RIO TABASARA PEDESTRIAN BRIDGE
FINAL DESIGN REPORT

December 14, 2018
CEE4916 Fall 2018
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Michigan Technological University 1885

To Kathryn and all other relevant parties,

PanaMac Engineering has assembled this final design report, detailing the structural members, supports, and connections of the proposed suspension bridge at the Tabasara River crossing location. Enclosed are drawing details of the structure and design calculations in the appendix. We wish to thank Kathryn Douglass, Kiko de Melo e Silva, Dr. David Watkins, Prof. Mike Drewyor, the residents of Llano Miranda and Bajo Mosquito, and everyone else who assisted us with this project. All questions regarding the information enclosed can be directed to our project manager, Erin Lau.

Sincerely,

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DISCLAIMER:

This report, titled "Tabasara River Crossing Final Design Report", 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 UNTIL PLANS HAVE BEEN APPROVED BY A PROFESSIONALLY LICENCED ENGINEER.**

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

Executive Summary

PanaMac Engineering's mission is to create a lasting solution to improve access to education and food in an underserved community. This team consisting of four structural engineering students travelled to Llano Miranda, Panama to assess and investigate a possible river crossing construction project. The purpose of this river crossing is for members of the Llano Miranda and Bajo Mosquito communities to have safer and faster access to the people and amenities of the opposing side. This report is a culmination of the work completed by PanaMac including data collection and analysis, discussion of design constraints, watershed analysis, design calculations, and a construction estimate and schedule.

Data was first collected on site using an Abney level and tape measure. Additionally, a GPS was used to record approximate global locations for each surveyed point. A digital rangefinder was used to check if measured distances were reasonable, but no values from the rangefinder were recorded or used otherwise. The team assessed the landscape and chose the primary site based on its location between the two communities, a similar elevation on both sides, and the existence of paths on each side.

Several bridge types were considered initially, but location, equipment availability, and cost were the primary constraints leading to the decision to design a suspension bridge. A 3-cable suspension bridge was considered as a possible cost-reducing alternate design, but the team's analysis led to the conclusion that it would only reduce the cost slightly while severely decreasing safety. The final design detailed in this report is a 270 foot long suspension bridge with 30 foot tall towers and 25 feet of sag in the cables.

A construction estimate and schedule are also included to show the expected cost and timeline of the project. The schedule follows a typical five-day work week but could easily be shortened if six days per week are deemed necessary. The schedule assumes the availability of an excavator, since large amounts of soil are needed to build earth ramps to the bridge itself. The estimate also assumes the availability of local workers to assist with non-skilled tasks at a reduced labor rate.

The team recommends this report and design be delivered to possible stakeholders including Panama's Ministry of Public Works after being reviewed by a professional engineer.

1. INTRODUCTION

PanaMac Engineering traveled to the Comarca Ngobe-Bugle of Panama in late August of 2018 with the goal of investigating a possible river crossing between the Llano Miranda and Bajo Mosquito communities (Figure 1). The purpose of this investigation was ultimately to devise a plan for a bridge to aid the members of these communities in reaching education and the markets more safely and easily. These two communities are divided by a river that becomes especially dangerous to cross during the rainy season (May-December). Nonetheless, many community members traverse the river on a regular basis, and several have died as a result.

Members of Llano Miranda and Bajo Mosquito cross the river regularly for three main reasons: to purchase food or supplies, to attend school, and to visit family and friends. The market on the Llano Miranda side is stocked every two weeks, while the market in Bajo Mosquito is stocked every other day. Currently, the children of Llano Miranda hike to school daily, about an hour trip one way. That trip would be reduced to about 10 to 15 minutes if a bridge were constructed.

The team was directed to a proposed location by a member of the Bajo Mosquito community. Land surveying was conducted in order to analyze the floodplain and identify elevations for bridge design constraints. With the help of the local Peace Corp Volunteer, the team communicated with the local community members to assess the needs of the community and understand previous flooding levels.

Figure 1. Llano Miranda Site Location

Several conclusions were made as a result of this investigation. First, there is a general consensus among community members that a bridge at this location is a necessity. During the rainy season, the river becomes especially dangerous after heavy rains in the afternoon when there is a high volume of runoff. The flooding at the proposed site typically reaches above the banks in the pasture along the river (Figure 2 shows the pasture on the Llano Miranda side, with the river behind the tree line.) During these high flow time periods, the river is deadly to cross, but that rarely prevents community members from attempting to cross anyway. The people in these communities know the risk when crossing this river and continue to do so because of the amenities and family ties on the opposite side.

Second, the team concluded that the proposed site is the optimal bridge location for several reasons. Both sides of the river are relatively flat and easy to walk across, so community members would not have to traverse difficult terrain in order to use the bridge. The elevation on both sides is nearly equal, so the bridge would not have to be built up significantly on one side or the other. Lastly, this location is also where community members typically cross the river, so there are already paths leading to the site.

Figure 2. Pasture on Llano Miranda Side of River

Other significant information to note is the ongoing construction of a dirt road in Llano Miranda. Throughout the team's stay in the community there was constant work being done. It is assumed that the road will run all the way to the existing bridge in Llano Ñopo that crosses the Tabasara River. The progress of this road construction will determine what construction equipment can reach the project site, indirectly influencing the design parameters.

2. DATA COLLECTION

Figure 3. Site Map

Data collection began by exploring the area around the river and asking locals about possible bridge locations and historical flood levels. The team had difficulty obtaining a clear answer with regards to flood levels and began by performing a preliminary survey of the river cross section at a spot thought to be feasible. Since the team resided in Llano Miranda, that side was surveyed first, then the Bajo Mosquito side the next day (Figure 3). Two bamboo poles of a member's eye level were fashioned to sight to and from locations. All surveying was completed with the previously mentioned poles, an Abney Level, a one-hundred foot tape measure, and a compass. A digital laser rangefinder was also used to check that the team's distance measurements were reasonable, but values from the rangefinder were not recorded or used for calculations. GPS coordinates were also taken at each point to aid in relating points back to real (approximate) global locations. After quickly analyzing the initial cross-section and comparing it to even the most conservative estimate of flooding, it was found that a bridge would need to span roughly 700 feet to stay clear of the floodplain. This was concluded to be infeasible, so more information on historic flood levels was needed.

The team asked the shop owner in Bajo Mosquito for his recollection of flood levels. A clear answer was not obtained, other than that the past year's levels were very high. He did say that in Llano Ñopo, the water rose within two or three meters below the existing bridge. To investigate this, a trip to Llano Ñopo was made to collect data on the bridge there (Figure 4). It was found to be a 277 foot suspension bridge (support to support) with approximately 15 feet of sag in the cables. The level of flooding there was reported by locals to be at the top of a pronounced rock under the bridge, measured to be approximately 20 feet higher than the current level. The bridge was found to be approximately 40 feet above the river. Information about all structural components was also collected for reference. Since the bridge in Llano Ñopo is located at a much narrower point in the river than the Llano Miranda site, this information was only somewhat useful. It was expected that the flood levels at Llano Miranda should be significantly lower than at Llano Ñopo because of the wider floodplain.

Later, a member of Bajo Mosquito showed the team the site where he thought the bridge would be best located and gave a much different account of the flooding than given by other members.

Figure 4. Suspension Bridge in Llano Ñopo

The amount of flooding that he said occurred was much more compatible with the level of flooding at the Llano Ñopo bridge. Using this new information, a different site near the original proposed river cross section was chosen. A preliminary survey of that site was taken immediately; more data was collected from both sides of the river. This data was used to make a topographic map of the site (Figure 5).

Qualititative visual soil analysis was performed at each site. This analysis consisted of estimating the grain size classification of the soil and estimating the depth of each layer. The top of the soil was clay with many large rocks on the Llano Miranda side. The banks of the river showed the layer of clay was no more than five feet deep, then giving way to sand. The top surface was clay on the Bajo Mosquito side. It was difficult to ascertain the exact nature and depth of the lower layers, but the bedrock seemed much shallower on that side, while there did not seem to be any sand below the clay.

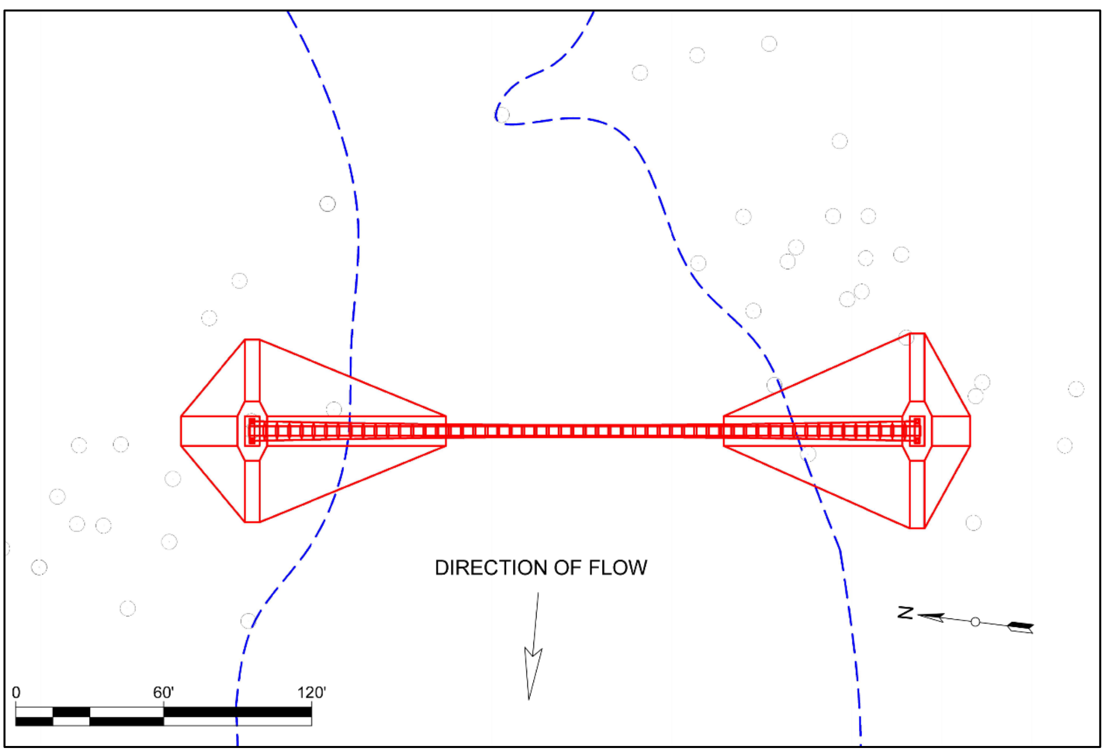


Figure 5. Survey Data Collected at Proposed Bridge Location (Plan View)

3. WATERSHED

Due to the fact that there is no hydrologic data relating to the project site, the team approximated flowrate calculations based on a nearby watershed. The watershed area was calculated using an AutoCAD drawing superimposed over a topographic map to determine overall area, as shown in Figure 6. Peak flow rate for a nearby watershed was scaled by the ratio of the two watersheds' drainage areas. Using this watershed area and scaled flowrate, an estimate of the flowrate at the project site was determined. These calculations can be seen in Appendix G. The overall watershed area was calculated to be 24,840 hectares.

Based on these calculations and observing the surrounding floodplain, the team determined there is very little risk of the river or transported debris reaching the design height of the bridge. Further, there is little risk of scour around the bridge anchor blocks, but the team specified rip rap to protect them as an extra precaution.

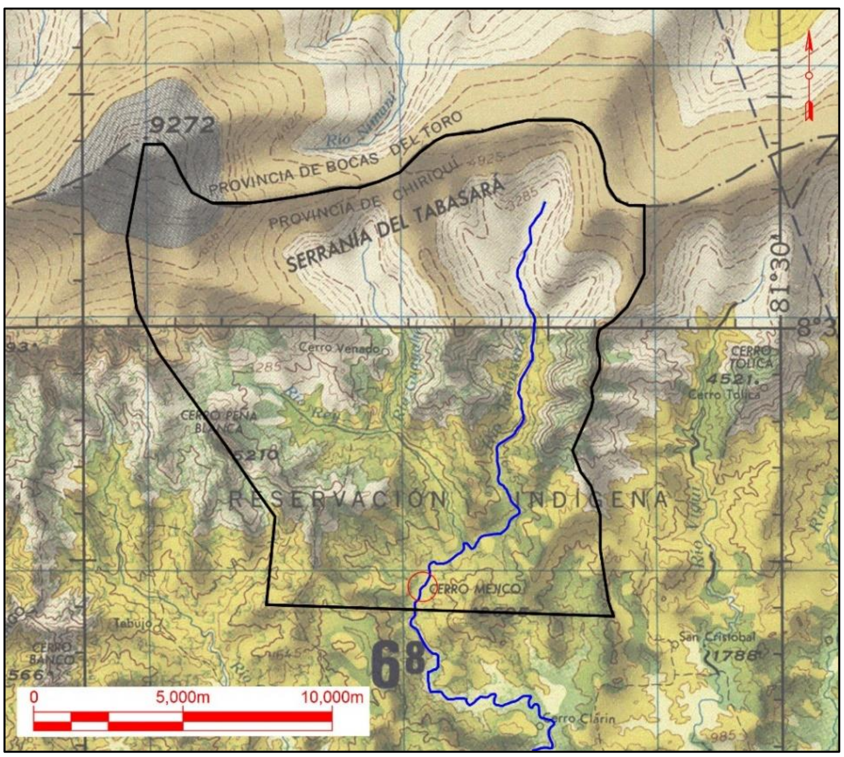


Figure 6. Approximate Watershed Area Draining to Bridge Site

4. DESIGN OVERVIEW

A suspension bridge configuration was selected as a result of site conditions and erection constraints. Erection limitations and environmental impacts made the placement of a pier in the river unlikely because piles would have to be driven, requiring a crane. Unless site and road conditions are dramatically improved, it is unrealistic for a crane to reach the construction location.

Because piers cannot be used, the only configurations that could potentially span 270 feet are a suspension bridge, a suspended bridge, an arch, or a truss. The lack of availability of a crane at the job site makes construction of an arch or truss bridge nearly impossible. Those configurations would be very inefficient as well, as the dead load of the bridge structure would far exceed any live load, and would also exceed the dead loads for the suspended and suspension configurations.

The primary parameter that drove the decision between a suspension bridge and a suspended bridge was clearance above the river. The site at which the bridge must be built has one evenly flat side where a large ramp must be constructed. A suspended bridge would require the deck to sag down many feet below its initial height in order to lower the force on the anchor block. This would require a ramp to be much higher than with a suspension bridge, and obtaining good soil to build a ramp at the jobsite will require trucking. The primary advantage of a suspended bridge is that towers do not need to be constructed. This advantage is far outweighed by the cost of a larger ramp.

A 3-cable bridge in which there are two hand cables and one cable to walk on was also considered as a much cheaper alternative to any full bridge. A weakness in this design is its difficulty to traverse. It would be very dangerous for five to twelve-year-old children to cross this type of bridge every day for school.

A 3-cable bridge would also suffer the same problem that a suspended bridge does, as it is really just a suspended bridge with a cable for a deck. The bridge would sag below its starting point, requiring a much larger ramp. The cable forces would also be smaller because there is less material needed and fewer people would cross the bridge at a time. However, even the need for a five foot increase in ramp height increases the volume of soil by a factor of 3.5. The size of the ramps would increase dramatically, and the cost would come close to the cost of a full suspension bridge, which would better serve the community.

A suspension bridge was chosen for its ability to span large distances with a low dead weight, while also allowing the deck to stay level across the entire span. It is also constructible with only an excavator, which could be brought to site without significant prior site improvement. A suspension bridge presents a cheap and safe crossing appropriate for the needs of Llano Miranda and Bajo Mosquito.

Several factors affect the viability of a bridge design, including loads, equipment availability, cost, intended use, soil, and flood levels. The team considered all of these constraints to produce a suspension bridge design to meet the needs of the community members safely and efficiently.

The appropriate design codes (steel, concrete, timber) were used to design the corresponding structural members, and the Bridges to Prosperity (B2P) design manual [1] was also used as a guideline for the overall bridge design.

Table 1. Bridge Weight Calculations

Density of Wood	48.33	lb/ft^3
Width of Bridge	4.00	ft
Depth of Decking	0.29	ft
Weight of Crossmember	6.50	plf
Length of Crossmember	5.00	plf
Depth of Wood Crossmember	0.13	ft
Breadth of Wood Crossmember	0.60	ft
Spacing	5.00	ft
Hanger Weight	0.67	lb/ft
Cable Weight	4.51	lb/ft
Average Hanger Length	13.10	ft
Decking	56.39	plf
Crossmember	10.15	plf
Hanger	3.51	plf
Cable	9.02	plf
TOTAL	**80**	**plf**

When considering design loads, the team identified the possibility of community members using the bridge to move livestock. The team was made aware that cattle have crossed the suspension bridge in Llano Ñopo, so the team determined it was prudent to design this bridge with that in mind. The loads used for design calculations were a dead load of 80 pounds per linear foot (plf) and a live load of 260 plf. The dead load is simply the weight of the structure, calculated as shown in Table 1. The live load is based on the B2P manual, which references the AASHTO Guide Specification for Design of Pedestrian Bridges, 1997 [2]. A wind load of 100 miles per hour was initially considered as well, but it was found to be negligible. The B2P design manual states that bridges under 394 feet in length do not practically need to consider wind unless in a high wind speed area. Therefore, the dead load and live load were the sole design loads used in the team's calculations.

The curve of a suspension bridge cable closely follows a catenary curve. However, the difference between catenary and parabolic profiles is negligible in the range of sag values used for suspended cable bridges. Therefore, a parabolic profile was used to calculate the hanger lengths at five foot intervals, as shown in Figure 7. The graph shows half of the bridge, since it is symmetrical.

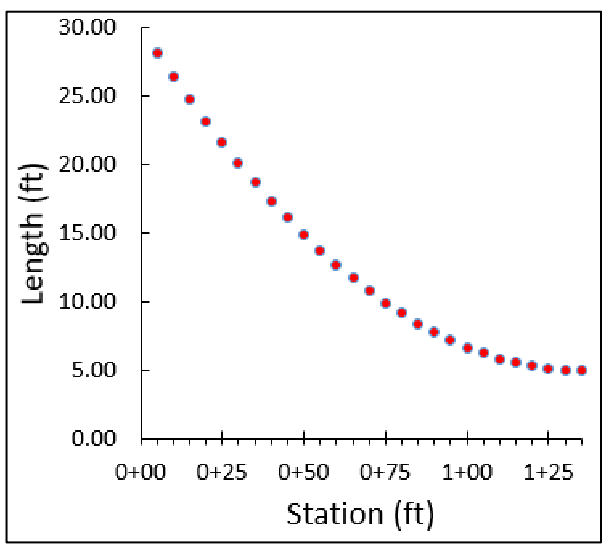


Figure 7. Hanger Lengths

Equipment availability is unknown to some extent. As mentioned before, a road is being constructed that reaches Llano Miranda, but that does not necessarily mean heavy equipment could reach the river. Bajo Mosquito already has a road that is traversable to the river, and during the dry season the river could possibly be crossed by an excavator. Much of the bridge was designed to be constructible without heavy equipment, but it would certainly be faster, easier, and safer to construct in the event it can be used. The project estimate and schedule were created under the assumption of heavy equipment being available.

The historic flood levels at the project site played a large role in the overall bridge design. The bridge must clear the water at the highest flood level, but it also must be high enough that no damage is inflicted by passing trees or brush. Flooding also causes concern for scour of the tower foundations. Considering these requirements, the team designed the bridge to be 10 feet above the assumed 20-year flood level, based on the information gathered from community member interviews. Also, the team specified large stones or rip rap to surround the foundations, preventing damage to the structure.

5. DESIGN DETAILS

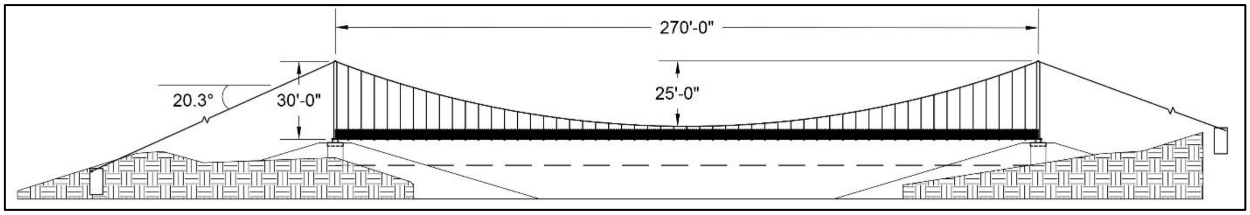


Figure 8. Full Bridge Design

The individual structural components of the bridge were designed per the design codes of the respective materials. The strengths of the materials used were also conservative per B2P recommendations. A compressive stress of concrete of 1500 pounds per square inch (psi) and a yield stress of steel of 35,000 psi were used. The concrete compressive stress is significantly lower than most concrete as aggregate and mixing quality cannot be guaranteed. The steel yield stress ensures that all structural components are safe if a lower grade of structural steel is used. The grades of steel recommended, however, are consistent with the American Institute of Steel Construction [3] recommendations. This represents an additional safety factor above those shown in calculations and makes buckling calculations significantly more conservative.

