Honduras Bridge
Final Report

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I. INTRODUCTION

For several communities near the rural village of Bacadillas, Honduras, access to the Predisan medical clinic is restricted by a steep riverbed which becomes impassable during the rainy season. In recent years, locals have annually constructed makeshift bridges to allow access to the clinic, only to have them washed away as the river level inevitably rises. The team aims to eliminate this issue by producing construction-ready documents for a long-term pedestrian bridge.

The design will be founded upon input from the community and direct measurements taken of the area, gathered during a team trip to the village in the Fall of 2019. Furthermore, the design emphasizes safety, constructability, economic feasibility, resiliency, and sustainability. An environmental assessment and community impact assessment were performed to ensure sustainability and safety in the design. A cost estimate was created to determine the economic feasibility of the project. A final pitch was then performed in conjunction with the Lipscomb Design Team to Predisan Health Ministries, owners of the clinic. The team hopes that the preliminary design documents and subsequent bridge construction will allow for safe and sustained travel to the clinic and consequently an improved quality of life for the surrounding communities.

II. PRE-DESIGN

Literature Review Overview

An extensive literature review was performed by all members of the team. During the first round of the literature review, information was found on numerous methods of footbridge design. Articles were found pertaining to modular footbridges, unconventional materials for footbridges such as thinning logs, and footbridge design in foreign countries. Information was also gathered on Bridges to Prosperity, including their design manuals and past projects. The information collected by each member was shared at a team meeting and compiled for use in brainstorming and design.

The team then performed a second round of literature review to cover some gaps in knowledge. More information on the land conditions in Honduras, specifically the local geology, hydrology, topology, available materials, and the flora/fauna, was found. This round of literature review focused more on learning about the site conditions rather than potential bridge ideas. Both of these reviews resulted in valuable information relating to international bridge design and site specific details. The information was later used in assessments during the pre-design and further informed decisions throughout the project.

Codes & Technical Specifications

The Bridges to Prosperity Bridge Builder Manual was referred to throughout the project. It provided valuable information and commonly accepted standards for portions of our bridge design including ramps, foundation placement, loadings, allowable deflections, and site sample collection advice. Creating quality plans was an initial objective during the planning aspects of this project. Following these standards that were made for international design, maintains the bridge's integrity. During the initial design phases, the specialization in suspension and suspended bridges made this manual invaluable.

The AASHTO Pedestrian Bridge Design standards were also referred to throughout the project. The full set of standards were not accessible, instead the mentors provided the necessary pages used in design. These standards were used to help with loadings, thermal expansion calculations, railing opening design, and railing strength design. The American Institute of Steel Construction Manual was referenced for steel member sizing and strength calculations, and the AISC Steel Bridge Alliance splice calculator was used in tandem with hand calculations to size splices.

Site Assessment

In November 2019, three of the five members of the team traveled to the site for three days of assessment work in conjunction with the Lipscomb Design Team. The goal of the visit was to procure engineering information concerning the topography, hydrology, geology, and material availability. An investigation into community wants and needs was conducted, ensuring that the community is a part of the decision process. By giving community members a stake in the project, they will be active members in the design and upkeep of the bridge.

The assessments performed were surveying, geological sample analysis, discussions with a local contractor, and discussions with members of the community. The team surveyed the proposed bridge site, including the topography near the clinic, the houses by the roadway, and the riverbed. The surveying data was then imported into AutoCAD Civil 3D and a surface was made for design. During the geotechnical investigation, geological samples were taken, 3 on the clinic side and 1 on the roadway side. Auger sampling and cone penetration tests were performed on site. Of the samples taken, only two samples made it through customs. These samples were used for further analysis in the geotechnical assessment. Onsite, multiple community meetings occurred. Learning more about the community and their desires and needs ensures that the community is actively involved and invested in this project.

