner of House
195	W Corner of House
196	Bottom of windowsill (House on Site) - Max flood ht (100-yr flood)
197	SW of ditch (W Corner of Tree-to)
198	SW of ditch (W Corner of Tree-to)
199	SW of ditch (W Corner of Tree-to)
200	Slope of Ditch (SW)
201	Bottom of Ditch

202	Slope of Ditch (NE)
203	Slope of Ditch (NE)
204	NE of Ditch
205	NE of Ditch
206	NE of Ditch
207	SW of Ditch
208	Top of hill SW
209	SW of Ditch
210	SW of Ditch
211	Slope of Ditch (SW)
212	Bottom of ditch
213	North East of Ditch slope
214	North East of Ditch edge
215	North East of Ditch
216	Tree 7
217	Tree 6
218	Tree 5
219	Tree 1
220	Tree 2 and 3
221	Tree 4
222	Test pit location
223	Traverse
224	South side of house
225	Large Tree on Soccer side shore
226	Water Edge
227	Top row opp. Trees 2/3
228	Top row to South
229	Top row to South
230	Top row to South
231	Top row to South
232	Top row to South
233	Top row to South
234	Top row to South
235	Top row 2
236	Top row to South
237	Top row to north
238	Top row to north
239	Top
240	Top
241	Top
242	Top
243	Top
244	Top (behind Tree in 225)
245	Top (behind Tree in 225)
246	Top (behind Tree in 225)
247	Top (behind Tree in 225)
248	Top (behind Tree in 225)
249	Top (behind Tree in 225)
250	Top (behind Tree in 225)
251	Top (behind Tree in 225)
252	Top (behind Tree in 225)
253	Shore, South side

Appendix B

ASTM Standard D-2488 - Visual Classification of Soils

Visual Identification Process for Soil

1.) Identify color, odor and texture
2.) Identify the major constituent, using Table 1 in Experiment 7 handout, as coarse gravel, fine gravel, coarse sand, medium sand, fine sand or fines.
3.) Estimate the percentages of all other soil constituents using Table 1 and the following terms: Trace (0-10%), Little (10-20%), Some (20-30%), And (30-50%).
4.) If the major soil constituent is sand or gravel: Identify the particle distribution (describe as well graded or poorly graded) and particle distribution using Figure 1 and Table 2 in Experiment 7 Handout.
5.) If the major soil constituent are fines, perform the following tests:
    - Dry strength test: mold a sample into 1/8” size ball and let it dry. Test the strength of the dry sample by crushing it between the fingers. Describe the strength as none, low, medium, high or very high depending on the results of the test as shown in Table 3a in Experiment 7 handout.
    - Dilatancy Test: make a sample of soft putty consistency in your palm. Then observe the reaction during shaking, squeezing and vigorous tapping. The reaction is rapid, slow or none according to the tst results given in Table 3b in Experiment 7 handout.
    - Plasticity Test: Roll the samples into a thread about 1/8” in diameter. Fold the thread and reroll it repeatedly until the thread crumbles at a diameter of 1/8”. Note the following:
        a.) The pressure required to roll the thread when it is near crumbling.
        b.) Whether it can support its own weight.
        c.) Whether it can be molded back into a coherent mass.
        d.) Whether it is tough during kneading.

      Describe the plasticity and toughness according to the criteria in Tables 3c and 3d in the Experiment 7 handout.
6.) Identify moisture condition using Table 5 in the Experiment 7 handout.
7.) Record visual classification of the soil in the following order: color, major constituent, minor constituents, particle distribution, particle shape (if major constituent is coarse-grained), plasticity (if major constituent is fine-grained), moisture content, and soil symbol (if major constituent is fine-grained).

Appendix C

Soil Classifications

SOIL A-1

Classified By : EJA Date: 8/18/09

Color	Odor	Texture	Major Soil Constituent	Minor Soil Constituents	Moisture Condition	Shape and Gradation	Type
Med. Brown	Earthy, Organic, Musty	Coarse	Sand	Fines (silts)	Moist	Rounded Poorly Graded (GP)	90% Sand 10% Fines

Classification: Medium Brown, medium Sand, trace clay, poorly graded, rounded, moist (GP)

Notes: Picture #413

SOIL A-2

Classified By : EJA Date: 8/18/09

Color	Odor	Texture	Major Soil Constituent	Minor Soil Constituents	Moisture Condition	Dry Strength	Low
						Dilatancy	Slow
Light Brown	none	Fine	Fines 70%	Fine Sand 30%	Moist	Plasticity	Low
						Toughness	Low
						Soil Symbol	ML

Classification: Light brown, silt, some fine sand, low plasticity, moist (ML)

Notes: Picture #414

SOIL B-1

Classified By : EJA Date: 8/18/09

Color	Odor	Texture	Major Soil Constituent	Minor Soil Constituents	Moisture Condition	Dry Strength	Med
						Dilatancy	None
Reddish Brown	none	Fine	Fine Clay 85%	Fine Sand 10% Fine Agg 5%	Moist	Plasticity	High
						Toughness	Med
						Soil Symbol	CL

Classification: Red brown silty clay, trace fine sand, trace fine gravel, plastic, moist (CL)

Notes: Picture #415

SOIL B-2

Classified By : EJA Date: 8/18/09

Color	Odor	Texture	Major Soil Constituent	Minor Soil Constituents	Moisture Condition	Shape and Gradation	Type
Med. Brown	Earthy, Organic, Musty	Coarse	Medium Sand	Coarse Gravel	Moist	Sub-Rounded Poorly Graded (GP)	80% Med Sand 15% Fine Agg 5% Coare Agg

Classification: Medium Brown, medium Sand, some gravel, trace coarse gravel, poorly graded, sub-rounded, moist (GP)

Notes: Picture #416

SOIL C-1

Classified By : EJA Date: 8/18/09

Color	Odor	Texture	Major Soil Constituent	Minor Soil Constituents	Moisture Condition	Dry Strength	Low
						Dilatancy	Slow
Light Brown	none	Fine	Fines	Fine Sand	Moist	Plasticity	Low
						Toughness	Low
						Soil Symbol	ML

Classification: Light brown, silt, some fine sand, low plasticity, moist (ML)

Notes: Picture #418

SOIL D-1

Classified By : EJA Date: 8/18/09

Color	Odor	Texture	Major Soil Constituent	Minor Soil Constituents	Moisture Condition	Dry Strength	Low
						Dilatancy	Slow
Light Brown	none	Fine	Fines	Fine Sand	Moist	Plasticity	Low
						Toughness	Low
						Soil Symbol	ML

Classification: Light brown, silt, some fine sand, low plasticity, moist (ML)

Notes: Picture #420

SOIL Riverbed

Classified By : HBW Date: 8/18/09

Color	Odor	Texture	Major Soil Constituent	Minor Soil Constituents	Moisture Condition	Shape and Gradation	Type
Grey/Blue	Fishy	Smooth	Coarse Agg	Medium Sand	Wet	Sub-Rounded Well Graded (GW)	60%CoarseAgg 30% Cobbles 10% Med Sand

Classification: Blue-Grey coarse gravel, some cobble, trace medium sand, well graded, wet

Notes: Sample is from middle of river bed, Picture #419

SOIL E-1 and Soil F-1

Classified By : HBW Date: 8/18/09

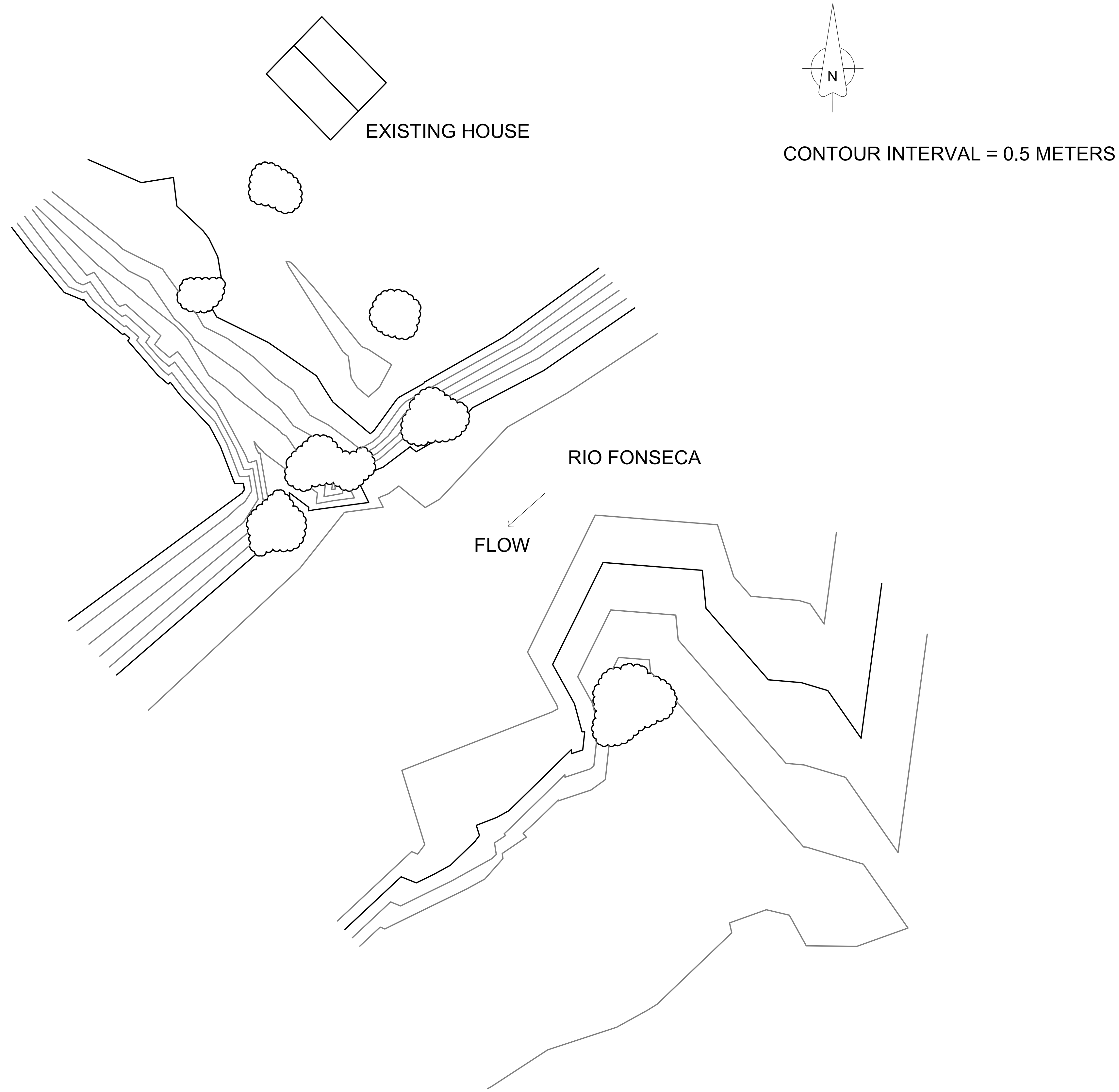
Color	Odor	Texture	Major Soil Constituent	Minor Soil Constituents	Moisture Condition	Shape and Gradation	Type
Grey/Brown	Organic	Coarse	Medium Sand	Fine Sand	Wet	Sub-Angular Poorly Graded (GP)	80% Med Sand 20% Fine Sand

Classification: Blue-Grey coarse gravel, some cobble, trace medium sand, well graded, wet

Notes: Sample is from middle of river bed, Picture #419

Appendix D

Construction Drawings



INDEX OF SHEETS

LOS AGUA NINOS

ERYK ANDERSON
NICK CHILDS
JONATHAN DOBREI
BEN WARING

INTERNATIONAL SENIOR DESIGN

PROJECT

THE SOLOY BRIDGE PROJECT

SOLOY, PANAMA

SHEET TITLE

SITE TOPOGRAPHY

SCALE 1:10

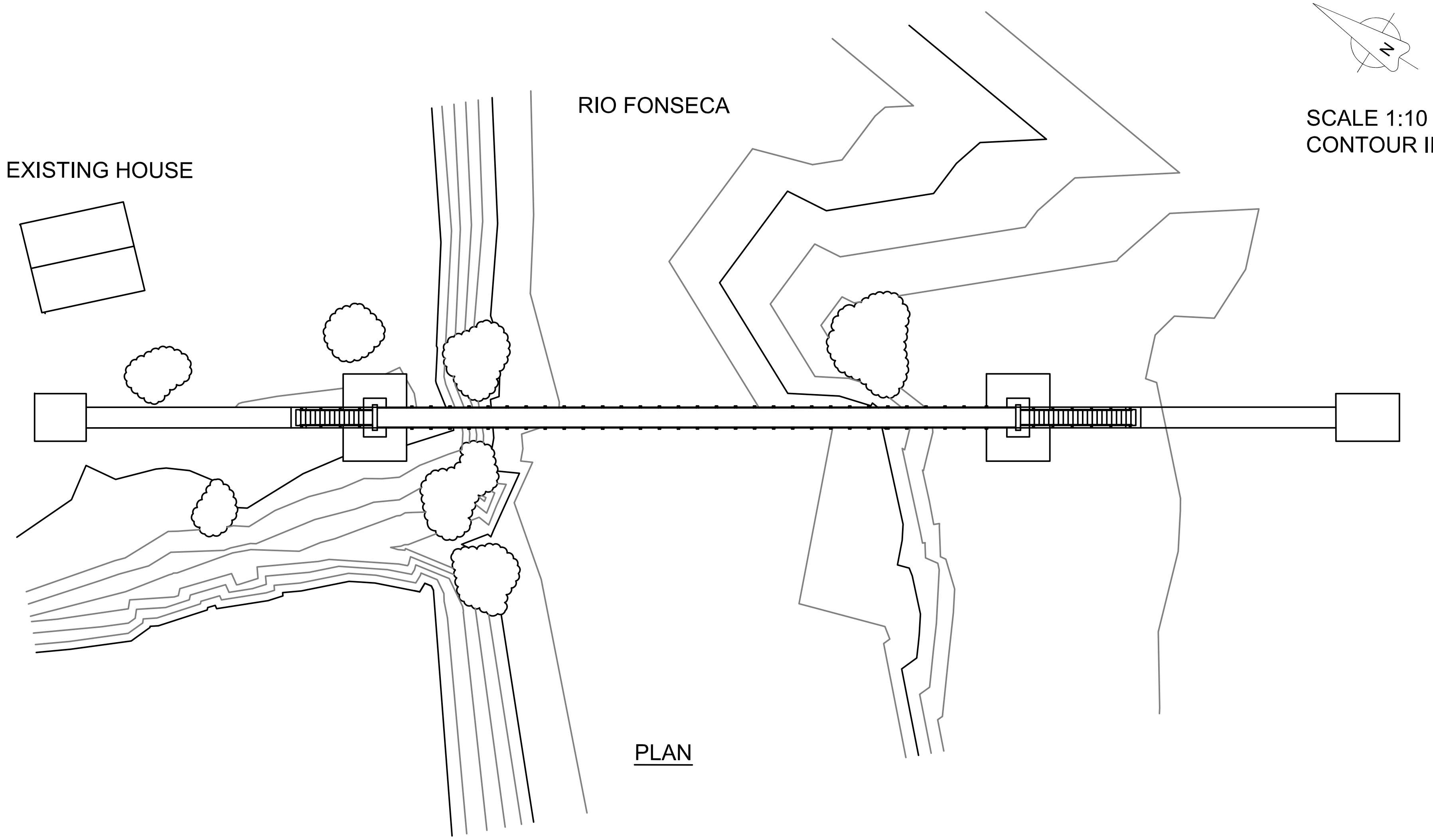
SHEET

1 OF 9

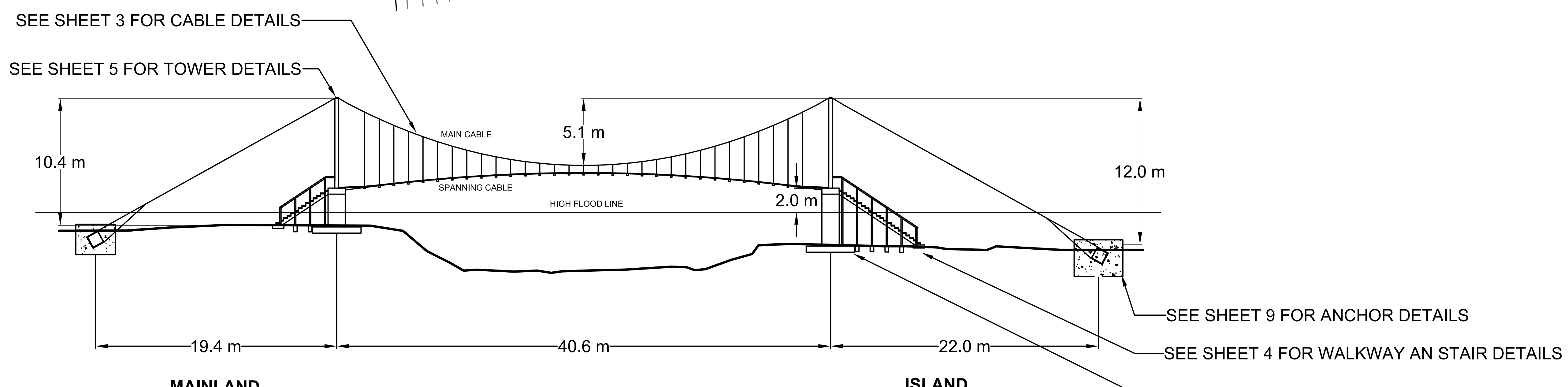
DATE

12/11/2009

SCALE 1:10
CONTOUR INTERVAL = 0.5 METERS



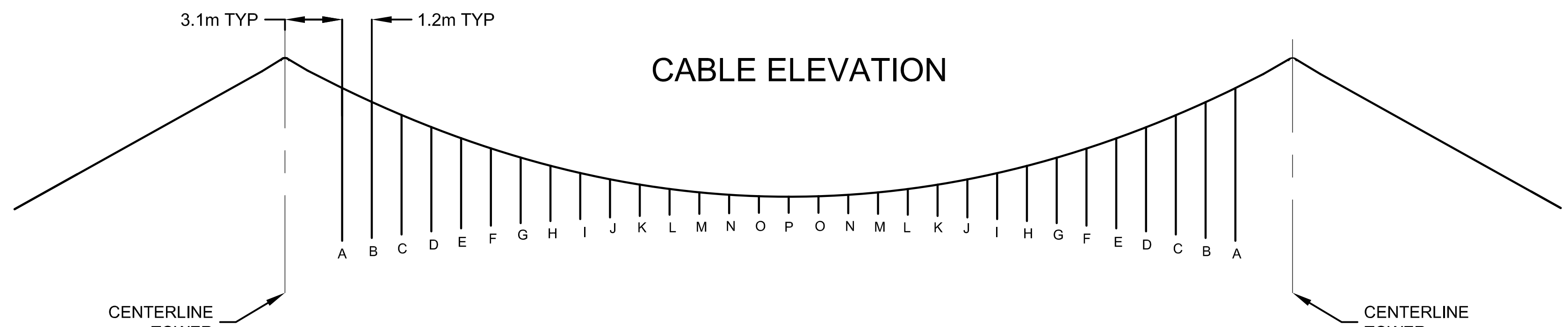
PLAN



CROSS SECTION

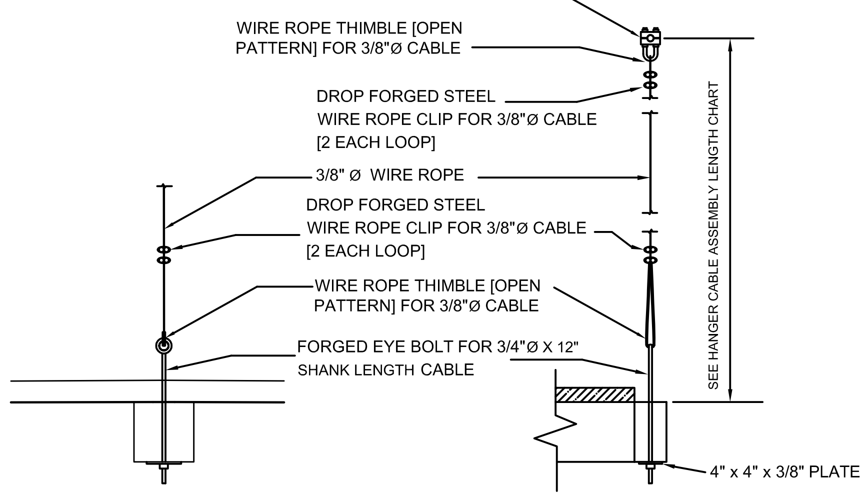
LOS AGUA NINOS ERYK ANDERSON NICK CHILDS JONATHAN DOBREI BEN WARING INTERNATIONAL SENIOR DESIGN	PROJECT THE SOLOY BRIDGE PROJECT FOOT BRIDGE TO RECREATION AREA SOLOY, PANAMA	SHEET TITLE BRIDGE LAYOUT SCALE AS INDICATED	SHEET 2 OF 9 DATE 12/11/2009

CABLE ELEVATION



HANGER CABLE LENGTH CHART	
ASSEMBLY LOCATION	LENGTH
A	5.69m
B	5.05m
C	4.45m
D	3.90m
E	3.40m
F	2.94m
G	2.52m
H	2.15m
I	1.82m
J	1.54m
K	1.30m
L	1.11m
M	0.95m
N	0.85m
O	0.78m
P	0.76m

SUSPENDER CABLE DETAIL

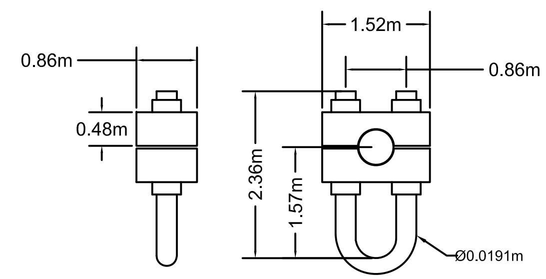


SIDE ELEVATION

FRONT ELEVATION

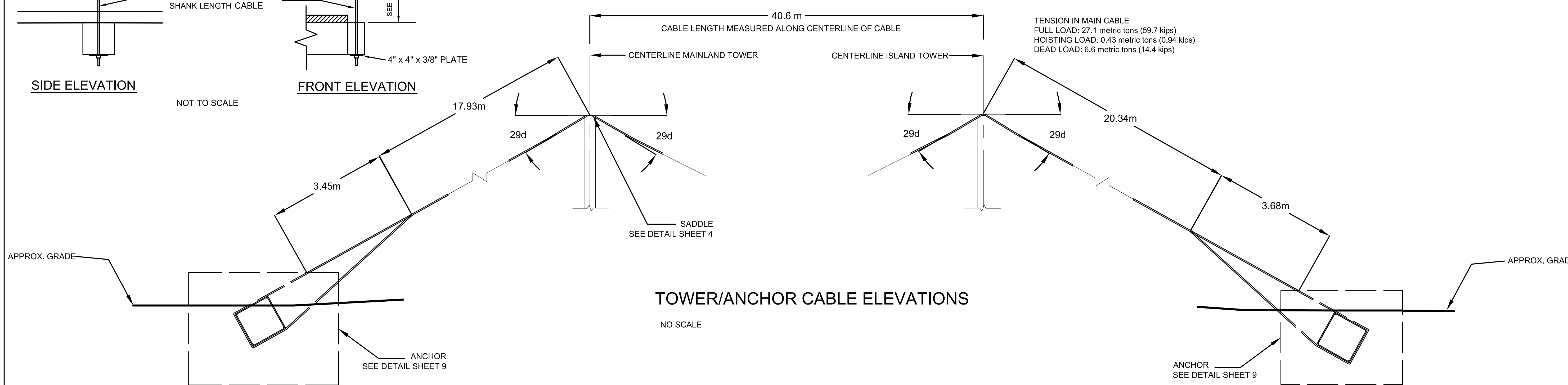
NOT TO SCALE

SUSPENDER CLAMP DETAIL



*NOT TO SCALE

FABRICATE FROM DROP FORGED STEEL TO DIMENSIONS INDICATED



TENSION IN MAIN CABLE
FULL LOAD: 27.1 metric tons (59.7 kips)
HOISTING LOAD: 0.43 metric tons (0.94 kips)
DEAD LOAD: 6.6 metric tons (14.4 kips)