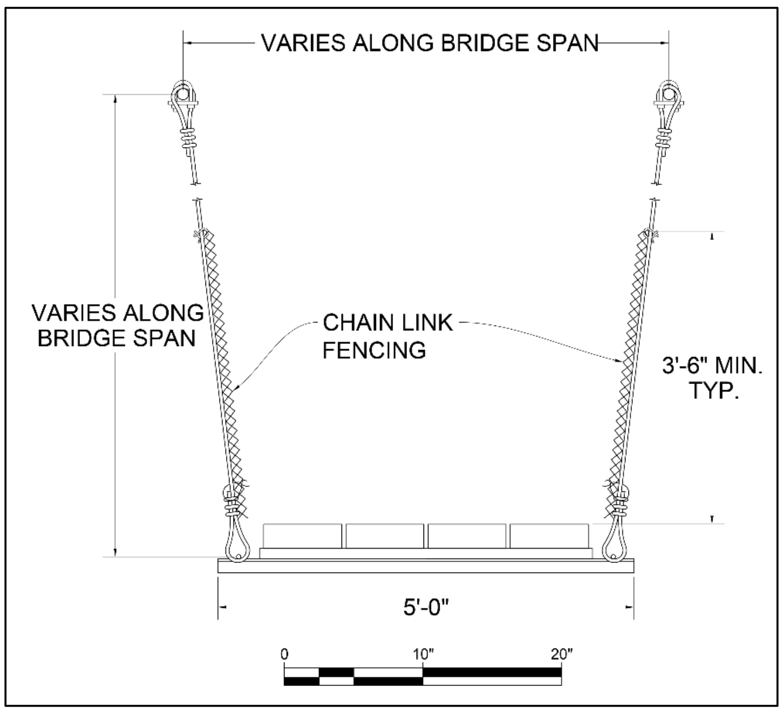


Figure 9. Decking and Hangers Detail

The decking design was controlled by the point load due to a cattle hoof (500 lbs), leading to the selection of 4”x12” wood planks. The rest of the bridge design was controlled by the distributed load. The cables were designed with a safety factor of three, due to the difficulty in replacing them and the possibility of the cables being previously used in rigging.

Since the local soil contains many boulders (making excavation difficult), minimizing the excavation for the anchor blocks was determined to be very important. The towers were designed to be 30 feet tall to allow for more sag, minimizing the forces at the anchor blocks. This led to a reasonable anchor size, and maintained a constructible tower height. The anchors were designed to maximize the passive pressure provided by the soil in front of them, as friction between the anchor and the soil was assumed low due to the clay. This led to a design that required a ten foot deep concrete block.

The shear capacity of the concrete determined the minimum thickness of the anchor block as six feet. The strength of the concrete-cable connection will be developed by the embedment of a large steel beam in the concrete. The pull-out strength of this connection was determined by calculating the stress in the concrete over planes determined by a 38° internal friction angle of the concrete.

The foundations for the towers were designed for a worst-case soil, correlating to the clay and sand observed at the site. They were designed using the general bearing capacity theory, with a safety factor of four to represent the uncertainty in the actual soil conditions. These calculations should be repeated with soil properties determined by testing of soil samples from the site. The structural analysis and design of the foundations were completed using the rigid method and were designed as reinforced concrete slabs.

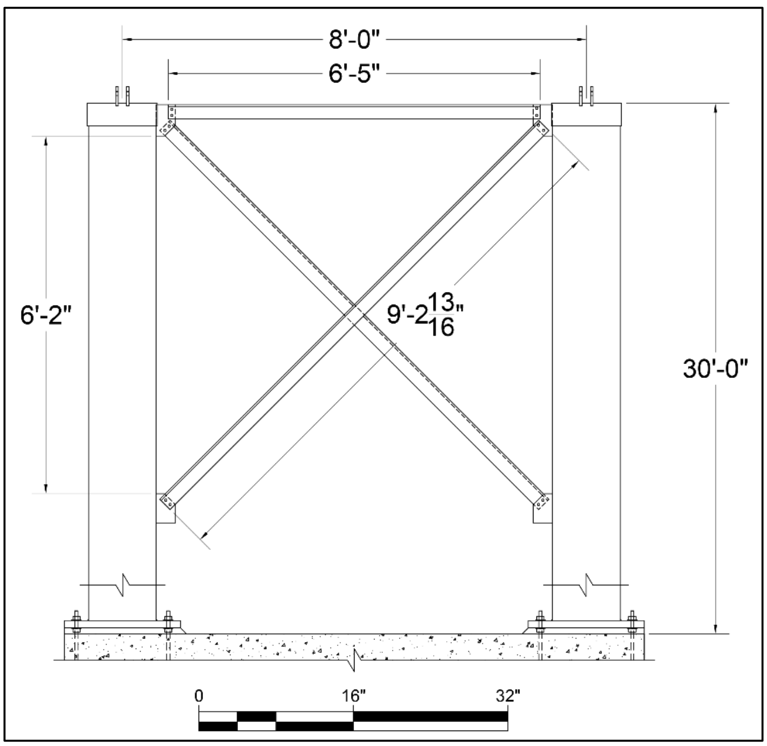


Figure 10. Towers and Foundation Detail

6. MAINTENANCE

Regular maintenance is recommended to preserve the structural integrity and usability of the bridge. The largest concerns are degradation due to moisture and issues with slope stability due to flooding. The degradation due to moisture will largely take the form of rust on steel components and rot in wooden components. These should be addressed in a timely manner to ensure safety and longevity of the structure.

The wooden components—the decking and the cross-member boards—should be checked for rot each month. A board must be replaced as soon as any rot is found. If a board is showing excessive deflection, it should be monitored and replaced once deemed unusable. Heavy rot-resistant wood should be used for all replacements.

Slopes must be monitored for any erosion or slope failures. When any degradation is noted, it should be repaired with local soil and stones. A detailed inspection by local community members should be completed after high floods fully recede. If slope stability problems are found that cannot be repaired by community members, an engineer shall inspect and assess the damages and determine a plan to repair the slope.

All steel components must be inspected regularly for signs of corrosion. Paint should be maintained on all non-galvanized components when possible. If all paint cannot be maintained, any rust that is found must be removed by sanding, and then painted over. As the steel components have been designed with some consideration to corrosion, the strict suppression of corrosion is not primarily a life safety concern, but a structure longevity concern.

An engineer shall complete a full inspection of the structure every four years, inspecting each load-bearing component in detail. This includes the connections between the cross-member and decking, the hanger and cross-member, the cable and hanger, the tower and cable, the tower and foundation, and the cable and anchor. This also includes the decking, the cross-members, all members that comprise the towers, the cables, the hangers, the foundations, and the anchor blocks. The foundations and anchor blocks shall be inspected both for soil movement and for cracking of the concrete.

7. Cost Estimate Overview

The cost estimate created by the team consists of four main sections: materials, labor, equipment, and overhead/profit. Materials make up more than half of the estimate, as the primary cables, anchors, hangers, and decking are especially costly items for a bridge of this size. Additionally, the design requires large earthen ramps to reach the bridge, increasing the materials cost estimate significantly. The labor and equipment costs are based on the construction schedule (Section 8). Labor rates were estimated based on general pay information gathered from speaking with Kat, the Peace Corp Volunteer on site.

While creating an estimate and a construction schedule for a rural suspension bridge in Panama, certain considerations needed to be taken. Some of these considerations include: adjustments to sales tax and labor rates from United States values to estimated values for Panama, as well as estimating equipment rental costs. Unable to find equipment rental rates for Panama, an estimated value was assumed by dividing the equipment rental rates in the United States by two. In the field of general requirements, insurance was not taken into account, and an overall percentage for overhead and profit of fifteen percent of the overall cost. To reduce cost, it is recommended that local community members be used to collect some of the material that can be found nearby. Local community members could also be used for basic tasks like breaking apart the rock and clearing the path for the equipment to get to the site. While in Panama, the team also received a price table for typical contruction materials available in the Comarca. These prices were used in the estimate where applicable. The full cost estimate can be found in Appendix H.

Table 2. Cost Estimate Breakdown

Division	Estimated Cost ($)
Materials	126,000
Labor	31,000
Equipment	38,000
O&P	29,000
TOTAL	**224,000**

8. CONSTRUCTION SCHEDULE OVERVIEW

The construction schedule, as shown in Appendix I, follows a general logic for suspension bridge construction and includes the major task divisions. The schedule was created under the assumption that activities could be completed on both sides of the river simultaneously, but in the event there are not enough workers for multiple crews, the project duration would increase significantly. The durations given for each activity are educated guesses based on the minimal information the team has collected regarding labor productivity in Panama. More time may need to be considered for extended periods of concrete curing time, depending on weather conditions. This construction project should take place during the dry season, preferably from January to March. A summary of the primary construction activities is shown in Table 3.

Table 3. Construction Schedule Summary

Activity	Estimated Duration (days)
Site Work	54
Foundation	11
Steel Erection	9
Abutments	5
Decking	3
TOTAL	**82**

9. CONCLUSION AND RECOMMENDATIONS

The PanaMac Engineering team travelled to Llano Miranda, Panama to determine the needs of the community regarding a possible river crossing. While in Panama, the team surveyed the land on each side of a possible crossing site, analyzed an existing bridge nearby, and communicated with locals to prepare for the design work to take place during the fall semester. Since visiting the site, the team has completed analysis of the survey data and developed the proposed bridge design, project cost estimate, and predicted construction schedule.

The survey data gave the team a rough estimate of how long the bridge would have to be, and this information was used to determine what type of bridge should be implemented. A topographic map was generated with the survey data, and the highest points of land near the river were determined. The expected flood level, as determined by interviews of community members, approximate hydrological analysis, and a large factor of safety were significant constraints. The team found that a 270-foot-long suspension bridge would be efficient, and created a design that will allow community members to cross the river safely.

Once the bridge design was complete, the team created a cost estimate and project schedule to predict the funding and manpower necessary to complete this project. The approximate cost estimate is 225,000 USD. This opinion of cost was made primarily using prices from USA-based material suppliers. The construction of the bridge will take an estimated three months. The team recommends constructing the bridge between January and March, focusing construction in the driest months.

The team recommends that this report and design be delivered to possible stakeholders including Panama's Ministry of Public Works after being reviewed by a professional engineer. As the bridge would span two administrative districts, it is recommended that both representatives are contacted. This should be carried out by community members of Llano Miranda and Bajo Mosquito, with the assistance of the local Peace Corps Volunteers.

The team recommends that the community check the bridge for obvious damage at monthly intervals to maintain the structural integrity of the bridge after its construction. This includes all connections and structural components on the bridge, foundations, and slope stability. Special attention should be paid to slope stability and the wooden decking, as they are the most susceptible to damage, and can be repaired by community members. Additionally, a full inspection by an engineer shall be completed once every four years.

REFERENCES

[1] *Bridges to Prosperity Bridge Builder Manual*. 5th ed., 2016.

[2] *AASHTO Guide Specification for Design of Pedestrian Bridges*. 1st ed., 1997.

[3] *American Institute of Steel Construction Steel Construction Manual*. 15th ed., 2017

Appendices

Appendix A

Cable Analysis & Design

Sheet Made By: Anthony Jaksa
Checked By: Erin Lau

Cable Design and Analysis

Bridge Parameters

$L := 270ft$

$sag := 25ft$

$\Delta H := 0ft$

$\Omega_{cable} := 3$

$width := 4ft$

Loads

Live load

$LL := 65psf$

$w_{live} := LL \cdot width = 260plf$

Calculate Dead Load

$\gamma_{wood} := 48.33pcf$ Assuming Beech-Birch-Hickory at 30% Water Content (NDS)

$d_{decking} := 3.5in$ Decking is four 4x12s

$d_{xmember} := 1.5in$ Wood Crossmember is a 2x8

$b_{xmember} := 7.25in$

$w_{steelxmember} := 6.5plf$ Steel Section 2L2x2x0.250 (AISC 14th Edition)

$L_{xmember} := 5ft$

$w_{hanger} := 0.67plf$ #4 rebar

$w_{cable} := 4.51plf$ https://catalog.lexcocable.com/item/all-categories-strand-bridge-rope/ope-galvanized-structural-bridge-rope-br-astm-a603/astm603-1-5-8

$spacing := 5ft$

$L_{hangeravg} := 13.1ft$ Converted by adding all hanger lengths and dividing by the number of hangers, conservative as the middle of the bridge would actually have shorter hangers

$n_{hanger} := 2$

$n_{cable} := 2$

Continued Dead Load Calculations

$$w_{cablef} := n_{cable} \cdot w_{cable} = 9.02\,\text{plf}$$

$$w_{hangerf} := \left(\frac{1}{spacing}\right) \cdot n_{hanger} \cdot L_{hangeravg} \cdot w_{hanger} = 3.511\,\text{plf}$$

$$w_{steelxmemberf} := w_{steelxmember} \cdot L_{xmember} \cdot \left(\frac{1}{spacing}\right) = 6.5\,\text{plf}$$

$$w_{woodxmember} := \gamma_{wood} \cdot d_{xmember} \cdot b_{xmember} \cdot L_{xmember} \cdot \left(\frac{1}{spacing}\right) = 3.65\,\text{plf}$$

$$w_{decking} := \gamma_{wood} \cdot d_{decking} \cdot width = 56.385\,\text{plf}$$

Full Dead Load

$$w_{dead} := w_{cablef} + w_{hangerf} + w_{steelxmember} + w_{woodxmember} + w_{decking} = 79.066\,\text{plf}$$

Dead Plus Live Load

$$w_{full} := w_{dead} + w_{live} = 339.066\,\text{plf}$$

Cable Dead Load Response

Cable Tension

$$P_h := \frac{\left(w_{dead} \cdot L^2\right)}{(8 \cdot sag)} = 28.819\,kip$$

$$\theta_{high} := atan\left(\frac{4 \cdot sag + \Delta H}{L}\right) = 20.323 \cdot deg$$

$$P_{vhigh} := P_h \cdot tan\left(\theta_{high}\right) = 10.674\,kip$$

$$P_{thigh} := \frac{P_h}{cos\left(\theta_{high}\right)} = 30.733\,kip$$

$$\theta_{low} := atan\left(\frac{4 \cdot sag - \Delta H}{L}\right) = 20.323 \cdot deg$$

$$P_{vlow} := P_h \cdot tan\left(\theta_{low}\right) = 10.674\,kip$$

$$P_{tlow} := \frac{P_h}{cos\left(\theta_{low}\right)} = 30.733\,kip$$

Reactions at Towers and Anchors

$$P_{tback} := \frac{P_h}{cos\left(\theta_{high}\right)} = 30.733\,kip$$

$$P_{vback} := P_{tback} \cdot sin\left(\theta_{high}\right) = 10.674\,kip$$

$$P_{tmain} := max\left(P_{thigh}, P_{tlow}\right) = 30.733\,kip$$

$$P_{vmain} := P_{tmain} \cdot sin\left(\theta_{high}\right) = 10.674\,kip$$

$$R_{tower} := P_{vback} + P_{vmain} = 21.348\,kip$$

$$R_{singletower} := \frac{R_{tower}}{2} = 10.674\,kip$$

Cable Live Load Response

Cable Tension

$$P_h := \frac{\left(w_{live} \cdot L^2\right)}{(8 \cdot sag)} = 94.77\ kip$$

$$\theta_{high} =: atan\left(\frac{4 \cdot sag + \Delta H}{L}\right) = 20.323 \cdot deg$$

$$P_{vhigh} := P_h \cdot tan\left(\theta_{high}\right) = 35.1\ kip$$

$$P_{thigh} := \frac{P_h}{cos\left(\theta_{high}\right)} = 101.061\ kip$$

$$\theta_{low} =: atan\left(\frac{4 \cdot sag - \Delta H}{L}\right) = 20.323 \cdot deg$$

$$P_{vlow} := P_h \cdot tan\left(\theta_{low}\right) = 35.1\ kip$$

$$P_{tlow} := \frac{P_h}{cos\left(\theta_{low}\right)} = 101.061\ kip$$

Reactions at Towers and Anchors

$$P_{tback} := \frac{P_h}{cos\left(\theta_{high}\right)} = 101.061\ kip$$

$$P_{vback} := P_{tback} \cdot sin\left(\theta_{high}\right) = 35.1\ kip$$

$$P_{tmain} := max\left(P_{thigh}, P_{tlow}\right) = 101.061\ kip$$

$$P_{vmain} := P_{tmain} \cdot sin\left(\theta_{high}\right) = 35.1\ kip$$

$$R_{tower} := P_{vback} + P_{vmain} = 70.2\ kip$$

$$R_{singletower} := \frac{R_{tower}}{2} = 35.1\ kip$$

Cable Live + Dead Load Response

Cable Tension

$$P_h := \frac{\left(w_{full} \cdot L^2\right)}{(8 \cdot sag)} = 123.589\ kip$$

$$\theta_{high} := atan\left(\frac{4 \cdot sag + \Delta H}{L}\right) = 20.323 \cdot deg$$

$$P_{vhigh} := P_h \cdot tan\left(\theta_{high}\right) = 45.774\ kip$$

$$P_{thigh} := \frac{P_h}{cos\left(\theta_{high}\right)} = 131.794\ kip$$

$$\theta_{low} := atan\left(\frac{4 \cdot sag - \Delta H}{L}\right) = 20.323 \cdot deg$$

$$P_{vlow} := P_h \cdot tan\left(\theta_{low}\right) = 45.774\ kip$$

$$P_{tlow} := \frac{P_h}{cos\left(\theta_{low}\right)} = 131.794\ kip$$

Reactions at Towers and Anchors

$$P_{tback} := \frac{P_h}{cos\left(\theta_{high}\right)} = 131.794\ kip$$

$$P_{vback} := P_{tback} \cdot sin\left(\theta_{high}\right) = 45.774\ kip$$

$$P_{tmain} := max\left(P_{thigh}, P_{tlow}\right) = 131.794\ kip$$

$$P_{vmain} := P_{tmain} \cdot sin\left(\theta_{high}\right) = 45.774\ kip$$

$$R_{tower} := P_{vback} + P_{vmain} = 91.548\ kip$$

$$R_{singletower} := \frac{R_{tower}}{2} = 45.774\ kip$$

$$P_{tsinglecable} := \frac{P_{tmain}}{2} = 65.897\ kip$$

Design Cable for Dead + Live

Desired SF of 3

$\Omega := 3$

$P_{req} := \Omega \cdot P_{tsinglecable} = 197.691 \text{ kip}$

For 1 and 5/8 cable

$P := 224\text{kip}$

A 1 and 5/8 inch cable is safe

Sheet Made By: Anthony Jaksa
Checked By: Erin Lau

Wind loading

Per ASCE 7-10

Bridge Parameters

$L_{bridge} := 270ft$

$Spacing := 5ft$

$t_{deck} := 3.5in$

$A_{xmember} := 1.5in \cdot 7.25in + 0.938in^2 \cdot 2 = 12.751\ in^2$ 2x8 wood plus Steel 2L2x2x0.250

$t_{cable} := 1.625in$ 1 and 5/8 inch steel wire rope

$t_{hanger} := 0.5in$ 1/2 inch wire rope hangers

$A_{avghanger} := 13.1ft \cdot t_{hanger} = 78.6\ in^2$

$A_{cable} := t_{cable} \cdot L_{bridge} \cdot 1.3 = 6.845 \times 10^3\ in^2$ 1.3 conservatively for length of cable compared to length of bridge

Drag Parameters from ASCE 7-10

$C_{dflat} := 2$

$C_{dround} := 1.3$

Calculate Wind Loads

$V := 100mph = 1.76 \times 10^3\ \frac{in}{s}$

$P := 0.00256 \cdot V^2 \cdot \left(\frac{1}{mph^2}\right) \cdot psf = 1.778 \times 10^{-4}\ ksi$

$F_2 := \frac{L_{bridge}}{Spacing} \cdot \left(A_{avghanger} \cdot C_{dround} + A_{xmember} \cdot C_{dflat}\right) \cdot P + A_{cable} \cdot P \cdot C_{dround} + t_{deck} \cdot L_{bridge} \cdot P$

$\frac{F_2}{2 \cdot 2} = 1.206\ kip$ Two connections per tower, two towers, this load will be applied to each column

$t_{tower} := 14in$

$w_{tower} := P \cdot t_{tower} \cdot C_{dround} = 38.827\ plf$ Distributed Load for the Side of the Tower

Appendix B

Tower Design

Created By: Anthony Jaksa

Checked By: Erin Lau

Tower Column Design

All Table and Equation References are to AISC Steel Construction Manual 15th edition unless otherwise stated

Material Properties

$F_y := 35\text{ksi}$ **Spec A500 Grade C Minimum** (B2P Section 3 Pg 11)

$E := 29000\text{ksi}$

Loads

Maximum from analysis considering wind, dead, and live loading in RISA

$P_{required} := 46.43\text{kip} = 46.43\text{ kip}$

$M_{max} := 15.75 \cdot \text{kip} \cdot \text{ft}$

$\Omega_M := 1.67$ (Section E1)

$\Omega_c := 1.67$

Conditions

$L := 30\text{ft}$

$r := 4.83\text{in}$

$A_g := 15\text{in}^2$

$w_{self} := 54.62\text{plf}$ HSS 14.000x0.375

$Z := 65.1\text{in}^3$

$S := 49.8\text{in}^3$

$I := 349\text{in}^4$

$D_t := 40.1$

$P_{required} := P_{required} + w_{self} \cdot L = 48.069\text{ kip}$

$k := 2$ (Table C-A-7.1, Idealized Flagpole, consider correct as there will be some restraint at the top)

Design

$$L_c := k \cdot L = 720\ \text{in}$$

$$F_e := \frac{\pi^2 \cdot E}{\left(\frac{L_c}{r}\right)^2} = 12.88\ \text{ksi}$$ (Eq E3-4)

$$4.71 \cdot \sqrt{\frac{E}{F_y}} = 135.577$$

$$\frac{L_c}{r} = 149.068 \qquad 52 < 135$$

$$F_{cr} := \left[0.658^{\left(\frac{F_y}{F_e}\right)}\right] \cdot F_y = 11.224\ \text{ksi}$$ (Eq E3-2)

$$P_n := F_{cr} \cdot A_g = 168.353\ \text{kip}$$ (Eq E3-1)

Compression Design Capacity

$$P_{nc\Omega} := \frac{P_n}{\Omega_c} = 100.81\ \text{kip}$$

$$P_{required} = 48.069\ \text{kip}$$

The member is good in compression

Moment Capacity

$$D_t < \frac{0.45E}{F_y} = 1$$

$$M_{ny} := F_y \cdot Z = 189.875\ \text{ft} \cdot \text{kip}$$

$$M_{nlb} := \left(\frac{0.021E}{D_t} + F_y\right) \cdot S = 208.276\ \text{ft} \cdot \text{kip}$$

$$M_{n\Omega} := \frac{M_{nlb}}{\Omega_M} = 124.716\ \text{ft} \cdot \text{kip}$$

$$M_{max} = 15.75\ \text{ft} \cdot \text{kip}$$

Combined Loading

$$\frac{P_{required}}{P_{nc\Omega}} = 0.477$$

$$\frac{P_{required}}{P_{nc\Omega}} + \frac{8}{9} \cdot \left(\frac{M_{max}}{M_{n\Omega}}\right) = 0.589$$ is below 1

HSS 14.00x0.375 is good

Made by: Erin Lau
Checked by: Anthony Jaksa

Tower Bracing Design

Purpose: Design the cross bracing for the towers. Towers are 8 ft on center apart.