Hydrological Assessment

The hydrological assessment is attached in Appendix B. This assessment utilized the hydrological modeling system HEC-HMS. Storm information was found from Hurricane Mitch, a 1 in 500 year storm that hit Honduras in 1998. Using this storm data and topographical data found online, a computer model on HEC-HMS was made of the watershed feeding into the bridge site. The maximum water flow was determined from the model, assuming large amounts of runoff from the surrounding hills into the stream system. From this flow, the high water mark was calculated using Manning's equation. The other variables were determined from the survey data and by estimating the roughness of the stream bed. The high water mark was calculated to be 100.4 feet from the model. However, testimonies from community members stated the water had risen as high as 101.4 feet, so the larger number was used to set the high water mark. Using the high water mark and 3 feet of freeboard, the bottom bridge elevation was set to 104.4 feet, resulting in a deck elevation of 106.6 feet. This data was later used for the foundation placement.

Geotechnical Assessment

The geological assessment is attached in Appendix D. This assessment was conducted by the Lipscomb Design Team. Both onsite and offsite testing results are summarized. This includes all estimates and reasonings in determining the geological layout of the site. Onsite testing consisted of auger sampling and the dynamic cone penetration test. Then the auger samples we packed to bring back to the US. The samples that made it through customs were analyzed in Lipscomb's lab. In the lab, the Lipscomb Design Team ran an atterberg limit tests and a sieve analysis. The type of soil was classified under USCS as poorly graded sand. The ultimate bearing capacity was an estimated 2000 pounds per square-foot. This data was later used for the foundation design.

III. DESIGN

Bridge Choice

Following the initial assessments, the team looked towards similar projects to find guidance as to which bridge type would best suit the site. The design standards initially chosen were those of Bridges to Prosperity (B2P), a well-established international pedestrian bridge design and implementation organization. B2P specializes in suspension and suspended bridges and so those were the primary bridge options originally pursued. Alternatives, such as the truss bridge, were also entertained, but initially ruled out in comparison to the less expensive and more constructable nature of suspended and suspension bridges.

Upon delving deeper into the design process for these wire rope bridges, some significant challenges arose. The most problematic issues encountered included limited international availability and high expense of steel cables, difficulties in meeting freeboard clearance requirements due to bridge sag, and minimal space for tie-back foundations on the road-side. Upon notification of Predisan's preference for a steel-beam bridge, the team decided to pivot the design intentions to accommodate the wants and needs of those the bridge would be serving.

Before resorting to a truss bridge design, the team thoroughly researched all available options and discovered the stringer bridge. Reservations with the truss bridge primarily laid with the relatively complex on-site assembly involved. These concerns were solved by the stringer bridge, which basically consists of two simply supported beams spanning the entire length of the bridge. Not only are on-site assembly and construction efforts reduced when compared to the truss bridge, but difficulties in satisfying foundation spacing and freeboard clearance requirements were resolved as well. Additional benefits of the stringer bridge design include an overall smaller footprint because of its minimized weight, no need for heavy equipment or special tools during construction, and a design that can primarily be constructed by local residents to support community involvement. It became clear that a stringer bridge was the best option for this project.

Beam Selection

The beam selection calculations are attached in Appendix E.1. As the primary load-bearing component of the bridge, choosing the beam was an important decision that would influence the rest of the bridge's configuration.

To begin, some initial assumptions had to be made. Informed by the hydrological and topographical analyses of the site, a span length of 80 feet was decided upon. Three equally-spaced lateral bracings were assumed for an unbraced length of 20 feet.

As for loadings, conservative values were used as safety and durability were top priorities. For the dead loads, a 30 pound per linear foot uniform load was chosen for the decking components, a 40 pound per linear foot uniform load was chosen for the railing components, and a 70 pound per linear foot uniform load was chosen for the beams and additional steel components. For the live load, a uniform load of 255 pounds per linear foot was used for strength design. While this may seem excessive for a pedestrian footbridge, this loading accounts for community members traversing the bridge using motorcycles within reason. Live load deflection checks account for twenty-five, 200-pound people and motorcycle use. Beam deflection was also checked for a 500 pound point load acting at midspan to represent a motorcycle. While the use of motor vehicles is strongly discouraged by the rest of the bridge's design, it was decided to accommodate the occasional vehicle on the bridge for safety purposes.