TOWER/ANCHOR CABLE ELEVATIONS

NO SCALE

LOS AGUA NINOS

ERYK ANDERSON
NICK CHILDS
JONATHAN DOBREI
BEN WARING

INTERNATIONAL SENIOR DESIGN

PROJECT: THE SOLOY BRIDGE PROJECT
FOOT BRIDGE TO RECREATION AREA

SOLOY, PANAMA

SHEET TITLE: CABLE DETAILS

SCALE: N/A

SHEET: 3 OF 9

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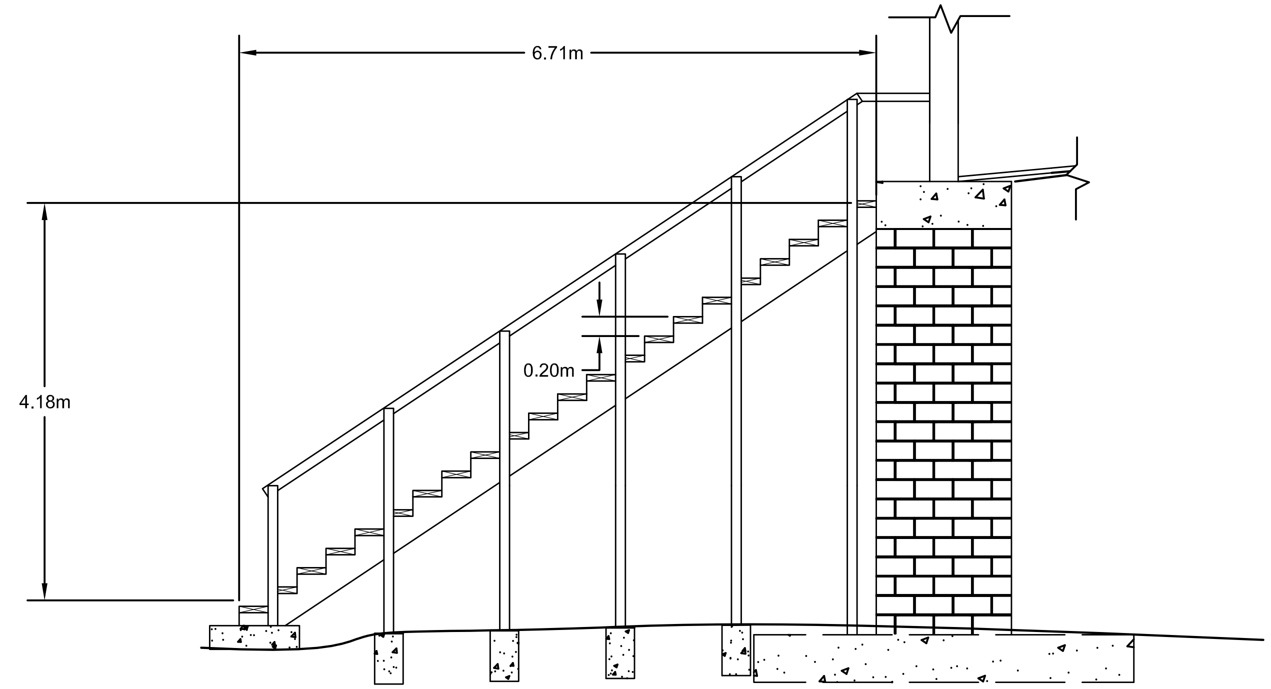
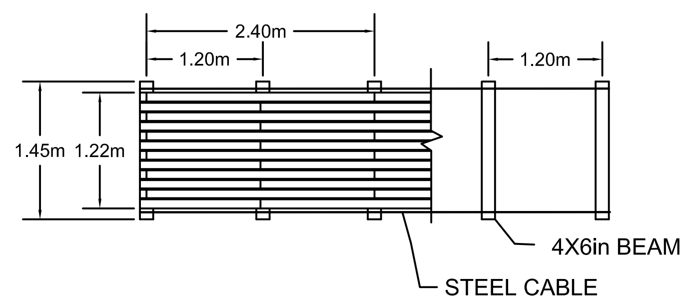
WALKWAY LOADS
LIVE LOAD: 480 kg/m² (98.3 PSF)
DEAD LOAD + LIVE LOAD: 607 kg/m² (124.3 PSF)



USE SAME CONCRETE PAD POSTS AND RAILING SIZE AS MAINLAND STAIRS

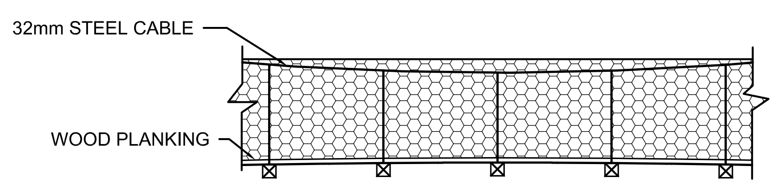
ELEVATION ISLAND ABUTMENT STAIRS

PARTIAL DECK PLAN



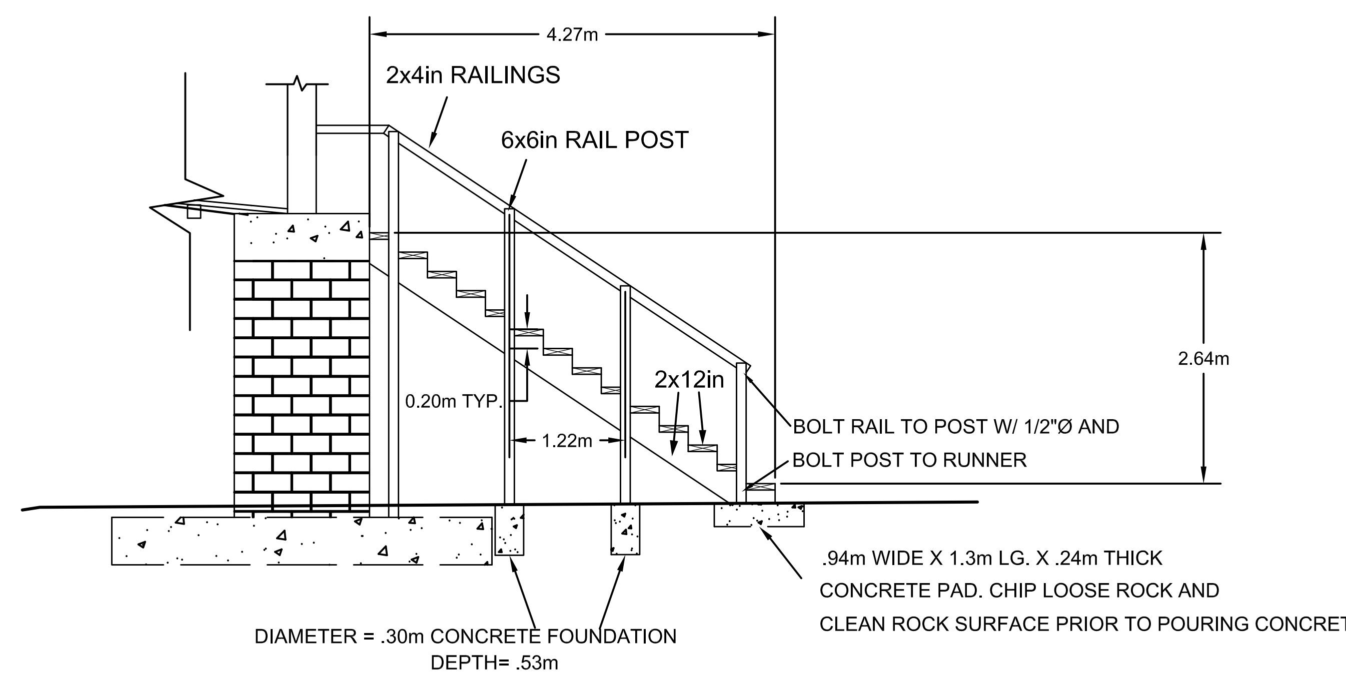
2X4 INCH NOMINAL 1.5" x 3.5" ACTUAL WOODEN PLANKING
1/2 in GAP BETWEEN LONGITUDINAL DECKING

TOP VIEW



3 1/2 ft HIGH FENCING

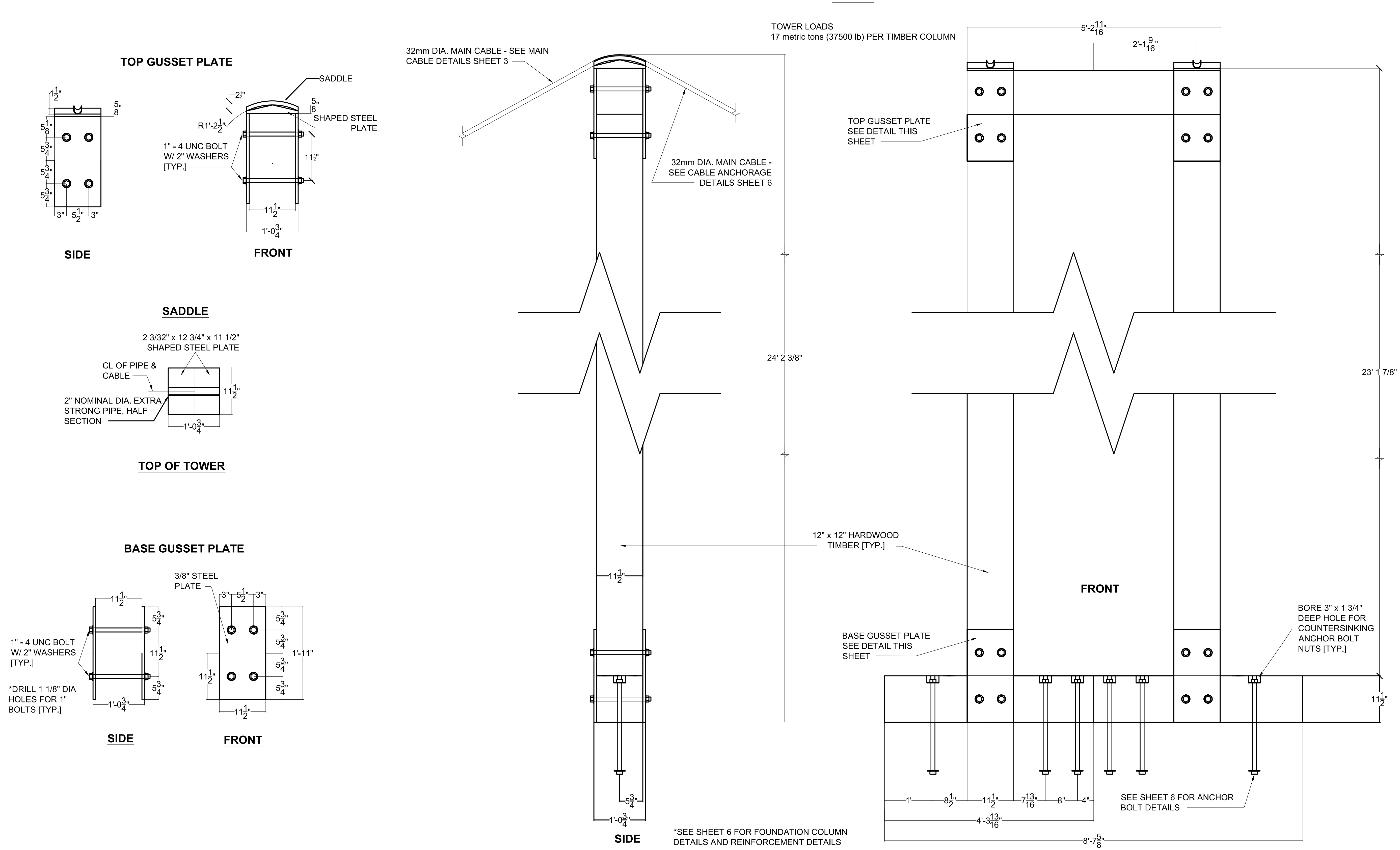
SIDE VIEW



ELEVATION MAINLAND ABUTMENT STAIRS

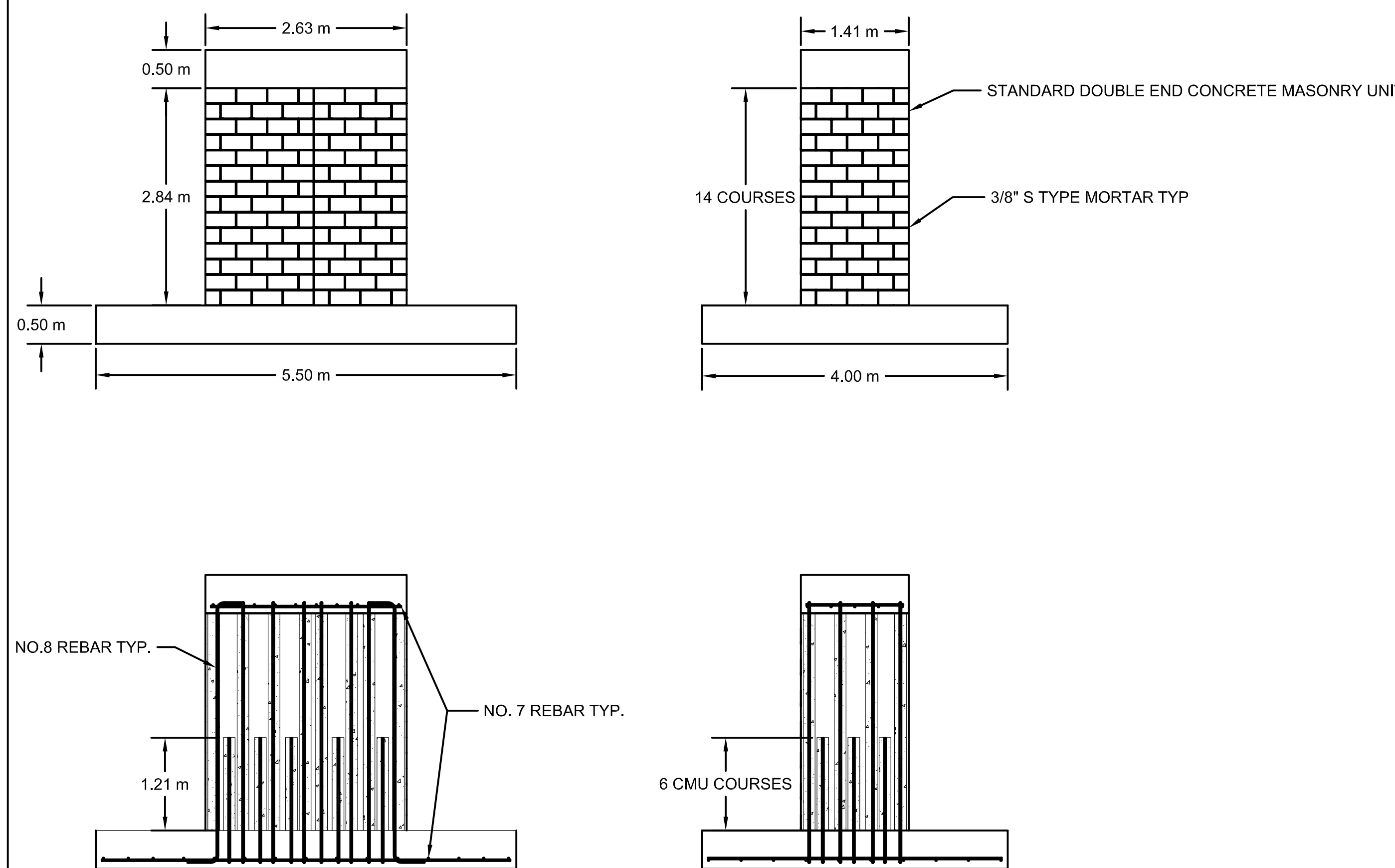
LOS AGUA NINOS	PROJECT	SHEET TITLE	SHEET
ERYK ANDERSON NICK CHILDS JONATHAN DOBREI BEN WARING	THE SOLOY BRIDGE PROJECT FOOT BRIDGE TO RECREATION AREA	STAIR/WALKWAY DETAILS	4 OF 9
INTERNATIONAL SENIOR DESIGN	SOLOY, PANAMA	SCALE: N/A	DATE 12/11/2009

TOWER



LOS AGUA NINOS	PROJECT	SHEET TITLE	SHEET
ERYK ANDERSON NICK CHILDS JONATHAN DOBREI BEN WARING	THE SOLOY BRIDGE PROJECT FOOT BRIDGE TO RECREATION AREA	TOWER DETAILS	5 OF 9
INTERNATIONAL SENIOR DESIGN	SOLOY, PANAMA	SCALE: N/A	DATE 12/11/2009

MAINLAND FOUNDATION

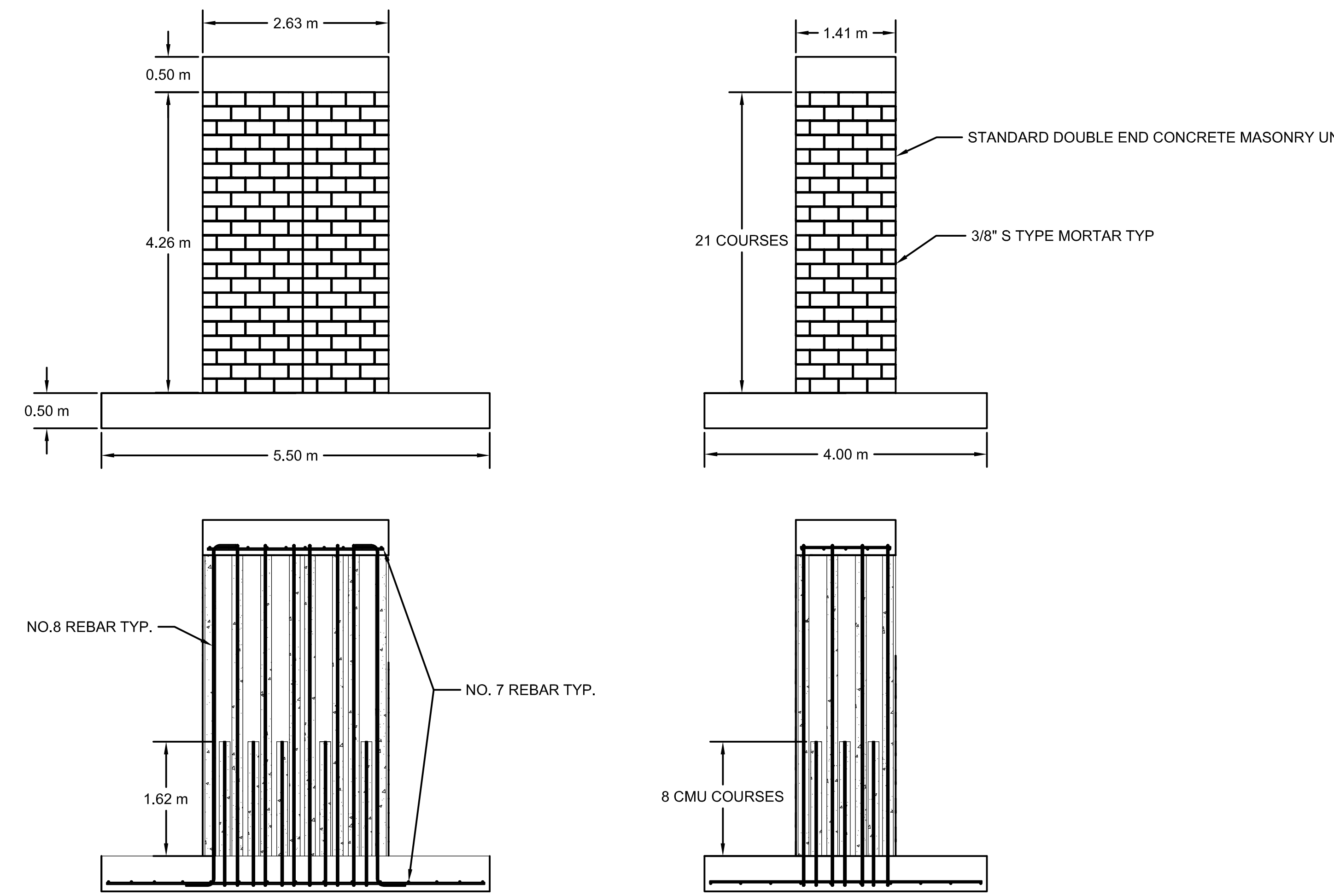


FRONT

SIDE

ISLAND FOUNDATION

FOUNDATION LOADS:
34 metric tons (75000 lb) PER FOUNDATION

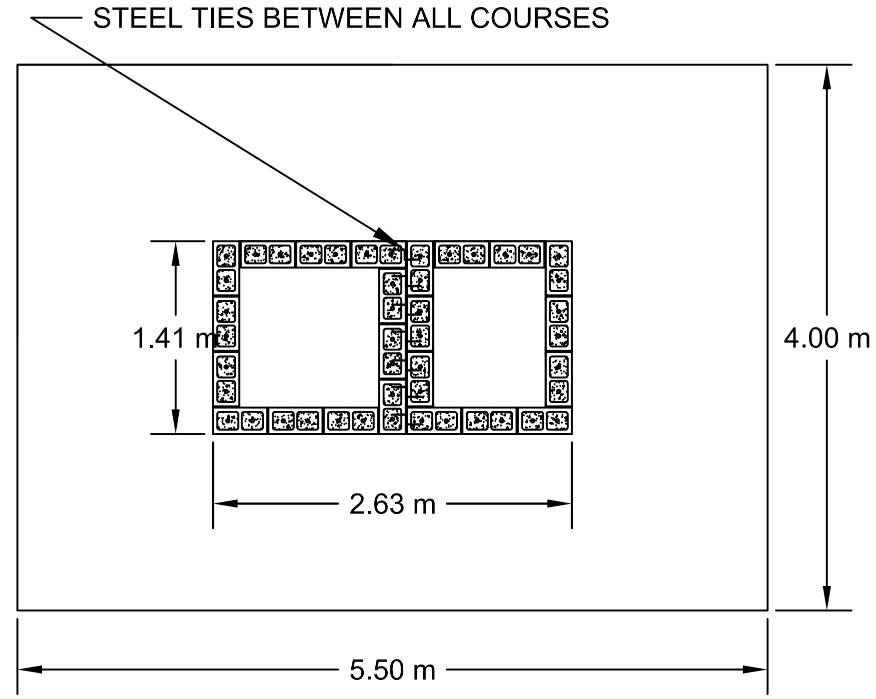


FRONT

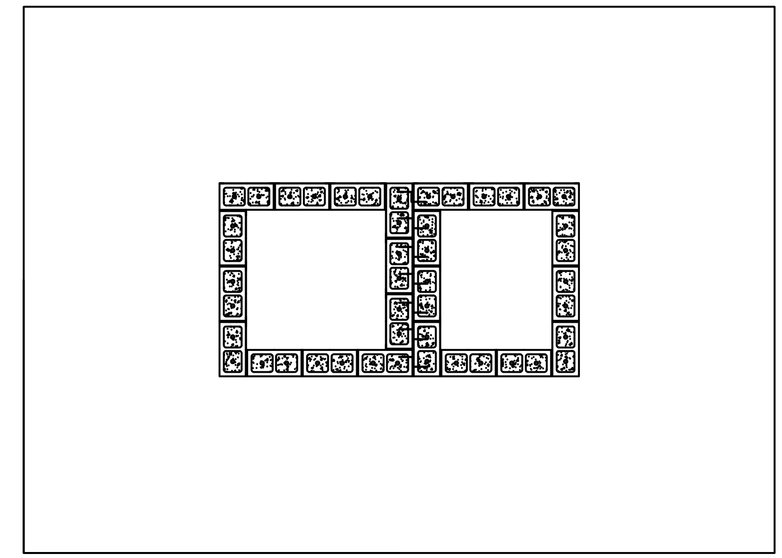
SIDE

LOS AGUA NINOS	PROJECT	SHEET TITLE	SHEET
ERYK ANDERSON NICK CHILDS JONATHAN DOBREI BEN WARING	THE SOLOY BRIDGE PROJECT FOOT BRIDGE TO RECREATION AREA	FOUNDATION DETAILS	6 OF 9
INTERNATIONAL SENIOR DESIGN	SOLOY, PANAMA	SCALE N/A	DATE 12/11/2009

DIMENSIONS TYP. SEE SHEET 8 FOR SLAB DETAILS.

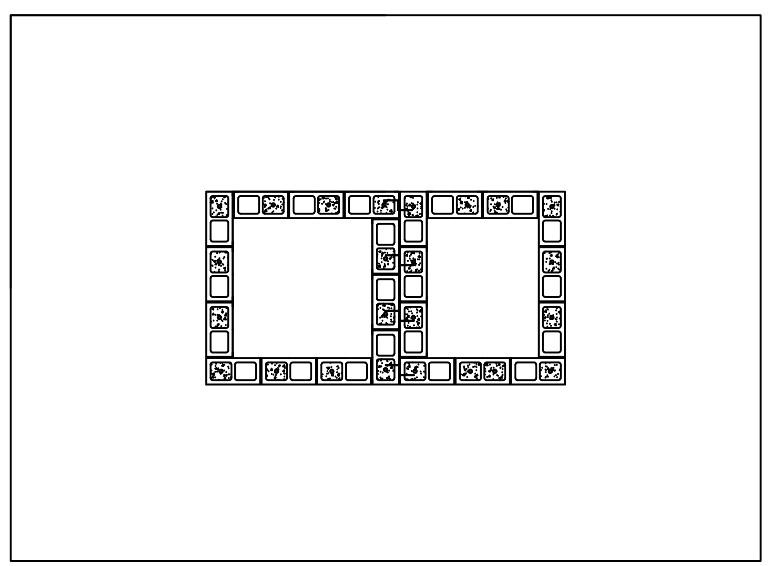


COURSE 1A

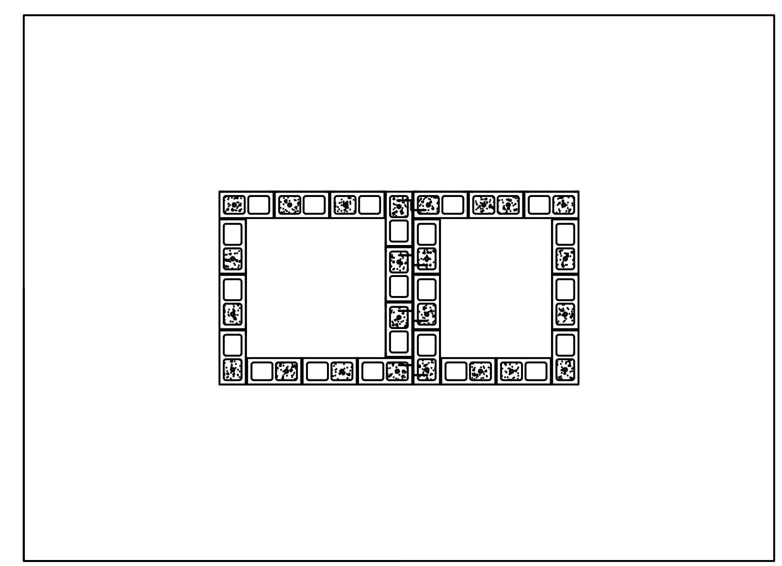


COURSE 2A

ALTERNATE COURSES 1A & 2A. REPEAT FOR 6 COURSES (ISLAND FOUNDATION) AND 8 COURSES (MAINLAND FOUNDATION).