Required Loading:

$Mem_1 := 1.8kip$

$Mem_3 := 3.9kip$ From Risa tower file.

$Mem_4 := 2.9kip$ compression

Select L3x3x1/4 Assume hole diameter to be .25in

$wt := 4.9\frac{lb}{ft}$ Table 1-7

$A_g := 1.44in^2$

$Ix := 1.23in^4$

$Sx := .569in^3$

$rx := .926in$

$F_y := 36ksi$ Table 2.4

$Fu := 58ksi$

$E := 29000ksi$

$t := \frac{1}{4}in$

$dia := .5in$

Tension Capacities

$\Omega := 1.67$

Yielding of the Gross Section

$Pny := F_y \cdot A_g = 51.84 \cdot kip$ D2-1

$\frac{Pny}{\Omega} = 31.042 \cdot kip$

Rupture of the Net Section

$U := 0.6$ Table D3.1 Case 8

$A_n := A_g - 2\left(dia + \frac{1}{8}in\right)t = 1.128 \cdot in^2$

$A_e := A_n \cdot U = 0.677 \cdot in^2$ D3-1

$Pnr := Fu \cdot A_e = 39.237 \cdot kip$ D2-2

$\Omega_{tr} := 2$

$\frac{Pnr}{\Omega_{tr}} = 19.619 \cdot kip$

Block Shear

$A_{gv} := .75in \cdot t = 0.187 \cdot in^2$

$A_{nv} := A_{gv} - .5 \cdot \left(dia + \frac{1}{8}in\right) \cdot t = 0.109 \cdot in^2$

$A_{nt} := (1.5in + .75in) \cdot t - 1.5\left(dia + \frac{1}{8}in\right) \cdot t = 0.328 \cdot in^2$

$U_{bs} := 1$

$Rn_1 := 0.6 \cdot F_y \cdot A_{gv} + U_{bs} \cdot Fu \cdot A_{nt} = 23.081 \cdot kip$ J4-5

$Rn := .6 \cdot Fu \cdot A_{nv} + U_{bs} \cdot Fu \cdot A_{nt} = 22.837 \cdot kip$

$\Omega_{bs} := 2$

$\frac{Rn}{\Omega_{bs}} = 11.419 \cdot kip$

$Mem_3 = 3.9 \cdot kip$ Member is good in tension

Compression Capacities

$\Omega_c := 1.67$

$K := 1.0$ Table C-A-7.1

$L_1 := 9.664\,ft$

$Lc_1 := K \cdot L_1 = 115.968 \cdot in$

$\dfrac{Lc_1}{rx} = 125.235$

$\dfrac{L_1}{rx} = 125.235$

$adj\_L := 32 + 1.25 \cdot \left(\dfrac{L_1}{rx}\right) = 188.544$

$4.71 \cdot \sqrt{\dfrac{E}{F_y}} = 133.681$

$F_e := \dfrac{\pi^2 \cdot E}{(adj\_L)^2} = 8.051 \cdot ksi$

$F_{cr} := .658^{\frac{F_y}{F_e}} \cdot F_y = 5.54 \cdot ksi$

$Pn_c := F_{cr} \cdot A_g = 7.978 \cdot kip$

$\dfrac{Pn_c}{\Omega_c} = 4.777 \cdot kip$

$Mem_4 = 2.9 \cdot kip$

Member is good in compression

$L_2 := 6.883\,ft$

$Lc_2 := K \cdot L_2 = 82.596 \cdot in$

$\dfrac{Lc_2}{rx} = 89.197$

$\dfrac{L_2}{rx} = 89.197$

$adj\_L1 := 32 + 1.25 \cdot \left(\dfrac{L_2}{rx}\right) = 143.496$ E5-2

$F_{e1} := \dfrac{\pi^2 \cdot E}{(adj\_L1)^2} = 13.9 \cdot ksi$

$F_{cr2} := .658^{\frac{F_y}{F_{e1}}} \cdot F_y = 12.177 \cdot ksi$

$P_{nc} := F_{cr2} \cdot A_g = 17.534 \cdot kip$

$\dfrac{P_{nc}}{\Omega_c} = 10.5 \cdot kip$

Use a L3x3x1/4 for the x-bracing of the tower.

Sheet Made By: Anthony Jaksa
Checked By: Erin Lau

Tower - Brace Connection Design

Material Properties

$F_y := 35ksi$

$F_u := 58ksi$

$E_{xx} := 60ksi$

$E := 29000ksi$

$F_{nt} := 60ksi$

Group A bolt threads included

$F_{nv} := 54ksi$

Parameters

$t := \frac{1}{4}in$ Half inch 4" wide by 6" tall steel plate

$t_w := 0.5in$

$d_{bolt} := \frac{1}{2}in$

$n_{bolt} := 2$

$l_{plate} := 6in$

$s_{edge} := 0.75in$ $L_{plate} := 4in$

Loads

$C_{maxtop} := .44kip$

$T_{maxbot} := 3.9kip$ $C_{maxbot} := 3.0kip$

$Split_T := \cos(45deg) \cdot T_{maxbot} = 2.758\ kip$ $Split_C := \cos(45deg) \cdot C_{maxbot} = 2.121\ kip$

$M_1 := L_{plate} \cdot Split_T = 0.919\ ft \cdot kip$ $M_2 := L_{plate} \cdot Split_C = 0.707\ ft \cdot kip$

Minimum Spacing

$s_{min}\ =:2.66 \cdot d_{bolt}\ =1.333\ in$

$s := 1.5in$

Hole Diameter

$d_{hole} := d_{bolt} + \frac{1in}{8} = 0.625\ in$

Design of bolts

$$A_b := \pi\left(\frac{d_{bolt}}{2}\right)^2 = 0.196\ in^2$$

$$R_n := F_{nv} \cdot A_b \cdot n_{bolt} = 21.206\ kip$$

$$\Omega := 2$$

$$\frac{R_n}{\Omega} = 10.603\ kip$$

$$T_{maxbot} = 3.9\ kip$$

Two 1/2 in Bolts are good

Design of Plate

Yielding of the gross section

$$R_{ny} := F_y \cdot \left(t \cdot l_{plate}\right) = 52.5\ kip$$

$$\Omega_{ty} := 1.67$$

$$R_{nty\Omega} := \frac{R_{ny}}{\Omega_{ty}} = 31.437\ kip$$

Rupture of the net section

$$A_e := t \cdot \left(l_{plate} - 3 \cdot d_{hole} - 1in\right) = 0.781\ in^2$$

$$R_{ntr} := F_u \cdot A_e = 45.312\ kip$$

$$\Omega_{tr} := 2$$

$$R_{ntr\Omega} := \frac{R_{ntr}}{\Omega_{tr}} = 22.656\ kip$$

$$R_{nt\Omega} := \min\left(R_{ntr\Omega}, R_{nty\Omega}\right) = 22.656\ kip$$

Shear yielding of the gross section

$$R_{nvy} := 0.6 \cdot F_y \cdot \left(t \cdot l_{plate}\right) = 31.5\ kip$$

$$\Omega_{vy} := 1.5$$

$$R_{nvy\Omega} := \frac{R_{nvy}}{\Omega_{vy}} = 21\ kip$$

Shear rupture of the net section

$R_{nvr} := 0.6 \cdot F_u \cdot A_e = 27.187\,\text{kip}$

$\Omega_{vr} := 2$

$R_{nvr\Omega} := \dfrac{R_{nvr}}{\Omega_{vr}} = 13.594\,\text{kip}$

$R_{nv\Omega} := \min\left(R_{nvy\Omega}, R_{nvr\Omega}\right) = 13.594\,\text{kip}$

Block Shear

$A_{nt1} := t \cdot \left(s_{edge} + s - 1.5 \cdot d_{hole}\right) = 0.328\,\text{in}^2$

$A_{nv1} := t \cdot \left(s_{edge} - 0.5 \cdot d_{hole}\right) = 0.109\,\text{in}^2$

$A_{gv1} := t \cdot \left(s_{edge}\right) = 0.187\,\text{in}^2$

$U_{bs} := 1$

$R_{nbs1} := \min\left(0.60 \cdot F_u \cdot A_{nv1} + U_{bs} \cdot F_u \cdot A_{nt1}, 0.6 \cdot F_y \cdot A_{gv1} + U_{bs} \cdot F_u \cdot A_{nt1}\right) = 22.837\,\text{kip}$

$A_{nt2} := t \cdot \left(1 \cdot d_{hole}\right) = 0.156\,\text{in}^2$

$A_{nv2} := 2t \cdot \left(s_{edge} - 0.5 \cdot d_{hole}\right) = 0.219\,\text{in}^2$

$A_{gv2} := 2t \cdot \left(s_{edge}\right) = 0.375\,\text{in}^2$

$R_{nbs2} := \min\left(0.60 \cdot F_u \cdot A_{nv2} + U_{bs} \cdot F_u \cdot A_{nt2}, 0.6 \cdot F_y \cdot A_{gv2} + U_{bs} \cdot F_u \cdot A_{nt2}\right) = 16.675\,\text{kip}$

$R_{nbs} := \min\left(R_{nbs1}, R_{nbs2}\right) = 16.675\,\text{kip}$

$\Omega_{bs} := 2$

$R_{nbs\Omega} := \dfrac{R_{nbs}}{\Omega_{bs}} = 8.337\,\text{kip}$

$T_{maxbot} = 3.9\,\text{kip}$

Check rupture of the net section for diagonal case, all other cases irrelevant or already calculated

$$h_r := l_{plate} - \left(s_{edge} + 2s - \sqrt{2 \cdot s_{edge}^2}\right) = 3.311\ in$$

$$l_{rupture} := \sqrt{2 \cdot h_r^2} = 4.682\ in$$

$$A_{e2} := t \cdot \left(l_{rupture} - 2 \cdot d_{hole}\right) = 0.858\ in^2$$

$$R_{ntr2} := F_u \cdot A_{e2} = 49.764\ kip$$

$$R_{ntr2} := \frac{R_{ntr}}{\Omega_{tr}} = 22.656\ kip$$

Check plate compressive buckling

$$r := \frac{t}{\sqrt{12}} = 0.072\ in$$

$$\frac{L_{plate}}{r} = 55.426$$

$$F_e := \frac{\pi^2 \cdot E}{\left(\frac{L_{plate}}{r}\right)^2} = 93.17\ ksi$$

$$4.71 \cdot \sqrt{\frac{E}{F_y}} = 135.577$$

$$F_{cr} := \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y = 29.908\ ksi$$

$$P_n := F_{cr} \cdot \left(t \cdot l_{plate}\right) = 44.862\ kip$$

$$\Omega_c := 1.67$$

$$\frac{P_n}{\Omega_c} = 26.863\ kip$$

Plate is safe in buckling

Weld Design

$A_w := 2 \cdot 0.707 \cdot t_w \cdot l_{plate} = 4.242\ in^2$

$F_{nw} := 0.60 \cdot E_{xx} = 36\ ksi$

$R_{nw} := A_w \cdot F_{nw} = 152.712\ kip$

$\Omega_w := 2$

$R_{nw\Omega} := \dfrac{R_{nw}}{\Omega_w} = 76.356\ kip$

$\tau_{momentpos} := \dfrac{3 \cdot M_1}{t_w \cdot l_{plate}^{\ 2}} = 1.838\ ksi$

$\tau_{momentneg} := \dfrac{3 \cdot M_2}{t_w \cdot l_{plate}^{\ 2}} = 1.414\ ksi$

$\tau_{directshear} := \dfrac{Split_T}{A_w} = 0.65\ ksi$

$\tau_{directload} := \dfrac{\max\left(Split_T - C_{maxtop}, Split_C + C_{maxtop}\right)}{A_w} = 0.604\ ksi$

Total possible stress from eccentric loading

$\tau_{max} := \tau_{momentpos} + \tau_{directshear} + \tau_{directload} = 3.092\ ksi$

$\tau_{max} \cdot \Omega_w = 6.185\ ksi$

This is far lower than the yield stresses of the welds and the steel, safe

Check Tower Shearing

$t_{tower} := 0.25in$

$A_{gvtower} := l_{plate} \cdot t_{tower} = 1.5\ in^2$

$R_{nv3} := 0.60 \cdot F_y \cdot A_{gvtower} = 31.5\ kip$

$R_{nv3\Omega} := \dfrac{R_{nv3}}{\Omega_{vy}} = 21\ kip$ $Split_T = 2.758\ kip$ Safe

Created by: Erin Lau

Checked by: Anthony Jaksa

Base Plate Design

Purpose: Design a base plate for each of the towers.

Loads

$Mu := 15.42 kip \cdot ft$

$Pu := 45 kip$

$\Omega_b := 2.31$

$\Omega_c := 1.67$

Assume Dimensions of Base Plate

$B := 2 ft$

$N := B$

$A_1 := B \cdot N = 4 \cdot ft^2$

Material

$F_y := 36 ksi$ Table 2.4

$Fu := 58 ksi$

Bearing J8-1

$P_p := 0.85 \cdot 1500 psi \cdot A_1 = 734.4 \cdot kip$ $1.7 \cdot 1000 psi \cdot A_1 = 979.2 \cdot kip$

$\frac{P_p}{\Omega_c} = 439.76 \cdot kip$

$Pu < P_p = 1$ True, the concrete will not crush.

$d := 14 in$

$b_f := d$

Base Plate

$$X := \left[\frac{4 \cdot d \cdot b_f}{\left(d + b_f\right)^2}\right] \cdot \frac{Pu}{\frac{P_p}{\Omega_c}} = 0.102$$

$$m_1 := \frac{N - .95 \cdot d}{2} = 5.35 \cdot in$$

$$n := \frac{B - .8 \cdot b_f}{2} = 6.4 \cdot in$$

$$n' := \frac{\sqrt{d \cdot b_f}}{4} = 3.5 \cdot in$$

$$\lambda := \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} = 0.329$$ less than one -> good

$$l := \max\left(m_1, n, n'\right) = 6.4 \cdot in$$

$$t_{min} := l \cdot \sqrt{\frac{2 \cdot Pu}{.9 \cdot F_y \cdot B \cdot N}} = 0.444 \cdot in$$

Assume thickness of 1/2in

$$t := \frac{1}{2} in$$

$$Mn := \frac{F_y \cdot B \cdot t^2}{4} = 4.5 \cdot kip \cdot ft$$

$$\frac{Mn}{\Omega_b} = 1.948 \cdot kip \cdot ft$$

Choose larger thickness

$$t_1 := 1.5in = 1.5 \cdot in$$

$$Mn_1 := \frac{F_y \cdot B \cdot {t_1}^2}{4} = 40.5 \cdot kip \cdot ft$$

$$\frac{Mn_1}{\Omega_b} = 17.532 \cdot kip \cdot ft$$

$$Mu < 17.532 kip \cdot ft = 1$$

A base plate with B and N of 2ft and a thickness of 1.5 in is sufficient to support the column.

Sheet Made By: Anthony Jaksa
Checked By: Erin Lau

Tower Base Plate Anchor Design

Material Properties

$f'_c := 1500psi$

Conditions

$d_{cover} := 3in \quad d_{stirrup} := 0.375in$

$c_a := d_{cover} + 6d_{stirrup} = 5.25\,in$

Loads

$M_{req} := 27.1kip\cdot ft$

$V_{req} := 2.4kip$

Design Parameters

$h_{ef} := 12in$

$d_a := 0.625in$

$d_h := 1.25in$

Tower Anchors

$$T_{req} := \frac{\frac{1}{2}M_{req}}{2\cdot 8in} = 10.163\,kip$$

$$A_{Nc} := \left(3\cdot h_{ef}\right)^2 = 1.296\times 10^3\,in^2$$

$$A_{Nco} := 9\cdot h_{ef}^{\;2} = 1.296\times 10^3\,in^2$$

$k_c := 24$

$$N_b := k_c\cdot\sqrt{\frac{f'_c}{psi}}\cdot\left(\frac{h_{ef}}{in}\right)^{1.5}\cdot lbf = 38.639\cdot kip$$

$\psi_{edN} := 0.7 + 0.3 \cdot \frac{c_a}{h_{ef}} = 0.831$

$\psi_{cN} := 1$ Assume cracking conservatively

$\psi_{cpN} := 1$

$N_{cb} := \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{edN} \cdot \psi_{cN} \cdot \psi_{cpN} \cdot N_b = 32.119\,\text{kip}$ is below $T_{req} = 10.163\,\text{kip}$

$A_{brg} := \pi \cdot \left(\frac{d_h}{2}\right)^2 - \pi \cdot \left(\frac{d_a}{2}\right)^2 = 0.92\,\text{in}^2$

$N_p := 8 \cdot A_{brg} \cdot f'_c = 11.045\,\text{kip}$

$\psi_{cP} := 1$

$N_{pn} := N_p \cdot \psi_{cP} = 11.045\,\text{kip}$ is below $T_{req} = 10.163\,\text{kip}$

$A_{Vc} := 1.5 \cdot 3 \cdot c_a^{\,2} = 124.031\,\text{in}^2$

$A_{Vc0} := 4.5 \cdot c_a^{\,2} = 124.031\,\text{in}^2$

$\psi_{edV} := 0.7 + 0.3 \cdot \frac{c_a}{1.5 \cdot c_a} = 0.9$

$\psi_{cV} := 1$

$\psi_{hV} := 1$

$V_b := \min\left[\left[7 \cdot \left(\frac{h_{ef}}{d_a}\right)^{0.2} \cdot \sqrt{\frac{d_a}{\text{in}}}\right] \cdot \sqrt{\frac{f'_c}{\text{psi}}} \cdot \left(\frac{c_a}{\text{in}}\right)^{1.5}, 9 \cdot \sqrt{\frac{f'_c}{\text{psi}}} \cdot \left(\frac{c_a}{\text{in}}\right)^{1.5}\right] \cdot \text{lbf} = 4.193\,\text{kip}$

$V_{bn} := V_b \cdot \psi_{edV} \cdot \psi_{cV} \cdot \psi_{hV} = 3.774\,\text{kip}$ is below $V_{req} = 2.4\,\text{kip}$

$\Phi := 0.7$

$\frac{T_{req}}{N_{cb} \cdot \Phi} + \frac{0.5 \cdot V_{req}}{V_{bn} \cdot \Phi} = 0.906$ Anchors are good

12 inch deep 5/8 inch headed bolts, head size 1.25 inches

Bolt Design

Material Properties

Assume Group A bolts, threads not excluded

$F_{nt} := 90ksi$ AISC Table J3.2

$F_{nv} := 54ksi$

Design by Section J3.7

$\Omega := 2$

$$A_b := \pi \cdot \left(\frac{d_a}{2}\right)^2 = 0.307\ in^2$$

$$F_{nt'} := 1.3 \cdot F_{nt} - \frac{\Omega \cdot F_{nt}}{F_{nv}} \cdot \frac{V_{req}}{A_b} = 90.924\ ksi$$

$$R_n := F_{nt'} \cdot A_b = 27.895\ kip$$

$$R_{n\Omega} := \frac{R_n}{\Omega} = 13.948\ kip$$

$$T_{req} = 10.163\ kip$$

Safe

Appendix C

Decking Design

Sheet Made By: Anthony Jaksa
Checked By: Erin Lau

Decking Design

Loads

$$w_{live} := 85 \frac{lbf}{ft^2}$$

$$P_{max} := 500 lbf$$

Conditions

$$L := 5 ft$$

$$Width := 4 ft$$

Material Properties

Assume Beech-Birch-Hickory (Standard)

$$F_b := 650 \frac{lbf}{in^2}$$

$$G := 0.71$$

$$E := 1300000 \frac{lbf}{in^2}$$

$$mc := 30\%$$

$$\gamma_{self} := 62.4 \frac{lbf}{ft^3} \cdot \left[\frac{G}{1 + G \cdot (0.009) \cdot (30)} \right] \cdot \left(1 + \frac{mc}{100\%} \right) = 48.33 \text{ pcf}$$

Parameters

$$b := 11.25 in$$ Nominal 4x12

$$d := 3.5 in$$

Combos

$$w_{self} := \gamma_{self} \cdot (Width)(d) = 56.385 \text{ plf}$$

$$w_D := w_{self} = 56.385 \text{ plf}$$

$$w_L := w_{live} \cdot b = 79.688 \text{ plf}$$

Loading

See diagrams, each board spans 3 bays

$M_{distributedD} := -0.100(w_D) \cdot L^2 = -0.141 \cdot kip \cdot ft$ (AISC Table 3-23 Case 39)

$M_{distributedDandL} := -0.117(w_L) \cdot L^2 - 0.100 \cdot w_D \cdot L^2 = -0.374 \cdot kip \cdot ft$ (AISC Table 3-23 Cases 37 and 39)

$M_{concentratedDandL} := -0.505 \cdot kip \cdot ft = -0.505 \cdot kip \cdot ft$ (RISA)

Concentrated live load case controls

$$f_{blive} := \frac{(6 \cdot M_{concentratedDandL})}{b \cdot d^2} = -0.264 \cdot ksi$$

Design

Factors

$C_D := 1$

$C_m := 0.85$

Allowable Bending Stress

$F_b := C_D \cdot C_m \cdot F_b = 0.553 \cdot ksi$

Design Capacities

$F_b = 0.553 \text{ ksi}$

$|f_{blive}| = 0.264 \text{ ksi}$

$F_b > |f_{blive}| = 1$

A 4x12 Beech Birch Hickory is safe

Total self weight per length bridge

$w_{total} := w_{self} \cdot Width = 0.226 \text{ kip}$

Sheet Made By: Anthony Jaksa

Checked By: Erin Lau

Cross-member Design

All Table and Equation References are to AISC Steel Construction Manual 15th edition unless otherwise stated

Loads

$w_{live} := 65\frac{lbf}{ft^2} = 4.514 \times 10^{-4}\,ksi$

$P_{max} := 500lbf$

$\Omega_m := 1.67$

Conditions

$Spacing := 5ft$ Length between cross members along bridge

$L := 5ft$ Length of cross-member

$L_b := \frac{L}{3} = 20\,in$ Bolted at 3rd points

(Sect F1 (a))