Using a factor of safety of 1.25 for the dead load and 1.75 for the live load, a uniform load of 0.3106 kips per linear foot was required per beam. Multiple beams were tried and checked for moment capacity and live load deflection serviceability. After meticulous calculation and careful consideration, W24x68 Grade 50 steel beams were chosen.

The bridge span will be 80 feet, with an additional 2 feet on each end to connect the beams to the abutments. It is not feasible to transport 2 84-foot beams to the bridge site that each weigh over 5,700 pounds. It was expedient to divide the bridge into 3 smaller sections, each 28 feet long. These members will fit into a tractor trailer and will be transported to their final location with some methods discussed below.

Superstructure

Splices

Since each beam will be divided into three sections, it was necessary to design beam splice connections. The connections consisted of bolted plates on the top and bottom flanges and on the webs of the W-shape steel members. Using 3 beams of equal length, the splices would occur at the third points of the span. Using an Excel Spreadsheet from the *American Institute of Steel Constructio*n (AISC) in tandem with hand calculations for assurance, the specifications were input for the bridge. In return, the number of bolts needed for both the flange and web splice plates were given. In this case, the flanges required two rows of 3 bolts each and the webs required two rows of 5 bolts each. Conceptual drawings can be seen below in Figures 1 and 2:

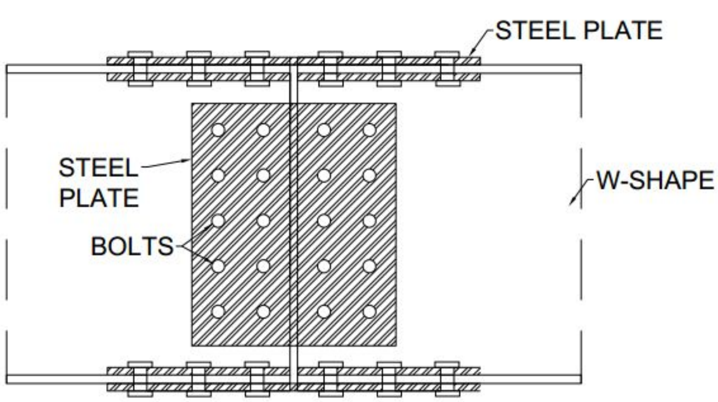

Figure 1: Splice Elevation View

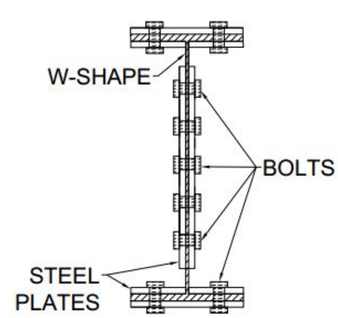

Figure 2: Splice Cross Section

Cross-Frames
In the process of sizing the steel members, an unbraced length of 20 feet was assumed. Unbraced length is a key component in the lateral-torsional buckling failure mode. The cross-framing every 20 feet along the span prevents this failure mode from occurring and allows the choice of smaller steel sections for the superstructure. Channel sections (C12x20.7) were used for each brace in the design. For added stability against any unexpected lateral load on the structure such as wind, cross-framing was used at each end of the span. These elements consisted of angles connected together in a cross shape to additionally support the span. For these steel pieces, L4x4x3/8 members were chosen. The layout of these elements can be seen in Figure 3 below, and the design details can be found in Appendix I.

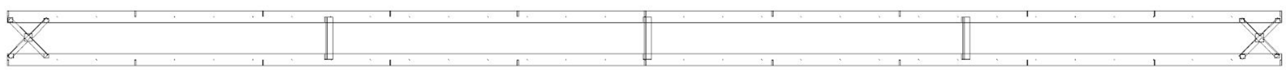

Figure 3: Cross Frame Layout

Decking
The walkway design can be composed of either composite material or treated wood. Each option has advantages and disadvantages. However, composite material is more durable and long-lasting so it is the recommended choice for this design. The decking must be water-resistant and must be able to span 2 feet with the design loadings without failure. Most composite materials meet this standard. The local availability of composite materials for decking is a possible issue. Treated wood would be a viable substitute given the low environmental impact, but it is not widely available locally.