COURSE 1B



COURSE 2B

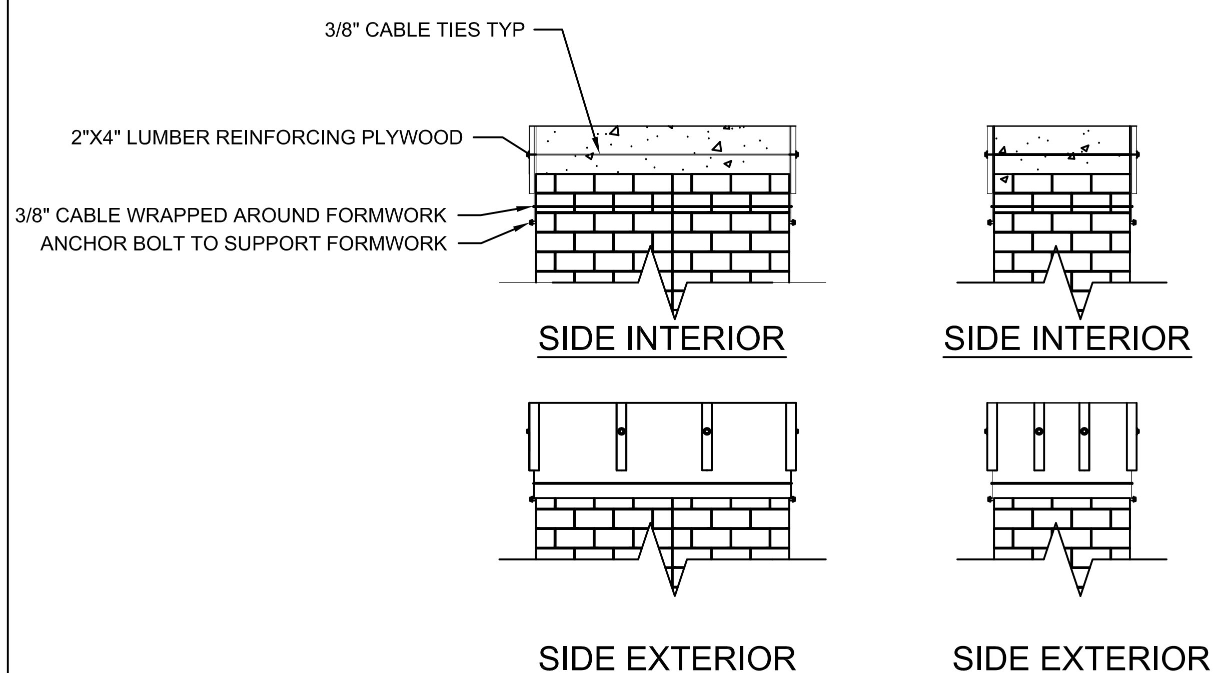
LAY ON TOP OF COMPLETED 'A' COURSES. ALTERNATE COURSES 1B & 2B. REPEAT FOR SPECIFIED NUMBER OF COURSES.

LOS AGUA NINOS
ERYK ANDERSON
NICK CHILDS
JONATHAN DOBREI
BEN WARING
INTERNATIONAL SENIOR DESIGN

PROJECT	SHEET TITLE	SHEET
THE SOLOY BRIDGE PROJECT FOOT BRIDGE TO RECREATION AREA SOLOY, PANAMA	CMU COURSES SCALE N/A	7 OF 9 DATE 12/11/2009

FORMWORK FOR TOP SLAB

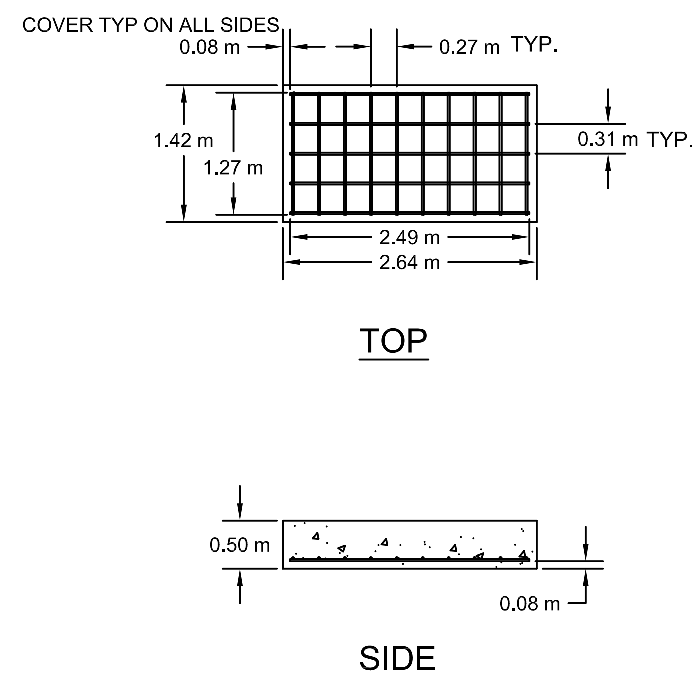
SEE SHEET 6 FOR FOUNDATION DIMENSIONS



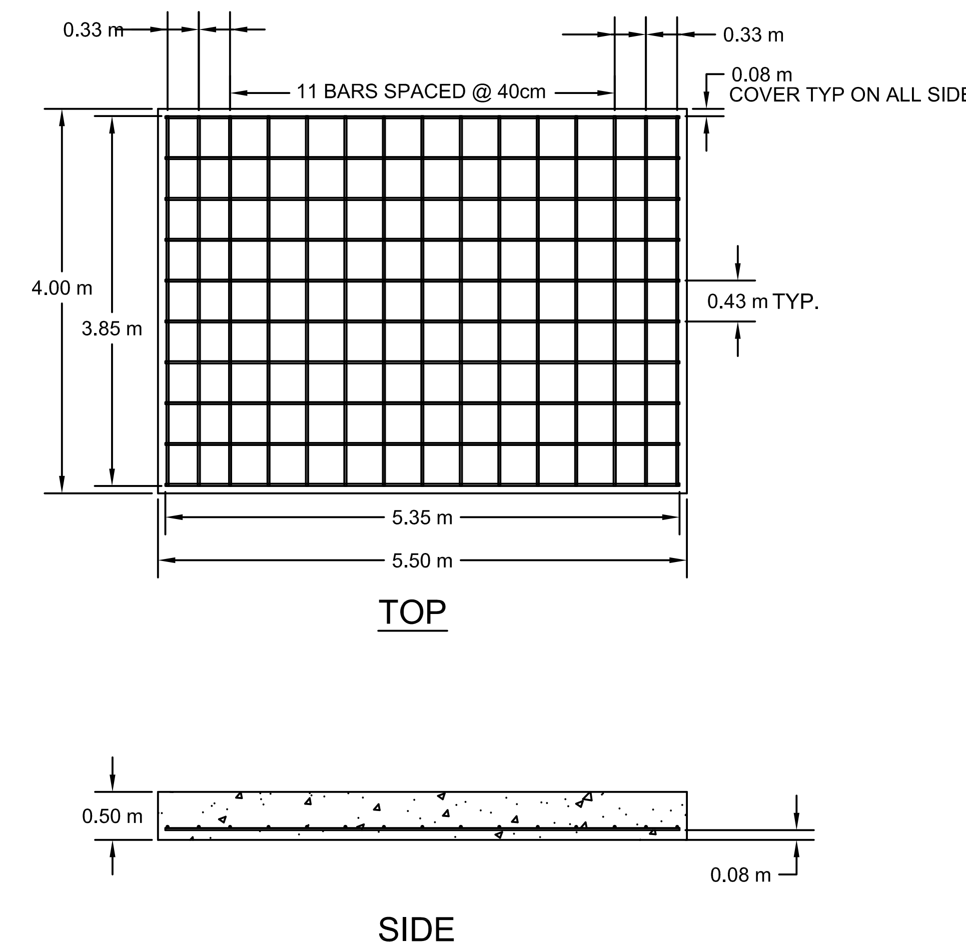
ALL REBAR NO.7

SLAB LOADS
75.4 metric tons (166.2 KIPS) PER SLAB

TOP SLAB



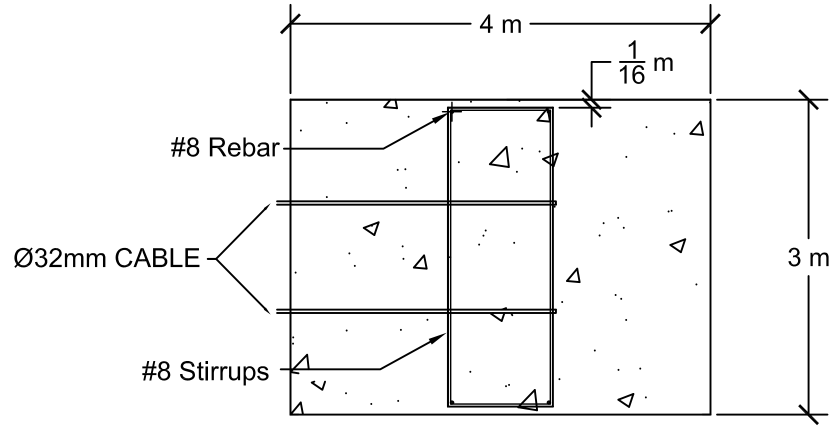
BOTTOM SLAB



LOS AGUA NINOS	PROJECT	SHEET TITLE	SHEET
ERYK ANDERSON NICK CHILDS JONATHAN DOBREI BEN WARING	THE SOLOY BRIDGE PROJECT FOOT BRIDGE TO RECREATION AREA	FOUNDATION SLAB DETAILS	8 OF 9
INTERNATIONAL SENIOR DESIGN	SOLOY, PANAMA	SCALE N/A	DATE 12/11/2009

ISLAND DEADMAN ANCHOR

TOP VIEW

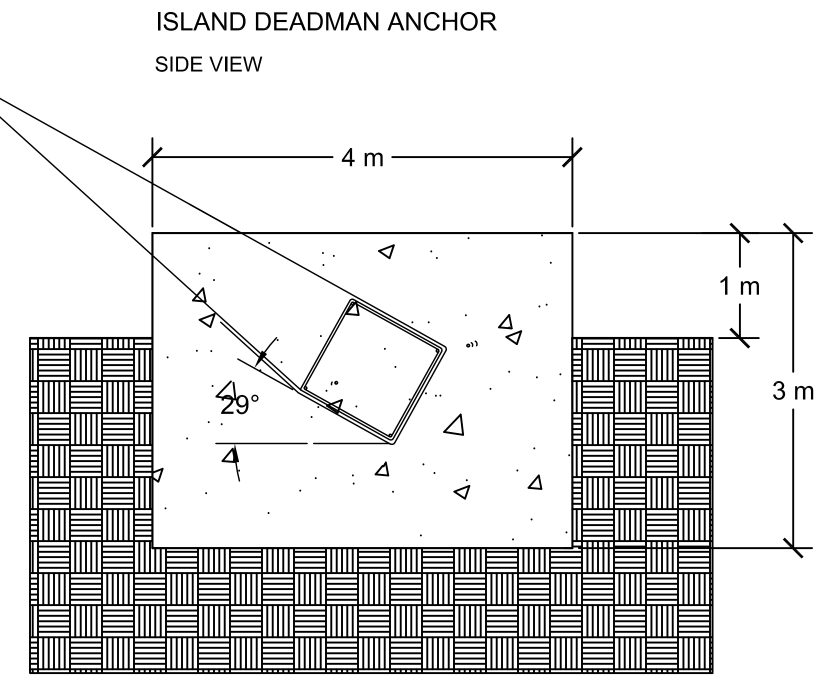


NOTE: CLEAR COVER IS 80 MM

CONCETE ANCHOR WITH # 8 REBAR CAGE

ISLAND DEADMAN ANCHOR

SIDE VIEW

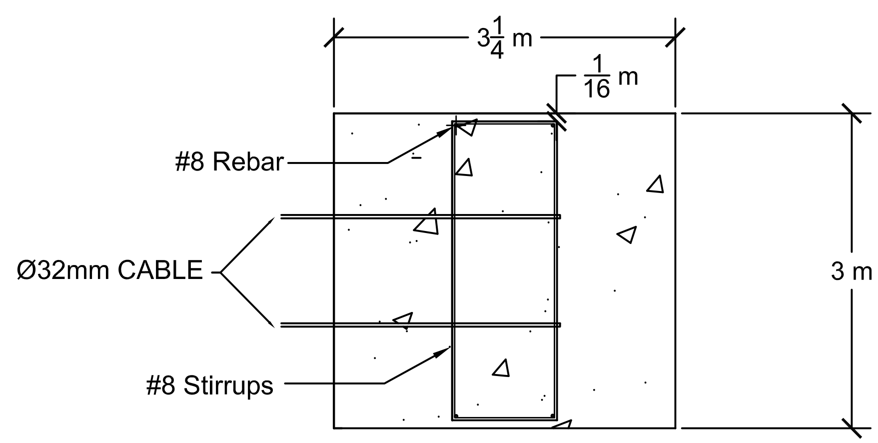


ANCHOR WILL BE 1 METER ABOVE GROUND

ANCHOR LOADS
TENSION IN CABLE UNDER
FULL LOAD: 27.1 metric tons (59.7 kips)
HOISTING LOAD: 0.43 metric tons (0.94 kips)
DEAD LOAD: 6.6 metric tons (14.4 kips)

MAINLAND DEADMAN ANCHOR

TOP VIEW

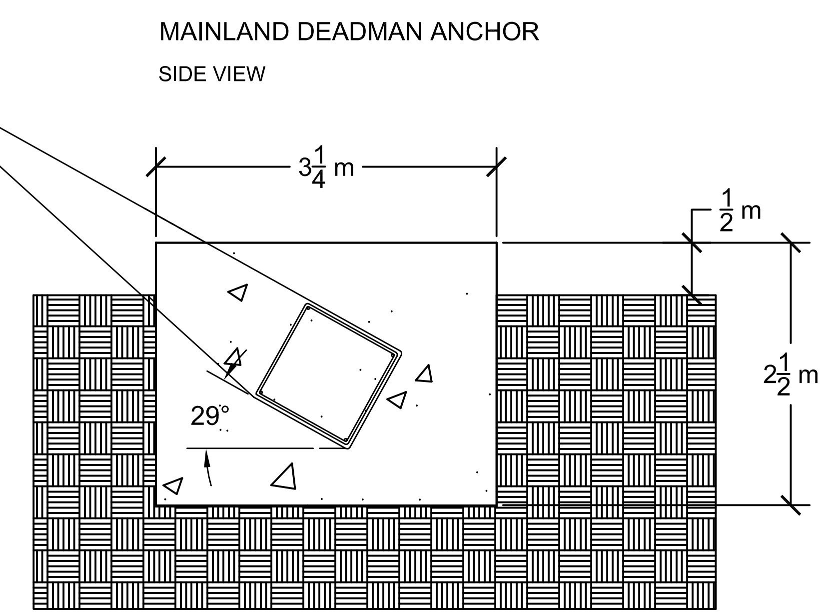


NOTE: CLEAR COVER IS 80 MM

CONCETE ANCHOR WITH # 8 REBAR CAGE

MAINLAND DEADMAN ANCHOR

SIDE VIEW



ANCHOR WILL BE 1/2 METER ABOVE GROUND

LOS AGUA NINOS
ERYK ANDERSON
NICK CHILDS
JONATHAN DOBREI
BEN WARING
INTERNATIONAL SENIOR DESIGN

PROJECT
THE SOLOY BRIDGE PROJECT
FOOT BRIDGE TO RECREATION AREA
SOLOY, PANAMA

SHEET TITLE
ANCHOR DETAILS
SCALE N/A

SHEET
9 OF 9
DATE
12/11/2009

Appendix E

Foundation Slab Design

Foundation Slab Design: Variable Identification List

λ_w	Unit weight of water
Q_b	Allowable bearing pressure
λ	Lateral bearing
ι	Lateral sliding resistance
$side_{col.s}$	Length of the short side of the column
$side_{col.l}$	Length of the long side of the column
h_{cmax}	Tallest column height
λ_c	Modification factor based on concrete weight from ACI
β_c	Ratio of column long side to short side
α_s	Constant used to determine V_c in slabs
ϕ_s	Strength reduction factor for shear
ϕ_{ft}	Strength reduction factor for flexure assuming tension controlled
j	Moment arm reduction factor
f'_c	Compressive strength of concrete
f_y	Yield strength of reinforcement steel
$cover_{conc}$	Concrete cover required for reinforcement
P_{cab}	Vertical load produced on foundation by tower (excluding tower dead weight)
D_t	Dead weight of tower
D_c	Dead weight of foundation column
P_u	Total factored load acting on slab
t_f	Thickness of slab
q_n	Net soil bearing pressure
A_{req}	Minimum allowable slab area
$side_{req}$	Minumum allowable side of a square slab
$side_{act.s}$	Actual length for the short side of slab
$side_{act.l}$	Actual length for the long side of slab
A_{act}	Actual area of slab
q_{act}	Actual bearing pressure applied
$d_{b.l}$	Diameter of a reinforcement bar running in the long direction

$d_{b.s}$	Diameter of a reinforcement bar running in the short direction
$A_{b.l}$	Cross-sectional area of a reinforcement bar running in the long direction
$A_{b.s}$	Cross-sectional area of a reinforcement bar running in the short direction
d_{avg}	Average distance from extreme compression fiber to centroid of tension reinforcement in both directions
V_{uu}	Applied two way shear force based upon factored loading
b_o	Length of critical shear perimeter
V_{cc}	Nominal two way shear strength provided by concrete
$V_{u.l}$	Applied one way shear force based upon factored loading in the long direction
$V_{c.l}$	Nominal one way shear strength provided by concrete in the long direction
$V_{u.s}$	Applied one way shear force based upon factored loading in the short direction
$V_{c.s}$	Nominal one way shear strength provided by concrete in the short direction
$M_{u.l}$	Applied moment based upon factored loading in the long direction
$M_{n.l}$	Nominal flexural strength in the long direction
$A_{sreq.l}$	Area of reinforcing steel required to meet flexural requirements in the long direction
$A_{smin.l}$	Minimum area of steel in the long direction required based upon ACI code
$A_{s.l}$	Governing area of steel needed in the long direction
$n_{b.l}$	Number of bars required to meet $A_{s.l}$
$n_{bact.l}$	Actual whole number of bars chosen for design
$A_{sact.l}$	Actual designed area of steel in the long direction
$spac_{min.l}$	Minumum spacing requirements in the long direction based upon ACI code
$spac_{max.1}$	Maximum spacing requirements in the long direction based upon ACI code
$spac_l$	Spacing between bars in the long direction
a_l	Depth of equivalent rectangular stress block in the long direction
$M_{nact.l}$	Actual nominal flexural strength in the long direction given designed A_s
$\beta_{1.l}$	Ratio of depth of rectangular stress block, a, to depth to neutral axis, c; for the long direction
c_1	Distance from extreme-compression fiber to neutral axis for the long direction
ψ_e	Factor used to modify development length based upon coating
$l_{d.l}$	Development length required in the long direction
$M_{u.s}$	Applied moment based upon factored loading in the short direction

$M_{n.s}$	Nominal flexural strength in the short direction
$A_{sreq.s}$	Area of reinforcing steel required to meet flexural requirements in the short direction
$A_{smin.s}$	Minimum area of steel in the short direction required based upon ACI code
$A_{s.s}$	Governing area of steel needed in the short direction
$spac_{min.s}$	Minumum spacing requirements in the short direction based upon ACI code
$spac_{max.s}$	Maximum spacing requirements in the short direction based upon ACI code
$n_{b.s}$	Number of bars required to meet $A_{s.s}$
$n_{bc.s}$	Number of bars required in the center square
$n_{bo.s}$	Number of bars required outside of the center square on each side
$n_{bact.s}$	Actual whole number of bars chosen for design
$A_{sact.s}$	Actual designed area of steel in the short direction
a_s	Depth of equivalent rectangular stress block in the short direction
$M_{nact.s}$	Actual nominal flexural strength in the short direction given designed A_s
$\beta_{1.s}$	Ratio of depth of rectangular stress block, a, to depth to neutral axis, c; for the short direction
c_s	Distance from extreme-compression fiber to neutral axis for the short direction
$l_{d.s}$	Development length required in the short direction
ϕ_b	Bearing strength reduction factor
A_1	Loaded area
A_2	Area of the lower base of the largest frustum of a pyramid contained wholly within the support and having its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal
P_{bmax}	Maximum allowable bearing pressure between column and slab
h_{bmax}	Height from the base of the foundation to the top of the flood line
F_b	Buoyant force produced by flood waters
W_f	Weight of entire foundation and tower

Foundation Calcs from 15-5 Spread Footings in Reinforced Concrete by Wight and MacGregor:

Knowns

Blue denotes manually modified numbers.

Unit weight of water

$\gamma_w := 62.4 \frac{lb}{ft^3}$ $\gamma_w = 999.552 \frac{kg}{m^3}$

Factored allowable foundation pressure

$Q_b := 1.6 \cdot 1500 \frac{lb}{ft^2}$ $Q_b = 11717.826 \frac{kg}{m^2}$ $Q_b = 2400 \frac{lb}{ft^2}$

Factored lateral bearing (below natural grade)

$\lambda := 1.6 \cdot 100 \frac{\frac{lb}{ft^2}}{ft}$ $\lambda = 2562.954 \frac{\frac{kg}{m^2}}{m}$ $\lambda = 160 \frac{\frac{lb}{ft^2}}{ft}$

Factored lateral sliding resistance

$\iota := 1.6 \cdot 130 \frac{lb}{ft^2}$ $\iota = 1015.545 \frac{kg}{m^2}$ $\iota = 208 \frac{lb}{ft^2}$

Column dimensions: 1.5m x 2.5m

$side_{col.s} := 3.5 \cdot 16in$ $side_{col.s} = 1.4224\,m$ $side_{col.s} = 4.667\,ft$

$side_{col.l} := 6.5 \cdot 16in$ $side_{col.l} = 2.6416\,m$ $side_{col.l} = 8.667\,ft$

$h_{cmax} := 21 \cdot 8in + .5m$ $h_{cmax} = 4.767\,m$ $h_{cmax} = 15.64\,ft$

$\lambda_c := 1.0$ $\beta_c := \frac{side_{col.l}}{side_{col.s}}$ $\beta_c = 1.857$ $\alpha_s := 40$

Lambda based upon normal weight concrete

$\phi_s := .75$ $\phi_{ft} := .9$ $j := .95$

phi.ft initially assumed to be tension controlled

$f'_c := 3000$ $f_y := 60000 \frac{lb}{in^2}$

Assuming grade 60 steel

$cover_{conc} := 3in$

Concrete cover is 3 in. as defined in 7.7.1a

Slab design (to be placed with center of the top at the ground level)

1. Compute the factored loads and the resistance factors

$P_{cab} := 71500lb$ $D_t := 3500lb$ $D_t = 1587.573\,kg$

P.t and D.t assumed from tower calcs

$D_c := 75412lb$ $D_c = 34206.308\,kg$

D.c manually taken from W.c in the column calcs below

$P_u := P_{cab} + 1.2(D_c + D_t)$

2. Estimate the footing size and the factored net soil pressure

$t_f := .5m$ **t.f manually input**

$q_n := Q_b - t_f \cdot 150 \frac{lb}{ft^3}$ $\quad q_n = 10516.442 \frac{kg}{m^2}$

$A_{req} := \frac{P_u}{q_n}$ $\quad A_{req} = 7.168\, m^2$

$side_{req} := \sqrt{A_{req}}$ $\quad side_{req} = 2.677\, m$

$side_{act.s} := 4m$ $\quad side_{act.s} = 13.123\, ft$

$side_{act.l} := 5.5m$ $\quad side_{act.l} = 18.045\, ft$

side.act.l and side.act.s chosen manually

$A_{act} := side_{act.s} \cdot side_{act.l}$ $\quad A_{act} = 22\, m^2$

$$q_{act} := \frac{P_u + A_{act} t_f \cdot 150 \frac{lb}{ft^3}}{A_{act}}$$ $\quad q_{act} = 4627.953 \frac{kg}{m^2}$

3. Check the thickness for two-way shear

Assuming No. 7 bars in the long direction $\quad d_{b.l} := .875in$ and $A_{b.l} := .6in^2$

Assuming No. 7 bars in the short direction $\quad d_{b.s} := .875in$ and $A_{b.s} := .6in^2$

$$d_{avg} := t_f - cover_{conc} - \frac{d_{b.l} + d_{b.s}}{2}$$ $\quad d_{avg} = 0.402\, m$

$$V_{uu} := q_{act} \cdot \left[A_{act} - \left(\frac{d_{avg}}{2}\right)^2\right]$$ $\quad V_{uu} = 101628.398\, kg$

Length of critical shear perimeter

$b_o := 2 \cdot (side_{col.s} + d_{avg}) + 2 \cdot (side_{col.l} + d_{avg})$ $\quad b_o = 9.734\, m$

V.cc is the smallest of the following 3 equations

a) $V_{cc1} := \left(2 + \frac{4}{\beta_c}\right) \cdot \lambda_c \cdot \sqrt{f'_c \frac{lb}{in^2}} \cdot b_o \cdot d_{avg}$ $\quad V_{cc1} = 625288.925\, kg$

b) $V_{cc2} := \left(\frac{\alpha_s \cdot d_{avg}}{b_o} + 2\right) \cdot \lambda_c \cdot \sqrt{f'_c \frac{lb}{in^2}} \cdot b_o \cdot d_{avg}$ $\quad V_{cc2} = 549465.422\, kg$ <-- Governs therefore:

$V_{cc} := V_{cc2}$

c) $V_{cc3} := 4 \cdot \lambda_c \cdot \sqrt{f'_c \frac{lb}{in^2}} \cdot b_o \cdot d_{avg}$ $\quad V_{cc3} = 602130.076\, kg$

$V_{cc} = 549465.422\, kg$

$\phi_s \cdot V_{cc} = 412099.066\, kg$ $\quad \phi_s \cdot V_{cc} = 908522.924\, lb$

4. Check the one-way shear

Long Direction

$$V_{u.l} := q_{act} \cdot \left[side_{act.s} \cdot \left(\frac{side_{act.l}}{2} - \frac{side_{col.l}}{2} - d_{avg} \right) \right]$$ $V_{u.l} = 19023.203\ kg$

$$V_{c.l} := 2 \cdot \lambda_c \cdot \sqrt{f'_c \cdot \frac{lb}{in^2}} \cdot side_{act.s} \cdot d_{avg}$$ $\phi_s \cdot V_{c.l} = 92784.804\ kg$

Short Direction

$$V_{u.s} := q_{act} \cdot \left[side_{act.l} \cdot \left(\frac{side_{act.s}}{2} - \frac{side_{col.s}}{2} - d_{avg} \right) \right]$$ $V_{u.s} = 22583.198\ kg$

$$V_{c.s} := 2 \cdot \lambda_c \cdot \sqrt{f'_c \cdot \frac{lb}{in^2}} \cdot side_{act.l} \cdot d_{avg}$$ $\phi_s \cdot V_{c.s} = 127579.105\ kg$