Material Properties

$F_y := 35\frac{kip}{in^2} = 35\,ksi$ (B2P Section 3 Pg 11)

Spec A36 Steel

$E := 29000ksi = 2.9 \times 10^4\,ksi$

Section Properties 2L2x2x1/4

(Table 1-7)

$Z_y := 2 \cdot 0.440in^3 = 0.88\,in^3$

$S_y := 2 \cdot 0.244in^3 = 0.488\,in^3$

$r_y := 1.37in$ (Table 1-15)

$I_y := 2 \cdot 0.346in^4 = 0.692\,in^4$ From Parallel Axis Theorum

$J := 2 \cdot 0.0209in^4 = 0.042\,in^4$

$h := 2in$

$t_w := \frac{1}{4} \cdot in = 0.25\,in$

Loading

Assume simply supported beam

$w_{self} := 2 \cdot 3.19 \text{plf}$

$w_{factored} := w_{live} \cdot \text{Spacing} + w_{self} = 331.38 \text{ plf}$

(Table 3-23)

$$M_{distributed} := \frac{(w_{factored}) \cdot L^2}{8} = 1.036 \text{ ft} \cdot \text{kip}$$

$$M_{concentrated} := \frac{P_{max} \cdot L}{4} + \frac{w_{self} \cdot L^2}{8} = 0.645 \text{ ft} \cdot \text{kip}$$

$$V_{distributed} := \frac{w_{factored} \cdot L}{2} = 0.828 \text{ kip}$$

$$V_{concentrated} := P_{max} + \frac{w_{self} \cdot L}{2} = 0.516 \text{ kip}$$

Design

Moment

Limit state of yielding

$M_n := F_y \cdot Z_y = 2.567 \text{ ft} \cdot \text{kip}$ (Eq F9-1) is below or equal to $1.6 \cdot F_y \cdot S_y = 2.277 \text{ ft} \cdot \text{kip}$ (Eq F9-2) FALSE

$M_n := 1.6 F_y \cdot S_y = 2.277 \text{ ft} \cdot \text{kip}$

Limit State of Lateral Torsional Buckling

$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 69.406 \text{ in}$ is above $L_b = 20 \text{ in}$ (Eq F9-8)

LTB does not apply

Shear

$\Omega_v := 1.67$

$k_v := 5$ (Section G4)

$\frac{h}{t_w} = 8$ is below $1.1 \cdot \sqrt{\frac{k_v \cdot E}{F_y}} = 70.802$ (Section G2-2)

$C_{v2} := 1$ (Eq G2-9)

$A_w := 2 \cdot h \cdot t_w = 1\ in^2$

$V_n := 0.6 \cdot F_y \cdot A_w \cdot C_{v2} = 21\ kip$ (Eq G4-1)

Design Capacity

$\frac{M_n}{\Omega_m} = 1.364\ ft \cdot kip$

$\frac{V_n}{\Omega_v} = 12.575\ kip$

Required Strength

$M_{distributed} = 1.036\ ft \cdot kip$

$\frac{M_n}{\Omega_m} > M_{distributed} = 1$

$\frac{V_n}{\Omega_v} > V_{distributed} = 1$

A 2L2x2x1/4 with 1.5 inches separation is good

Design welds and rod to connect to hanger

Design welds for "Tension" ie force at 90 degrees to weld

Loads

$P_{hanger} := \max(0.706\text{kip}, 1.2\cdot 65\text{psf}\cdot 4\text{ft}\cdot 5\text{ft}) = 1.56\,\text{kip}$ From deck beam concentrated load and 3 span beam distributed load

Conditions and Parameters

$d_{rod} := 0.625\text{in}$ 5/8 inch min dimension

$t_{weld} := 0.25\text{in}$ Minimum yield stress of 35ksi, minimum Group A bolt material or 40ksi rebar material

$n_{weld} := 2$

$l_{weld} := 2\text{in}$

$E_{xx} := 60\text{ksi}$

$$A_{rod} := \pi\cdot\left(\frac{d_{rod}}{2}\right)^2 = 0.307\,\text{in}^2$$

$A_{weld} := n_{weld}\cdot t_{weld}\cdot 0.707\cdot l_{weld} = 0.707\,\text{in}^2$

Check shear in rod

$V_{nr} := 0.6\cdot F_y\cdot A_{rod} = 6.443\,\text{kip}$ (AISC G2-1)

Use SF of 3 to prevent progressive failure

$\Omega_{hanger} := 3$

$$V_{nr\Omega} := \frac{V_{nr}}{\Omega_{hanger}} = 2.148\,\text{kip}$$

Check Shear in weld

$F_{nw} := 0.6\cdot E_{xx}\cdot\left(1 + 0.5\cdot\sin(90\text{deg})^{1.5}\right) = 54\,\text{ksi}$

$R_{nw} := F_{nw}\cdot A_{weld} = 38.178\,\text{kip}$

$$R_{nw\Omega} := \frac{R_{nw}}{\Omega_{hanger}} = 12.726\,\text{kip}$$

A 5/8in rod with two 1/4inch welds 2 in long is good

$V_{nr\Omega} > P_{hanger} = 1$ $R_{nw\Omega} > P_{hanger} = 1$

Made by: Erin Lau
Checked by: Anthony Jaksa

Decking Cross Member Connection Design

Purpose: Design the connections between the decking and the wood plank and the wood plank and the steel double angle members of the cross supports.

Using a 4x12 Beech Birch Hickory spanning 15ft (3 hanger widths spaced at 5ft each) for the decking, and a 2L2x2x1/4 with 0.75 inches of speparation. The cross members are 5ft in width extending 6in past the edge on each side. This allows for adequate space for a hanger connection and a welded bar to connect the angles to each other.

Dimensions from Table 1B ANSI supplement

$w_s := .25in$

$b_1 := 1.5in$

$w_1 := 7.25in$

$b_2 := 3.5in$

$w_2 := 11.25in$

$\frac{w_1}{2} = 3.625 \cdot in$

$w_1 - 2 \cdot 3.5in = 0.25 \cdot in$

$spacing := .25in$

$4 \cdot w_2 = 3.75 \cdot ft$ just shy of 4 ft in width, 4 planks of decking will span the width of the bridge

Lateral Load Capacities

Wood to Wood

$Zx_1 := 193lb$ .216, 1.5in side member Table 12L wood screw

$C_g := 1$ Section 11.3.6

$C_\Delta := 1.0$ Section 12.5.1

Steel to Wood

$Z_x := 180lb$ Z perp, 1/4 thickness, 1/4 in dia table 12K, lag screw

$C_d := 2.0$ Appendix B.3

$C_m := 0.7$ Table 11.3.3

$C_t := 0.7$ Table 11.3.4

$Z_{1'} := Zx_1 \cdot C_d \cdot C_m \cdot C_t \cdot C_g \cdot C_\Delta = 189.14 \cdot lb$

$Z' := Z_x \cdot C_d \cdot C_m \cdot C_t \cdot C_g \cdot C_\Delta = 176.4 \cdot lb$

Required Loading

$W := 68lb$

From previous design calculations

$Wind := 43lb$

$\boxed{Wind < Z' < Z_{1'} = 1}$

True lateral loading is good

Withdrawl Capacity

Wood to Wood

$Wx_1 := 310 \frac{lb}{in}$

Table 12.2B G=.71 and #12 Screw

$W1' := Wx_1 \cdot C_d \cdot C_m \cdot C_t = 303.8 \cdot \frac{lb}{in}$

$D_1 := .225in$

$Pmin := 6 \cdot D_1 = 1.35 \cdot in$

$L_1 := 5in$ Table L3

$P_1 := L_1 - 3.5in = 1.5 \cdot in$

$W' \cdot P_1 = 560.07 \cdot lb$

$W' \cdot P_1 \cdot 2 = 1.12 \times 10^3 \cdot lb$

Steel to Wood

$W_x := 381 \frac{lb}{in}$

Table 12.2a G=.71 and 1/4"

$W' := W_x \cdot C_d \cdot C_m \cdot C_t = 373.38 \cdot \frac{lb}{in}$

$D := .25in$

$Pmin := 6 \cdot D = 1.5 \cdot in$

$L := 3in$ Table L2

$P := L - 1.5in = 1.5 \cdot in$

$W1' \cdot P = 455.7 \cdot lb$

$W1' \cdot P \cdot 1 = 455.7 \cdot lb$

$\boxed{W < W1' \cdot P < W' \cdot P_1 = 1}$

True, design is good for withdrawl

Spacing Requirements

Table C12.1.5.7

Wood Side members

$\text{Edge\_ditance} := 2.5 \cdot D_1 = 0.563 \cdot \text{in}$

$\text{End\_distance} := 10 \cdot D_1 = 2.25 \cdot \text{in}$

$\text{space} := 15 \cdot D_1 = 3.375 \cdot \text{in}$

$\text{btw\_rows} := 5 \cdot D_1 = 1.125 \cdot \text{in}$

Steel Side Members

$\text{Edge\_distance}_s := 2.5 \cdot D = 0.625 \cdot \text{in}$

$\text{End\_distance}_s := 10 \cdot D = 2.5 \cdot \text{in}$

$\text{space}_s := 10 \cdot D = 2.5 \cdot \text{in}$

$\text{btw\_row}_s := 3 \cdot D = 0.75 \cdot \text{in}$

Sheet Made By: Anthony Jaksa
Checked By: Erin Lau

All Table and Equation References are to AISC Steel Construction Manual 15th edition unless otherwise stated

Deck Hanger Design

Loads

$P := \max(0.706\text{kip}, 1.2 \cdot 65\text{psf} \cdot 4\text{ft} \cdot 5\text{ft}) = 1.56\,\text{kip}$

Maximum reaction from concentrated load beam RISA file, maximum reaction from 3 span beam 2 loaded

$w_{self} := 0.376\text{plf}$

$\Omega_1 := 3$

$\Omega_2 := \Omega_1 \cdot \left(\frac{2}{1.67}\right) = 3.593$

Increase SF by judgement, hanger cable breakage could cause progressive failure

Material Properties

$F_y := 35\text{ksi}$ **Spec Grade 40 Rebar Minimum** (B2P Section 3 Pg 11)

$F_u := 58\text{ksi}$ (Table 2-4)

$E := 29000\text{ksi}$

Size

$L_{max} := 30\text{ft}$

$d := \frac{1}{2} \cdot \text{in}$ #5 Rebar (Minimum per B2P Section 4.3)

$A := \pi \cdot \left(\frac{d}{2}\right)^2 = 0.196\,\text{in}^2$

Design

Loading

$P_{required} := P + w_{self} \cdot L_{max} = 1.571\ \text{kip}$

Tensile Yielding

$P_{n1} := F_y \cdot A = 6.872\ \text{kip}$ (Eq D2-1)

Rupture of the net section

$P_{n2} := F_u \cdot A = 11.388\ \text{kip}$ (Eq D2-2)

Design Capacity

$$\frac{P_{n1}}{\Omega_1} = 2.291\ \text{kip}$$

$$\frac{P_{n2}}{\Omega_2} = 3.17\ \text{kip}$$

Tensile Yielding Controls

$P_{required} = 1.571\ \text{kip}$

$$\frac{P_{n1}}{\Omega_1} > P_{required} = 1$$

A 1/2 inch deformed steel bar is good

Appendix D

Foundation Design

Created By: Anthony Jaksa

Checked By: Erin Lau

Tower Foundation Design

Loads

$SF := 4$

$P_2 := 48.069kip = 48.069\,kip \qquad P := P_2 \cdot 2 = 96.138\,kip$

Check for both sand and clay soils as depth of sand is not known

Design Parameters

Rectangular Foundation

$B := 6ft$

$L := 12ft$

$D_f := 2ft$

$\beta := 0deg$

$Pr := \frac{P}{B \cdot L} = 9.273 \times 10^{-3}\,ksi$ Actual Pressure

Pressure Per Unit Length

$w := Pr \cdot B = 8.011\,klf$

$inset := 2ft$ Inset of column on foundation

Minimum thickness

$\frac{L}{20} = 7.2\,in \qquad good$

$A_{smin} := 0.0020 \cdot 6ft \cdot 14in = 2.016\,in^2$

Max spacing

$3 \cdot 14in = 42\,in$

18 in actual

Clay - High Plasticity

$c := 10\text{kPa} = 1.45 \times 10^{-3}\,\text{ksi}$

Approximate worst-case values

$\Phi := 17\text{deg} = 0.297$

$\gamma := 115\text{pcf} = 115\,\text{pcf}$

Design

$$N_q := \tan\left(45\cdot\text{deg} + \frac{\Phi}{2}\right)^2 \cdot e^{\pi\cdot\tan(\Phi)} = 4.772$$ (Das 4.27)

$$N_c := \left(N_q - 1\right)\cdot\cot(\Phi) = 12.338$$ (Das 4.28)

$$N_\gamma := 2\cdot\left(N_q + 1\right)\cdot\tan(\Phi) = 3.529$$ (Das 4.29)

(Das Table 4.3)

$$F_{cs} := 1 + \left(\frac{B}{L}\right)\cdot\left(\frac{N_q}{N_c}\right) = 1.193$$

$$F_{qs} := 1 + \left(\frac{B}{L}\right)\tan(\Phi) = 1.153$$

$$F_{\gamma s} := 1 - 0.4\left(\frac{B}{L}\right) = 0.8$$

$$F_{qd} := 1 + 2\cdot\tan(\Phi)\cdot\left(1 - \sin(\Phi)\right)^2\cdot\left(\frac{D_f}{B}\right) = 1.102$$

$$F_{cd} := F_{qd} - \frac{1 - F_{qd}}{N_c\cdot\tan(\Phi)} = 1.129$$

$F_{\gamma d} := 1$

$F_{ci} := 1$

$F_{qi} := 1$

$F_{\gamma i} := 1$

$q := D_f\cdot\gamma = 1.597 \times 10^{-3}\,\text{ksi}$

$$q_u := c\cdot N_c\cdot F_{cs}\cdot F_{cd}\cdot F_{ci} + q\cdot N_q\cdot F_{qs}\cdot F_{qd}\cdot F_{qi} + \frac{1}{2}\cdot\gamma\cdot B\cdot N_\gamma\cdot F_{\gamma s}\cdot F_{\gamma d}\cdot F_{\gamma i} = 0.041\,\text{ksi}$$ (Das 4.26)

$$q_{actual} := \frac{P}{B\cdot L} = 9.273 \times 10^{-3}\,\text{ksi}$$

$$SF_{actual} := \frac{q_u}{q_{actual}} = 4.374$$

Sand

$c := 0\text{kPa} = 0$

$\Phi := 34\text{deg} = 0.593$ **Approximate sand values**

$\gamma := 110\text{pcf} = 110\,\text{pcf}$

Design

$$N_q := \tan\left(45\cdot\text{deg} + \frac{\Phi}{2}\right)^2\cdot e^{\pi\cdot\tan(\Phi)} = 29.44$$ (Das 4.27)

$$N_c := \left(N_q - 1\right)\cdot\cot(\Phi) = 42.164$$ (Das 4.28)

$$N_\gamma := 2\cdot\left(N_q + 1\right)\cdot\tan(\Phi) = 41.064$$ (Das 4.29)

$$F_{cs} := 1 + \left(\frac{B}{L}\right)\cdot\left(\frac{N_q}{N_c}\right) = 1.349$$ (Das Table 4.3)

$$F_{qs} := 1 + \left(\frac{B}{L}\right)\tan(\Phi) = 1.337$$

$$F_{\gamma s} := 1 - 0.4\left(\frac{B}{L}\right) = 0.8$$

$$F_{qd} := 1 + 2\cdot\tan(\Phi)\cdot\left(1 - \sin(\Phi)\right)^2\cdot\left(\frac{D_f}{B}\right) = 1.087$$

$$F_{cd} := F_{qd} - \frac{1 - F_{qd}}{N_c\cdot\tan(\Phi)} = 1.09$$

$F_{\gamma d} := 1$

$F_{ci} := 1$

$F_{qi} := 1$

$F_{\gamma i} := 1$

$$q := D_f\cdot\gamma = 1.528\times10^{-3}\,\text{ksi}$$

$$q_u := c\cdot N_c\cdot F_{cs}\cdot F_{cd}\cdot F_{ci} + q\cdot N_q\cdot F_{qs}\cdot F_{qd}\cdot F_{qi} + \frac{1}{2}\cdot\gamma\cdot B\cdot N_\gamma\cdot F_{\gamma s}\cdot F_{\gamma d}\cdot F_{\gamma i} = 0.141\,\text{ksi}$$ (Das 4.26)

$$q_{actual} := \frac{P}{B\cdot L} = 9.273\times10^{-3}$$ $$SF_{actual} := \frac{q_u}{q_{actual}} = 15.172$$

Structural Design

By Rigid Method

Spanning the long way

Loading from beam analogy

$V_{maxpos} := inset \cdot w = 16.023\,kip$

$V_{maxneg} := inset \cdot w - P_2 = -32.046\,kip$

$M_{maxpos} := V_{maxpos} \cdot 0.5 \cdot inset = 16.023\,ft \cdot kip$

$M_{maxneg} := V_{maxpos} \cdot 0.5 \cdot inset - V_{maxneg} \cdot 0.5 \cdot \left[\frac{(L - 2inset)}{2}\right] = 80.115\,ft \cdot kip$

$b := B = 72\,in$

Material Properties

$f_y := 35ksi$ (B2P Section 3 Page 11)

$f'_c := 1.5ksi$

$\varepsilon_{cu} := 0.003$ (Assumed Per ACI)

$E := 29000ksi$

Parameters

$h := 15in$

$d := h - 3in$

$A_{sneg} := 4.5in^2$ Negative Moment Reinforcement

$A_{spos} := 1in^2$ Positive Moment Reinforcement

$\frac{A_{sneg}}{\left(0.44in^2\right)} = 10.227$

12 (conservatively) #6 Bars at 6 inches OC

$\frac{A_{spos}}{\left(0.20in^2\right)} = 5$

6 #4 bars at 12 inches OC

Check Strength

Negative Moment Condition

$$a := \frac{A_{sneg} \cdot f_y}{0.85 \cdot f'_c \cdot b} = 1.716 \text{ in}$$

$$M_n := A_{sneg} \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 146.241 \text{ ft} \cdot \text{kip}$$

$$\beta_1 := 0.85$$

$$c := \frac{a}{\beta_1} = 2.018 \text{ in}$$

$$\varepsilon_t := \frac{\varepsilon_{cu}}{c} \cdot (d - c) = 0.015$$

$$\varepsilon_{ty} := \frac{f_y}{E} = 1.207 \times 10^{-3}$$

As the actual strain in the tensile reinforcement is greater than 0.005, this is a tension controlled section and a phi of 0.90 may be used. A corresponding safety factor for ASD design was obtained working backwards by using a live to dead ratio of 3

$$\Omega_m := \frac{1.2 \cdot \frac{1}{4} + 1.6 \cdot \frac{3}{4}}{0.90} = 1.667$$

$$M_{design} := \frac{M_n}{\Omega_m} = 87.744 \text{ ft} \cdot \text{kip}$$

Good in moment

$$M_{maxneg} = 80.115 \text{ ft} \cdot \text{kip}$$

Shear

$$V_c := 2 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi \cdot b \cdot d = 66.925 \text{ kip}$$

$$5V_c = 334.626 \text{ kip}$$

$$\Omega_v := \frac{1.2 \cdot \frac{1}{4} + 1.6 \cdot \frac{3}{4}}{0.75} = 2$$

$$V_{design} := \frac{V_c}{\Omega_v} = 33.463 \text{ kip}$$

Good in shear

$$V_{maxneg} = -32.046 \text{ kip}$$

Positive Moment Condition

$$a := \frac{A_{spos} \cdot f_y}{0.85 \cdot f'_c \cdot b} = 0.381 \text{ in}$$

$$M_n := A_{spos} \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 34.444 \text{ ft} \cdot \text{kip}$$

$$\beta_1 := 0.85$$

$$c := \frac{a}{\beta_1} = 0.449 \text{ in}$$

$$\varepsilon_t := \frac{\varepsilon_{cu}}{c} \cdot (d - c) = 0.077$$

$$\varepsilon_{ty} := \frac{f_y}{E} = 1.207 \times 10^{-3}$$

As the actual strain in the tensile reinforcement is greater than 0.005, this is a tension controlled section and a phi of 0.90 may be used. A corresponding safety factor for ASD design was obtained working backwards by using a live to dead ratio of 3

$$\Omega := \frac{1.2 \cdot \frac{1}{4} + 1.6 \cdot \frac{3}{4}}{0.90} = 1.667$$

$$M_{design} := \frac{M_n}{\Omega} = 20.666 \text{ ft} \cdot \text{kip}$$

good in moment

$$M_{maxpos} = 16.023 \text{ ft} \cdot \text{kip}$$

Spanning the short way

Loading from beam analogy

$$V_{max} := w \cdot \frac{B}{2} = 24.035\,\text{kip}$$

$$M_{max} := V_{max} \cdot \left(\frac{B}{2}\right) \cdot 0.5 = 36.052\,\text{ft}\cdot\text{kip}$$

$$b := B = 72\,\text{in}$$

Parameters

$A_{sneg} := 1\text{in}^2$ Negative Moment Reinforcement

$A_{spos} := 2\text{in}^2$ Positive Moment Reinforcement

$\frac{A_{sneg}}{0.20\text{in}^2} = 5$ 18 inch max spacing $\frac{(L - 6\text{in})}{18\text{in}} = 7.667$ 8 bars at approx 18" OC

$\frac{A_{spos}}{0.20\text{in}^2} = 10$ 5 #4 at 6 inch OC below each column, 4 additional at 12 inch OC throughout

Check Strength

Negative Moment Condition

$$a := \frac{A_{sneg} \cdot f_y}{0.85 \cdot f'_c \cdot b} = 0.381 \text{ in}$$

$$M_n := A_{sneg} \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 34.444 \text{ ft} \cdot \text{kip}$$

$$\beta_1 := 0.85$$

$$c := \frac{a}{\beta_1} = 0.449 \text{ in}$$

$$\varepsilon_t := \frac{\varepsilon_{cu}}{c} \cdot (d - c) = 0.077$$

$$\varepsilon_{ty} := \frac{f_y}{E} = 1.207 \times 10^{-3}$$

As the actual strain in the tensile reinforcement is greater than 0.005, this is a tension controlled section and a phi of 0.90 may be used. A corresponding safety factor for ASD design was obtained working backwards by using a live to dead ratio of 3

$$\Omega := \frac{1.2 \cdot \frac{1}{4} + 1.6 \cdot \frac{3}{4}}{0.90} = 1.667$$

$$M_{design} := \frac{M_n}{\Omega} = 20.666 \text{ ft} \cdot \text{kip}$$

No actual negative moment

The member is good in negative moment

Positive Moment Condition

$$a := \frac{A_{spos} \cdot f_y}{0.85 \cdot f'_c \cdot b} = 0.763\ \text{in}$$

$$M_n := A_{spos} \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 67.776\ \text{ft} \cdot \text{kip}$$

$$\beta_1 := 0.85$$

$$c := \frac{a}{\beta_1} = 0.897\ \text{in}$$

$$\varepsilon_t := \frac{\varepsilon_{cu}}{c} \cdot (d - c) = 0.037$$

$$\varepsilon_{ty} := \frac{f_y}{E} = 1.207 \times 10^{-3}$$

As the actual strain in the tensile reinforcement is greater than 0.005, this is a tension controlled section and a phi of 0.90 may be used. A corresponding safety factor for ASD design was obtained working backwards by using a live to dead ratio of 3

$$\Omega := \frac{1.2 \cdot \frac{1}{4} + 1.6 \cdot \frac{3}{4}}{0.90} = 1.667$$

$$M_{design} := \frac{M_n}{\Omega} = 40.666\ \text{ft} \cdot \text{kip}$$

good in moment

$$M_{max} = 36.052\ \text{ft} \cdot \text{kip}$$

$$M_{design} > M_{max} = 1$$

The member is good in positive moment

Shear

$$V_c := 2 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi \cdot b \cdot h = 83.656\,kip$$

$$V_{design} := \frac{V_c}{\Omega_v} = 41.828\,kip$$

$$V_{max} = 24.035\,kip$$

Member is good in shear

Summary

6'x12' 14" thick concrete foundation with #6 bars at 6" OC on top and #4 bars at 12" OC on bottom spanning the long direction, and #4 bars at 6" OC under the columns as 12" OC elsewhere on bottom, and #4 bars at 18" OC on top spanning the short way.

Sheet Made By: Anthony Jaksa
Checked By: Erin Lau

Pedastal Design

Loads

$P_{req} := 48.069kip$ From Tower Analysis

$M_{req} := 27.1kip \cdot ft$

$V_{req} := 2.4kip$

Material Properties

$f'_c := 1500psi$

$\gamma := 165pcf$ (B2P Section 3 Page 4)

$f_y := 40ksi$

$E := 29000ksi$

$\varepsilon_{cu} := 0.003$

Parameters

$b := 2ft$

$w := 2ft$

$h := 2ft$

$A_{st} := 6 \cdot 0.44in^2$

$d_{stirrup} := 0.375in$

$d_{cover} := 3in$

$d_{bar} := 0.750in = 0.75\,in$ Number 6 bars at 4.5 inches oc

$$\frac{(2ft - 2 \cdot 6 \cdot d_{stirrup} - 2 \cdot d_{cover})}{3} = 4.5\,in$$

minimum spacing

$A_{tr} := 2 \cdot .11in^2 = 0.22\,in^2$ Number 3 Stirrups at 12 inches oc

$\max(1in, 1.5 \cdot d_{bar}, 1.33 \cdot 1.5in) = 1.995\,in$

good spacing

Derived Parameters

$h := \min(b, w) = 24\,in$

$c_a := d_{cover} + 6 \cdot d_{stirrup} = 5.25\,in$

$d := h - d_{cover} = 21\,in$

$d' := d_{cover}$

$A_s := 3 \cdot 0.44in^2$

$A_{s'} := 3 \cdot 0.44in^2$

Design

First point (Total Compression)

$P_o := 0.85 \cdot f'_c \cdot (b \cdot w - A_{st}) + f_y \cdot A_{st} = 836.634\ \text{kip}$ (ACI 22.4.2.2)

$P_{nmax} := 0.8 P_o = 669.307\ \text{kip}$ (ACI Table 22.4.2.1)

$$\Omega_c := \frac{1.2 \cdot \frac{1}{4} + 1.6 \cdot \frac{3}{4}}{0.65} = 2.308$$

$$P_\Omega := \frac{P_{nmax}}{\Omega_c} = 290.033\ \text{kip}$$

Second point (Total Moment)

$$a := \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = 1.725\ \text{in}$$

$$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 88.604\ \text{ft} \cdot \text{kip}$$

Conservatively ignore compression reinforcement

$$c := \frac{a}{\beta_1} = 2.03\ \text{in}$$

$$\varepsilon_t := \frac{\varepsilon_{cu}}{c} \cdot (d - c) = 0.028$$

$$\varepsilon_{ty} := \frac{f_y}{E} = 1.379 \times 10^{-3}$$

$$\Omega_M := \frac{1.2 \cdot \frac{1}{4} + 1.6 \cdot \frac{3}{4}}{0.9} = 1.667$$

Tension Controlled

$$M_\Omega := \frac{M_n}{\Omega_M} = 53.162\ \text{ft} \cdot \text{kip}$$

Only consider two points conservatively

$$\text{slope} := \frac{P_\Omega}{M_\Omega} = 0.455\ \frac{1}{\text{in}}$$

$P_\Omega - \text{slope} \cdot M_{req} = 142.186\ \text{kip}$ $P_{req} = 48.069\ \text{kip}$ compression capacity Good

Development length

$A_{circ} := d_{bar} \cdot 2 \cdot \pi = 4.712 \text{ in}$

$\psi_e := 1$ ACI Table 25.4.2.4 Assume no epoxy

$\psi_s := 1$

$\psi_t := 1$

$K_{tr} := \frac{40 \cdot A_{tr}}{6in} = 1.467 \text{ in}$ 4 in oc spacing, 4 stirrups

$c_b := d_{cover}$

$$\left(\frac{c_b + K_{tr}}{d_{bar}}\right) = 5.956$$

$$l_d := \left(\frac{3}{40} \cdot \frac{f_y}{\sqrt{f'_c \cdot \frac{1}{psi}} \cdot psi} \cdot \frac{\psi_t \cdot \psi_e \cdot \psi_s}{2.5}\right) \cdot d_{bar} = 23.238 \text{ in}$$

With hook

$\psi_c := 0.7$

$\psi_r := 1$ ACI Table 25.4.3.2

$$l_d := \left(\frac{f_y \cdot \psi_e \cdot \psi_c \cdot \psi_r}{50 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi}\right) \cdot d_{bar} = 10.844 \text{ in}$$

$l_{ext} := 12 \cdot d_{bar} = 9 \text{ in}$

Embed with hook 11 inches into footing, 9 inches of standard 90 degree hook

Shear

$$V_c := 2 \cdot \sqrt{\frac{f'_c}{psi}} \, psi \cdot b \cdot d = 39.04 \, kip$$

$$\Omega_V := \frac{1.2 \cdot 0.25 + 1.6 \cdot 0.75}{0.75} = 2$$

$$\frac{V_c}{\Omega_V} = 19.52 \, kip$$

Member is good with no steel reinforcement, provide minimal stirrups at 6" oc

Summary

A 2x2x2' minimum pedastal will be provided for the steel tower columns with #6 bars at 4.5" OC on the sides parallel to the length of the bridge, with 11" of embedment into the footing terminating in a 90 degree hook with 9 inches of extension.

Appendix E

Anchor Design

Sheet Made By: Anthony Jaksa
Checked By: Erin Lau

Anchor Design

Parameters

$B := 6ft$

$L := 12ft$

$D := 10ft$

Material Properties

$\gamma_{conc} := 165pcf$

$f'_c := 1500psi$

$\phi_{conc} := 37.5deg$

Loads

$F_v := 46.43kip$

$F_{total} := 133.69kip$

$F_h := \sqrt{F_{total}^2 - F_v^2} = 125.369\,kip$

$\theta_{cable} := 20.3deg$

Soil Properties

Assume stiff clay as worst-case estimate

$c_p := 10kPa = 1.45 \times 10^{-3}\,ksi$

$\Phi := 17deg = 0.297$ Typical Stiff Clay

$\gamma := 115pcf = 115\,pcf$

$\delta := 12deg$ (Das Pg 655)

$c_{soilconc} := 0.3 \cdot c_p = 4.351 \times 10^{-4}\,ksi$

A Safety Factor of 1.5 is desired against pull-out and sliding, and a safety factor of 2 is desired in shear

Design

Vertical Forces

Force due to weight

$F_{vcap} := B \cdot L \cdot D \cdot \gamma_{conc} = 118.8\ \mathrm{kip}$ is below $F_v = 46.43\ \mathrm{kip}$

Resultant vertical force

$F_{vr} := F_v - F_{vcap} = -72.37\ \mathrm{kip}$

$FS_{actual} := \dfrac{F_{vcap}}{F_v} = 2.559$ is above 1.5

Horizontal Forces

Rankine Passive Earth Pressure

$K_p := \tan\left(45\mathrm{deg} + \dfrac{\Phi}{2}\right)^2 = 1.826$ (Das 12.57)

$P_p := \dfrac{1}{2} \cdot K_p \cdot \gamma \cdot D^2 + 2 \cdot c_p \cdot \sqrt{K_p} \cdot D = 1.615 \times 10^4\ \mathrm{plf}$ (Das 12.59)

$F_p := P_p \cdot L = 193.758\ \mathrm{kip}$

Sliding friction

$F_{f1} := -F_{vr} \cdot \tan(\delta) = 15.383\ \mathrm{kip}$ (Das 13.8)

$F_{f2} := B \cdot L \cdot c_{soilconc} = 4.511\ \mathrm{kip}$

Active Earth Pressure

$K_a := \tan\left(45\mathrm{deg} - \dfrac{\Phi}{2}\right) = 0.74$

$z_c := \dfrac{2 \cdot c_p}{\gamma \cdot \sqrt{K_a}} = 50.67\ \mathrm{in}$

$P_a := \dfrac{1}{2} \cdot \left(D - z_c\right) \cdot \left(\gamma \cdot D \cdot K_a - 2 \cdot c_p \cdot \sqrt{K_a}\right) = 1.42 \times 10^3\ \mathrm{plf}$

$F_a := P_a \cdot L = 17.043\ \mathrm{kip}$

Sliding Capacity and Factor of Safety

$$F_{hcap} := F_p + F_{f1} + F_{f2} = 213.652\ \text{kip}$$

$$FS_{actual} := \frac{F_{hcap}}{F_h + F_a} = 1.5$$ is above 1.5

A block of concrete 12 feet in length by 10 feet in depth, with a sidelength perpendicular to loading of 6 feet is good

Connection and Structural Design

Cable - Beam - Concrete Connection

$cableinset := 6in \qquad beaminset := 18in$

$d_{embed} := (B - cableinset - beaminset)\cdot\tan(\theta_{cable}) = 17.756\ in$

$\theta_{up} := \phi_{conc} + \theta_{cable} = 57.8\cdot deg$

$\theta_{down} := \theta_{cable} - \phi_{conc} = -17.2\cdot deg$

$$L_{upshearplane} := \frac{d_{embed}}{\sin(\theta_{up})} = 20.983\ in$$

$A_{upshearplane} := L_{upshearplane}\cdot L = 3.022 \times 10^{3}\ in^{2}$

Use Geometry to calculate shear planes as the area above and below an internal angle of friction in the concrete from the pull direction

$depth_{down} := (B - beaminset)\cdot\tan(\theta_{down}) - d_{embed} = -34.472\ in$

$$L_{downshearplane} := \frac{(B - beaminset)}{\cos(\theta_{down})} = 56.528\ in$$

$A_{downshearplane} := L_{downshearplane}\cdot L = 8.14 \times 10^{3}\ in^{2}$

$A_{shear} := A_{upshearplane} + A_{downshearplane} = 1.116 \times 10^{4}\ in^{2}$

$$\tau_{avgbeam} := \frac{F_{total}}{A_{shear}} = 0.012\ ksi$$

Shear strength for concrete

$$\tau_{cap} := \sqrt{\frac{f'_c}{psi}}\ psi = 0.039\ ksi$$

Actual Safety Factor

$$SF_{connectionshear} := \frac{\tau_{cap}}{\tau_{avgbeam}} = 3.234$$

desired SF of 2

The connection between the beam and the concrete is good

Shear Across the Concrete Section Along its Depth

$$A_{totalshear} := B \cdot L = 1.037 \times 10^{4}\,in^{2}$$

$$\tau_{avgtotal} := \frac{F_{total}}{A_{totalshear}} = 0.013\,ksi$$

$$SF_{totalshear} := \frac{\tau_{cap}}{\tau_{avgtotal}} = 3.004$$ desired SF of 2

The embedment at 20.3 degrees to the back of the anchor block while maintaining clear cover produces a satisfactory shear strength, and the 6' wide anchor block is strong enough itself as well

Sheet Made By: Anthony Jaksa
Checked By: Erin Lau

Internal Anchor Steel Beam

Loads

$F := 66\text{kip}$

$\Omega_M := 1.667$

$\Omega_V := 2$

Material Properties

$f'_c := 1500\text{psi}$

Per B2P Section 3 Page 6

$F_y := 35\text{ksi}$

Conditions

$L := 11\text{ft}$

$\text{inset} := 1.5\text{ft}$

Loading

Approximate analysis acting as if the concrete reaction is a distributed load on the beam

$q := \frac{2 \cdot F}{L} = 12 \cdot \text{klf}$

Approximate reaction from the concrete

$V_{peak1} := q \cdot \text{inset} = 18\,\text{kip}$

$V_{peak2} := -q \cdot \text{inset} + F = 48\,\text{kip}$

$M_{peak1} := -V_{peak1} \cdot \text{inset} \cdot \frac{1}{2} = -13.5\,\text{ft} \cdot \text{kip}$

$M_{peak2} := M_{peak1} + V_{peak2} \cdot \frac{(L - 2 \cdot \text{inset})}{2} \cdot \frac{1}{2} = 82.5\,\text{ft} \cdot \text{kip}$

Beam Design

For Moment

Assume full lateral restraint

Design Parameters

Shape: W12x106

$S_y := 49.3 in^3$

$b_f := 12.2 in$

$t_f := .990 in$

$d := 12.9 in$

Design

$M_n := F_y \cdot S_y = 143.792\ ft \cdot kip$ (No LTB)

$M_{n\Omega} := \frac{M_n}{\Omega_M} = 86.258\ ft \cdot kip$ $M_{peak2} = 82.5\ ft \cdot kip$ Safe

For Shear

Assume flange shear buckling cannot happen because of concrete confinement

$V_n := 0.6 \cdot F_y \cdot b_f \cdot t_f = 253.638\ kip$

$V_{n\Omega} := \frac{V_n}{\Omega_V} = 126.819\ kip$ $V_{peak2} = 48\ kip$ Safe

Crushing of concrete

Average stress over surface of concrete

$\sigma := \frac{2 \cdot F}{d \cdot L} = 0.078\ ksi$

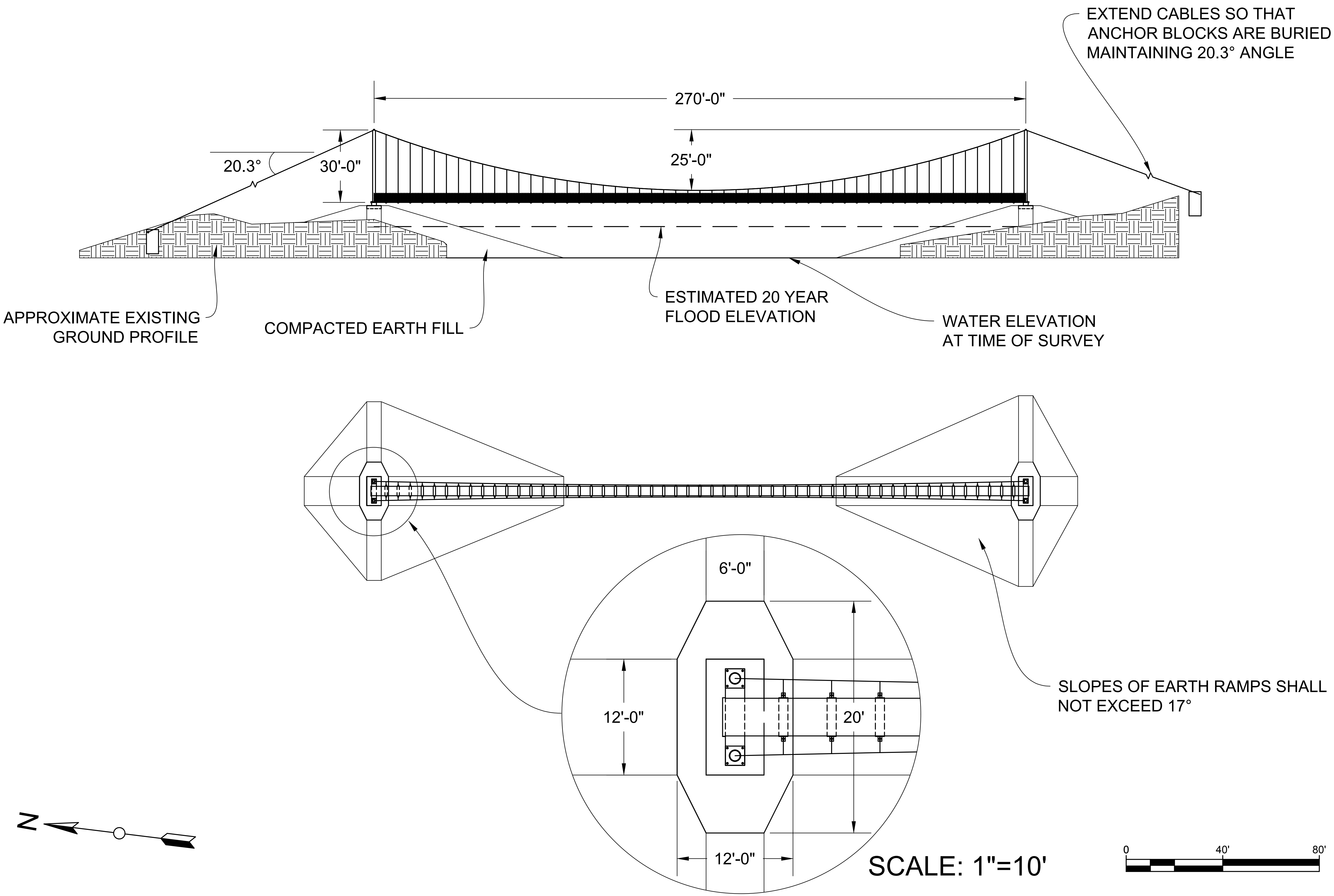
$\frac{f'_c}{3} = 0.5\ ksi$ Use SF of 3 against concrete crushing

Safe

A W12x106 is safe as a anchor beam within the concrete anchor block

Appendix F

Design Drawings



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FALL 2018 INTERNATIONAL SENIOR DESIGN - MICHIGAN TECHNOLOGICAL UNIVERSITY

LANO MIRANDA
TO
BAJO MOSQUITO
SUSPENSION BRIDGE

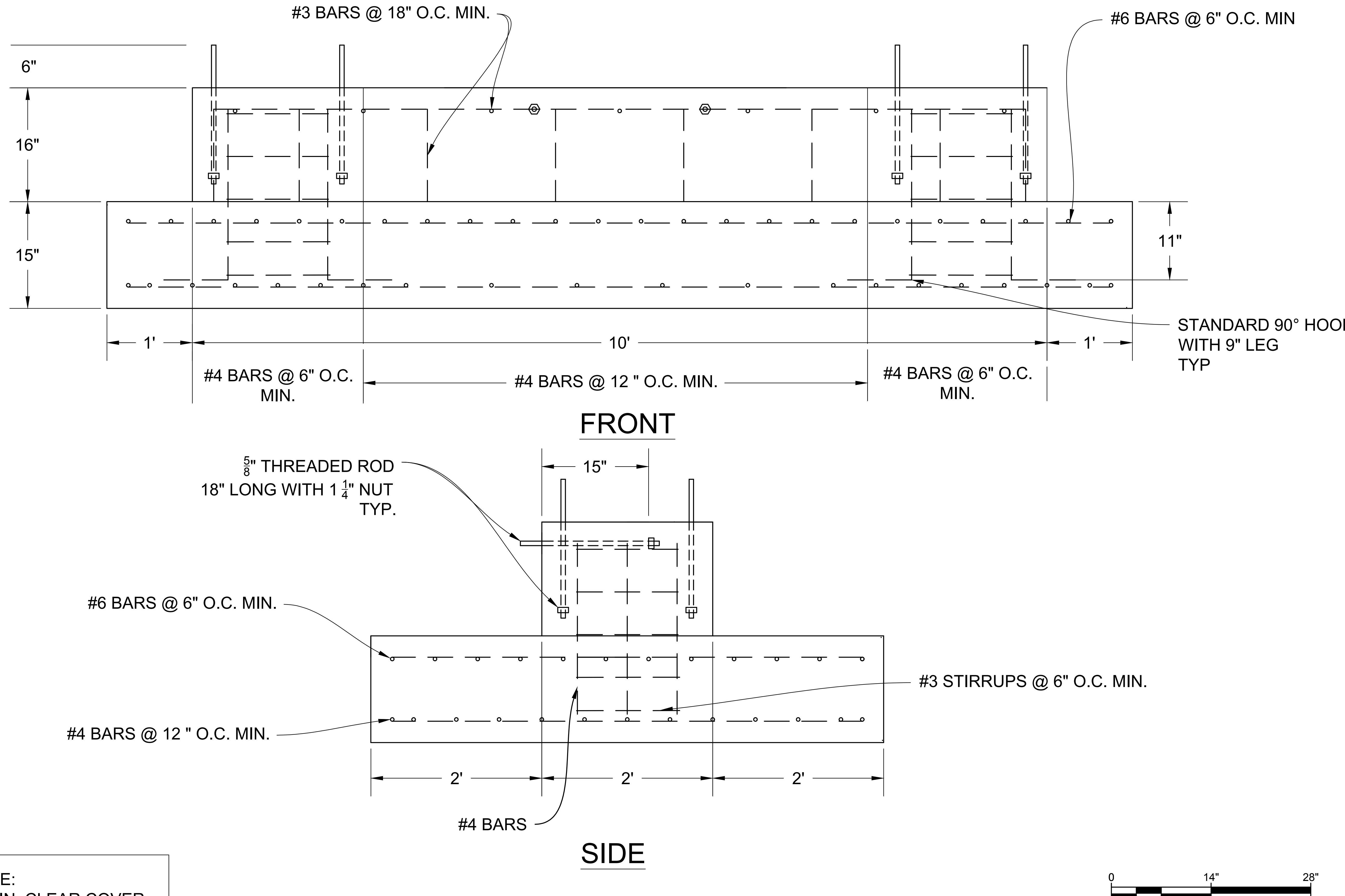
DATE:	NOVEMBER 5, 2018
DESIGNED BY:	TJJ
DRAWN BY:	NMP
CHECKED BY:	EAL
SCALE:	AS NOTED