5. Design the reinforcement in the long direction

$$M_{u.l} := q_{act} \cdot side_{act.s} \cdot \frac{\left(\frac{side_{act.l}}{2} - \frac{side_{col.l}}{2} \right)^2}{2}$$ $M_{u.l} = 18906.233\ kg \cdot m$

$$M_{n.l} := \frac{M_{u.l}}{\phi_{ft}}$$ $M_{n.l} = 21006.925\ kg \cdot m$ Assuming tension controlled

$$A_{sreq.l} := \frac{M_{n.l}}{f_y \cdot j \cdot d_{avg}}$$ $A_{sreq.l} = 2.023\ in^2$

Minimum A.s A.smin from ACI sections 10.5.4 and 7.12.2.1

$$A_{smin.l} := .0018 \cdot side_{act.s} \cdot t_f$$ $A_{smin.l} = 5.58\ in^2$ <-- Governs, therefore: $A_{s.l} := A_{smin.l}$

Trying No.7 Bars

$$n_{b.l} := \frac{A_{s.l}}{A_{b.l}}$$ $n_{b.l} = 9.3$

$n_{bact.l} := 10$ **Manually rounded up to nearest whole number.**

$$A_{sact.l} := n_{bact.l} \cdot A_{b.l}$$ $A_{sact.l} = 6\ in^2$

$spac_{min1.l} := 1in$

or

$spac_{min2.l} := d_{b.l}$ $spac_{min2.l} = 0.875\ in$ spac.min is max of spac.min1 and spac.min2

$spac_{max.l} := 18in$

$$spac_l := \frac{side_{act.s} - 2cover_{conc}}{n_{bact.l} - 1}$$ $spac_l = 16.831\ in$ $spac_l = 42.751\ cm$

spac.l is on center spacing

$$a_l := \frac{A_{sact.l} \cdot f_y}{.85 \cdot f'_c \cdot \frac{lb}{in^2} \cdot side_{act.s}}$$ $a_l = 0.023\,m$

$$M_{nact.l} := A_{sact.l} \cdot f_y \cdot \left(d_{avg} - \frac{a_l}{2} \right)$$ $M_{nact.l} = 63715.366\,kg \cdot m$

$\phi_{ft} \cdot M_{nact.l} = 57343.829\,kg \cdot m$ phi.ft x M.nact > M.u therefore it checks

Check tension controlled assumption

$f'_c = 3000$ psi < 4000 psi Therefore, $\beta_{1.l} := .85$

$$c_l := \frac{a_l}{.85}$$

$\frac{c_l}{d_{avg}} = 0.067$ << .375 Therefore tension controlled assumption checks

Check the Development

$\psi_e := 1$

Equation based upon f'.c, f.y, and rebar size.
See Appendix Table A-6. Currently Case 2

$$l_{d.l} := 54.8 \frac{\psi_e}{\lambda_c} \cdot d_{b.l}$$ $l_{d.l} = 47.95\,in$

$$\frac{side_{act.l}}{2} - \frac{side_{col.l}}{2} = 56.268\,in$$ OK

6. Design the reinforcement in the short direction.

$$M_{u.s} := q_{act} \cdot side_{act.l} \cdot \frac{\left(\frac{side_{act.s}}{2} - \frac{side_{col.s}}{2} \right)^2}{2}$$ $M_{u.s} = 21139.404\,kg \cdot m$

$$M_{n.s} := \frac{M_{u.s}}{\phi_{ft}}$$ $M_{n.s} = 23488.226\,kg \cdot m$ Assuming tension controlled

$$A_{sreq.s} := \frac{M_{n.s}}{f_y \cdot j \cdot d_{avg}}$$ $A_{sreq.s} = 2.262\,in^2$

Minimum A.s A.smin from ACI sections 10.5.4 and 7.12.2.1

$A_{smin.s} := .0018 \cdot side_{act.l} \cdot t_f$ $A_{smin.s} = 7.673\,in^2$ <-- Governs, therefore: $A_{s.s} := A_{smin.s}$

$spac_{min1.s} := 1in$ Spacing requirements from ACI 318-08 section 7.6

or

$spac_{min2.s} := d_{b.l}$ $spac_{min2.s} = 0.875\,in$ spac.min is max of spac.min1 and spac.min2

$spac_{max.s} := 18in$

Trying No.7 Bars

$n_{b.s} := \frac{A_{s.s}}{A_{b.s}}$ $\qquad n_{b.s} = 12.788$

Bar Arrangement

$\beta := \frac{side_{act.l}}{side_{act.s}}$ $\qquad \beta = 1.375$

Bars required in the center square of 2.5m by 2.5m

$n_{bc.s} := \frac{2}{\beta + 1} \cdot ceil(n_{b.s})$ $\qquad n_{bc.s} = 10.947$ $\qquad$ Use 11 bars in the 2.5m by 2.5m center square

$\frac{side_{act.s}}{ceil(n_{bc.s}) - 1} = 40\,cm$ o.c. $\qquad$ Denominator based on the rounded result of the previous equation

Bars required beyond middle square $\qquad$ Section relies on manual input of denominators

$n_{bo.s} := \frac{\frac{side_{act.l} - side_{act.s}}{2}}{spac_{max.s}}$ $\qquad n_{bo.s} = 1.64$ $\qquad$ Use 2 bars and thus 3 spaces

Place 2 bars on each side spaced 25cm on center

$\frac{\frac{side_{act.l} - side_{act.s}}{2} - cover_{conc}}{ceil(n_{bo.s})} = 33.69\,cm$ $\qquad 33.69\,cm = 13.264\,in$

$n_{bact.s} := ceil(n_{bc.s}) + 2\,ceil(n_{bo.s})$ $\qquad n_{bact.s} = 15$

$A_{sact.s} := n_{bact.s} \cdot A_{b.s}$ $\qquad A_{sact.s} = 9\,in^2$

$a_s := \frac{A_{sact.s} \cdot f_y}{.85 \cdot f'_c \cdot \frac{lb}{in^2} \cdot side_{act.l}}$ $\qquad a_s = 0.025\,m$

$M_{nact.s} := A_{sact.s} \cdot f_y \cdot \left(d_{avg} - \frac{a_s}{2}\right)$ $\qquad M_{nact.s} = 95319.532\,kg \cdot m$

$\phi_{ft} \cdot M_{nact.s} = 85787.579\,kg \cdot m$ $\qquad$ phi.ft x M.nact > M.u therefore it checks

Check tension controlled assumption

$f'_c = 3000$ psi $\quad < \quad 4000$ psi $\qquad$ Therefore, $\qquad \beta_{1.s} := .85$

$c_s := \frac{a_s}{.85}$

$\frac{c_s}{d_{avg}} = 0.073$ $\quad << \quad .375$ $\qquad$ Therefore tension controlled assumption checks

Check the Development

$l_{d.s} := 54.8 \frac{\psi_e}{\lambda_c} \cdot d_{b.s}$ $\qquad$ **Equation based upon f'.c, f.y, and rebar size. See Appendix Table A-6. Currently Case 2** $\qquad$ psi.e as before

$$\frac{side_{act.s}}{2} - \frac{side_{col.s}}{2} = 50.74\ in$$ $l_{d.s} = 47.95\ in$ OK

7. Design the column-footing joint

$P_u = 75384.512\ kg$

Maximum Bearing Strength (P.bmax)

$\phi_b := .65$

$A_1 := side_{col.s} \cdot side_{col.l}$

$A_2 := \left(side_{col.s} + 4 \cdot t_f\right) \cdot \left(side_{col.l} + 4 \cdot t_f\right)$

A.2 is dependent upon column and slab dimensions and must be reviewed upon modification of those dimensions

$$\frac{A_2}{A_1} = 4.228$$

$$\sqrt{\frac{A_2}{A_1}} = 2.056$$

$\sqrt{\frac{A_2}{A_1}}$ Cannot be greater than 2 therefore,

$$P_{bmax} := .85 \cdot \phi_b \cdot f'_c \cdot \frac{lb}{in^2} \cdot A_1 \cdot 2$$ $P_{bmax} = 8757308.307\ kg$

Since $P_{bmax} > P_u$ no additional reinforcement is needed for bearing strength.

8. Flooding checks

Vertical Force Check

$h_{bmax} := h_{cmax} - 2m + t_f$

$F_b := h_{bmax} \cdot \gamma_w \cdot side_{act.l} \cdot side_{act.s}$ $F_b = 71846.207\ kg$ $F_b = 158393.773\ lb$

$$W_f := \frac{P_{cab}}{1.6} + D_t + D_c + \left(150\ \frac{lb}{ft^3} \cdot side_{act.l} \cdot side_{act.s} \cdot t_f\right)$$ $W_f = 82494.255\ kg$ $W_f = 181868.7\ lb$

W_f > F_b Checks

Appendix F

Foundation Column Design

Foundation Column Design: Variable Identification List

*Variables missing from this list can be found in ACI 530-05

t_{ts}	Thickness of the top slab
h_{cw}	Height of the west column
h_{ce}	Height of the east column
h_{fw}	Height of the flood above ground level on the west side
h_{fe}	Height of the flood above ground level on the east side
A_{soil}	Area of soil to be placed in the center of the column
$f'm$	Compressive strength of masonry
A_{mg}	Gross area of masonry layout
A_{core}	Area of one core in a CMU
n_{bl}	Number of blocks per course
n_{core}	Number of cores left open per course
A_{tn}	Net area of one course
P_{max}	Maximum compressive force allowed by masonry layout
P_t	Total force acting on the foundation
C_d	Drag coefficient
A_c	Flood water area on column
v_w	Velocity of the flood water
D	Drag force applied by flood water
W_{creq}	Weight of the column required to keep the column from toppling
W_c	Weight of the column resisting the flood force

Column Design (center of base is to be at ground level)

See Appendix E: Foundation Slab Design for previous values

Dimensions

Thickness of top slab

$t_{ts} := .5m$

$h_{cw} := 14 \cdot 8in + .5m$ $\qquad h_{cw} = 3.345\,m$ $\qquad h_{cw} = 10.974\,ft$

$h_{ce} := 21 \cdot 8in + .5m$ $\qquad h_{ce} = 4.767\,m$ $\qquad h_{ce} = 15.64\,ft$

$h_{fw} := 1.2m$ $\qquad h_{fw} = 1.2\,m$

$h_{fe} := 2.6m$ $\qquad h_{fe} = 2.6\,m$

$A_{soil} := 40in \cdot (36in + 40in)$

Compression check

$f'_m := 1800 \frac{lb}{in^2}$

Based upon 8 x 8 x 16

$A_{mg} := 7.625in \cdot 15.625in$ $\qquad A_{mg} = 0.077\,m^2$ $\qquad A_{mg} = 119.141\,in^2$

$A_{core} := \left(\frac{15.625in - 2 \cdot 1.25in - 1in}{2}\right)(7.625in - 2 \cdot 1.25in)$ $\qquad A_{core} = 0.02\,m^2$ $\qquad A_{core} = 31.07\,in^2$

$n_{bl} := 23$

Based upon number of blocks and empty holes

$n_{core} := 22$

$A_{tn} := n_{bl} \cdot A_{mg} - n_{core} \cdot A_{core}$ $\qquad A_{tn} = 1.327\,m^2$ $\qquad A_{tn} = 2056.687\,in^2$

$P_{max} := f'_m \cdot A_{tn}$ $\qquad P_{max} = 1679215.963\,kg$ $\qquad P_{max} = 3702037.5\,lb$

$P_t := P_{cab} + D_t$ $\qquad P_t = 34019.428\,kg$ $\qquad P_t = 75000\,lb$

Flood Force Check

$C_d := 2$

$\rho := \gamma_w$ $\qquad A_c := h_{fe} \cdot side_{col.s}$

$v_w := 3 \frac{ft}{s}$

$$D := \frac{C_d \cdot \rho \cdot A_c \cdot \frac{v_w^2}{2}}{g}$$

$D = 694.843\,lb$

Weight required to resist flood force (based on moment analysis)

$$W_{creq} := \frac{\frac{h_{cmax}}{2} \cdot D}{side_{col.l}}$$

$W_{creq} = 284.393\,kg$ $\qquad W_{creq} = 626.979\,lb$

Column weight resisting overturning moment provided by the flood force

$$W_c := \left(h_{cmax} - t_{ts}\right)\cdot A_{tn}\cdot 150\,\frac{lb}{ft^3} + t_{ts}\cdot side_{col.s}\cdot side_{col.l}\cdot 150\,\frac{lb}{ft^3} + \left(h_{cmax} - t_{ts}\right)A_{soil}\cdot 120\,\frac{lb}{ft^3}$$

$W_c = 34206.266\ kg$ $\qquad$ $W_c = 75411.907\ lb$

$W_{creq} = 626.979\ lb$ $\qquad$ < $\qquad$ $W_c = 75411.907\ lb$ $\qquad$ Checks

Foundation Single Wall Check: This check will figure where the fully grouting is no longer necessary in the East side foundation column (ACI 530-05). An axial/bending check on the steel spaced at 16" will follow.

EASTSIDE:

$k := 1000lb$

$Wall := 41in$

$\gamma_{soil} := 120 \frac{lb}{ft^3}$

$K_a := 0.3333$

$h_{column} := 21 \cdot 8in$ *21 courses tall w/ a 0.5 thick top slab

$F_{soil} := 0.5 \cdot h_{column} \cdot \gamma_{soil} \cdot K_a$ $F_{soil} = 279.972 \frac{lb}{ft^2}$

$Soil_{loading} := F_{soil} \cdot Wall$

$Soil_{loading} = 956.571 \frac{lb}{ft}$

$Resultant := \frac{Soil_{loading} \cdot h_{column}}{2}$ $Resultant = 6695.997\,lb$

$M_{req} := Resultant \cdot h_{column} \cdot \frac{1}{3}$ $M_{req} = 31247.986\,lb \cdot ft$

Masonry Properties: #8 rebar spaced at 8"

$A_s := 0.79in^2$ # 8 rebar $t_f := 1.25in$ $S_{steel} := 8in$

$f_m := 1800 \frac{lb}{in^2}$

$E_s := 29000000 \frac{lb}{in^2}$ $E_m := 900 \cdot f_m$

$n := \frac{E_s}{E_m}$ $n = 17.901$

$F_b := \frac{f_m}{3}$ $F_b = 600 \frac{lb}{in^2}$

$F_s := 24000 \frac{lb}{in^2}$

$b := 15.625in$ $d := \frac{7.625in}{2}$

$\rho := \frac{A_s}{b \cdot d}$ $\rho = 0.013$

$$k := \frac{n \cdot \rho + \frac{\left(\frac{t_f}{d}\right)^2}{2}}{n \cdot \rho + \left(\frac{t_f}{d}\right)} \qquad k = 0.515$$

$k \cdot d = 1.964\,in$

$f_M := \frac{F_s}{n} \cdot \left(\frac{k}{1-k}\right) \qquad f_M = 1423.976\ \frac{lb}{in^2}$

$j := 1 - \left(\frac{k}{3}\right)$

$M_{wall} := \frac{A_s \cdot F_s \cdot j \cdot d}{S_{steel}} \cdot Wall \qquad M_{wall} = 25571.429\ lb \cdot ft \qquad M_{steel} > M_{req}$ *Therefore section passes this check.

$h_{reqfg} := \frac{1}{3} \cdot h_{column} \qquad h_{reqfg} = 4.667\ ft$

$CMUCourses_{reqfg} := \frac{h_{reqfg}}{8in} \qquad CMUCourses_{reqfg} = 7$

***Atleast 7 courses are required to be fully grouted w/#8 rebars. So, after 8 courses the grout and steel will be spaced every 16".**

<u>Partially Grouted w/Steel Spaced @ 16" Section of Column Axial/Bending Capacity Check: (ACI530-05 Specs).</u>

$P_u := 75000lb$ *Loading from Tower and Cable

$M_u := 700lb \cdot \frac{h_{column}}{2} \qquad M_u = 4900\ lb \cdot ft$

*Mu uses the drag force acting on it during a flood

$A_n := 55.625in \cdot 103.625in - (40.375in)^2 - (32.375in \cdot 40.375in) - 48 \cdot (5.125in \cdot 6.0625in)$

$A_n = 1335.484\ in^2$

*An was calculated with info from AutoCad Plan view of foundation column.

$A_s := 24 \cdot 0.79in^2 \qquad A_s = 18.96\ in^2$

$0.0025 \cdot A_n = 3.339\ in^2$

$0.04 \cdot A_n = 53.419\ in^2 \qquad 0.0025A_n < A_s < 0.04A_n$

*Amount of Steel Checks out

$b := 55.625in$
$h := 103.625in$

$A_{tfv} := 104in \cdot 1.25in \cdot 4 \qquad A_{tfh} := (5.5in \cdot 2 + 40in) \cdot 1.25in$

$A_{tv} := 5.5in \cdot 1.25in \qquad A_{th} := 2 \cdot 5.5in \cdot 1.25in$

$$y_{bar} := \frac{A_{tfv} \cdot \frac{h}{2} + A_{tfh} \cdot (.625in + 7in + 40in + 2 \cdot 48in + 56in + 96in + 104in) + 7 \cdot A_{tv} \cdot (4in + 44in + 52in + 100in) + 2 \cdot A_{th} \cdot (8in + 16in + 2 \cdot 24in + 32in + 2 \cdot 64in + 72in + 2 \cdot 80in + 88in + 96in)}{A_n}$$

$y_{bar} := 59.801in$

$A_c := 64in^2$

$A_e := 6.0625in \cdot 5.125in$

$A_{ec} := 64in^2 - (6.0625in \cdot 5.125in)$

$$I := (56in)^2 \cdot (b \cdot 8in - 3 \cdot A_e) + 2 \cdot A_{ec} \cdot \left(48.5^2 in^2 + 24^2 in^2 + 16^2 in^2 + 32^2 in^2\right) + (40in)^2 \cdot (b \cdot 8in - 3 \cdot A_e) + 2 \cdot A_c \cdot \left(40^2 in^2 + 32.5^2 in^2 + 8^2 in^2 + 24^2 in^2\right)$$

$I := 2365145.715in^4$

$r := \sqrt{\frac{I}{A_n}} \qquad r = 42.083\ in \qquad r > 99$

$\rho := \frac{A_s}{A_n} \qquad \rho = 0.014$

$n := \frac{E_s}{E_m}$ $\quad n = 17.901$

$k := \sqrt{2 \cdot n \cdot \rho + (n \cdot \rho)^2} - n \cdot \rho$ $\quad k = 0.503$

$j := 1 - \frac{k}{3}$ $\quad j = 0.832$

$f_s := n \cdot f_m \cdot \frac{1 - k}{k}$ $\quad f_s = 31870.543 \frac{lb}{in^2}$ $\quad f_s > 24000psi$ *Tension Controls

$f_s := 24000 \frac{lb}{in^2}$

Moment Capacity: Ma

$M_a := A_s \cdot f_s \cdot j \cdot d$

$M_a = 1444113.48\,lb \cdot in$

$h_{eff} := 1.2 \cdot (h_{column} - 8 \cdot 8in)$ $\quad h_{eff} = 10.4\,ft$

$\frac{h_{eff}}{b} = 2.24< 25$ *Clears Check

$\frac{h_{eff}}{r} = 2.96< 99$ *Clears Check

$$P_a := (0.25 \cdot f_m \cdot A_n + 0.65 \cdot A_s \cdot f_s)\left[1 - \left(\frac{h_{eff}}{140 \cdot r}\right)^2\right]$$

$P_a = 896341.602\,lb$

$\frac{P_u}{P_a} + \frac{M_u}{M_a} = 0.124$ **0.12 < 1.0 Therefore this check passes**

Foundation Single Wall Check: This check will figure where the fully grouting is no longer necessary in the West side foundation column (ACI 530-05). Since the taller column passed the axial/bending check on the steel spaced at 16", it is not necessary to check the smaller column.

WESTSIDE: $k := 1000lb$

$Wall := 41in$

$\gamma_{soil} := 120\,\frac{lb}{ft^3}$

$K_a := 0.3333$

$h_{column} := 14 \cdot 8in$ *14 courses tall w/ a 0.5 thick top slab

$F_{soil} := 0.5 \cdot h_{column} \cdot \gamma_{soil} \cdot K_a$ $F_{soil} = 186.648\,\frac{lb}{ft^2}$

$Soil_{loading} := F_{soil} \cdot Wall$

$Soil_{loading} = 637.714\,\frac{lb}{ft}$

$Resultant := \frac{Soil_{loading} \cdot h_{column}}{2}$ $Resultant = 2975.999\,lb$

$M_{req} := Resultant \cdot h_{column} \cdot \frac{1}{3}$ $M_{req} = 9258.663\,lb \cdot ft$

Masonry Properties: #8 rebar spaced at 8"

$A_s := 0.79in^2$ # 8 rebar $t_f := 1.25in$ $S_{steel} := 8in$

$f_m := 1800\,\frac{lb}{in^2}$

$E_s := 29000000\,\frac{lb}{in^2}$ $E_m := 900 \cdot f_m$

$n := \frac{E_s}{E_m}$ $n = 17.901$

$F_b := \frac{f_m}{3}$ $F_b = 600\,\frac{lb}{in^2}$

$F_s := 24000\,\frac{lb}{in^2}$

$b := 15.625in$ $d := \frac{7.625in}{2}$

$\rho := \frac{A_s}{b \cdot d}$ $\rho = 0.013$

$$k := \frac{n \cdot \rho + \frac{\left(\frac{t_f}{d}\right)^2}{2}}{n \cdot \rho + \left(\frac{t_f}{d}\right)} \qquad k = 0.515$$

$k \cdot d = 1.964\,in$

$f_M := \frac{F_s}{n} \cdot \left(\frac{k}{1-k}\right)$ $\quad f_M = 1423.976\,\frac{lb}{in^2}$

$j := 1 - \left(\frac{k}{3}\right)$

$M_{wall} := \frac{A_s \cdot F_s \cdot j \cdot d}{S_{steel}} \cdot Wall$ $\quad M_{wall} = 25571.429\,lb \cdot ft$ $\quad M_{steel} > M_{req}$ *Therefore section passes this check.

$h_{reqfg} := \frac{1}{3} \cdot h_{column}$ $\quad h_{reqfg} = 3.111\,ft$

$CMUCourses_{reqfg} := \frac{h_{reqfg}}{8in}$ $\quad CMUCourses_{reqfg} = 4.667$

***Atleast 5 courses are required to be fully grouted w/#8 rebars. So, after 6 courses the grout and steel will be spaced every 16".**

Appendix G

Anchor Design

Anchor Design: Variable Identification List

γ_w	Unit weight of water
Q_b	Allowable bearing pressure
λ	Lateral bearing (below natural grade). Assuming force distridution is linear.
ι	Lateral sliding resistance
T_f	Cable tension
β_f	Angle of Cable off of the horizontal
h_{fg}	Height of the flood line above ground level
h_{ag}	Height of the top of the anchor above ground level
T_{fx}	X component of cable tension
T_{fy}	Y component of cable tension
L_a	Length (parallel with the bridge line) of the anchorage block
w_a	Width (perpendicular to bridge line) of the anchorage block
d_a	Depth of the anchorage block
V_a	Volume of anchorage block
F_{lb}	Force produced by lateral bearing
F_{ls}	Force produced from lateral sliding resistance
F_{ex}	Excess force in the horizontal direction keeping the anchor from pulling out
h_{fb}	Height of the flood in relation to the base of the the foundation
W_a	Weight of anchorage block
F_b	Buoyant force
F_{ey}	Excess force in the vertical direction keeping the anchor from pulling out
W_{amax}	Maximum anchor weight able to be supported by the soil

Anchorage Design E (Island Side):

Blue indicates areas where manual input is needed

Knowns

Unit weight of water

$\gamma_w := 62.4 \frac{lb}{ft^3}$ $\gamma_w = 999.552 \frac{kg}{m^3}$

Factored allowable foundation pressure

$Q_b := 1.6 \cdot 1500 \frac{lb}{ft^2}$ $Q_b = 11717.826 \frac{kg}{m^2}$ $Q_b = 2400 \frac{lb}{ft^2}$

Factored lateral bearing (below natural grade)

$\lambda := 1.6 \cdot 100 \frac{\frac{lb}{ft^2}}{ft}$ $\lambda = 2562.954 \frac{\frac{kg}{m^2}}{m}$ $\lambda = 160 \frac{\frac{lb}{ft^2}}{ft}$

Factored lateral sliding resistance

$\iota := 1.6 \cdot 130 \frac{lb}{ft^2}$ $\iota = 1015.545 \frac{kg}{m^2}$ $\iota = 208 \frac{lb}{ft^2}$

Factored cable tension

$T_f := 27090 kg$ $T_f = 59723.227\, lb$

Angle of cable off of the horizontal

$\beta_f := 29.263\ deg$

Height of the flood line above ground level

$h_{fg} := 3.2m$ $h_{fg} = 10.499\, ft$

Height of the top of the anchor above ground level

$h_{ag} := 1m$ $h_{ag} = 3.281\, ft$

Cable tension components

$T_{fx} := T_f \cdot \cos(\beta_f)$ $T_{fx} = 23632.913\, kg$ $T_{fx} = 52101.654\, lb$

$T_{fy} := T_f \cdot \sin(\beta_f)$ $T_{fy} = 13242.112\, kg$ $T_{fy} = 29193.859\, lb$

Anchor block dimensions Adjust these manually and check to see that they meet requirements.

L.a refers to length (parallel with the bridge line) of the anchorage block

$L_a := 4m$ $L_a = 13.123\, ft$

w.a refers to the width (perpendicular to bridge line) of the anchorage block

$w_a := 3m$ $w_a = 9.843\,ft$

d.a refers to the depth of the anchorage block

$d_a := 3m$ $d_a = 9.843\,ft$

$V_a := L_a \cdot d_a \cdot w_a$ $V_a = 36\,m^3$ $V_a = 1271.328\,ft^3$

Horizontal check

$F_{lb} := .5\lambda \cdot \left(d_a - h_{ag}\right)^2 \cdot w_a$ $F_{lb} = 15377.725\,kg$ $F_{lb} = 33902.08\,lb$

$F_{ls} := \iota \cdot L_a \cdot w_a$ $F_{ls} = 12186.539\,kg$ $F_{ls} = 26866.72\,lb$

$F_{ex} := F_{lb} + F_{ls} - T_{fx}$ $F_{ex} = 3931.351\,kg$ $F_{ex} = 8667.146\,lb$

A positive value corresponds to excess pull-out resistance force and a negative value identifies that an adjustment to the dimensions needs to be made.