SHEET CONTENTS:

OVERALL
PLAN

SHEET NO:

S-101



NOTE:
3" MIN. CLEAR COVER

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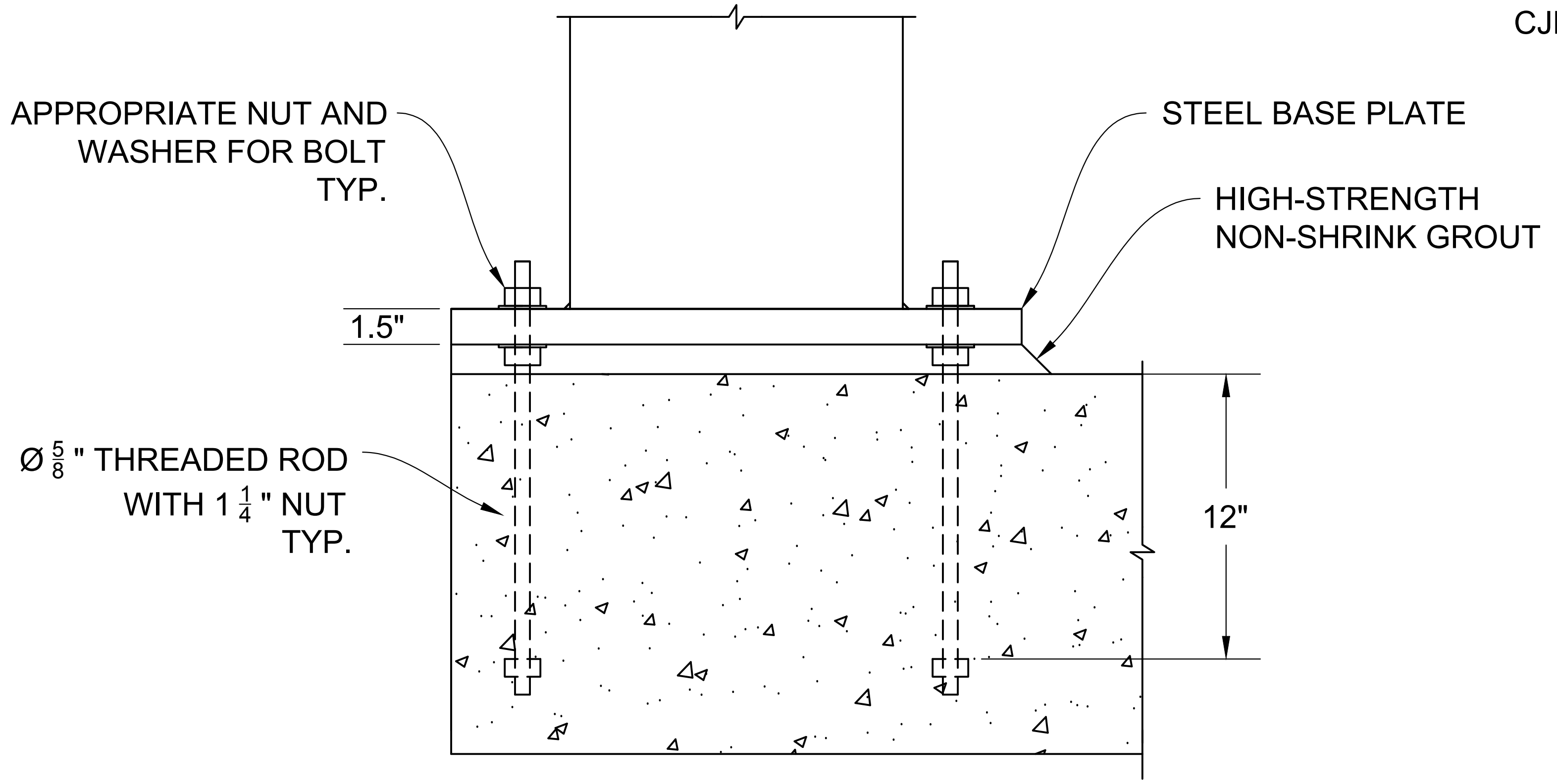
SCALE:	AS NOTED

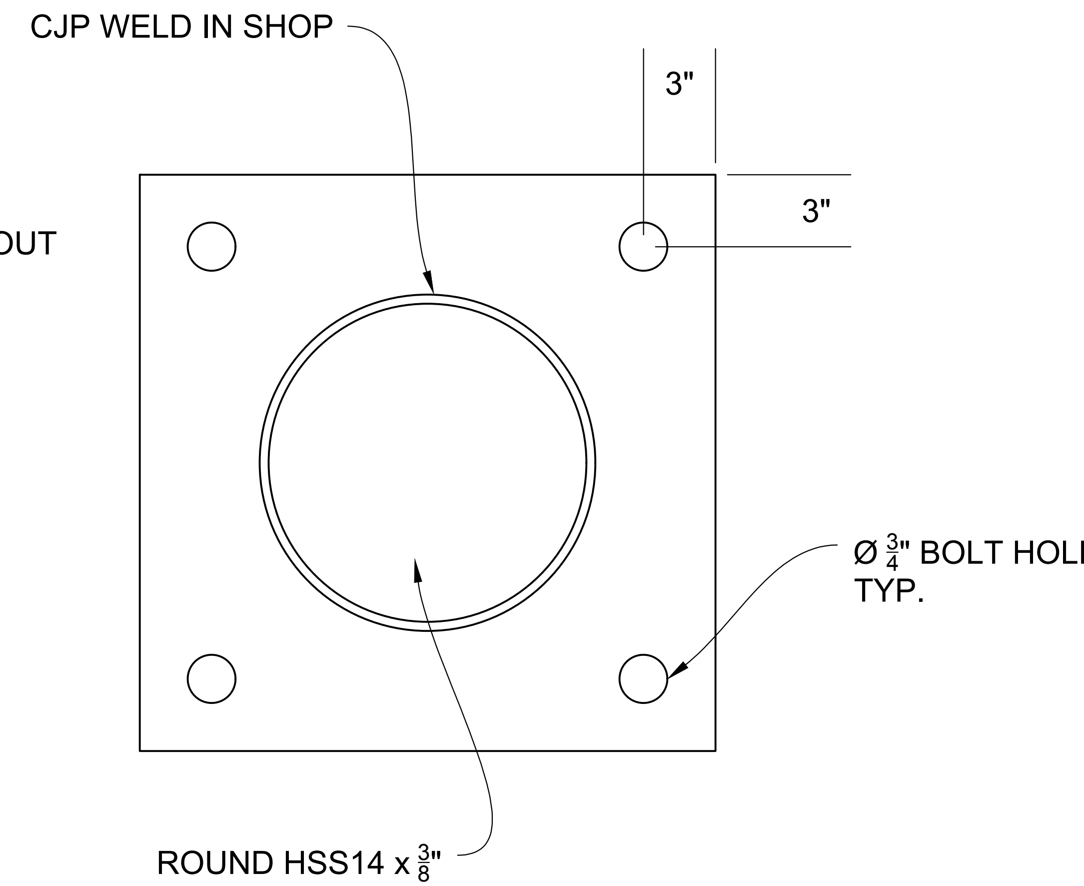
SHEET CONTENTS:

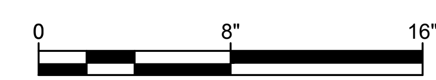
TOWER
FOUNDATION
DETAILS

SHEET NO:

D-101







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SCALE:	AS NOTED

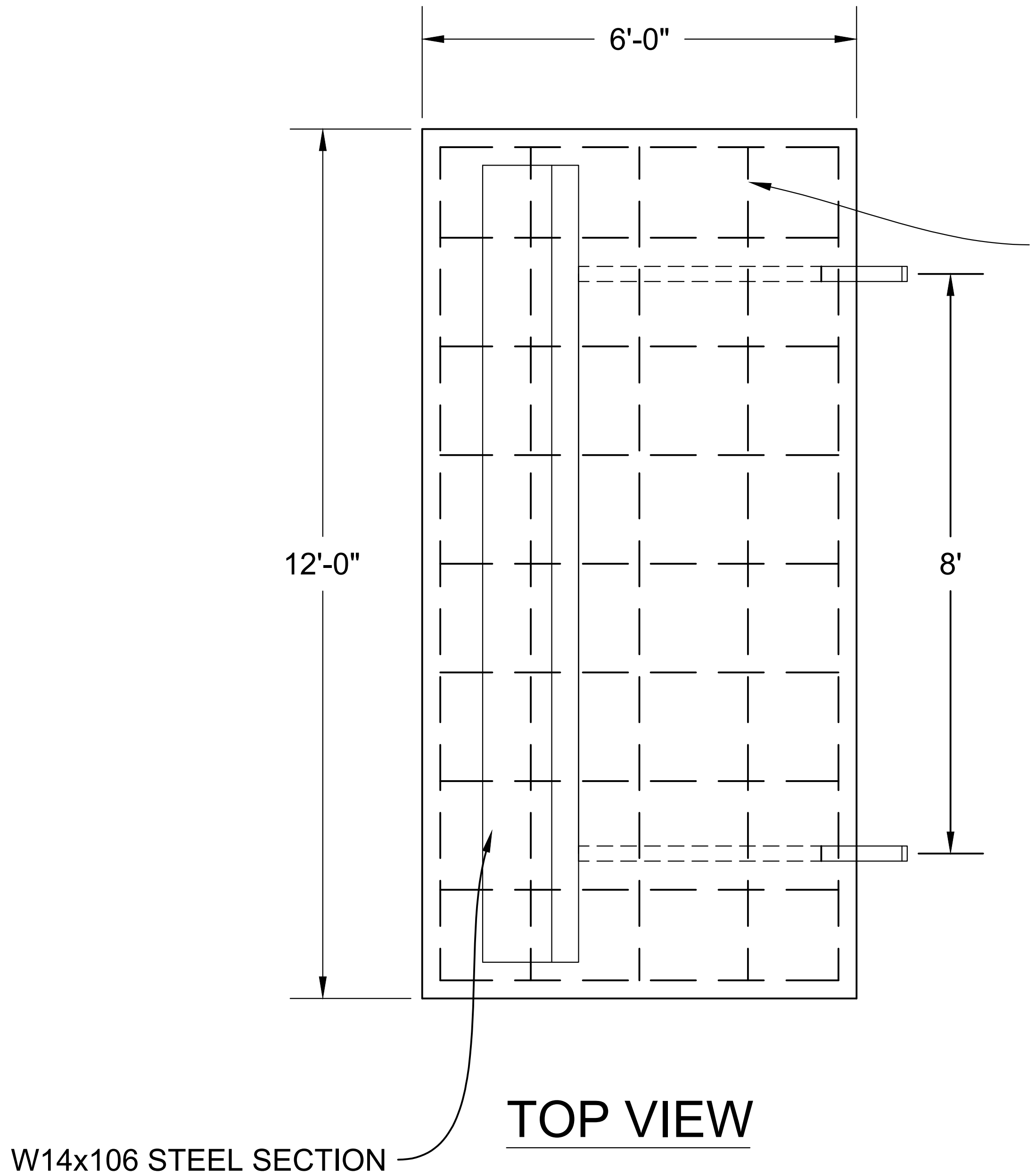
SHEET CONTENTS:

BASE PLATE
CONNECTION
DETAIL

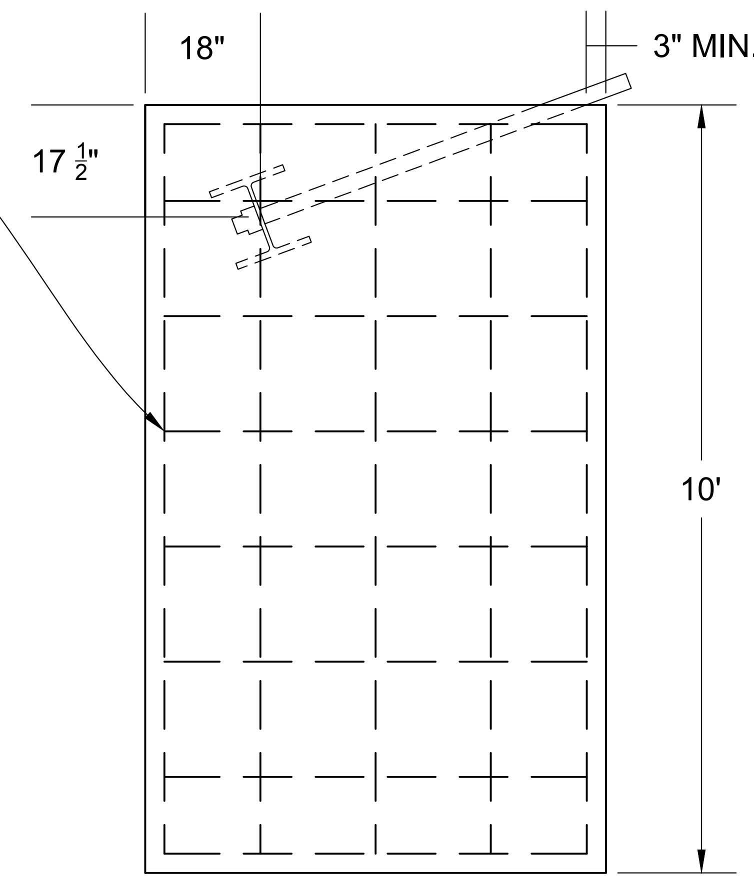
SHEET NO:

D-102

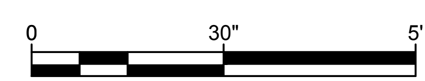
NOTE: 3" MIN. CLEAR COVER TYP.



TOP VIEW



SIDE VIEW



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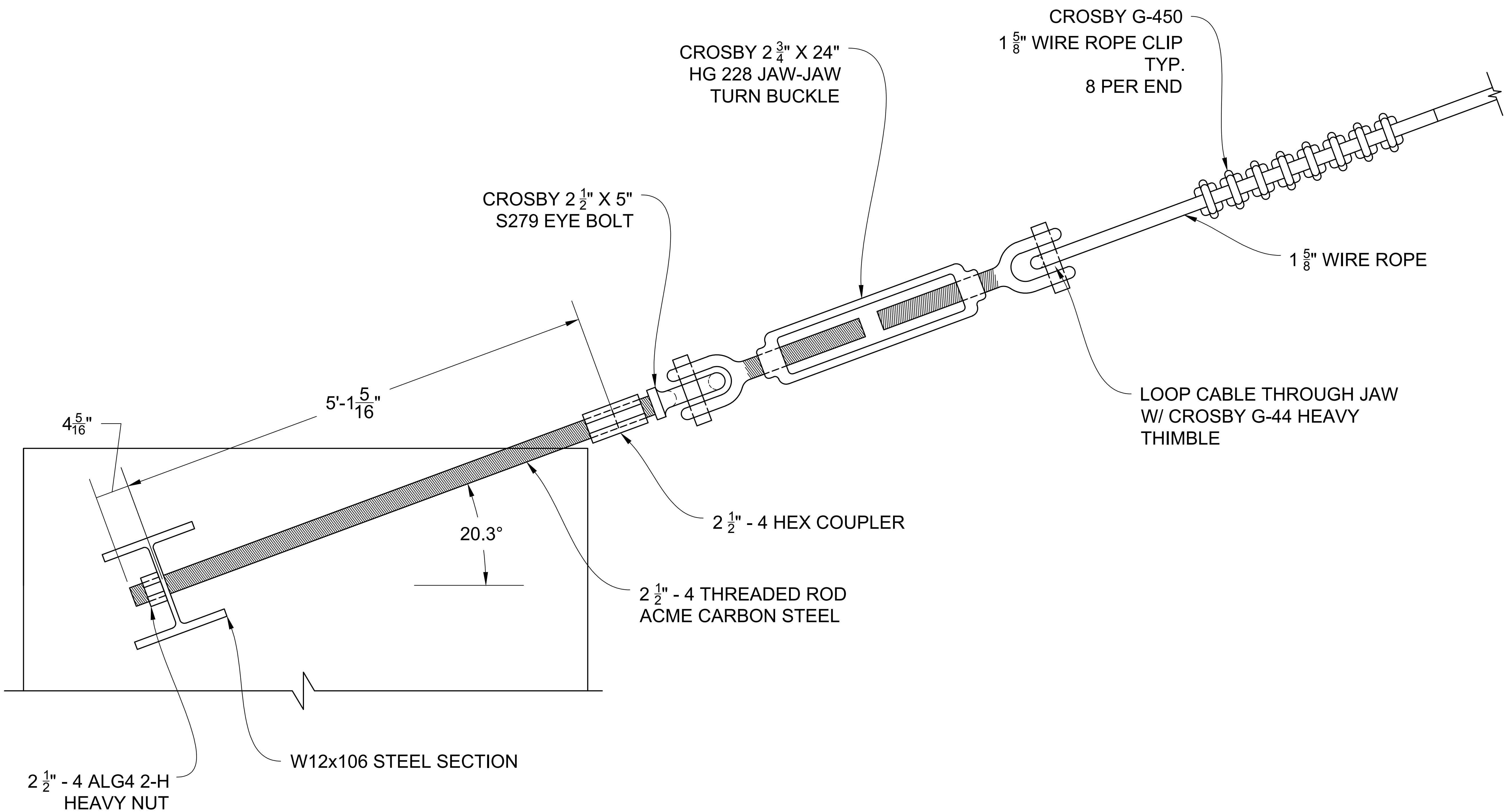
SCALE:	AS NOTED

SHEET CONTENTS:

ANCHOR BLOCK
DETAIL

SHEET NO:

D-103



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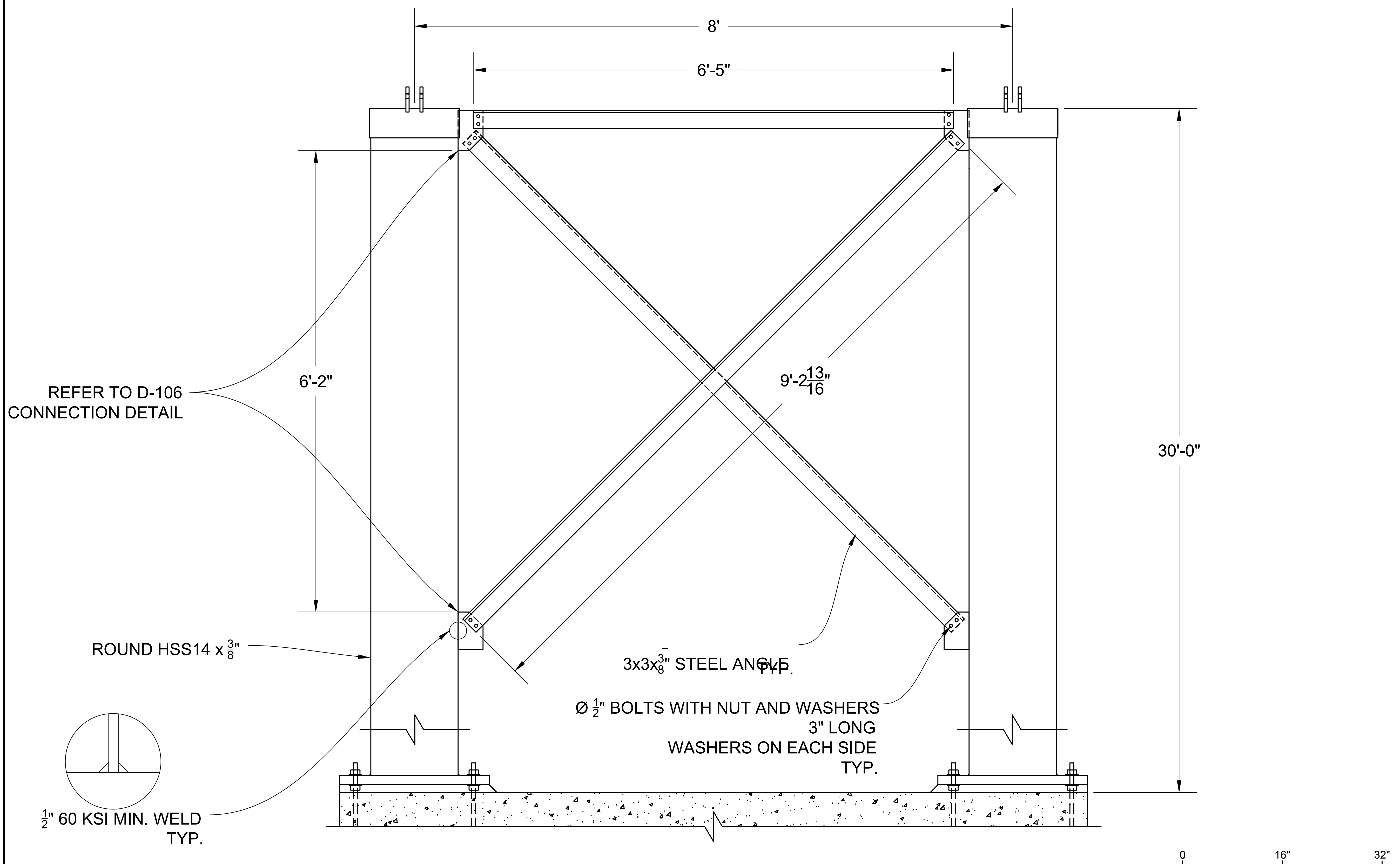
SHEET CONTENTS:

ANCHOR
ASSEMBLY
DETAIL

SHEET NO:

D-104



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SCALE:	AS NOTED

SHEET CONTENTS:

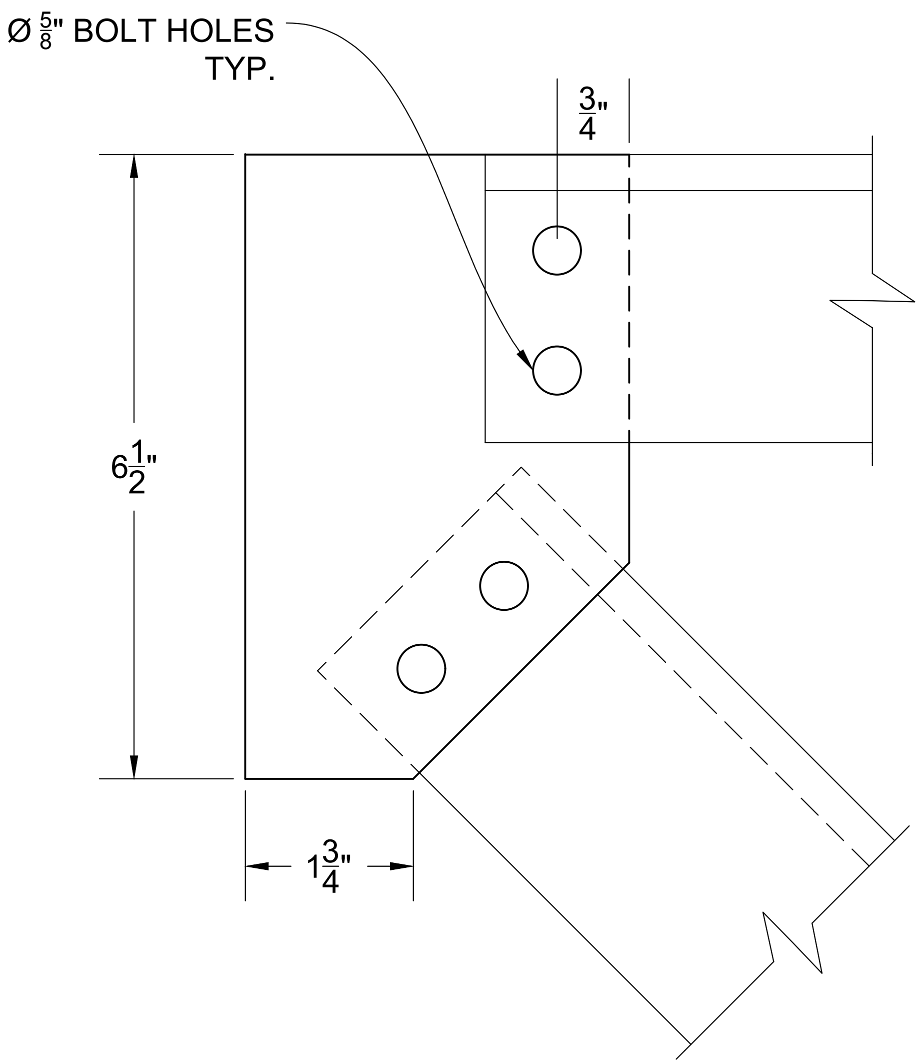
CROSS SUPPORT DETAIL

SHEET NO:

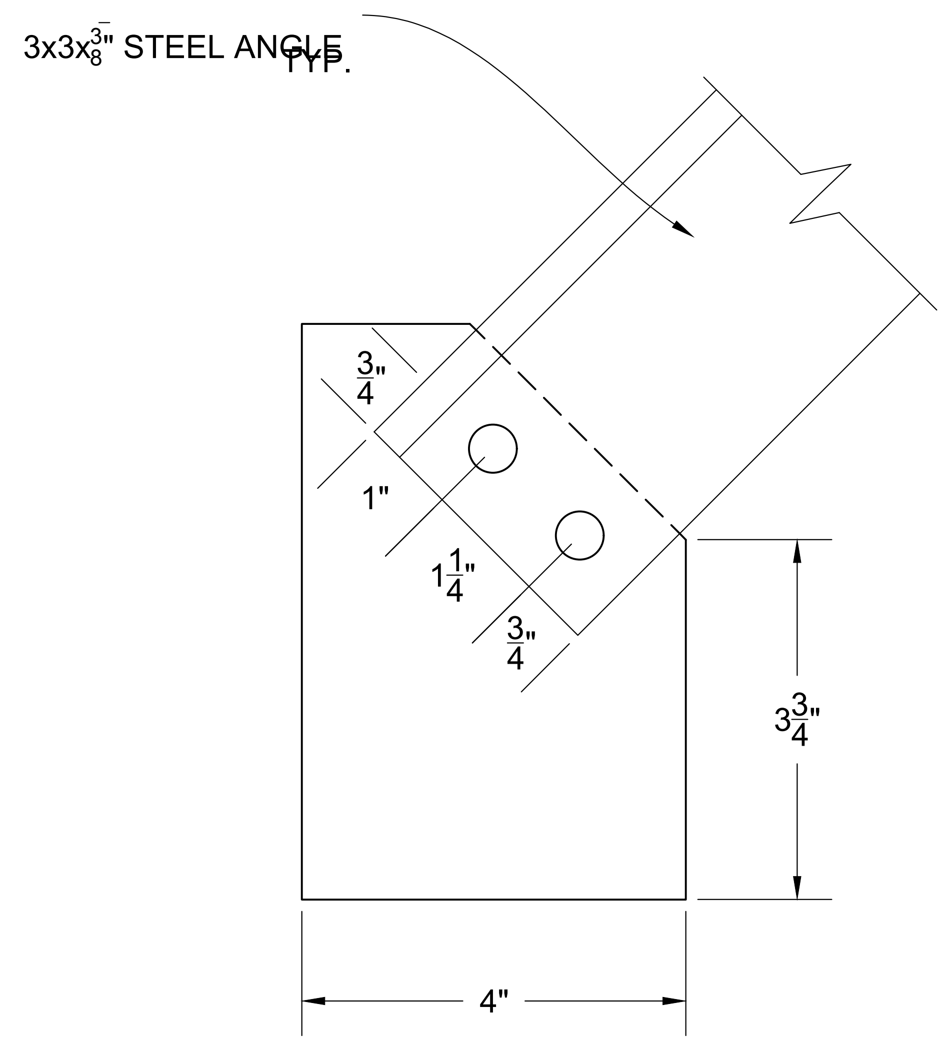
D-105

NOTES:
PLATES ARE 3/8" THICK

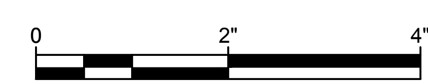
BRACKETS FOR RIGHT SIDE ARE MIRROR IMAGES



TOP BRACKET



BOTTOM BRACKET



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DATE:	NOVEMBER 5, 2018
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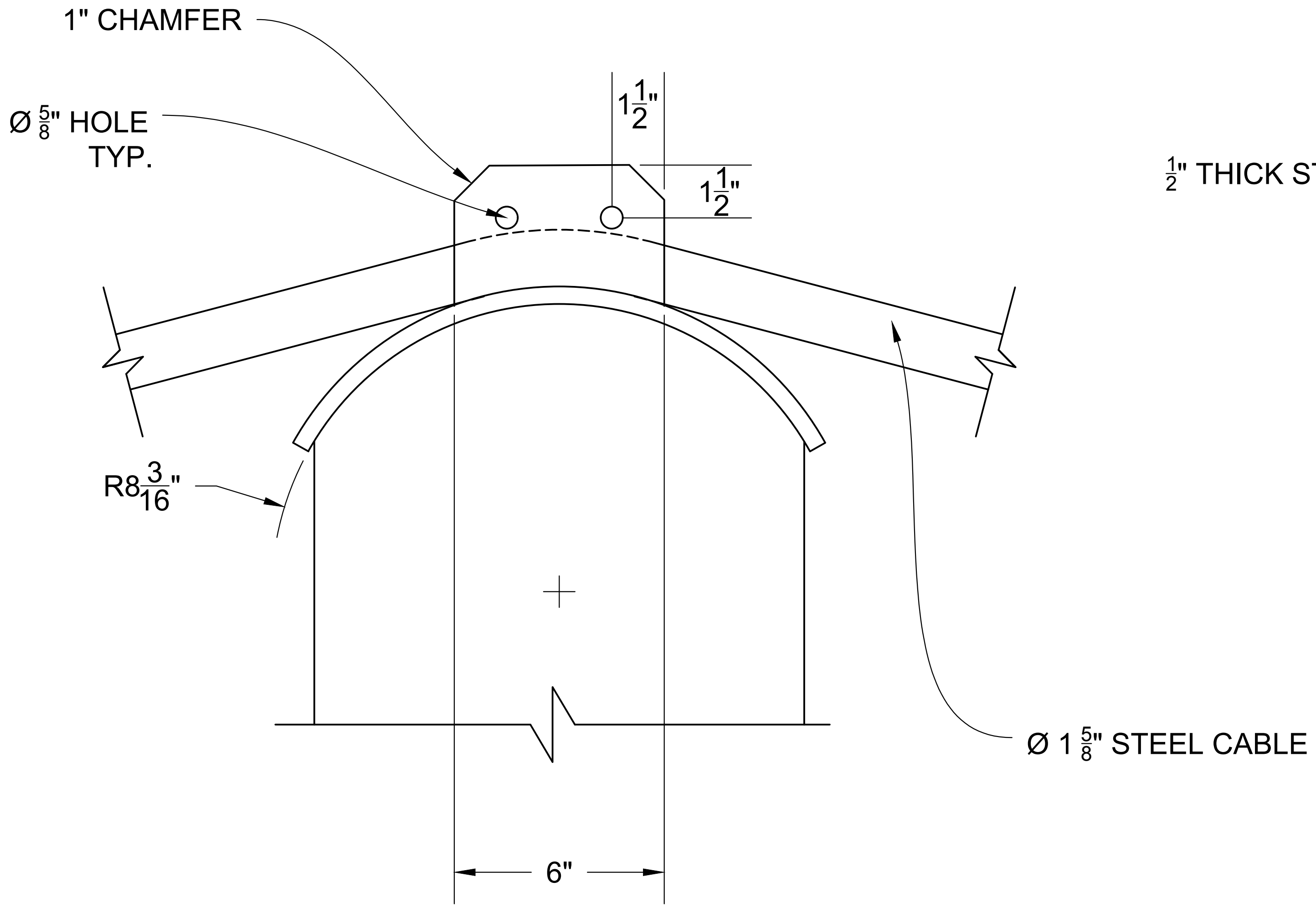
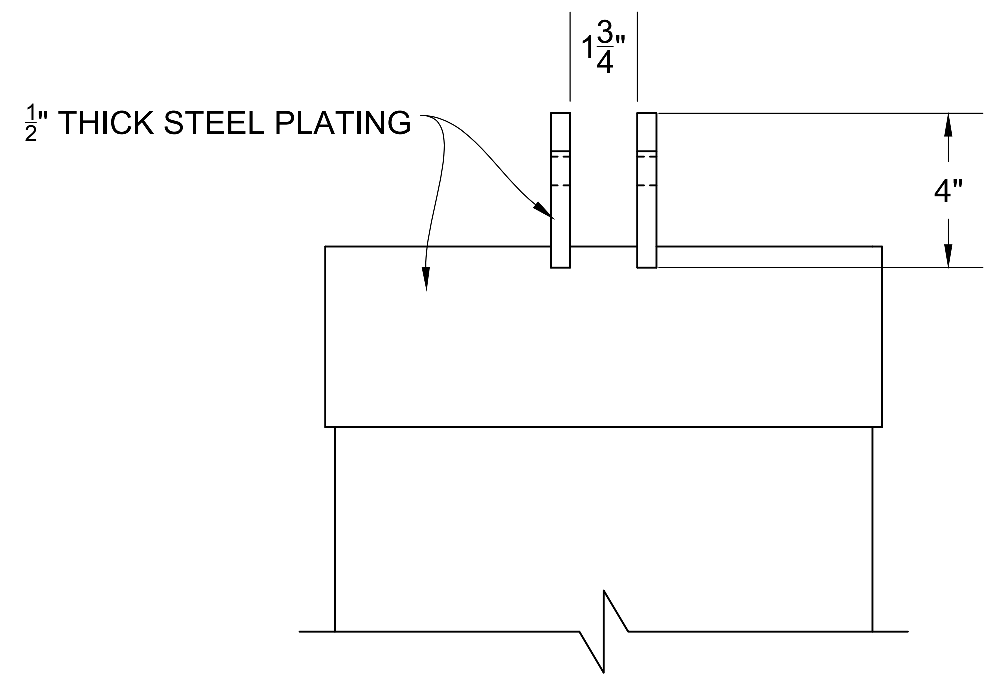
SCALE:	AS NOTED

SHEET CONTENTS:
CROSS SUPPORT CONNECTION BRACKET DETAIL

SHEET NO:
D-106





0 5" 10"

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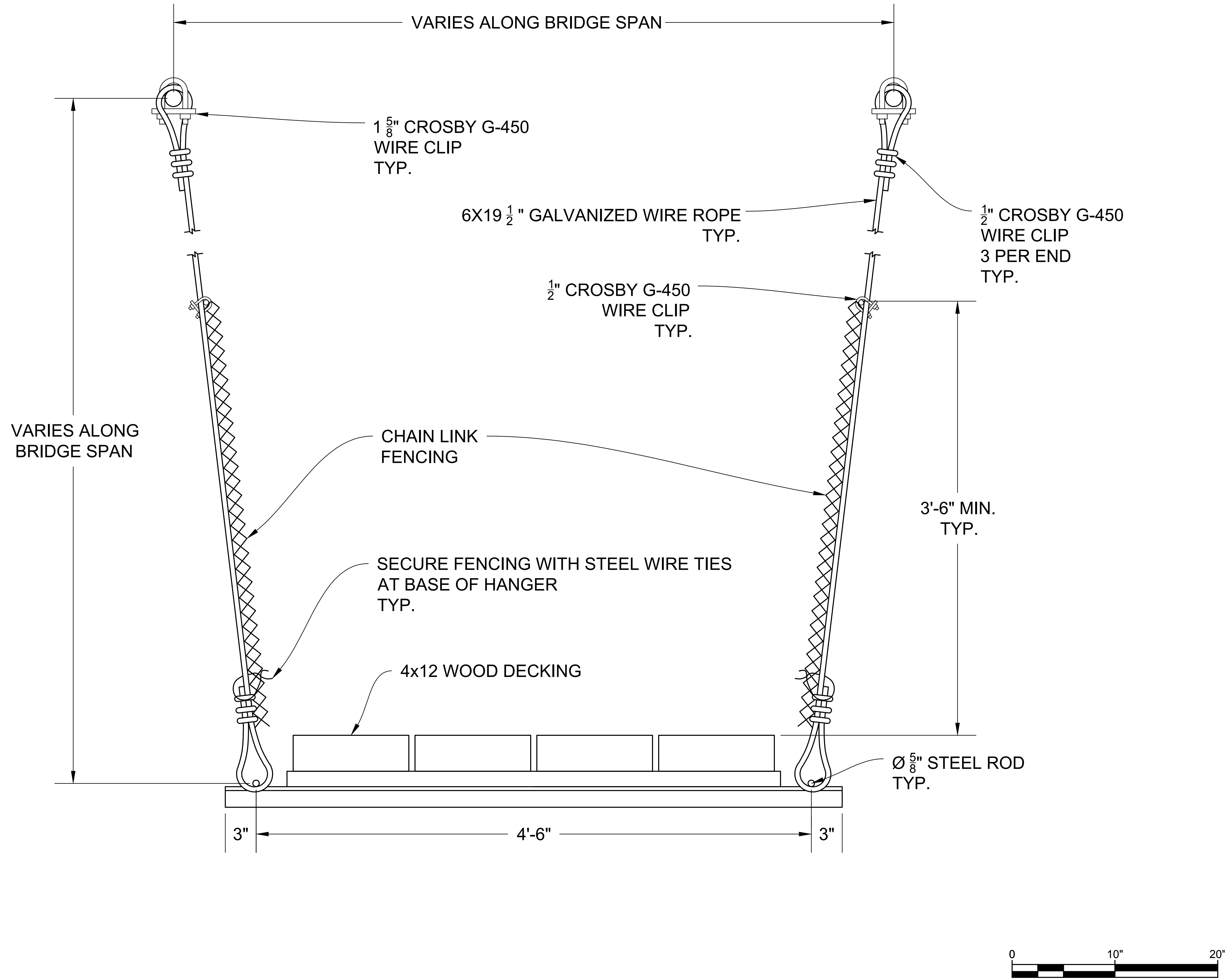
DESIGNED BY:	TJJ
DRAWN BY:	NMP
CHECKED BY:	EAL
SCALE:	AS NOTED

SHEET CONTENTS:

CABLE SADDLE
DETAIL

SHEET NO:

D-107



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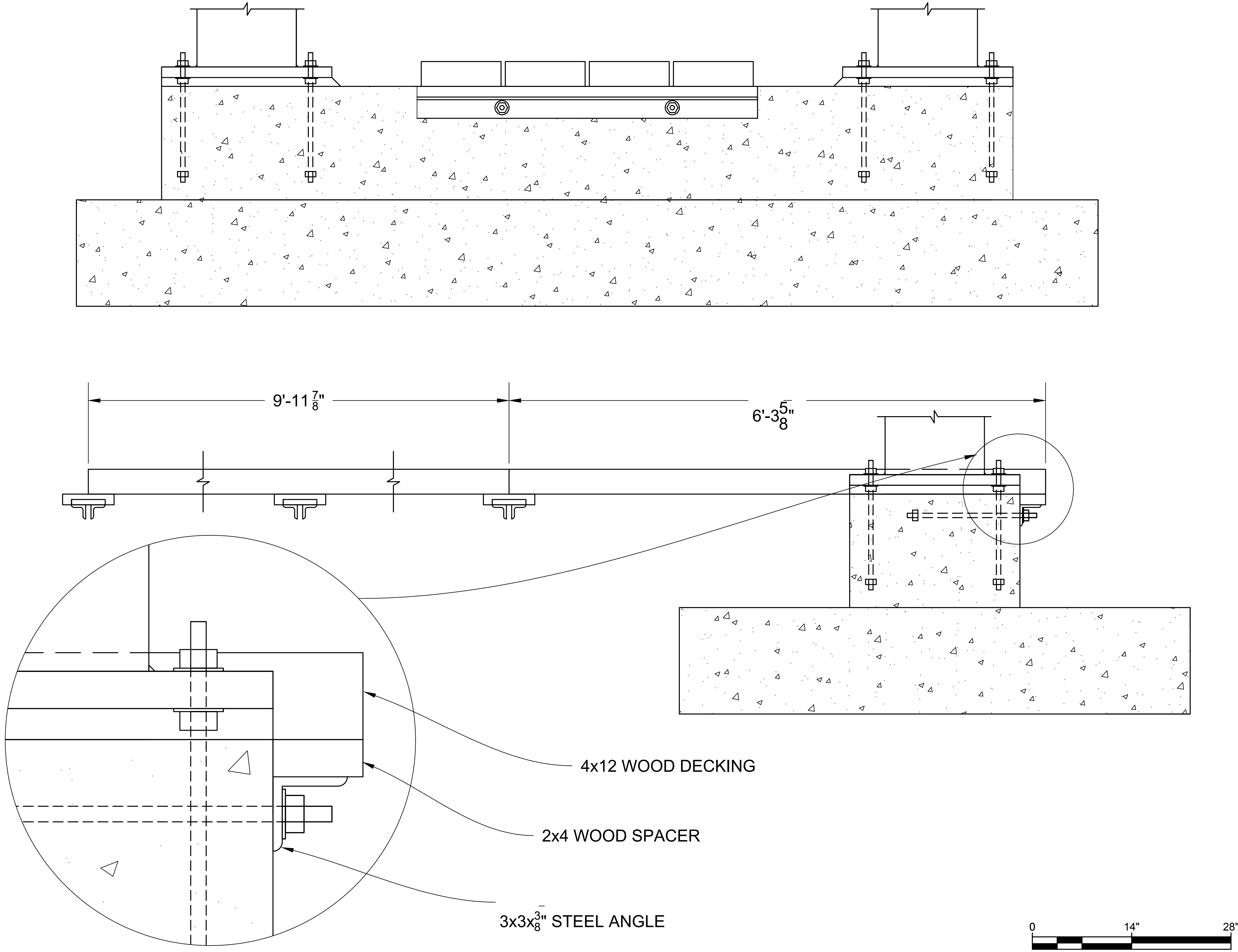
DATE:	NOVEMBER 5, 2018
DESIGNED BY:	TJJ
DRAWN BY:	NMP
CHECKED BY:	EAL
SCALE:	AS NOTED

SHEET CONTENTS:

DETAIL OF
HANGERS AND
CONNECTIONS

SHEET NO:

D-108



SCALE: 1"=4"