Vertical check (with bouyancy from flooding)

$h_{fb} := h_{fg} + d_a - h_{ag}$ $h_{fb} = 5.2\,m$ $h_{fb} = 17.06\,ft$

$W_a := V_a \cdot 150\,\frac{lb}{ft^3}$ $W_a = 86499.702\,kg$ $W_a = 190699.2\,lb$

$F_b := h_{fb} \cdot \gamma_w \cdot L_a \cdot w_a$ $F_b = 62372.052\,kg$ $F_b = 137506.837\,lb$

$F_{ey} := W_a - F_b - T_{fx}$ $F_{ey} = 494.737\,kg$ $F_{ey} = 1090.709\,lb$

Again, a positive value corresponds to excess pull out resistance force and a negative value identifies that an adjustment to the dimensions needs to be made.

Note: this does not take into account any size requirements for the placement of anything in the concrete.

Bearing check

$W_{amax} := Q_b \cdot L_a \cdot w_a$ $W_{amax} = 140613.916\,kg$ $W_{amax} = 310000.62\,lb$

$W_a = 86499.702\,kg$ < $W_{amax} = 140613.916\,kg$ Checks

Anchorage Design W (Mainland Side)

Blue indicates areas where manual input is needed

Knowns

Unit weight of water

$\gamma_w := 62.4 \frac{lb}{ft^3}$ $\gamma_w = 999.552 \frac{kg}{m^3}$

Factored allowable foundation pressure

$Q_b := 1.6 \cdot 1500 \frac{lb}{ft^2}$ $Q_b = 11717.826 \frac{kg}{m^2}$ $Q_b = 2400 \frac{lb}{ft^2}$

Factored lateral bearing (below natural grade)

$\lambda := 1.6 \cdot 100 \frac{\frac{lb}{ft^2}}{ft}$ $\lambda = 2562.954 \frac{\frac{kg}{m^2}}{m}$ $\lambda = 160 \frac{\frac{lb}{ft^2}}{ft}$

Factored lateral sliding resistance

$\iota := 1.6 \cdot 130 \frac{lb}{ft^2}$ $\iota = 1015.545 \frac{kg}{m^2}$ $\iota = 208 \frac{lb}{ft^2}$

Factored cable tension

$T_f := 27090kg$ $T_f = 59723.227\,lb$

Angle of cable off of the horizontal

$\beta_f := 29.263\ deg$

Height of the flood line above ground level

$h_{fg} := 1.5m$ $h_{fg} = 4.921\,ft$

Height of the top of the anchor above ground level

$h_{ag} := .5m$ $h_{ag} = 1.64\,ft$

Cable tension components

$T_{fx} := T_f \cdot \cos(\beta_f)$ $T_{fx} = 23632.913\,kg$ $T_{fx} = 52101.654\,lb$

$T_{fy} := T_f \cdot \sin(\beta_f)$ $T_{fy} = 13242.112\,kg$ $T_{fy} = 29193.859\,lb$

Anchor block dimensions Adjust these manually and check to see that they meet requirements.

L.a refers to length (parallel with the bridge line) of the anchorage block

$L_a := 3.25m$ $L_a = 10.663\,ft$

w.a refers to the width (perpendicular to bridge line) of the anchorage block

$w_a := 3m$ $w_a = 9.843\,ft$

d.a refers to the depth of the anchorage block

$d_a := 2.5m$ $d_a = 8.202\,ft$

$V_a := L_a \cdot d_a \cdot w_a$ $V_a = 24.375\,m^3$ $V_a = 860.795\,ft^3$

Horizontal check

$F_{lb} := .5\lambda \cdot \left(d_a - h_{ag}\right)^2 \cdot w_a$ $F_{lb} = 15377.725\,kg$ $F_{lb} = 33902.08\,lb$

$F_{ls} := \iota \cdot L_a \cdot w_a$ $F_{ls} = 9901.563\,kg$ $F_{ls} = 21829.21\,lb$

$F_{ex} := F_{lb} + F_{ls} - T_{fx}$ $F_{ex} = 1646.375\,kg$

$F_{ex} = 3629.636\,lb$

A positive value corresponds to excess pull-out resistance force and a negative value identifies that an adjustment to the dimensions needs to be made.

Vertical check (with bouyancy from flooding)

$h_{fb} := h_{fg} + d_a - h_{ag}$ $h_{fb} = 3.5\,m$ $h_{fb} = 11.483\,ft$

$W_a := V_a \cdot 150 \frac{lb}{ft^3}$ $W_a = 58567.507\,kg$ $W_a = 129119.25\,lb$

$F_b := h_{fb} \cdot \gamma_w \cdot L_a \cdot w_a$ $F_b = 34109.716\,kg$ $F_b = 75199.051\,lb$

$F_{ey} := W_a - F_b - T_{fx}$ $F_{ey} = 824.878\,kg$

$F_{ey} = 1818.544\,lb$

Again, a positive value corresponds to excess pull out resistance force and a negative value identifies that an adjustment to the dimensions needs to be made.

Note: this does not take into account any size requirements for the placement of anything in the concrete.

Bearing check

$W_{amax} := Q_b \cdot L_a \cdot w_a$ $W_{amax} = 114248.807\,kg$ $W_{amax} = 251875.504\,lb$

$W_a = 58567.507\,kg$ < $W_{amax} = 114248.807\,kg$ Checks

Appendix H

Timber Tower Design

Tower Design: Variable Identification List

*Variables missing from this list can be found in NDS-05

l	Bridge Span
f	Sag
c	Camber
h_t	Tower height
x	Horizontal displacement of the bridge center under wind load
c/c_1	Center distance of tower legs
c/c_2	Center distance of holding down bolts at tower legs
α_1	Inclination angle of the plane of the spanning cables under wind load in relation to the vertical
γ_1	Inclination angle of the plane of the main cables under wind load in relation to the vertical
β	Frontstay cable inclination at saddle
β_f	Backstay cable inclination = frontstay cable inclination at saddle for full load
L_M	Length of main cables between saddles
L_s	Length of spanning cables between tower axes
ϕ_M	Diameter of main cables
ϕ_S	Diameter spanning cables
n	Number of main cables
A_M	Total sectional area of main cables
A_S	Total sectional area of spanning cables
E	Modulus of elasticity
g_o	Vertical load in [t/m]
w	Wind load in [t/m]
p_s	Pretension of spanning cables in [t/m]
V	Vertical load on top of the tower
G_t	Dead weight of tower
H_w	Horizontal load on tower saddles due to wind
P_1	Vertical reaction at tower base, tower leg 1
P_2	Vertical reaction at tower base, tower leg 2
P_H	Horizontal reaction at tower base
T_S	Maximum tension in spanning cables
T_H	Horizontal component of the spanning cable tension in the direction of the bridge axis

T_V Vertical component of the spanning cable tension

T_{ps} Pretension in the sidestay cables

p_{M1} Load on main cables under wind [t/m]

p_{S1} Load on spanning cables under wind [t/m]

k Integral factor for average tension along cable

$$\int_0^L \frac{1}{EA}\, dT := \frac{k \cdot H}{EA}$$

Indices:

- o Initial loading case without considering wind loads, either dead load or full load
- f full load
- d dead load
- 1 load case 1, either A or B

Tower Calculation for level bridge, p.183 of
Survey, Design, Construction of Trail Suspension Bridges for Remote Areas by Grob, et al.

See Appendix I: Cable Design for previous values

Two load combinations are considered: A and B

$W_{thirdofwind} := 0.033 \frac{t}{m}$

$g_l := 350 \frac{lb}{ft}$ $\quad g_l = 0.521 \frac{t}{m}$ Note : g.l is assumed 100 psf w/ deck width of 3.5 ft.

Wind Loading Case A= Full wind load + dead load of bridge

$W_{wind} := 0.1 \frac{t}{m}$

$w_A := W_{wind} \cdot \left(0.232m + 0.0075 \cdot h_T\right)$ $\quad w_A = 0.029\,t$

$w_A = 63.3\,lb$

$A_{case} := w_A + g_d \cdot l$ $\quad A_{case} = 5.185\,m \frac{t}{m}$

$w_{1a} := W_{wind} \cdot (.232 + .0075 \cdot 5.5)$ $\quad w_{1a} = 0.027 \frac{t}{m}$ Note: Wind load supported by bridge=w.1a

Vertical load on the bridge=g.d

$W_{cablesA} := .25 \cdot W_{wind}$ $\quad W_{cablesA} = 0.025 \frac{t}{m}$

$W_{suspendersA} := .015 \cdot W_{wind} \cdot \left(h_T - 2.40m\right)$ $\quad W_{suspendersA} = 7.425 \times 10^{-3}\,m \frac{t}{m}$

$W_{perpA} := 0.205 \frac{t}{m}$

Wind Loading Case B= 1/3 Wind load + Full load

$B_{case} := W_{thirdofwind} + g_l$ $\quad B_{case} = 0.554 \frac{t}{m}$

$w_{1b} := W_{thirdofwind} \cdot (.232 + .0075 \cdot 5.5)$ $\quad w_{1b} = 9.017 \times 10^{-3} \frac{t}{m}$ Note: Wind load supported by bridge=w.1b

Vertical load on the bridge=g.f

$W_{cablesB} := .25 \cdot W_{thirdofwind}$

$W_{suspendersB} := .015 \cdot W_{thirdofwind} \cdot \left(h_T - 2.40m\right)$

$W_{perpB} := 0.068 \frac{t}{m}$

Case A > Case B, therefore Case A controls the design

Calculation Case A (pg. 188)

$f_{OA} := f_d$ $\quad f_{OA} = 4.872\,m$ $\quad c_{OA} := c_d$ $\quad c_{OA} = 1.218\,m$ $\quad g_{OA} := g_d$

1) Calculate total sectional area A_M (mm^2) for all main cables (of n ϕ_M), and A_S (mm^2) for all spanning cables (of 2 ϕ_S), using Table 48: Sectional Area of Cables on p.175

$$A_M := 2\,\frac{\pi\cdot(32mm)^2}{4}\cdot 0.5278 \qquad A_M = 848.964\,mm^2$$

Cable diameters from sag calcs

$$A_S := 2\,\frac{\pi\cdot(32mm)^2}{4}\cdot 0.5278 \qquad A_S = 848.964\,mm^2$$

2) Calculate initial cable lengths:

$$L_{MOA} := l\cdot\left[1+\frac{8}{3}\cdot\left(\frac{f_{OA}}{l}\right)^2-\frac{32}{5}\cdot\left(\frac{f_{OA}}{l}\right)^4\right] \qquad L_{MOA} = 42.105\,m$$

(Main cables)

$$L_{SOA} := l\cdot\left[1+\frac{8}{3}\cdot\left(\frac{c_{OA}}{l}\right)^2\right] \qquad L_{SOA} = 40.697\,m$$

(Spanning cables)

3) Pretension in spanning cables

$$p_S := 0.1\frac{t}{m} - g_d \qquad p_S = -0.027\frac{t}{m}$$

4) Calculate displacement x and sag f.l by iteration method for both loading cases.

$x := 0.015\cdot l$ $\quad x = 0.609\,m$

$f_l := 1.002\cdot f_d$ $\quad f_l = 4.882\,m$

$f_d = 4.872\,m$

$$f_l := \begin{pmatrix}4.305\\4.296\end{pmatrix}m$$

$x := .556m$ messed with neg sign

Step 1:

$$\gamma_l := \operatorname{asin}\left(\frac{x}{f_l+1.30m}\right) \qquad \gamma_l = 0.09 \qquad \gamma_l = 5.16\,deg$$

$$b := \overrightarrow{\left[\cos(\gamma_l)\cdot(f_l+1.30m)\right]} \qquad b = 6.157\,m$$

$$\alpha_1 := \frac{x}{h_T+0.25m-b} \qquad \alpha_1 = 0.385 \qquad \alpha_1 = 22.072\,deg$$

$$c_l := \frac{x}{\sin(\alpha_1)}$$ $c_l = 1.48\,m$

Step 2: $$L_{M1} := l\cdot\left[1 + \frac{8}{3}\cdot\left(\frac{f_l}{l}\right)^2 - \frac{32}{5}\cdot\left(\frac{f_l}{l}\right)^4\right]$$ $L_{M1} = 42.111\,m$

$$L_{Sl} := l\cdot\left[1 + \frac{8}{3}\cdot\left(\frac{c_l}{l}\right)^2\right]$$ $L_{Sl} = 40.744\,m$

$$p_{Ml} := \frac{8\cdot f_l\cdot E\cdot A_M\cdot(L_{M1} - L_{MOA})}{1.04\cdot L_{MOA}\cdot l^2} + g_{OA} + p_S$$ $p_{Ml} = 0.128\,\frac{t}{m}$

$$p_{Sl} := \frac{8\cdot c_l\cdot E\cdot A_S\cdot(L_{Sl} - L_{SOA})}{1.003\cdot L_{SOA}\cdot l^2} + p_S$$ $p_{Sl} = 0.046\,\frac{t}{m}$

If $p_{S1} < 0$ take $p_{S1} = 0$

Step 3: $$new\_f_1 := f_{OA} + (f_l - f_{OA})\cdot\left(\frac{g_{OA}}{p_{Ml}\cdot\cos(\gamma_l) - p_{Sl}\cdot\cos(\alpha_1)}\right)$$ $new\_f_1 = 4.887\,m$

$$new\_x := x\cdot\frac{W_{wind}\cdot\left(0.232 + 0.0075\cdot\frac{h_T}{m}\right)}{p_{Ml}\cdot\sin(\gamma_l) + p_{Sl}\cdot\sin(\alpha_1)}$$ Messed with units here $new\_x = 0.556\,m$

$$x_{redo} := \frac{x + new\_x}{2}$$ $x_{redo} = 0.556\,m$

Step 4: if $|new\_f_1 - old\_f_1| \geq 0.005$ go to step 1 with f_1 = new f_1

$|new\_f_1 - old\_f_1| < 0.005$ stop calculation

All values are absolute values; disregard "-" signs

$$new\_f_1 - f_1 = \begin{pmatrix} 0.112 \\ 0.848 \\ 0.652 \\ 0.701 \\ 0.688 \end{pmatrix} m$$

> 0.005

< 0.005

Use first f1 to pass check, rounded to :

$f_h := 3.66m$ (rounded f_1 still passes check with a value of 0.002 < 0.005)

$f_1 := new\_f_1$ $f_1 = 4.887\,m$

4 cont.) Additional load on cables due to the wind is determined as follows:

$$\Delta P_{M1} := \frac{8\cdot f_1\cdot E\cdot A_M\cdot\left(L_{M1}-L_{MOA}\right)}{1.04\cdot L_{MOA}\cdot l^2} \qquad \Delta P_{M1} = 0.031\,\frac{ton}{m}$$

$$\Delta H := \frac{\Delta P_{M1}\cdot l^2}{8\cdot f_1} \qquad \Delta H = 1.304\,ton$$

$$\Delta L := \frac{1.04\cdot\Delta H}{E\cdot A_M}\cdot L_{MOA} \qquad \Delta L = 5.81\times 10^{-3}\,m$$

5.) Calculate the final data for loading case A

$$G_T := 0.4ton + 0.01\,\frac{ton}{m^2}\cdot h_T^{\,2} \qquad G_T = 0.94\,ton$$

$$t := \frac{t}{ton} \qquad g_2 := g\cdot\frac{s^2}{m} \qquad f_1 := \frac{f_1}{m} \qquad l_2 := \frac{l}{m}$$

$$V := \frac{p_{Ml}\cdot l}{2}\cdot\cos(\gamma_l)\cdot\left(1+\frac{l_2\cdot t\cdot g_2\cdot\beta_f}{4\cdot f_1\cdot\cos(\gamma_l)}\right) \qquad V = 71412.255\,lb$$

$$H_W := \frac{p_{Ml}\cdot l}{2}\cdot\sin(\gamma_l) \qquad H_W = 0.258\,ton$$

$$P_1 := \frac{V}{2}+\frac{G_T}{2}-\frac{H_W\cdot h_T}{3.5m}-\frac{1.025\cdot W_{wind}\cdot h_T^{\,2}}{3.5m} \qquad P_1 = 16.038\,ton$$

$$P_2 := \frac{V}{2}+\frac{G_T}{2}-\frac{H_W\cdot h_T}{3.5m}+\frac{1.025\cdot W_{wind}\cdot h_T^{\,2}}{3.5m} \qquad P_2 = 19.526\,ton$$

$$P_H := H_W + 2.05\cdot W_{wind}\cdot h_T + \frac{p_{Sl}\cdot l}{2}\cdot\sin(\alpha_1) \qquad P_H = 2.303\,ton$$

$$T_{Sv} := \frac{p_{Sl}\cdot l}{2}\cdot\cos(\alpha_1) \qquad T_{Sv} = 0.948\,ton$$

$$T_{Sh} := \frac{p_{Sl}\cdot l^2}{8\cdot c_l} \qquad T_{Sh} = 7.015\,ton$$

Tower Weight:

MC: Moisture Content in %
G: Specific Gravity as to NDS (Assuming White Oak characteristics)
Dwood: Density of wood in pcf

$G := 0.73$

$MC := 25$

$D_{wood} := 62.4\left[\frac{G}{1 + G\cdot(0.009)\cdot(MC)}\right]\cdot\left(1 + \frac{MC}{100}\right)$ $\quad D_{wood} = 48.907 \quad D_{wood} := D_{wood}\cdot\frac{lb}{ft^3}$

$D_{wood} = 48.907\,\frac{lb}{ft^3}$

$L_T := (7.35m - 2ft)\cdot 2 + (1m + 2ft) + 3m \quad L_T = 59.352\,ft$ ***L.T = Length of wood needed to construct one complete tower set-up.**

$G_T := L_T\cdot 12in\cdot 12in\cdot D_{wood} \quad G_T = 2902.715\,lb$

Loading placed upon foundation column: $T_T := G_T + V \qquad T_T = 74314.97\,lb$

<u>TOWER SPECIFICATION CHECKS: The following are various timber checks using NDS-05 code</u>

Column Specs:
12" X 12"
White Oak Characteristics

$k := 1000lb$

Column Design Checks: NDS 2005

$h_T := 7.35m$ -Tower Height

$B_{horzbeam} := 3m$

$d := 11.5in$ $b := 11.5in$ -Effective width and depth of wooden beams

$L := h_T - 2 \cdot d$ $L = 22.198\,ft$ -Length of vertical wooden beam

$k_e := 1.2$ NDS Appendix G: Table G1

$L_e := L \cdot k_e$ $L_e = 26.637\,ft$

$SR := \frac{L_e}{d}$ $SR = 27.795$ -Slenderness Ratio Requires: SR < 50

COMPRESSION DESIGN

$F_C > f_c$

$F_c := 825 \frac{lb}{in^2}$ NDS Table 4D: White Oak #1

$C_D := 0.9$ NDS Table 2.3.2

$C_M := 0.91$ NDS Table 4D Adjustment Factor

$C_t := 1.0$ NDS Table 2.3.3

$C_F := 1.0$ NDS Table 4D Adjustment Factor

$C_i := 1.0$ NDS 4.3.8

$C_T := 1.0$ NDS 4.4.2

Cp Calc: NDS 3.7.1

$E_{min} := 370000 \frac{lb}{in^2}$ NDS Table 4D: White Oak #2

$E_{MIN} := E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T \quad E_{MIN} = 336700 \frac{lb}{in^2}$

$F_{cstar} := F_c \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i \quad F_{cstar} = 675.675 \frac{lb}{in^2}$

$F_{cE} := \frac{0.822 \cdot E_{MIN}}{SR^2} \quad F_{cE} = 358.243 \frac{lb}{in^2}$

$c := 0.8$ NDS 3.7.1

$$C_p := \frac{1 + \left(\frac{F_{cE}}{F_{cstar}}\right)}{2 \cdot c} - \sqrt{\left[\frac{1 + \left(\frac{F_{cE}}{F_{cstar}}\right)}{2 \cdot c}\right]^2 - \frac{\frac{F_{cE}}{F_{cstar}}}{c}} \qquad C_p = 0.454$$

$F_C := F_c \cdot \left(C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i \cdot C_p\right) \quad F_C = 307.078 \frac{lb}{in^2}$

$P := 35000 lb$

$A := b \cdot d$

$f_c := \frac{P}{A} \qquad f_c = 264.65 \frac{lb}{in^2} \qquad F_C > f_c$ *Checks out

BEARING DESIGN OF TOP BEAM / BOTTOM BEAM @ POST:

$F_{Cperp} > f_{cperp}$ REQUIRED

$C_b := 1.0$ NDS 3.10.4

$F_{cperp} := 800 \frac{lb}{in^2}$

$F_{Cperp} := F_{cperp} \cdot C_M \cdot C_t \cdot C_i \cdot C_b$ $F_{Cperp} = 728 \frac{lb}{in^2}$

$f_{cperp} := f_c$ $f_{cperp} = 264.65 \frac{lb}{in^2}$ $F_{Cperp} > f_{cperp}$ *Passes Check

BEARING DESIGN: POST

$\frac{3}{4} F_{cstar} > f_c$ REQUIRED

$0.75 \cdot F_{cstar} = 506.756 \frac{lb}{in^2}$

$f_c = 264.65 \frac{lb}{in^2}$ $\frac{3}{4} F_{cstar} > f_c$ *Passes check so metal bearing plate is not required

BENDING/FLEXURE CHECK:

$F_b := 1050 \frac{lb}{in^2}$ NDS Table 4D: White Oak #1

$C_D = 0.9$

$C_M = 0.91$

$C_t = 1$

$C_L := 1.0$ NDS 3.3.3

$C_F = 1$

$C_{fu} := 1.0$ NDS 4.3.7

$C_i = 1$

$C_r := 1.0$ NDS 4.3.9

$F_B := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r$ $F_B = 859.95 \frac{lb}{in^2}$

$L_u := L$

$L_E := 1.44 \cdot L_u + 3 \cdot d$

$R_B := \sqrt{L_E \cdot \frac{d}{b^2}}$ $R_B = 6.029$

$E_{MIN} = 336700 \frac{lb}{in^2}$

$F_{bE} := \frac{1.2 \cdot E_{MIN}}{R_B^{\,2}}$ $F_{bE} = 11113.994 \frac{lb}{in^2}$

Actual Bending Stress:

$V_{wind} := 50 \frac{mile}{hr}$

$q_s := 0.00256 \cdot V_{wind}^{\,2}$ $q_s := 6.4 \frac{lb}{ft^2}$ Fundamentals of Structural Analysis (Leet, Gilbert, Uang) Eq. 2.4a

$I := 0.87$

$K_z := 1.17$

$K_{zt} := 1.0$

$K_d := 0.85$

$q_z := q_s \cdot I \cdot K_z \cdot K_{zt} \cdot K_d$ $q_z = 27.03 \frac{lb}{ft^2}$ Fundamentals of Structural Analysis (Leet, Gilbert, Uang) Eq. 2.6 and Tables 2.4, 2.5

$G := 0.85$

$C_{pWind} := 0.8$ $C_{pLeeward} := -0.5$ $C_{pSide} := -0.7$

$p_{windwall} := q_z \cdot G \cdot C_{pWind}$ $p_{windwall} = 0.026 \frac{lb}{in^2}$

$p_{Leewardwall} := q_z \cdot G \cdot C_{pLeeward}$ $p_{Leewardwall} = -0.016 \frac{lb}{in^2}$

$p_{Sidewall} := q_z \cdot G \cdot C_{pSide}$ $p_{Sidewall} = -0.023 \frac{lb}{in^2}$

$p_{wind} := p_{windwall} - p_{Leewardwall}$ $p_{wind} = 0.042 \dfrac{lb}{in^2}$

$f_b := p_{wind}$

$F_b > f_b$ Check Passes

Area of Bridge From N-S View:

$horzbeam := b \cdot d$ $vertbeam := (h_T - 2 \cdot d) \cdot b$

$A_{towerNS} := 2 \cdot (horzbeam) + (vertbeam)$ $A_{towerNS} = 23.109\,ft^2$

Main and spanning cables

$A := 32mm \cdot \dfrac{153m + 72m}{2}$ $A = 38.75\,ft^2$

Hanger beams

$B := 4in \cdot 6in \cdot 27$ $B = 4.5\,ft^2$

Walkway

$C := 2in \cdot \dfrac{72m}{2}$ $C = 19.685\,ft^2$

Suspenders

$D := 27 \cdot 10mm \cdot 2m$ $D = 5.813\,ft^2$

$A_{NS} := (A + B + C + D)$ $A_{NS} = 68.748\,ft^2$ Area of bridge exposed to wind from the North

Area of Tower From E-W View:

$A_{towerEW} := 2vertbeam$

$A_{horzbeamsEW} := 2 \cdot (d \cdot B_{horzbeam})$

$A_{EW} := A_{towerEW} + A_{horzbeamsEW}$ $A_{EW} = 61.41\,ft^2$ Area of bridge exposed to wind from the East

COMBINED BENDING AND AXIAL COMPRESSION

$F_{B1} := F_B$ $F_{B2} := F_B$ $F_{cE1} := F_{cE}$ $F_{cE2} := F_{cE}$ $f_{b1} := f_b$ $f_{b2} := f_b$

$$\left(\frac{f_c}{F_C}\right)^2 + \frac{f_{b1}}{F_{B1} \cdot \left[1 - \left(\frac{f_c}{F_{cE1}}\right)\right]} + \frac{f_{b2}}{F_{B2} \cdot \left[1 - \left(\frac{f_c}{F_{cE2}}\right) - \left(\frac{f_{b1}}{F_{bE}}\right)^2\right]} = 0.743$$

Value is less than 1.0 so this wood section passes this check.