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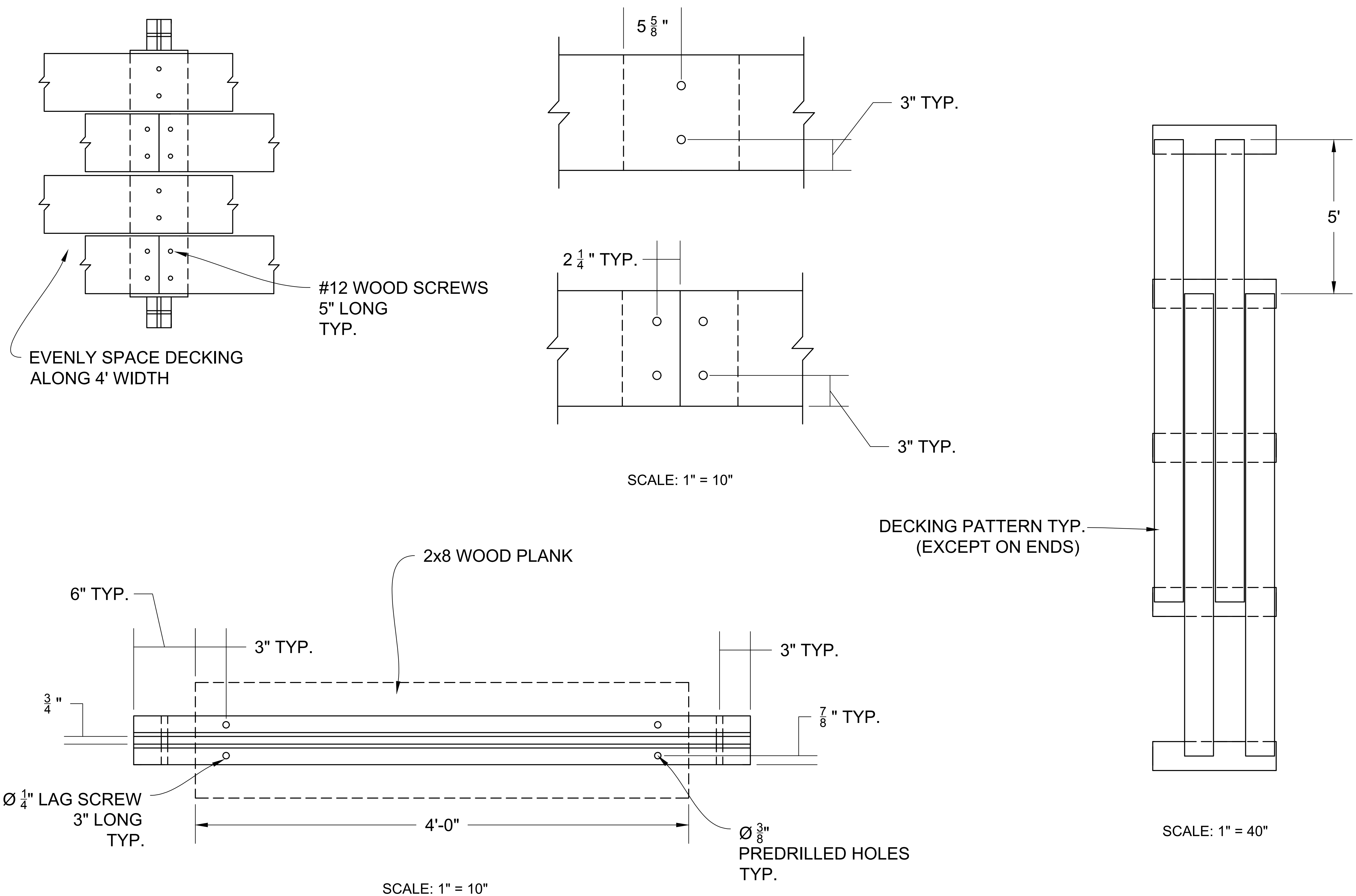
DESIGNED BY:	TJJ
DRAWN BY:	NMP
CHECKED BY:	EAL
SCALE:	AS NOTED

SHEET CONTENTS:

END DECKING DETAIL

SHEET NO:

D-109



BOTTOM OF CROSS MEMBER

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TO
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SUSPENSION BRIDGE

DATE:	NOVEMBER 5, 2018
DESIGNED BY:	TJJ
DRAWN BY:	NMP
CHECKED BY:	EAL
SCALE:	AS NOTED

SHEET CONTENTS:

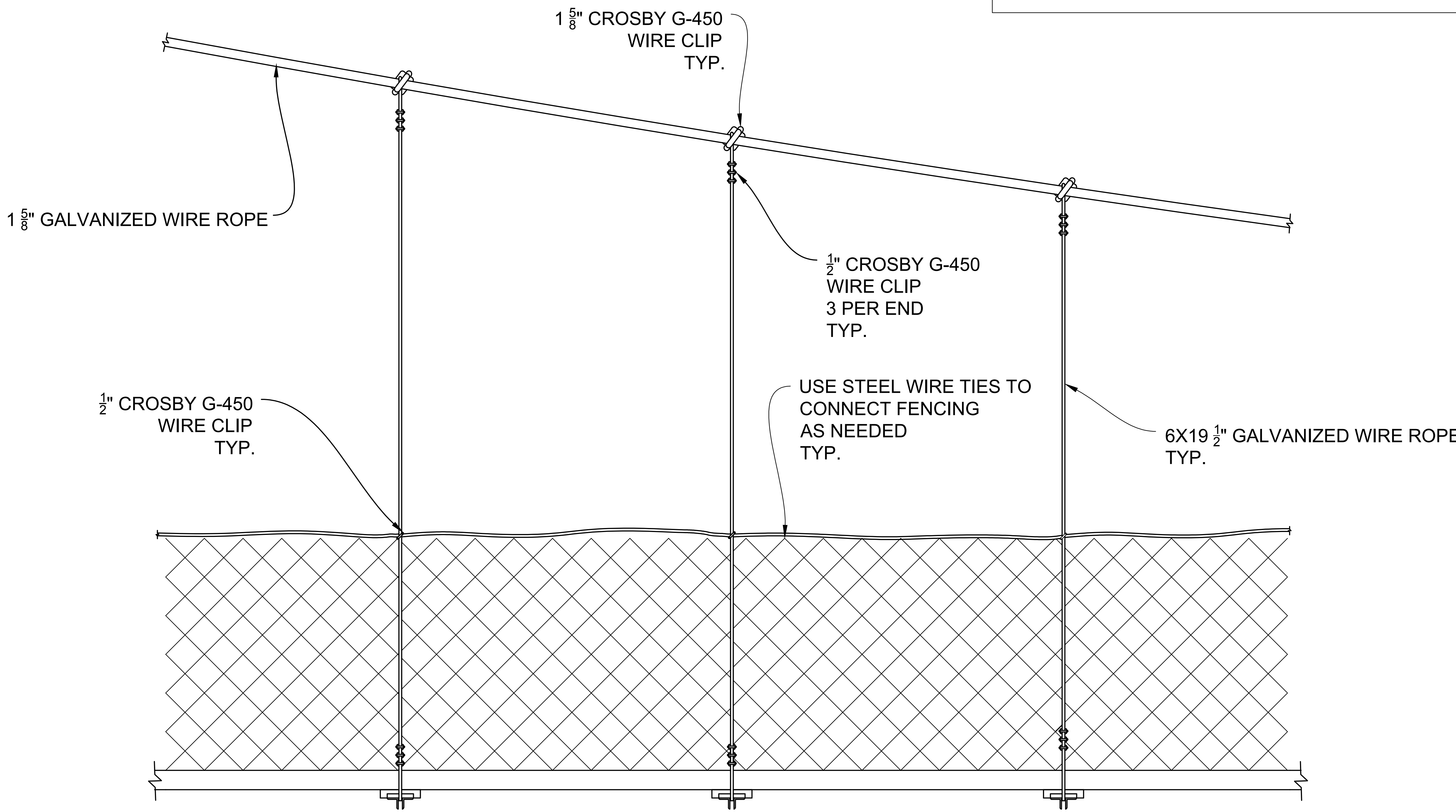
DECKING
DETAILS

SHEET NO:

D-110

NOTES:
HAND CABLE TO BE WRAPPED AROUND TOWER AND CLAMPED ONTO SELF

CHAIN LINK FENCE TO BE ATTACHED THROUGH WIRE CLIP OR WITH STEEL WIRE TIES AS NEEDED



0 20" 40"

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SUSPENSION BRIDGE

DATE:	NOVEMBER 5, 2018
DESIGNED BY:	TJJ
DRAWN BY:	NMP
CHECKED BY:	EAL
SCALE:	AS NOTED

SHEET CONTENTS:
HAND CABLE DETAIL

SHEET NO:
D-111

Appendix G

Watershed Calculations

Sheet made by : Ryan Olsen
Checked by: TJ Jaksa

Watershed

$$A_D := \frac{(270ft + 100ft) \cdot 30ft}{2} = 5550 \cdot ft^2$$

estimated cross section of river at project site

$$A_1 := 61380acre$$

estimated drainage area of watershed where project site is located

$$A_2 := 7.980 \cdot 10^5 acre$$

estimated drainage area of adjacent watershed

$$r := \frac{A_2}{A_1} = 13.001$$

ratio between adjacent watershed drainage areas

$$Q_1 := 200000 \frac{ft^3}{s}$$

100-year expected flow rate for adjacent area

$$Q_D := \frac{Q_1}{r} = 15383.459 \cdot \frac{ft^3}{s}$$

Design flow at bridge site scaled based on watershed area

$$V_D := \frac{Q_D}{A_D} = 2.772 \cdot \frac{ft}{s}$$

estimated velocity at project site

Due to this water velocity, the team recommends 10' rip rap on the east side of the bridge towers to prevent scour.

Appendix H

Cost Estimate

Engineer's Opinion of Cost - Summary

Materials	
Cables	44038
Hangers & Decking	25091
Towers	7056
Tower Foundations	1793
Cross Members	827
Anchors	15949
Earthen Ramps	21175
Site Preparation	0
General Requirements	10469

Labor	
Cables	368
Hangers & Decking	768
Towers	256
Tower Foundations	2192
Cross Members	288
Anchors	4928
Earthen Ramps	3280
Site Preparation	2400
General Requirements	16707

Equipment	
Cables	665
Hangers & Decking	0
Towers	3570
Tower Foundations	47
Cross Members	0
Anchors	2828
Earthen Ramps	25801
Site Preparation	3756
General Requirements	926

Total:	$ 195,178

Total with O&P:	$ 224,455

General Requirements

	Description	Quantity	Unit	Unit Price	Total
Materials					
	Taxes	1	Ea	5796.20	5796
	Testing	1	Ea	1051	1051
	Scafolding	2	Ea	140	280
	Small Tools	1	Ea	3341.43	3341
Subtotal					10469
Labor					
	General Contractor	1	Ea	16707.16	16707
Subtotal					16707
Equipment					
	Mobilization	1	Ea	925.5	926
Subtotal					926

Subtotal:	$ 28,101

Materials

	Description	Quantity	Unit	Unit Price	Total
Cables					
	6x25 1 5/8" ASTM 603 Galvenized Steel Rope	1400	Ft	5.00	7000
	6x19 1/2" Galvanized Wire Rope	600	Ft	1.07	642
	1 5/8" Wire Rope Clamps	148	Ea	108.80	16102
	Crosby 2-3/4" x 24" HG-288 Jaw & Jaw Turnbuckle	4	Ea	4883.00	19532
	Crosby G-414 Heavy Duty Thimble	4	Ea	190.50	762
Subtotal					44038
Deck					
	L2x2x1/4 (110 each 5' long)	550	Ft	2.74	1507
	#5 Gr. 40 Rebar	33	Lb	1.12	37
	#12 Wood Screws 5" Long	864	Ea	0.76	657
	1/4" Lag Screws, 3" Long	226	Ea	0.10	23
	2x8 Beech-Birch-Hickory Planks (54, 4ft long)	216	Ft	14.25	3078
	4x12 Beech-Birch-Hickory Decking	72	Ea	130.00	9360
	6x19 1/2" Galvanized Wire Rope	1950	Ft	1.07	2087
	Crosby G-450 Clips 1/2" Wire Clips	764	Ea	9.92	7579
	4' Chain Link Fencing	11	Ea	69.50	765
Subtotal					25091
Towers					
	Round HSS, 14" Diameter, 30ft tall	4	Ea	824.00	3296
	P1112 A36 Steel Plate 2'x2', 1.5in thick	4	Ea	387.56	1550
	Additional Fabrication	8	Ea	82.40	659
	Custom Saddle Fabrication	4	Ea	387.56	1550
Subtotal					7056
Foundations					
	Cement	41	Ea	10.70	439
	5/8" Washer	76	Ea	0.46	35
	5/8" Nuts	38	Ea	0.35	13
	5/8" Bolt	40	Ea	0.80	32
	River Water	27	Cf	0.00	0
	Locally Sourced Aggregate	134	Cbf	0.00	0
	Sand	4	Ton	25.00	100
	#3 Gr. 40 Rebar	3	30 lf	4.95	15
	#4 Gr. 40 Rebar	23	30 lf	8.50	196
	#6 Gr. 40 Rebar	2	30 lf	14.50	29
	Threaded Rods 18" L	20	Ea	10.07	201
	Forms	90	Sf	8.15	734
Subtotal					1793

Cross Members					
	L3X3X1/2	2	Ea	269.00	538
	6.5"x4"x3/8" Plates	8	Ea	15.19	122
	1/2" Bolts 3" Long	48	Ea	1.85	89
	1/2" Nuts	48	Ea	0.63	30
	1/2" Washers	96	Ea	0.50	48
Subtotal					827
Anchors					
	Excavated Soils	53	Cby	0.00	0
	Cement	238	Ea	10.70	2547
	Rebar #3	55	30 lf	4.95	271
	W12x106	2	Ea	1500.00	3000
	2 1/2" Threaded Rod 5'-1 5/16" Long	4	Ea	74.00	296
	Crosby 2 1/2" x5" S279 Eye Bolt	4	Ea	681.00	2724
	Sand	24	Ton	25.00	600
	Locally Sourced Aggregate	787	Cbf	0.00	0
	River Water	157	Cf	0.00	0
	2 1/2" Hex Coupler	4	Ea	99.00	396
	2 1/2" Heavy Nut	4	Ea	61.85	247
	Forms	720	Sf	8.15	5868
Subtotal					15949
Earthen Ramps					
	Soil Fill	847	Cby	25.00	21175
Subtotal					21175
Site Preperation					
	Brush & Tree Clearing				0
	Path Widening/Road Creation				0
Subtotal					0

Subtotal:	$ 115,929

Labor

	Description	Quantity	Unit Price	Hours	Total
Cables					
	Operator	1	10.00	8	80
	Laborer	6	6.00	8	288
Subtotal					368
Deck					
	Laborer	2	6.00	32	384
	Ironworker	2	8.00	24	384
Subtotal					768
Towers					
	Operator	1	10.00	16	160
	Laborer	2	6.00	8	96
Subtotal					256
Foundations					
	Laborer	2	6.00	36	432
	Ironworker	2	8.00	8	128
	Carpenter	2	6.00	8	96
	Local Worker	10	2.00	16	320
	Laborer	2	6.00	8	96
	Local Worker	10	2.00	56	1120
Subtotal					2192
Cross Members					
	Laborer	3	6.00	16	288
Subtotal					288
Anchors					
	Operator	1	10.00	16	160
	Laborer	1	6.00	16	96
	Ironworker	2	8.00	32	512
	Local Worker	20	2.00	96	3840
	Laborer	1	10.00	32	320
Subtotal					4928
Earthen Ramps					
	Operator	1	10.00	40	400
	Laborer	4	6.00	120	2880
Subtotal					3280
Site Preperation					
	Operator	2	10	40	800
	Local Worker	10	2	80	1600
Subtotal					2400

Subtotal:	$ 14,480

Equipment

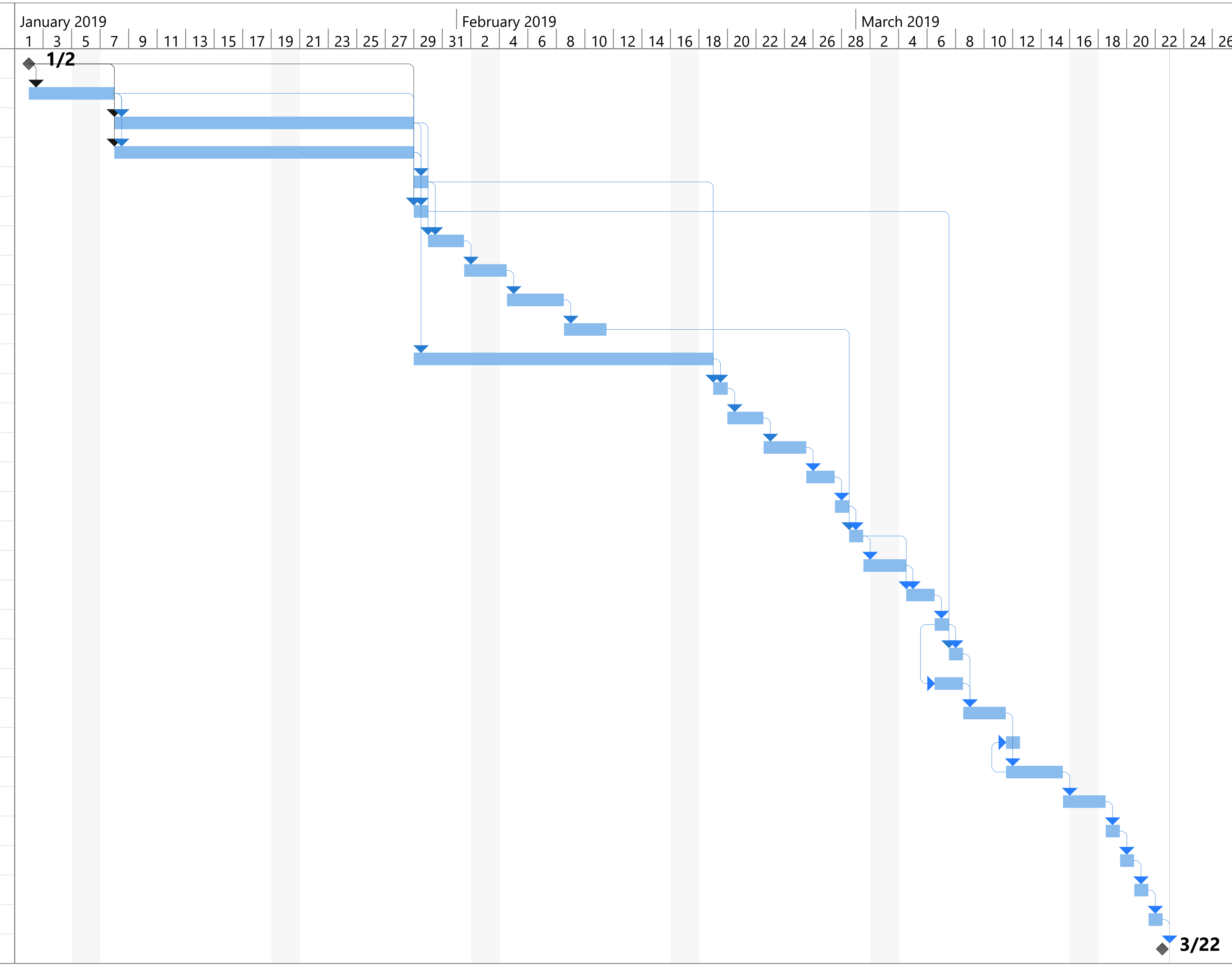
	Description	Quantity	Unit Price	Hours	Total
Cables					
	Excavator (329EL)	1	83.13	8	665
Subtotal					665
Deck					0
Subtotal					0
Towers					
	Excavator (329EL)	1	83.13	16	1330
	Trailer and Pickup for Tower Delivery	2	70.00	16	2240
Subtotal					3570
Tower Foundations					
	Dump Truck (14 yd. Tandem)	1	46.83	16	47
Subtotal					47
Cross Members					0
Subtotal					0
Anchors					
	Excavator (329EL)	1	83.13	16	1330
	Dump Truck (14 yd. Tandem)	1	46.83	32	1498
Subtotal					2828
Earthen Ramps					
	Excavator (329EL)	1	83.13	40	3325
	Dump Truck (14 yd. Tandem)	4	46.83	120	22476
Subtotal					25801
Site Preperation					
	Dozer D6N	2	46.95	40	3756
Subtotal					3756

Subtotal:	$ 36,668

Appendix I

Construction Schedule

ID	Task Mode	Task Name	Duration	Start	Finish
23		START	0 days	Wed 1/2/19	Wed 1/2/19
21		Mobilize	4 days	Wed 1/2/19	Mon 1/7/19
1		Transport Fill to Site	15 days	Tue 1/8/19	Mon 1/28/19
22		Clear Grub 1	15 days	Tue 1/8/19	Mon 1/28/19
2		Excavate Abutment Foundations 1	1 day	Tue 1/29/19	Tue 1/29/19
14		Place and Space Hangers	1 day	Tue 1/29/19	Tue 1/29/19
3		Compact Fill for Ramps 1	2.5 days	Wed 1/30/19	Fri 2/1/19
4		Form Tower Foundations 1	1 day	Fri 2/1/19	Mon 2/4/19
5		Place Rebar 1	4 days	Mon 2/4/19	Fri 2/8/19
7		Pedestal Forms 1	1 day	Fri 2/8/19	Mon 2/11/19
24		Clear Grub 2	15 days	Tue 1/29/19	Mon 2/18/19
25		Excavate Abutment Foundations 2	1 day	Tue 2/19/19	Tue 2/19/19
26		Compact Fill for Ramps 2	2.5 days	Wed 2/20/19	Fri 2/22/19
27		Form Tower Foundations 2	1 day	Fri 2/22/19	Mon 2/25/19
28		Place Rebar 2	2 days	Mon 2/25/19	Wed 2/27/19
29		Pedestal Forms 2	1 day	Wed 2/27/19	Thu 2/28/19
6		Pour Tower Foundation and Pedestal	1 day	Thu 2/28/19	Fri 3/1/19
31		Remaining Fill 1	1 day	Fri 3/1/19	Mon 3/4/19
8		Install Towers	2 days	Mon 3/4/19	Wed 3/6/19
9		Install Saddle	1 day	Wed 3/6/19	Thu 3/7/19
10		Hang Cable	1 day	Thu 3/7/19	Fri 3/8/19
15		Cross Members - Tower	2 days	Wed 3/6/19	Fri 3/8/19
11		Tension Cable	1 day	Fri 3/8/19	Mon 3/11/19
13		Place Angle	1 day	Mon 3/11/19	Tue 3/12/19
12		Pour Abutments	4 days	Mon 3/11/19	Fri 3/15/19
16		Cross Members - Decking	1 day	Fri 3/15/19	Mon 3/18/19
17		Wood Plank Decking	1 day	Mon 3/18/19	Tue 3/19/19
18		Metal Decking	1 day	Tue 3/19/19	Wed 3/20/19
20		Fence Guide Wire	1 day	Wed 3/20/19	Thu 3/21/19
19		Fence	1 day	Thu 3/21/19	Fri 3/22/19
30		FINISH	0 days	Fri 3/22/19	Fri 3/22/19



Project: Bridge Schedule
Date: Mon 12/10/18

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