GEOMETRY FOR CONNECTIONS:

$D := 1.0625in$

BOLTED CONNECTION GEOMETRY FOR BASE BEAM:

$D_{edgeBB} := 1.5 \cdot D$ $\quad D_{edgeBB} = 1.594\,in$

$D_{endBB} := 4 \cdot D$ $\quad D_{endBB} = 4.25\,in$ NDS Table 11.5.1E

$SpacingBB := 4 \cdot D$ $\quad SpacingBB = 4.25\,in$

BOLTED CONNECTION GEOMETRY FOR GUSSET PLATES:

$D_{edgeGP} := 1.5D$ $\quad D_{edgeGP} = 1.594\,in$

$D_{endGP} := 4 \cdot D$ $\quad D_{endGP} = 4.25\,in$ NDS Table 11.5.1A-D

$Spacing_{inrow} := 4 \cdot D$ $\quad Spacing_{inrow} = 4.25\,in$

$Spacing_{betweenrows} := 1.5D$ $\quad Spacing_{betweenrows} = 1.594\,in$

Distance from center of base beam to perpendicular bolt line: $D_{hbl} := 3.75in$

Tributary Area of each Anchor Bolt on Base Beam: $A_{tribAB} := \frac{b}{2} \cdot SpacingBB$ $\quad A_{tribAB} = 24.437\,in^2$

Yield Limits Equations Gusset Plate Bolts:

$L_m := d$

$L_s := \frac{3}{8} in$

$F_{em} := 8200 \frac{lb}{in^2}$ NDS Table 11.3.2

$F_{es} := 6300 \frac{lb}{in^2}$ NDS Table 11.3.2

$F_{yb} := 45000 \frac{lb}{in^2}$

$\theta := 90$

$K_\theta := 1 + 0.25\left(\frac{\theta}{90}\right)$

$R_{d1} := 4 \cdot K_\theta$

$R_{d2} := 3.2 \cdot K_\theta$

$R_e := \frac{F_{em}}{F_{es}}$ $R_e = 1.302$

$k_3 := -1 + \sqrt{2\frac{1 + R_e}{R_e} + \frac{2 \cdot F_{yb} \cdot (2 + R_e) \cdot D^2}{3 \cdot F_{em} \cdot L_s^2}}$ $k_3 = 9.025$

$Z_{Im} := \frac{D \cdot L_m \cdot F_{em}}{R_{d1}}$ $Z_{Im} = 20038.75\,lb$

$Z_{Is} := \frac{2 \cdot D \cdot L_s \cdot F_{es}}{R_{d1}}$ $Z_{Is} = 1004.062\,lb$ CONTROLS

$Z_{IIIs} := \frac{2 \cdot k_3 \cdot D \cdot L_s \cdot F_{em}}{(2 + R_e) \cdot R_{d2}}$ $Z_{IIIs} = 4465.565\,lb$

$Z_{IV} := \frac{2 \cdot D^2}{R_{d2}} \cdot \sqrt{\frac{2 \cdot F_{em} \cdot F_{yb}}{3(1 + R_e)}}$ $Z_{IV} = 5835.549\,lb$

Allowable Capacity = 1000 lbs / bolt

Area of Tower From E-W View Subjected to wind (for gusset plate moment calc):

$A_{towerEW} := 2vertbeam$

$A_{horzbeamsEW} := (d \cdot B_{horzbeam})$

$A_{EWgp} := A_{towerEW} + A_{horzbeamsEW}$ $A_{EWgp} = 51.978\,ft^2$ Area of bridge exposed to wind from the East

Vertical Beam Moment:

$M_{Vb} := 2\cdot\left[h_T\cdot\left(f_b\cdot d\right)\cdot\left(h_T + d\right)\right]$ $M_{Vb} = 7090.6\,lb\cdot ft$

Top Beam Moment:

$M_{Tb} := d\cdot\left(f_b\cdot B_{horzbeam}\right)\left(\frac{d}{2} + h_T + d\right)$ $M_{Tb} = 1474.716\,lb\cdot ft$

Total Tower Moment:

$M_{EWgp} := M_{Vb} + M_{Tb}$ $M_{EWgp} = 8565.317\,lb\cdot ft$

$D_{gp} := \frac{11.5}{2}in$

Required Capacity for Anchor Bolts located in Base Beam:

$P_{EWgp} := \frac{M_{EWgp}}{D_{gp}}$ $P_{EWgp} = 17875.444\,lb$

$Z_{perp} := 3000lb$ *Value is the perpendicular load a 1" bolt can withstand from NDS-05 Table 11G. Value chosen is lower to account as a safety factor.

$N_{bolts} := \frac{M_{EWgp}}{Z_{perp}\cdot D_{gp}}$ $N_{bolts} = 5.958$

***Atleast 6 Gusset Plate Bolts are required**

ANCHOR BOLT DESIGN: Using UNC 1"ϕ bolts

Required Moment Capacity in E-W direction for Base Beam Anchor Bolts:

Base Beam Moment:

$M_{Bb} := d\cdot\left(f_b\cdot B_{horzbeam}\right)\left(\frac{d}{2}\right)$ $\quad$ $M_{Bb} = 27.655\,lb\cdot ft$

Vertical Beam Moment:

$M_{Vb} := 2\cdot\left[h_T\cdot\left(f_b\cdot d\right)\cdot\left(h_T + d\right)\right]$ $\quad$ $M_{Vb} = 7090.6\,lb\cdot ft$

Top Beam Moment:

$M_{Tb} := d\cdot\left(f_b\cdot B_{horzbeam}\right)\left(\frac{d}{2} + h_T + d\right)$ $\quad$ $M_{Tb} = 1474.716\,lb\cdot ft$

Total Tower Moment:

$M_{EW} := M_{Bb} + M_{Vb} + M_{Tb}$ $\quad$ $M_{EW} = 8592.972\,lb\cdot ft$

$M_{NS} := A_{NS}\cdot f_b\cdot\frac{h_T}{2}$

$M_{NS} = 5071.85\,lb\cdot ft$

*** Mew>Mns and Mew has a smaller moment arm, so Mew will control in the design. Therefore, there is no need to check the number of anchor bolts required for the N-S moment.**

Required Capacity for Anchor Bolts located in Base Beam:

$P_{AB} := \frac{M_{EW}}{D_{hbl}}$ $\quad$ $P_{AB} = 27497.51\,lb$

Anchor Bolt Properties:

$\varepsilon_{AB} := 3.14\,\frac{in}{in}$

$\sigma_{concreteboltbond} := 200\,\frac{lb}{in^2}$

$\varepsilon_{AB}\cdot\sigma_{concreteboltbond} = 628\,\frac{lb}{in^2}$

Depth of Anchor Bolts in concrete: $Depth_{AB} := 12in$

$Diameter_{AB} := 1in$

$A_{ABcapacity} := Depth_{AB}\cdot Diameter_{AB}$ $\quad$ $A_{ABcapacity} = 12\,in^2$

Capacity each Anchor Bolt can withstand:

$P_{ABcapacity} := \varepsilon_{AB}\cdot\sigma_{concreteboltbond}\cdot A_{ABcapacity}$ $\quad$ $P_{ABcapacity} = 7536\,lb$

Moment from Anchor Bolts placed at center of base beam:

$M_{ABolt} := P_{ABcapacity}\cdot\frac{b}{2}$

$M_{ABolt} = 3611\,lb\cdot ft$

Number of Anchor Bolts required:

$\frac{M_{EW}}{M_{ABolt}} = 2.38$ $\quad$ **3 Anchor Bolts are required. 4 will be used.**

Appendix I

Cable Design

Cable Design: Variable Identification List

l	Bridge Span, distance between tower axes
f	Sag, vertical distance from tower saddle to the lowest point of the main cable
c	Camber, vertical distance from the spanning cable anchorage (top of the walkway and tower foundation minus 0.25m) and the highest point of the spanning cable
h_t	Tower height, vertical distance between top of walkway and the tower foundation and saddle
β	Frontstay cable inclination at saddle
β_f	Backstay cable inclination = frontstay cable inclination at saddle for full load
β_{fo}	Initial approximation of β_f
D_R, D_L	Backstay distance for main cable, distance between tower axis and hinge of the main cable anchorage
D	$= D_R + D_L$
L	Length of main cables between saddles
L_{total}	Total main cable length between main cable anchorages
Δf	Increase/decrease of f due to changing load
$\Delta D_R, \Delta D_L$	Increase/decrease of D_R, D_L due to changing load
ΔD	Increase/decrease of D due to changing load
ΔL	Increase/decrease of L due to changing load
ϕ	Cable Diameter
ϕ_M	Diameter of main cables
ϕ_S	Diameter spanning cables
n	Number of main cables
H	Horizontal component of the main cable tension (all main cables)
T	Main cable tension at saddle for frontstay (all main cables)
T_{perm}	Permissible cable tension
V	Vertical component of the frontstay main cable tension (all main cables)
V_{max}	Vertical component of the backstay main cable tension (all main cables)
P_s	Pretension of spanning cables (both spanning cables) in [t]
p_s	Pretension of spanning cables expressed as equally distributed load in [t/m]
A_M	Total sectional area of main cables
E	Modulus of elasticity of cables
g	Load in [t/m]

p	Live load in [t/m]
a, b	Constants in the formula for the calculation of sag increase/decrease

Indices:

h	hoisting load
d	dead load
f	full load
1	load case 1, either full or hoisting load
R	right bank
L	left bank

Sag calculations for level bridge, p.170 of
Survey, Design, Construction of Trail Suspension Bridges for Remote Areas by Grob, et al.

Define metric ton and kips $t := 1000kg$ $k := 1000lb$ Red = check/revise Blue = manual input

Define span and correct

$l_i := 40m$

$i := \frac{l_i - 2.20m}{2.40m}$ $i = 15.75$ $i := 16$ (rounded up)

$l := 2.40m \cdot i + 2.20m$ $l = 40.6\,m$

Dead load camber

$c_d := 0.03 \cdot l$ $c_d = 1.218\,m$

Tower height, number and diameter of main cables

1) Assume tentative dead load sag

$f_d := 0.12 \cdot l$ $f_d = 4.872\,m$

2) Calculate theoretical tower height

$h_T := f_d + c_d + 1.05m$ $h_T = 7.14\,m$

3) Select the nearest available tower heights above and below the theoretical tower height, using Table 45: Standard Towers (p.171)

Standard Tower Number	Total Tower Height (m)	Main Cables Number	Diam. (mm)	Diam. of Span. Cables (mm)	c/c1 (m)	c/c2 (mm)
1	5.50	2	26/32/36/40	26	3.50	383
2	7.35	2	26/32/36/40	26	3.50	383

$h_{T1} := 5.50m$

$h_{T2} := 7.35m$

4) Calculate effective dead load sag for both possibilities

$f_{d1} := h_{T1} - 1.05m - c_d$ $f_{d1} = 3.232\,m$

Check: $0.09 \cdot l = 3.654\,m$ < f_{d1} < $0.137 \cdot l = 5.562\,m$ ==> Not OK

$f_{d2} := h_{T2} - 1.05m - c_d \qquad f_{d2} = 5.082\,m$

Check: $0.09 \cdot l = 3.654\,m$ < f_{d2} < $0.137 \cdot l = 5.562\,m$ ==> OK

5) Calculate full load cable tension for both possiblities

Interpolate gf from Table 46: Approximate Full Load gf, p.172 (manual input: l = 40.6)

$vx := \begin{pmatrix} 25 \\ 50 \end{pmatrix} \qquad vy := \begin{pmatrix} 0.61 \\ 0.615 \end{pmatrix} \qquad g_f := \text{linterp}(vx, vy, 40.6)\frac{t}{m} \qquad g_f = 0.613\frac{t}{m}$

$$T_{f1} := \frac{g_f \cdot l^2}{8.4 \cdot f_{d1}} \cdot \sqrt{1 + 17.64 \cdot \left(\frac{f_{d1}}{l}\right)^2} \qquad T_{f1} = 39.252\,t$$

$$T_{f2} := \frac{g_f \cdot l^2}{8.4 \cdot f_{d2}} \cdot \sqrt{1 + 17.64 \cdot \left(\frac{f_{d2}}{l}\right)^2} \qquad T_{f2} = 26.747\,t$$

Corresponds to h_T = 7.35m - use this value for checks

6) Select required number and diameter of main cables for both possibilities with the condition:

$T_f \le T_{perm}$

ϕM: 26, 32, 36, or 40mm

n: 2, 4, 6, or 8

Select tower height based on above check for fd

$\boxed{h_T := 7.35m}$

Select ϕM and n to satisfy Tf < Tperm (Table 52: Permissible Tensions for Main Cables, p.180)

$\boxed{\phi_M := 32mm} \qquad \phi_M = 1.26\,in$ Bridges to Prosperity can donate 1.25in diameter steel cable

$\boxed{n := 2}$

using ***cable terminal with thimbles and bulldog grips***, Tperm = 30.6t > $T_f := 26.747t$ ==> OK

7) Draw approximate location of the main cable anchorages in the General Arrangement with:

$tg\beta_{fo} := \frac{4.2 \cdot f_d}{l} \qquad tg\beta_{fo} = 0.504 \qquad 0.504rad = 28.877\,deg$

$21 \cdot 8in + 0.5m = 4.767\,m \qquad h_{FE} := 4.77m$

$14 \cdot 8in + 0.5m = 3.345\,m \qquad h_{FW} := 3.35m$

Input designed footing and abutment height (# of masonry courses) x (height CMU) + concrete top for anchors

East bank (island): $D_E := \frac{h_T + h_{FE}}{\tan(tg\beta_{fo})} \qquad D_L := D_E \qquad D_L = 21.976\,m$

West bank (mainland): $D_W := \dfrac{h_T + h_{FW}}{\tan(tg\beta_{fo})}$ $\qquad D_R := D_W \qquad D_R = 19.401\,m$

Where DL and DR are assigned looking upstream (N)

Windguy cables will not be used

Diameter and number of spanning cables (Table 45)

$\phi_s := 32mm$ ***if Bridges to Prosperity donated cable is used, ϕ_s = 32mm = 1.25in***

$n_s := 2$

Modulus of elasticity

$E := 10.5\,\dfrac{t}{mm^2}$ ***according to the manufacturer; current value is assumed***

Backstay distances DL and DR on the General Arrangement

1) Fix cable elevations of the main cable anchorages on both banks

East bank (island): $E_E := 998.8m$

West bank (mainland): $E_W := 1000.5m$

2) Determine saddle elevation = elev. on top of walkway and tower foundation + hT

$E_S := 1003.95m + h_T \qquad E_S = 1011.3\,m$

3) DL/DR = (difference in elevation) * 1/(tgβfo)

$\text{Ratio} := (E_W - E_E)\cdot\dfrac{1}{tg\beta_{fo}} \qquad \text{Ratio} = 3.373\,m$

Hoisting load, gh

$g_h := 0.00038058\cdot n\cdot\phi_M^{\,2}$ [empirical equation; manual input with ϕM (cm)]

$\phi_M = 3.2\,cm$

$g_h := 0.00038058\cdot 2\cdot 3.2^2 \qquad g_h = 0.008 \qquad g_h := 0.008\,\dfrac{t}{m}$

Dead load, gd *(windties and windguy cables not included in design)*

$g_s := 0.00038058\cdot\left(2\cdot\phi_s^{\,2}\right)$ [empirical equation; manual input with ϕσ (cm)]

$\phi_s = 3.2\,cm$ ***If BtP donated cable is used, ϕ_s = 32mm = 1.25in***

$g_s := 0.00038058\cdot 2\cdot 2.6^2 \qquad g_s = 0.005 \qquad g_s := 0.005\,\dfrac{t}{m}$

gh	0.008	t/m
walkway incl. planks	0.088	t/m
handrail, fixation cables	0.003	t/m
wiremesh netting	0.006	t/m
suspenders (average)	0.017	t/m
~~windties (average)~~	~~0.004~~	~~t/m~~
spanning cables, ~~windguy cables~~	0.005	t/m

$$g_d := (0.008 + 0.088 + 0.003 + 0.006 + 0.017 + 0.005)\frac{t}{m} \quad g_d = 0.127\,\frac{t}{m}$$

Live load, p

span < 100m: $p := 0.480\,\frac{t}{m}$

Width of walkway: $w := 1m$ $\qquad \frac{p}{w} = 98.3\,\frac{lb}{ft^2}$

Full load, gf

$g_f := g_d + p \qquad g_f = 0.607\,\frac{t}{m}$

= dead load + live load

Pretension in spanning cables

(2 cables)

$p_s := 0.10 \cdot g_d \qquad p_s = 0.013\,\frac{t}{m}$

$p_{sl} := 0.42 \cdot g_d \cdot l \qquad p_{sl} = 2.166\,t$

Design Calculations (p.175)

1) Calculate total sectional area A_M (mm^2) for all main cables (of n ϕ_M),
using Table 48: Sectional Area of Cables on p.175

$$A_M := 2\,\frac{\pi \cdot (32mm)^2}{4} \cdot 0.5278 \qquad A_M = 848.964\,mm^2$$

2) Calculate length of dead loaded main cables between saddles

$$L_d := l \cdot \left[1 + \frac{8}{3} \cdot \left(\frac{f_d}{l}\right)^2 - \frac{32}{5} \cdot \left(\frac{f_d}{l}\right)^4\right] \qquad L_d = 42.105\,m$$

Calculate length of dead loaded sag cables between saddles (slight overestimate in length - sag cables will be attached to abutments)

$$L_S := l \cdot \left[1 + \frac{8}{3} \cdot \left(\frac{c_d}{l}\right)^2 - \frac{32}{5} \cdot \left(\frac{c_d}{l}\right)^4\right] \qquad \boxed{L_S = 40.697\,m}$$

3) Calculate dead load main cable tension

$$T_d := \frac{(g_d + p_s) \cdot l^2}{8 \cdot f_d} \cdot \sqrt{1 + 16 \cdot \left(\frac{f_d}{l}\right)^2} \qquad T_d = 6.554\,t$$

$$H_d := \frac{(g_d + p_s) \cdot l^2}{8 \cdot f_d} \qquad H_d = 5.908\,t$$

4) Calculate values of a, b, and βfo

$$a := 16 \cdot \frac{f_d}{l} \cdot \left[5 - 24 \cdot \left(\frac{f_d}{l}\right)^2\right] \qquad a = 8.936$$

$$b := 15 - 8 \cdot \left(\frac{f_d}{l}\right)^2 \cdot \left[5 - 36 \cdot \left(\frac{f_d}{l}\right)^2\right] \qquad b = 14.484$$

5) Calculate full load sag and hoisting sag by the iteration method (Table 49, p.177)

Begin iteration with the following values:

$f_1 = f_f$ = approx. 1.05 * f_d
$f_1 = f_h$ = approx. 0.98 * f_d

Iteration will be accurate after ~3 repetitions

$$\beta_{fo} := \frac{4.2 \cdot f_d}{l} \qquad \beta_{fo} = 0.504 \quad \text{(radians)}$$

Step 1: $f_1 = f_f$ and $g_1 = g_f$, full load

$$f_1 := 1.05 \cdot f_d \qquad f_1 = 5.116\,m$$

$$g_1 := g_f \qquad g_1 = 0.607\,\frac{t}{m} \qquad f_1 := \begin{pmatrix} 5.116 \\ 5.188 \\ 5.184 \end{pmatrix} m$$

5.116m is the starting value for l = 40.6m; all subsequent values of f1 are input from calculated new_f1 (below)

1) $$H_1 := \frac{g_1 \cdot l^2}{8 \cdot f_1} \qquad H_1 = \begin{pmatrix} 24.447 \\ 24.107 \\ 24.126 \end{pmatrix} t$$

$$T_i := \sqrt{1 + 16 \cdot \left(\frac{f_1}{l}\right)^2} \qquad T_i = \begin{pmatrix} 1.12 \\ 1.123 \\ 1.123 \end{pmatrix}$$

$T_1 := \overrightarrow{(H_1 \cdot T_i)}$ $\qquad \vec{T_1} = \begin{pmatrix} 27.377 \\ 27.074 \\ 27.091 \end{pmatrix} t$

2) $\Delta L_1 := \dfrac{(2 \cdot H_1 + T_1) \cdot L_d}{3 \cdot E \cdot A_M} \cdot \dfrac{g_1 - g_d - p_s}{g_1}$ $\qquad \Delta L_1 = \begin{pmatrix} 0.092 \\ 0.091 \\ 0.091 \end{pmatrix} m$

$\Delta D_{R1} := \dfrac{T_1 \cdot D_R}{E \cdot A_M} \cdot \dfrac{g_1 - g_d - p_s}{g_1} + \dfrac{{g_h}^2 \cdot {D_R}^3}{24 \cdot \cos(\beta_{fo})} \cdot \left(\dfrac{1}{{H_d}^2} - \dfrac{1}{{H_1}^2} \right)$ $\qquad \Delta D_{R1} = \begin{pmatrix} 0.046 \\ 0.046 \\ 0.046 \end{pmatrix} m$

$\Delta D_{L1} := \dfrac{T_1 \cdot D_L}{E \cdot A_M} \cdot \dfrac{g_1 - g_d - p_s}{g_1} + \dfrac{{g_h}^2 \cdot {D_L}^3}{24 \cdot \cos(\beta_{fo})} \cdot \left(\dfrac{1}{{H_d}^2} - \dfrac{1}{{H_1}^2} \right)$ $\qquad \Delta D_{L1} = \begin{pmatrix} 0.053 \\ 0.052 \\ 0.052 \end{pmatrix} m$

$\Delta D_1 := \Delta D_{R1} + \Delta D_{L1}$ $\qquad \Delta D_1 = \begin{pmatrix} 0.099 \\ 0.098 \\ 0.098 \end{pmatrix} m$

3) $\Delta f_1 := \dfrac{15 \cdot \Delta L_1}{a} + \dfrac{b \cdot \Delta D_1}{a}$ $\qquad \Delta f_1 = \begin{pmatrix} 0.316 \\ 0.312 \\ 0.313 \end{pmatrix} m$

4) $new\_f_1 := f_d + \Delta f_1$ $\qquad new\_f_1 = \begin{pmatrix} 5.188 \\ 5.184 \\ 5.185 \end{pmatrix} m$

5) if $\left| new\_f_1 - old\_f_1 \right| \geq 0.005$ go to step 1 with f_1 = new f_1

$\left| new\_f_1 - old\_f_1 \right| < 0.005$ stop calculation

All values are absolute values; disregard "-" signs

$new\_f_1 - f_1 = \begin{pmatrix} 0.072 \\ -0.004 \\ 0.001 \end{pmatrix} m$ $\qquad$ > 0.005 < 0.005

Use first f1 that passes check:

$f_f := 5.184 m$

Step 2: $f_1 = f_h$ and $g_1 = g_h$, hoisting load

$f_1 := 0.98 \cdot f_d$ $\qquad f_1 = 4.775\, m$

$g_1 := g_h \qquad g_1 = 0.008\,\frac{t}{m} \qquad f_1 := \begin{pmatrix} 4.775 \\ 4.039 \\ 4.235 \\ 4.186 \\ 4.199 \end{pmatrix} m$

4.775m is the starting value for l = 40.6m; all subsequent values of f1 are input from calculated new_f1 (below)

1) $$H_1 := \frac{g_1 \cdot l^2}{8 \cdot f_1} \qquad H_1 = \begin{pmatrix} 0.345 \\ 0.408 \\ 0.389 \\ 0.394 \\ 0.393 \end{pmatrix} t$$

$$T_i := \sqrt{1 + 16 \cdot \left(\frac{f_1}{l}\right)^2} \qquad T_i = \begin{pmatrix} 1.105 \\ 1.076 \\ 1.084 \\ 1.082 \\ 1.082 \end{pmatrix}$$

$$T_1 := \overrightarrow{\left(H_1 \cdot T_i\right)} \qquad T_1 = \begin{pmatrix} 0.381 \\ 0.439 \\ 0.422 \\ 0.426 \\ 0.425 \end{pmatrix} t$$

2) $$\Delta L_1 := \frac{\left(2 \cdot H_1 + T_1\right) \cdot L_d}{3 \cdot E \cdot A_M} \cdot \frac{g_1 - g_d - p_s}{g_1} \qquad \Delta L_1 = \begin{pmatrix} -0.028 \\ -0.033 \\ -0.031 \\ -0.031 \\ -0.031 \end{pmatrix} m$$

$$\Delta D_{R1} := \frac{T_1 \cdot D_R}{E \cdot A_M} \cdot \frac{g_1 - g_d - p_s}{g_1} + \frac{{g_h}^2 \cdot {D_R}^3}{24 \cdot \cos\left(\beta_{fo}\right)} \cdot \left(\frac{1}{{H_d}^2} - \frac{1}{{H_1}^2}\right) \qquad \Delta D_{R1} = \begin{pmatrix} -0.2 \\ -0.149 \\ -0.161 \\ -0.158 \\ -0.159 \end{pmatrix} m$$

$$\Delta D_{L1} := \frac{T_1 \cdot D_L}{E \cdot A_M} \cdot \frac{g_1 - g_d - p_s}{g_1} + \frac{{g_h}^2 \cdot {D_L}^3}{24 \cdot \cos\left(\beta_{fo}\right)} \cdot \left(\frac{1}{{H_d}^2} - \frac{1}{{H_1}^2}\right) \qquad \Delta D_{L1} = \begin{pmatrix} -0.286 \\ -0.211 \\ -0.23 \\ -0.225 \\ -0.226 \end{pmatrix} m$$

$$\Delta D_1 := \Delta D_{R1} + \Delta D_{L1}$$

$$\Delta D_1 = \begin{pmatrix} -0.485 \\ -0.36 \\ -0.391 \\ -0.383 \\ -0.385 \end{pmatrix} m$$

3) $$\Delta f_1 := \frac{15 \cdot \Delta L_1}{a} + \frac{b \cdot \Delta D_1}{a}$$

$$\Delta f_1 = \begin{pmatrix} -0.833 \\ -0.637 \\ -0.686 \\ -0.673 \\ -0.677 \end{pmatrix} m$$

4) $$new\_f_1 := f_d + \Delta f_1$$

$$new\_f_1 = \begin{pmatrix} 4.039 \\ 4.235 \\ 4.186 \\ 4.199 \\ 4.195 \end{pmatrix} m$$

5) if $\left| new\_f_1 - old\_f_1 \right| \geq 0.005$ go to step 1 with f_1 = new f_1

$\left| new\_f_1 - old\_f_1 \right| < 0.005$ stop calculation

All values are absolute values; disregard "-" signs

$$new\_f_1 - f_1 = \begin{pmatrix} -0.736 \\ 0.196 \\ -0.049 \\ 0.013 \\ -0.004 \end{pmatrix} m$$

> 0.005

< 0.005

Use first f1 that passes check: $f_h := 4.199 m$

Calculate hoisting load main cable tension

$$T_h := \frac{(g_h + p_s) \cdot l^2}{8 \cdot f_h} \cdot \sqrt{1 + 16 \cdot \left(\frac{f_h}{l}\right)^2} \qquad T_h = 1.099\,t$$

$$H_h := \frac{(g_h + p_s) \cdot l^2}{8 \cdot f_h} \qquad H_h = 1.016\,t$$

Calculate final data METRIC ENGLISH

1) Cable inclination at saddles

$$\beta_f := \frac{4 \cdot f_f}{l}$$ $\boxed{\beta_f = 0.511}$ $\boxed{\beta_f = 29.263\,\text{deg}}$

2) Cable tensions in main cables (all main cables)

$$T_f := \frac{g_f \cdot l^2}{8 \cdot f_f} \cdot \sqrt{1 + 16 \cdot \left(\frac{f_f}{l}\right)^2}$$ $\boxed{T_f = 27.1\,\text{t}}$ $\boxed{T_f = 59.7\,\text{k}}$

$$T_h := \frac{g_h \cdot l^2}{8 \cdot f_h} \cdot \sqrt{1 + 16 \cdot \left(\frac{f_h}{l}\right)^2}$$ $\boxed{T_h = 0.425\,\text{t}}$ $\boxed{T_h = 0.937\,\text{k}}$

3) Displacement of saddles (for cable hoisting)

$$\Delta D_R := \frac{T_h \cdot D_R}{E \cdot A_M} \cdot \frac{g_h - g_d - p_s}{g_h} + \frac{g_h^{\,2} \cdot D_R^{\,3}}{24 \cdot \cos(\beta_f)} \cdot \left(\frac{1}{H_d^{\,2}} - \frac{1}{H_h^{\,2}}\right)$$ $\boxed{\Delta D_R = -0.036\,\text{m}}$ $\boxed{\Delta D_R = -1.43\,\text{in}}$

$$\Delta D_L := \frac{T_h \cdot D_L}{E \cdot A_M} \cdot \frac{g_h - g_d - p_s}{g_h} + \frac{g_h^{\,2} \cdot D_L^{\,3}}{24 \cdot \cos(\beta_f)} \cdot \left(\frac{1}{H_d^{\,2}} - \frac{1}{H_h^{\,2}}\right)$$ $\boxed{\Delta D_L = -0.048\,\text{m}}$ $\boxed{\Delta D_L = -1.88\,\text{in}}$

4) Cable length (excluding overlapping length at terminals) for one main cable

$$L_{total} := L_d + \frac{D_R + D_L}{\cos(\beta_f)}$$ $\boxed{L_{total} = 89.5\,\text{m}}$ $\boxed{L_{total} = 293.75\,\text{ft}}$

Appendix J

Suspender Design

Suspender Design: Variable Identification List

c/c_n	Distance from center of main cables to center of spanning cables for suspender No. n
l_n	Total suspender length for suspender No. n
n	Running suspender number
n_{max}	Maximum n
x_n	Distance of suspender No. n from bridge center
c	Constant in the formula for c/c_n
k	Constant in the formula for c/c_n
e	Euler Constant
N	Total number of required suspenders (for one bridge)

Suspender Calculations:

Blue signifies a comment

1) Compile l, f_d, h_t from sag calculation for dead load

$l := 40.6\ m$

$f_d := 5.082\ m$

$h_t := 7.35\ m$

2) Calculate for suspender number n = 1 to n_{max} continuously:

--Center distance of cables:

$$n_{max} := \frac{l - 4.60}{2.40} + 1$$

$n_{max} = 16$

$n := 1 .. n_{max}$

n =

1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16

$x_n := (n - 1) \cdot 1.20$

$x_n =$

0
1.2
2.4
3.6
4.8
6
7.2
8.4
9.6
10.8
12
13.2
14.4
15.6
16.8
18

$$c := \frac{8 \cdot f_d}{l^2}$$

$c = 0.025$

$$k := \frac{2 \cdot c \cdot f_d}{e^{\frac{c \cdot l}{2}} + e^{\frac{-c \cdot l}{2}} - 2}$$

$k = 0.979$

$$c\_c_n := \left[\frac{k}{2 \cdot c} \cdot \left(e^{c \cdot x_n} + e^{-c \cdot x_n} - 2\right) + \frac{4 \cdot \left(x_n\right)^2}{l^2} \cdot \left(h_t - f_d - 1.05\right) + 1.3\right] m$$

$c\_c_n =$

c_c$_n$ is equivalent to c/c$_n$ in the book

1.3	m
1.322	
1.387	
1.495	
1.647	
1.842	
2.081	
2.364	
2.691	
3.062	
3.478	
3.938	
4.444	
4.995	
5.592	
6.236	

Total suspender length:

$l_n := c\_c_n - 542mm$

$l_n =$

0.758	m
0.78	
0.845	
0.953	
1.105	
1.3	
1.539	
1.822	
2.149	
2.52	
2.936	
3.396	
3.902	
4.453	
5.05	
5.694	

$$t_l := \left[\sum_n \left(2\, l_n\right) - .758m\right]$$

$t_l = 77.641\,m$

Appendix K

Walkway Design

Walkway Design: Variable Identification List

*All variables can be found in NDS-05

Walkway Timber Checks: The following will check the actual capacity of the wood planking using the NDS-05 Code.

Wood Loads:
MC: Moisture Content in %
G: Specific Gravity as to NDS (Assuming White Oak characteristics)
Dwood: Density of wood in pcf

$G := 0.73$

$MC := 25$

$D_{wood} := 62.4\left[\frac{G}{1 + G\cdot(0.009)\cdot(MC)}\right]\cdot\left(1 + \frac{MC}{100}\right)$ $\quad D_{wood} = 48.907$ $\quad D_{wood} := D_{wood}\cdot\frac{lb}{ft^3}$

$D_{wood} = 48.907\frac{lb}{ft^3}$

$Live_{load} := 1.6\cdot 80.6\frac{lb}{ft^2}$ $\quad Live_{load} = 128.96\frac{lb}{ft^2}$

PLANKING CALCULATIONS: Using 2X4 Boards

$b := 3.5in$ $\quad d := 1.5in$

$Deck_{width} := 1.22m$

$L_P := 1.2m$

$A_P := b\cdot d$

$W_P := D_{wood}\cdot A_P + Live_{load}\cdot Deck_{width}$ $\quad W_P = 517.962\frac{lb}{ft}$

***The distributed weight acting on the planking comes out to be 518.0 lbs/ft which acts on a single plank.**

$M_P := \frac{W_P\cdot L_P^2}{8}$ $\quad M_P = 1003.553\,lb\cdot ft$

$I := \frac{Deck_{width}\cdot d^3}{12}$ $\quad I = 13.509\,in^4$

$c := \frac{d}{2}$

$S_P := \frac{I}{c}$

$f_b := \frac{M_P}{S_P}$ $\quad f_b = 668.596\frac{lb}{in^2}$

BENDING/FLEXURE CHECK:

$F_b := 1200 \frac{lb}{in^2}$ NDS Table 4D: White Oak #1

$C_D := 0.9$ $C_M := 0.91$

$C_t := 1$ $C_T := 1$

$C_{fu} := 1$ $C_F := 1$

$C_{fu} := 1.0$ NDS 4.3.7

$C_i := 1$

$C_r := 1.15$ NDS 4.3.9

$L_u := L_P$

$L_E := 1.44 \cdot L_u + 3 \cdot d$ $L_E = 1.842\,m$

$R_B := \sqrt{L_E \cdot \frac{d}{b^2}}$ $R_B = 2.98$

$E_{min} := 370000 \frac{lb}{in^2}$ NDS Table 4D: White Oak #1

$E_{MIN} := E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T$

$E_{MIN} = 336700 \frac{lb}{in^2}$

$F_{bE} := \frac{1.2 \cdot E_{MIN}}{{R_B}^2}$ $F_{bE} = 45492.788 \frac{lb}{in^2}$

$F_{bstar} := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i \cdot C_r$

$$C_L := \frac{1 + \frac{F_{bE}}{F_{bstar}}}{1.9} - \sqrt{\left(\frac{1 + \frac{F_{bE}}{F_{bstar}}}{1.9}\right)^2 - \frac{\frac{F_{bE}}{F_{bstar}}}{0.9}} \qquad C_L = 1.056$$

$F_B := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r$ $F_B = 1193.094 \frac{lb}{in^2}$

$F_b > f_b$ Check Passes

HANGERBEAMS CALCULATIONS: Using 5X6 Posts

BOLTED CONNECTION GEOMETRY:

$D := 0.5in + \frac{1}{8}in \qquad D = 0.625\,in$

*This value is the diameter of the pre-drilled hole for the suspender cable (1/2 in diameter) to fit through.

$D_{edgeHB} := 1.5 \cdot D \qquad D_{edgeHB} = 0.938\,in$

$D_{endHB} := 4 \cdot D \qquad D_{endHB} = 2.5\,in$ NDS Table 11.5.1E

$SpacingHB := 4 \cdot D \qquad SpacingHB = 2.5\,in$

$b := 4.5in \qquad d := 5.5in$

$L_{HB} := 1.3m + 0.5in + D_{endHB}$

$A_{hb} := b \cdot d$

$W_P := D_{wood} \cdot d \cdot L_P + Live_{load} \cdot L_P$

$W_{hb} := D_{wood} \cdot A_{hb} + W_P \qquad W_{hb} = 604.373\,\frac{lb}{ft}$

***Each Hangerbeam has a distributed weight of 605 lbs/ft acting upon it.**

$M_{hb} := \frac{W_{hb} \cdot L_{HB}^2}{8} \qquad M_{hb} = 1540.098\,lb \cdot ft$

$I := \frac{b \cdot d^3}{12} \qquad I = 62.391\,in^4$

$c := \frac{d}{2}$

$S_{hb} := \frac{I}{c}$

$f_b := \frac{M_{hb}}{S_{hb}} \qquad f_b = 814.597\,\frac{lb}{in^2}$

BENDING/FLEXURE CHECK:

$F_b := 1050\,\frac{lb}{in^2}$ NDS Table 4D: White Oak #1

$C_D := 0.9 \qquad C_M := 0.91$

$C_t := 1 \qquad C_T := 1$

$C_{fu} := 1 \qquad C_F := 1$

$C_{fu} := 1.0$ NDS 4.3.7

$C_i := 1$

$C_r := 1.15$ NDS 4.3.9

$L_u := L_{HB}$

$L_E := 1.44 \cdot L_u + 3 \cdot d \qquad L_E = 2.401\,m$

$R_B := \sqrt{L_E \cdot \frac{d}{b^2}} \qquad R_B = 5.067$

$E_{min} := 370000 \frac{lb}{in^2}$ NDS Table 4D: White Oak #1

$E_{MIN} := E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T$ $E_{MIN} = 336700 \frac{lb}{in^2}$

$E_{MIN} = 336700 \frac{lb}{in^2}$

$F_{bE} := \frac{1.2 \cdot E_{MIN}}{R_B^{\ 2}}$ $F_{bE} = 15738.356 \frac{lb}{in^2}$

$F_{bstar} := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i \cdot C_r$

$$C_L := \frac{1 + \frac{F_{bE}}{F_{bstar}}}{1.9} - \sqrt{\left(\frac{1 + \frac{F_{bE}}{F_{bstar}}}{1.9}\right)^2 - \frac{\frac{F_{bE}}{F_{bstar}}}{0.9}}$$ $C_L = 1.056$

$C_L := 1.0$

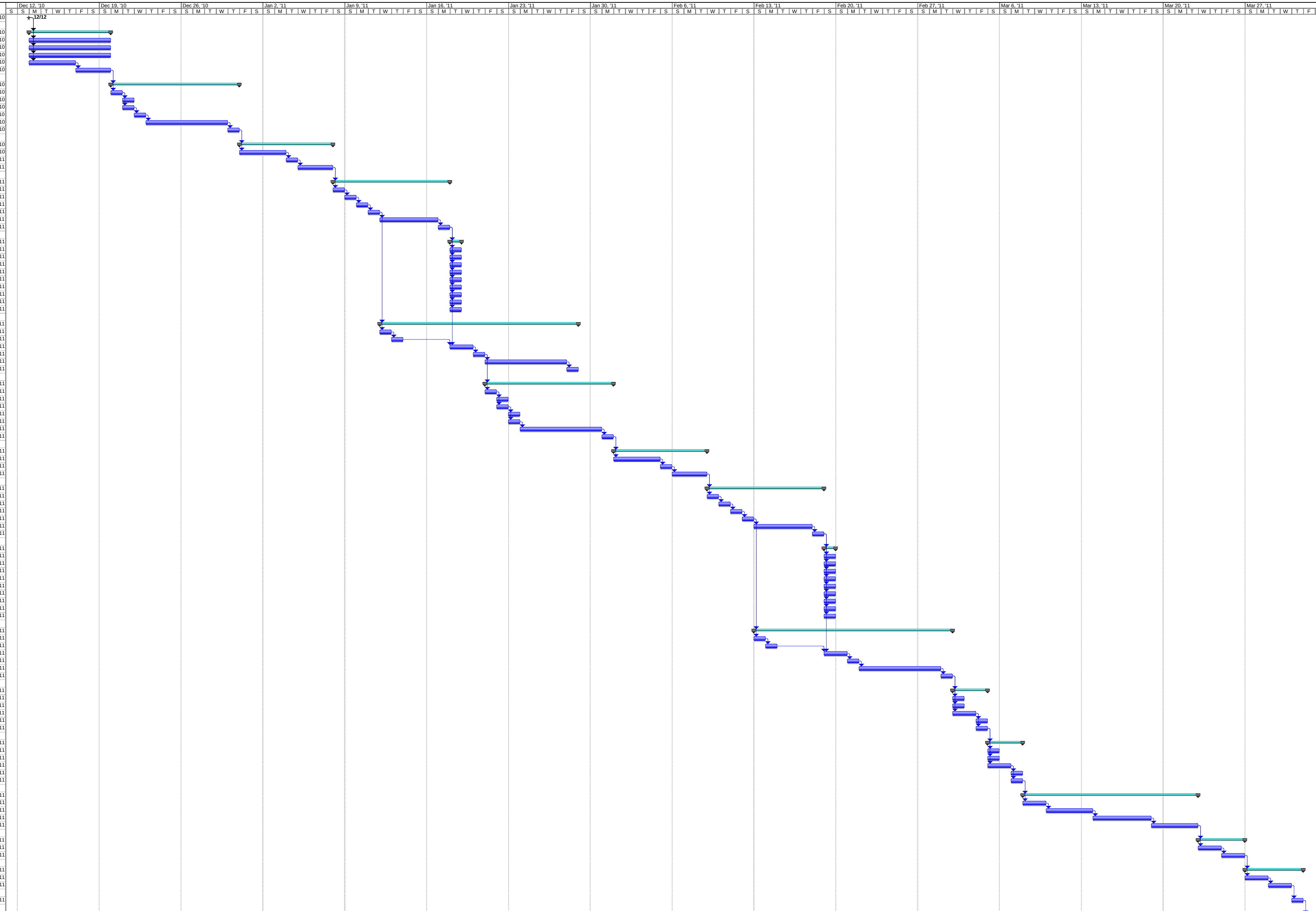
$F_B := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r$ $F_B = 988.942 \frac{lb}{in^2}$

$F_b > f_b$ Check Passes

Appendix L

Construction Schedule

ID	Task Name	Start
1	**Project Start**	Sun 12/12/10
2		
3	**Initial Preparations**	Mon 12/13/10
4	Mobilization	Mon 12/13/10
5	Initial Earthwork	Mon 12/13/10
6	Material Transportation	Mon 12/13/10
7	Site Clearing	Mon 12/13/10
8	Site Survey and Point Layout	Fri 12/17/10
9		
10	**Base Slab East**	Mon 12/20/10
11	Base Slab Excavation	Mon 12/20/10
12	Formwork Base Slab	Tue 12/21/10
13	Lay Steel Reinforcement (Grid and Vertical)	Tue 12/21/10
14	Mix and Pour Concrete for Base Slab	Wed 12/22/10
15	Cure Base Slab Concrete	Thu 12/23/10
16	Formwork Stripping and Cleaning	Thu 12/30/10
17		
18	**Foundation Column East**	Fri 12/31/10
19	Place CMU's for Masonry Columns	Fri 12/31/10
20	Grout and Reinforce	Tue 1/4/11
21	Fill Columns with Soil and Compact	Wed 1/5/11
22		
23	**Top Slab East**	Sat 1/8/11
24	Construct Forms and Place Wall Ties	Sat 1/8/11
25	Place and Tie Forms	Sun 1/9/11
26	Place Flexural Reinforcement	Mon 1/10/11
27	Mix and Pour Concrete for Top Slab	Tue 1/11/11
28	Cure Top Slab Concrete	Wed 1/12/11
29	Formwork Stripping and Cleaning	Mon 1/17/11
30		
31	**Towers East**	Tue 1/18/11
32	Cut Wood to Specifications	Tue 1/18/11
33	Place Base Timber Beam	Tue 1/18/11
34	Place Base Gusset Plates	Tue 1/18/11
35	Place Vertical Columns	Tue 1/18/11
36	Place Top Gusset Plate/Saddle	Tue 1/18/11
37	Place Top Timber Beam	Tue 1/18/11
38	Place Saddles	Tue 1/18/11
39	Hoist Complete Tower Set Up into Place	Tue 1/18/11
40	Tighten Anchor Bolts to Base Beam	Tue 1/18/11
41		
42	**Anchors East**	Wed 1/12/11
43	Anchor Earthwork	Wed 1/12/11
44	Construct Reinforcement Cage	Thu 1/13/11
45	Construct and Place Forms	Tue 1/18/11
46	Mix and Pour Concrete	Thu 1/20/11
47	Cure Anchor Concrete	Fri 1/21/11
48	Formwork Stripping and Cleaning	Fri 1/28/11
49		
50	**Base Slab West**	Fri 1/21/11
51	Base Slab Excavation	Fri 1/21/11
52	Formwork Base Slab	Sat 1/22/11
53	Lay Steel Reinforcement (Grid and Vertical)	Sat 1/22/11
54	Mix Concrete for Base Slab	Sun 1/23/11
55	Concrete Pour Base Slab	Sun 1/23/11
56	Base Slab Concrete Cure	Mon 1/24/11
57	Formwork Stripping and Cleaning	Mon 1/31/11
58		
59	**Foundation Column West**	Tue 2/1/11
60	Place CMU's for Masonry Columns	Tue 2/1/11
61	Grout and Reinforce	Sat 2/5/11
62	Fill Columns with Soil and Compact	Sun 2/6/11
63		
64	**Top Slab West**	Wed 2/9/11
65	Construct Forms and Place Wall Ties	Wed 2/9/11
66	Place and Tie Forms	Thu 2/10/11
67	Place Flexural Reinforcement	Fri 2/11/11
68	Mix and Pour Concrete for Top Slab	Sat 2/12/11
69	Cure Top Slab Concrete	Sun 2/13/11
70	Formwork Stripping and Cleaning	Fri 2/18/11
71		
72	**Towers West**	Sat 2/19/11
73	Cut Wood to Specifications	Sat 2/19/11
74	Place Base Timber Beam	Sat 2/19/11
75	Place Base Gusset Plates	Sat 2/19/11
76	Place Vertical Columns	Sat 2/19/11
77	Place Top Gusset Plate/Saddle	Sat 2/19/11
78	Place Top Timber Beam	Sat 2/19/11
79	Place Saddles	Sat 2/19/11
80	Hoist Complete Tower Set Up into Place	Sat 2/19/11
81	Tighten Anchor Bolts to Base Beam	Sat 2/19/11
82		
83	**Anchors West**	Sun 2/13/11
84	Anchor Earthwork	Sun 2/13/11
85	Construct Reinforcement Cage	Mon 2/14/11
86	Construct and Place Forms	Sat 2/19/11
87	Mix and Pour Concrete	Mon 2/21/11
88	Cure Anchor Concrete	Tue 2/22/11
89	Formwork Stripping and Cleaning	Tue 3/1/11
90		
91	**Main Cable North**	Wed 3/2/11
92	Unravel Cable From Spool	Wed 3/2/11
93	Fix Main Cable To East Anchor	Wed 3/2/11
94	Hoist Main Cable	Wed 3/2/11
95	Tension Main Cable	Fri 3/4/11
96	Connect Main Cable to West Side Turnbuckle	Fri 3/4/11
97		
98	**Main Cable South**	Sat 3/5/11
99	Unravel Cable From Spool	Sat 3/5/11
100	Fix Main Cable To East Anchor	Sat 3/5/11
101	Hoist Main Cable	Sat 3/5/11
102	Tension Main Cable	Mon 3/7/11
103	Connect Main Cable to West Side Turnbuckle	Mon 3/7/11
104		
105	**Walkway**	Tue 3/8/11
106	Measure and Cut Suspender Cables	Tue 3/8/11
107	Construct Suspender Set-Up (54)	Thu 3/10/11
108	Place Suspender Set-Ups Along Main Cables	Mon 3/14/11
109	Place Staggered Walkway Planking	Sat 3/19/11
110		
111	**Approachments West**	Wed 3/23/11
112	Approachment Earthwork	Wed 3/23/11
113	Wooden Step Construction/Placement	Fri 3/25/11
114		
115	**Approachments East**	Sun 3/27/11
116	Approachment Earthwork	Sun 3/27/11
117	Wooden Step Construction/Placement	Tue 3/29/11
118		
119	**Punch List**	Thu 3/31/11
120		
121	**Project End**	Thu 3/31/11



Project: Bridge Construction Schedule
Date: Wed 12/16/09

Task | Split | Progress | Milestone | Summary | Project Summary | External Tasks | External Milestone | Deadline

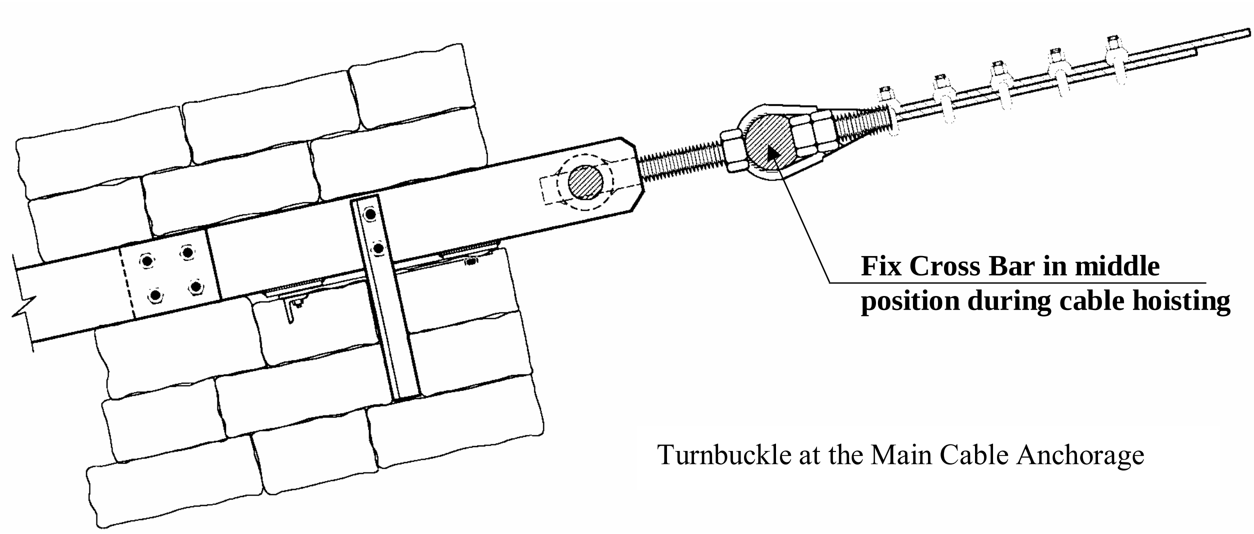
Appendix M

Selected Bridges to Prosperity Suspension Manual Pages

6.7.2 HOISTING OF MAIN CABLES AND SAG SETTING

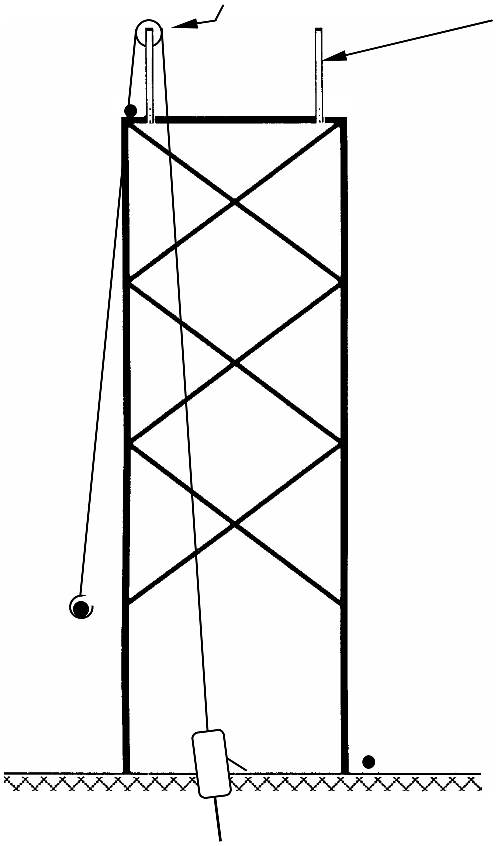
Usually, the main cables are pulled across the river with the help of nylon ropes. In case of a deep or turbulent river, attach an empty airtight plastic can (jerry can) at the end of the cable. This will prevent the cable-end from getting stuck between stones and rocks lying on the riverbed.

Make sure that the respective Main Cables are pulled on either side of the Tower and Walkway Foundations. Fix them temporarily at the respective Turnbuckle at the Main Cable Anchor.



Turnbuckle at the Main Cable Anchorage

Fix a pulley block

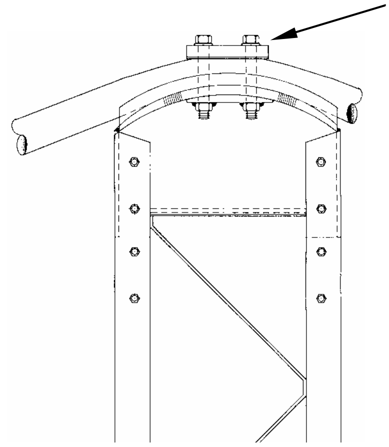


Main Cables at either side of the Tower

Temporary device for lifting the main cables (supplied with Top Element)

Lift the cables one by one, first the inner then the outer cables.

Once the cables are in the saddle groove, immediately secure them with the saddle cover plate, but do not tighten the bolts so that the cables can still slide during erection time.



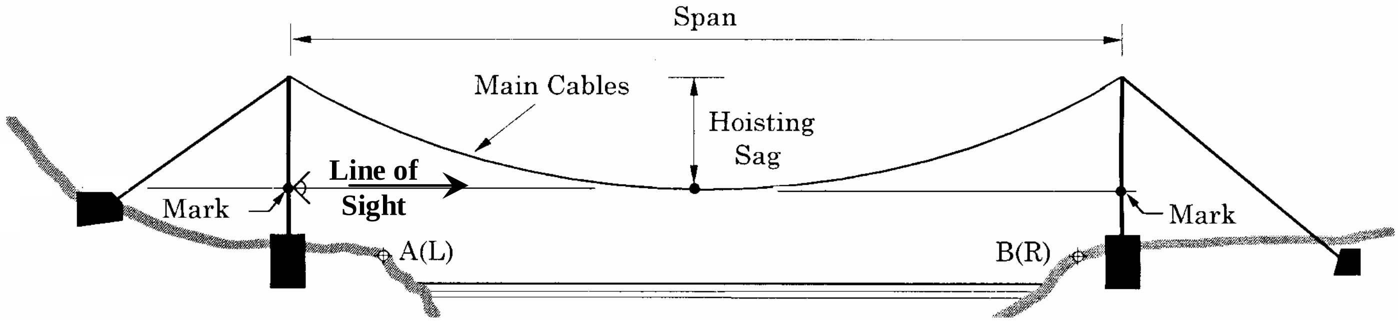
Keep saddle cover plate loose during bridge erection time, but tighten it firmly before removing the temporary side struts at the bottom of the tower.

The hoisting sag setting of the Main Cables is one of the most important tasks during the erection of the bridge.

The towers should stand exactly vertical, the saddle cover plates are loose, and the temporary side struts are fixed. With this arrangement, the main cables can slide over the saddles when the bridge is being erected and the cables become longer; and the towers remain in vertical position in dead load.

With a leveling instrument, the exact hoisting sag is fixed in the following way:

- Mark the elevation of the hoisting sag on both the towers with permanent paint.
- Now set up the leveling instrument on the tower foundation so that its line of sight matches with the mark on the tower across the river. Setting up the leveling instrument at the prescribed hoisting sag elevation has to be done by trial and error, and may require several attempts. Make use of the three adjustment wheels of the leveling instrument when the eyesight is close to the mark.
- Pull the Main Cables until they reach a level of about 20 cm higher than the hoisting sag.
- Clamp the cables around the thimbles at the cross bar of the Turnbuckle of the main cable anchorage. Make sure that the crossbar is **in the middle position** of the threaded anchor bars when clamping the main cables, secured with two nuts in the front and one in the back (see page 135).
- The Main Cables should be left in this "over pulled" position for at least 12 hours so that some relaxation can take place.
- Now move the Turnbuckles to achieve the exact sag setting. For compensating elongations due to change in air temperature, recheck the hoisting sag at different times of the day and make the necessary adjustments. It is recommended to adjust the final sag setting during the hot day after noon, when the cables have accumulated maximum heat, i.e., during maximum elongation condition.
- The hoisting sags of all the Main Cables must be identical at any point of time.



Also check the sags from time to time when the fitting works are going on. Different elongations may take place due to dissimilar hidden cable relaxations when the tension increases. Adjust possible sag differences with the help of the turnbuckles at the main cable anchor so that the Main Cables are always parallel and compare the dead load sag with the pre-calculated values.

6.7.3 HOISTING OF SPANNING CABLES

Fix the Spanning Cables at the Turnbuckles of the Tower and Walkway Anchorage on one river bank. Make sure that the crossbar of the turnbuckles are at the **outermost position** secured with two nuts each so that more tension can be applied when all the fitting work is completed.
Pull the cables across the river and secure them at the corresponding turnbuckles on the other bank (crossbar at the outermost position).

It is not necessary to achieve the sag corresponding to the required dead load camber, since this requires very high pulling forces. Just make sure that both the Spanning Cables are hanging approximately parallel and are high enough over the highest water-level of the river.
It is much easier to adjust the spanning cables when the suspender is being fitted (see Chapter 6.7.4).

6.7.4 FITTING SUSPENDERS AND CENTER ROW OF STEEL DECK

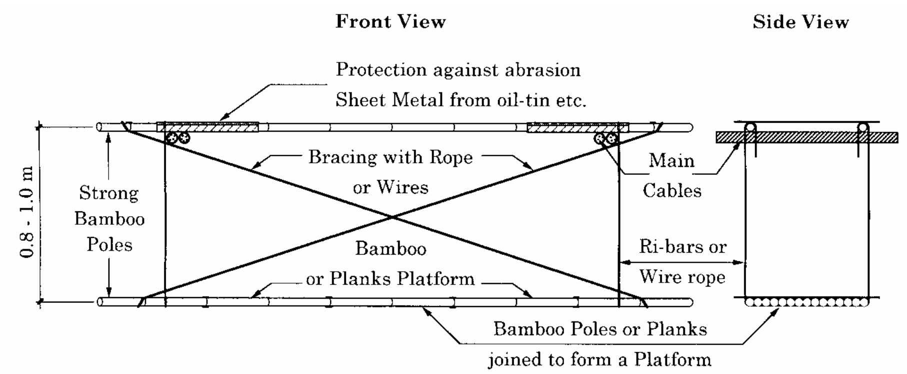
Fitting the suspenders and walkway elements is the most difficult and daring job.
As mentioned already in the beginning of this Chapter 6.7, adequate safety precautions should be strictly followed and the respective responsibilities should be clarified.

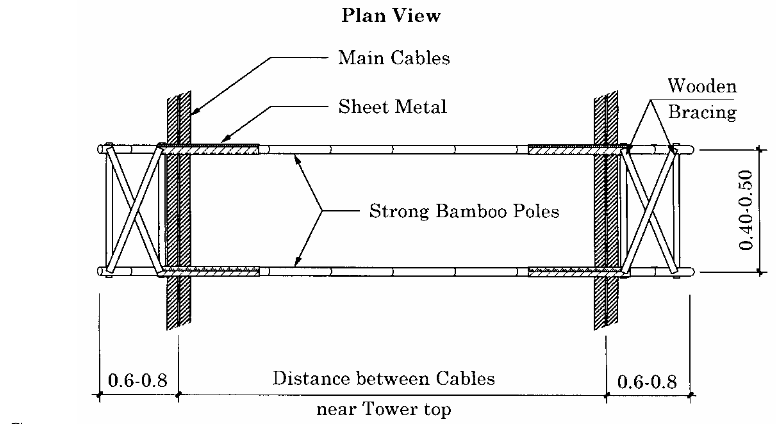
The suspender fitting work should start from **both** the towers and proceed towards the center of the bridge. This procedure is easier and has more advantages than starting the fitting work from the center. However, in order to achieve a proper symmetry of the suspenders, the **central suspender** must be fitted first.

The only disadvantage will arise when finishing the fitting works at the middle of the bridge. Due to inaccuracies, the remaining spacing at the center of the bridge might be either too long or too short. For minimizing this imprecision, the required distances to the towers and the center have to be rechecked after fitting 10 suspenders.

Preparation for Suspender Fitting Works:

- Lay out all the suspenders in sequence on the ground.
- Prepare all crossbeams, J-hooks and steel deck.
- Prepare two fitter platforms, one for the main cables and one for the spanning cables, and two gauged sticks of exactly 1.00m length.



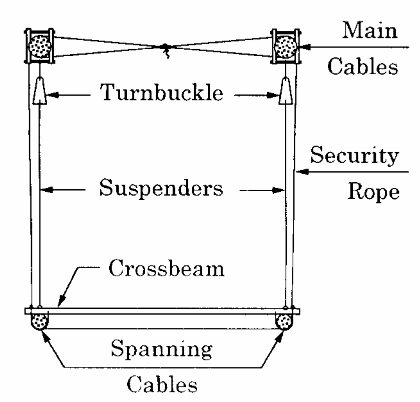


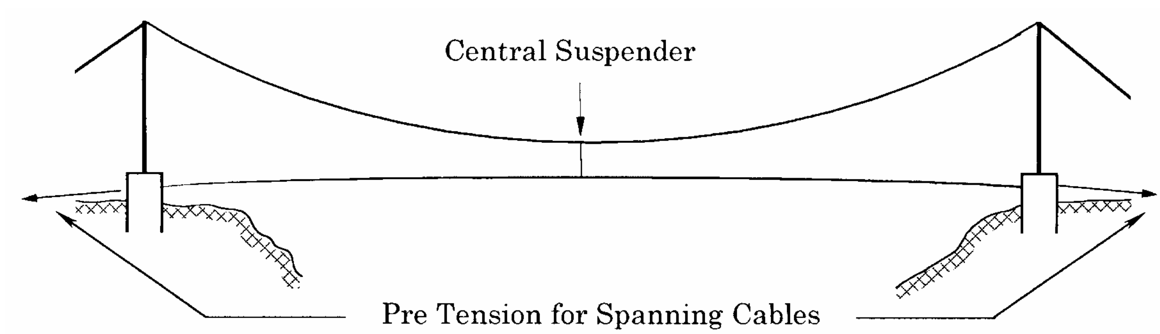
Assembly:
First fit the top portion on the main cables, then hang the platform under this by using either steel bars or cables.

Fitting the Central Suspender :

With the help of the fitting platform, the suspender in the center has to fitted first. Determine and reconfirm the center with a tape and level instrument, then fit the first suspender-pair at the center of the bridge. To avoid excessive load on the center suspender during erection time, bind all cables (spanning and main cables) together as shown in the sketch below with a security rope.

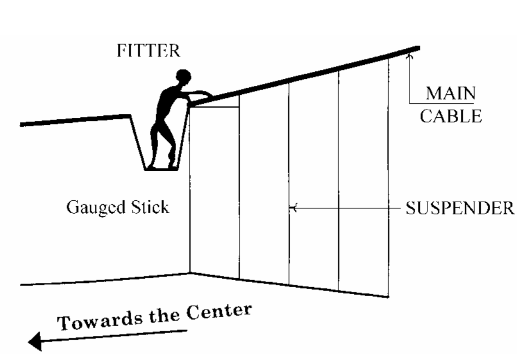
The **security rope** supports the suspender in the middle during erection work.

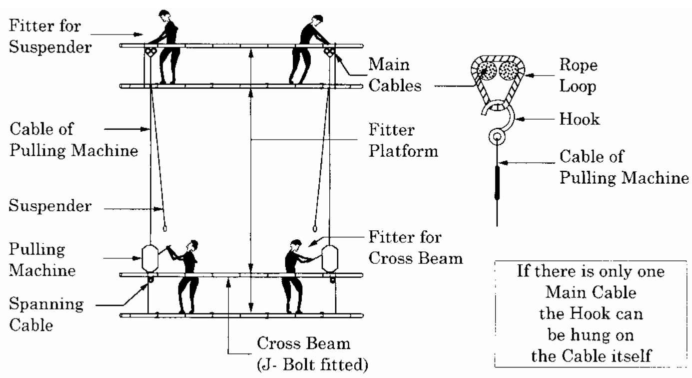




Tighten the spanning cable to some extent; now the cables are ready to be fixed to the suspenders.

Sketches and Procedures for Fitting Operations:





- Start the fitting work from both sides of the bridge and work towards the center of the bridge;
- Fix one cable car on top of the main cables and one on top of the spanning cables;
- Fix the first two suspenders to the main cables at the prescribed distance from the tower;
- Lift the spanning cable until the suspenders can be connected with the threaded rod of the walkway crossbeam.

Note: The first crossbeam at the bridge entrance is fitted without a suspender (see Drw. No. 19Ncon).

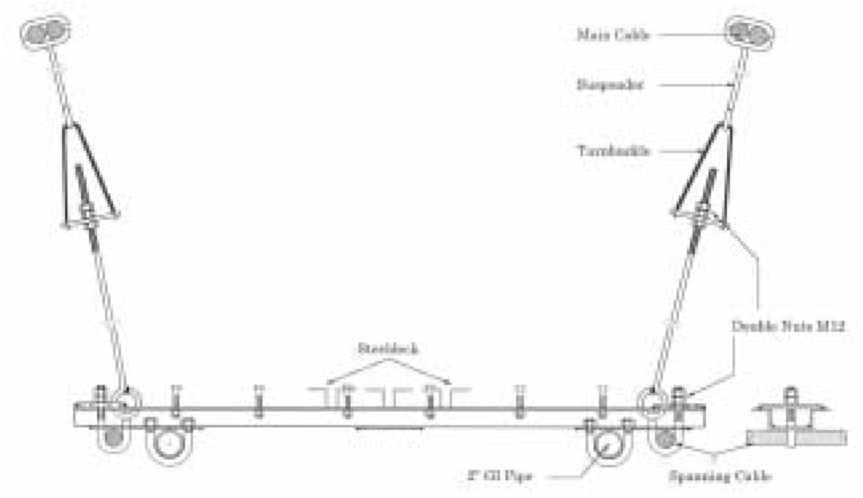
- In order that the suspenders are fixed exactly 1m apart, use gauged sticks for exact fitting;
- Re-adjust the spanning cables from both the banks as the suspenders are being fitted;
- After fitting ten pairs of suspenders, check the distances to the tower and to the center;
- Adjust only inaccuracies by moving the crossbeams;
- Gradually start fitting the **center row only** with standard steel deck panels as shown in Drawing No. 19 Ncon.

When the center is reached, there will be some extra length of spanning cable. For adjusting this, pull the spanning cables from both the banks with the tirefor machine through the loose J-hooks. Make sure that the middle row of the steel deck is fitted when doing this work.

- When all the suspenders have been fixed, tighten the spanning cables with the cable pulling machine as much as possible before fitting the 2" G.I. pipes below the crossbeams and before fitting the rest of the steel deck panels.
- Fix the handrail cables by pulling them through the suspender-rings just above the suspender turnbuckle, and secure them to the handrail posts by winding the cable end twice around the post.

Fitting the 2" G.I. Pipes :

Two 2" G.I. pipes have to be mounted from below to the steel deck cross beams. This provides additional vertical but also lateral stability to the entire walkway. These pipes can also be used for transferring water across the river as per local requirement.



The G.I. pipes have to be fitted before the outer rows of the steel decks are mounted in the following way:

- Lay two pipes of 6m length end to end on the ground and join them together firmly. Use a 2" die set and jute threads to make the joint water tight.
- In the same way, also fix half of the "union" at each end of the 12m piece.
- Now carry the 12m pipe to the bridge, pass it through the suspenders by securing it with nylon ropes until the entire 12m piece is on the outside of the suspenders.
- Now bring the pipe into proper position underneath the walkway, and secure it immediately with the U-clamps and join it with the "union".
- In case a union coincides with a crossbeam, cut the pipe and make a new thread with the die set.

This work requires special attention. While passing the pipe outside the suspenders, several workers are necessary and sufficient ropes are required to secure the pipe at all times.

Appendix N

Cost Estimate

Overall Estimate		
Item	Total Cost	Actual Cost (Donations Subtracted)
Materials	$32,162.97	$16,792.44
Labor	$19,926.50	$5,760.00
Equipment	$1,989.66	$850.66
Total	**$54,079.12**	**$23,403.10**

Foundation Estimate

Material Estimate							
Material Description	# of Material Units	Price/Unit	Transportation Costs	Cost of Materials	Total Cost	Donated?	Actual Cost
Cement (42.5 kg bag)	125	$7.80	$195.00	$975.00	$1,170.00	N	$1,170.00
Sand (m^3)	9	$13.90	$25.02	$125.10	$150.12	N	$150.12
Gravel (Tons)	34	$28.00	$190.40	$952.00	$1,142.40	Y	$0.00
CMU (block)	805	$2.12	$341.32	$1,706.60	$2,047.92	N	$2,047.92
No. 7 Steel Rebar (lft)	1005	$1.51	$341.70	$1,708.50	$2,050.20	N	$2,050.20
No. 8 Steel Rebar (lft)	1070	$1.70	$363.80	$1,819.00	$2,182.80	N	$2,182.80
3/4" Plywood (4' x 8' sheets)	9	$31.40	$56.52	$282.60	$339.12	N	$339.12
2 x 4 @ 12' (each)	11	$0.90	$1.98	$9.90	$11.88	N	$11.88
Nails (lb)	10	$0.85	$1.70	$8.50	$10.20	N	$10.20
Form oil (SFCA)	1014	$0.22	$44.62	$223.08	$267.70	N	$267.70
Total			$1,515.74	$7,578.70	$9,094.44		**$7,952.04**

Labor Estimate							
Labor Description	# of Laborers	Hrs/Laborer	Wage of Laborer	Cost of Labor	Total Cost	Donated?	Actual Cost
Form Builders and Strippers	3	20	$7.25	$435.00	$435.00	Y	$0.00
Concrete Mixers	2	80	$7.25	$1,160.00	$1,160.00	Y	$0.00
Rebar Layers	2	10	$7.25	$145.00	$145.00	Y	$0.00
Concrete Pourers	10	80	$7.25	$5,800.00	$5,800.00	Y	$0.00
Concrete Formers	4	80	$7.25	$2,320.00	$2,320.00	Y	$0.00
Excavators (People)	5	10	$7.25	$362.50	$362.50	Y	$0.00
Total			$43.50	$10,222.50	$10,222.50		**$0.00**

Equipment Estimate							
Equipment Description	# of Equipment Units	Price/Unit	Transportation Costs	Cost of Equipment	Total Cost	Donated?	Actual Cost
Metal Hacksaw	1	$25	$5.00	$25.00	$30.00	Y	$0.00
Replacement blade	5	$15	$15.00	$75.00	$90.00	Y	$0.00
Wheelbarrows	2	$120	$48.00	$240.00	$288.00	Y	$0.00
Metal Buckets	10	$10	$20.00	$100.00	$120.00	Y	$0.00
Hammers	2	$15	$6.00	$30.00	$36.00	Y	$0.00
Shovels	5	$25	$25.00	$125.00	$150.00	Y	$0.00
Total			$169.75	$595.00	$714.00		**$0.00**

Anchor Estimate

Material Estimate							
Material Description	# of Material Units	Price/Unit	Transportation Costs	Cost of Materials	Total Cost	Donated?	Actual Cost
Cement 42.5 kg (bags)	235	$7.80	$2,000.00	$1,833.00	$4,599.60	N	$4,599.60
Sand (m^3)	25	$13.90	$2,001.00	$347.50	$2,818.20	N	$2,818.20
Gravel (tons)	88	$28.00	$2,002.00	$2,464.00	$5,359.20	On Site	$0.00
Rebar No. 8	250	$1.70	$500.00	$425.00	$1,110.00	N	$1,242.60
Water (kg)	4695	$0.00	$0.00	$0.00	$0.00	On Site	$0.00
Total			$6,503.00	$5,069.50	$13,887.00		**$8,660.40**

Labor Estimate							
Labor Description	# of Laborers	Hrs/Laborer	Wage of Laborer	Cost of Labor	Total Cost	Donated?	Actual Cost
Concrete Mixers	4	40	$7.25	$290.00	$1,160.00	Y	$0.00
General Labor	4	20	$7.25	$580.00	$580.00	Y	$0.00
ReSteel Cage	3	10	$7.25	$217.50	$217.50	Y	$0.00
Formwork Labor	3	12	$7.25	$261.00	$261.00	Y	$0.00
Digging	6	5	$7.25	$217.50	$217.50	Y	$0.00
Project Manager*	1	576	$10.00	$5,760.00	$5,760.00	N	$5,760.00
Total			$46.25	$7,326.00	$8,196.00		**$5,760.00**

Equipment Estimate							
Equipment Description	# of Equipment Units	Price/Unit	Transportation Costs	Cost of Equipment	Total Cost	Donated?	Actual Cost
*See Equipment Estimate							

**Project manager estimate is for entire project duration*

Tower Estimate

Material Estimate							
Material Description	# of Material Units	Price/Unit	Transportation Costs	Cost of Materials	Total Cost	Donated?	Actual Cost
12"X12" Wooden Beam (Lft)	600	$24.45	$2,000.00	$14,670.00	$20,004.00	Yes	$0.00
2"X 4" Wood boards (Lft)	350	$0.70	$500.00	$245.00	$894.00	Yes	$0.00
2"X 6" Wood boards (Lft)	400	$0.95	$500.00	$380.00	$1,056.00	Yes	$0.00
Top Gusset Plates	4	$110.00	$50.00	$440.00	$588.00	No	$588.00
Base Gusset Plates	4	$110.00	$50.00	$440.00	$588.00	No	$588.00
Saddles	4	$15.00	$50.00	$60.00	$132.00	No	$132.00
Bolts and Fixings (Plates)	48	$2.00	$50.00	$96.00	$175.20	No	$175.20
Anchor Bolts	8	$25.00	$50.00	$200.00	$300.00	No	$300.00
Pulleys	2	$200.00	$50.00	$400.00	$540.00	No	$540.00
Pulley Cable	2	$200.00	$50.00	$400.00	$540.00	No	$540.00
Steel Cross Bracing	1	$100.00	$50.00	$100.00	$180.00	No	$180.00
Total			$3,400.00	$17,431.00	$24,997.20		**$3,043.20**

Labor Estimate							
Labor Description	# of Laborers	Hrs/Laborer	Wage of Laborer	Cost of Labor	Total Cost	Donated?	Actual Cost
Chainsaw Operator	1	12	$7.25	$87.00	$87.00	Yes	$0.00
General Labor	4	20	$7.25	$580.00	$580.00	Yes	$0.00
Hoisting Labor	8	30	$7.25	$1,740.00	$1,740.00	Yes	$0.00
Scaffolding Labor	6	72	$7.25	$3,132.00	$3,132.00	Yes	$0.00
Pulley Labor	6	3	$7.25	$130.50	$130.50	Yes	$0.00
Total			$36.25	$5,669.50	$5,669.50		**$0.00**

Equipment Estimate							
Equipment Description	# of Equipment Units	Price/Unit	Transportation Costs	Cost of Equipment	Total Cost	Donated?	Actual Cost
Truck	1	$250.00	$0.00	$50.00	$300.00	Yes	$0.00
Chainsaw	1	$100.00	$0.00	$25.00	$125.00	Yes	$0.00
Generator	1	$100.00	$100.00	$50.00	$250.00	No	$250.00
Drill	1	$25.00	$20.00	$25.00	$70.00	No	$70.00
Total			$120.00	$150.00	$745.00		**$320.00**

Main and Spanning Cables, Suspenders, Walkway, and Stairs Estimate

Material Estimate							
Material Description	# of Material Units	Price/Unit	Transportation Costs	Cost of Materials	Total Cost	Donated?	Actual Cost
Main cable d = 32mm (m)	172	$20.01	$688	$3,441.72	$4,130.06	Y	$0.00
Spanning cable d = 32mm (m)	82	$20.01	$328	$1,640.82	$1,968.98	Y	$0.00
Suspenders d = 3/8" or 10mm (m)	58	$91.20	$18	$91.20	$109.44	N	$91.20
Turnbuckles	2	$106.63	$43	$213.26	$255.91	N	$213.26
Hanger clamp	58	$14.78	$171	$857.24	$1,028.69	N	$857.24
Clips	232	$0.94	$44	$218.08	$261.70	N	$218.08
Wire rope thimble	116	$0.65	$15	$75.40	$90.48	N	$75.40
Eye bolt d = 3/4", L = 10"	58	$7.84	$91	$454.72	$545.66	N	$454.72
Plate A36 4"x4"x3/8"	58	$4.01	$47	$232.52	$279.02	N	$232.52
Hanger beam - wood, 4"x6" (Lft)	89	$1.90	$34	$168.32	$201.98	Y	$0.00
Walkway/stairs (2"x4"x12' boards)	120	$0.90	$22	$108.00	$129.60	Y	$0.00
Stairs (6"x6"x12' posts)	15	$1.20	$4	$18.00	$21.60	Y	$0.00
Stainless screws, #10, 3"	474	$0.43	$41	$203.82	$244.58	N	$203.82
Total			$1,500.25	$7,501.27	$9,001.53		**$2,142.42**

Labor Estimate							
Labor Description	# of Laborers	Hrs/Laborer	Wage of Laborer	Cost per Laborer	Total Cost	Donated?	Actual Cost
Unroll cable	5	2	$7.25	$14.50	$72.50	Y	$0.00
Hoist spanning cable	5	10	$7.25	$72.50	$362.50	Y	$0.00
Hoist main cable	5	10	$7.25	$72.50	$362.50	Y	$0.00
Fit suspenders	2	20	$7.25	$145.00	$290.00	Y	$0.00
Construct walkway	2	20	$7.25	$145.00	$290.00	Y	$0.00
Total					$1,377.50		**$0.00**

Equipment Estimate							
Equipment Description	# of Equipment Units	Price/Unit	Transportation Costs	Cost of Equipment	Total Cost	Donated?	Actual Cost
Pulley - see tower estimate	1		$0.00	$0.00	$0.00	M	$0.00
Drill	4	$51.49	$10.30	$205.96	$216.26	N	$216.26
Truck (to tension cable)	1	$0.00	$0.00	$0.00	$0.00	Y	$0.00
20' Ladder	1	$262.00	$52.40	$262.00	$314.40	N	$314.40
Total			$62.70	$467.96	$530.66		**$530.66**