Railings
Railings on the side of the bridge ensure the safety of the users. They are designed to support a lateral load in accordance with AASHTO Standards and maintain spacing such that a 6-in diameter sphere could not pass through the railing at any point (AASHTO).

The railings will consist of 4-inch by 4-inch posts spaced at 8 feet connected to a handrail and a toe board, each sized at 2 inches by 4 inches. Chain link fencing will be stapled to the horizontal members of the railings with heavy-duty staples. Wood was selected as the railing material because it is both less expensive and more environmentally friendly than steel. Chain link fence was selected to ensure safety of the users by minimizing gaps in the railings.

The railings will be connected to the bridge at two places. First, a C7x9.8 will be bolted to a 20-inch by 4.25-inch by 0.5-inch stiffener plate welded to the W24x68. The C-shape will also bolt to the 4-inch by 4-inch post. The post will also bolt to two L3x3x½'s that will connect the post to the nailer board.

Substructure

Approach Ramps
The Bridges to Prosperity Bridge Builder Manual (*Bridges to Prosperity*) was used to design the approach ramps. The ramps will consist of 8-inch concrete masonry block walls supported by a

1-foot-thick concrete footer. Inside the walls, levels of fill consisting of rock, gravel, and sand will meet the top of the masonry blocks. The ramp will be capped with a 4-inch-thick concrete slab.

Railings will be embedded in the ramp. The posts will be 4 inches by 4 inches and extend at least 42 inches above the surface of the ramp. Horizontal 2-inch by 4-inch members at the top and bottom of the posts will be attached to chain link fencing with heavy-duty staples.

The ramps will be no steeper than a 5:1 (20%) slope to ensure usability for clinic patients. The exact dimensions of the ramps will be determined in the field by the contractor in accordance with the given specifications. The base of the ramp will match the existing grade.

Foundation

The foundation design relies heavily on the information gathered in the geotechnical assessment. The maximum gross allowable bearing capacity of the soil is 2000 pounds per square foot. This value is potentially conservative, as every boring hit rock before the 6.5 feet specified by B2P, which qualifies the ground conditions as rock (*Bridges to Prosperity*). However, the cause of refusal could also be large boulders suspended throughout the subsurface profile, so the conservative value was used. The foundations are constructed of three elements, as seen in Figure 4: a footing (foundation), an abutment, and an endwall.

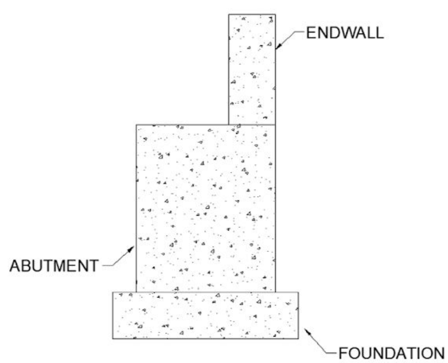

Figure 4: Bridge Foundation Design

The footing must be large enough to spread the weight of the bridge-foundation-combo over the soil, so as not to exceed 2000 pounds per square foot. The abutment raises the bridge elevation so that it is above the maximum water level determined in the hydrological assessment, and the endwall acts as a retaining wall to hold backfill material from under the ramps. The foundation will be made of concrete, and will need enough rebar for possible tension forces in the concrete.

The bridge is relatively light compared to the weight of large buildings that foundations often need to support. The calculations for the dimensions/thickness of the concrete and the area of steel were both calculated, as can be found in Appendix E.4. However, the calculated values were all below the minimum required footing thickness and area of steel, so the minimum specifications were utilized for the design. This resulted in a 1 foot thick footing, 5.3/5.8 foot tall abutments, and 458 feet of rebar.

Abutment Connections

Due to the variation in temperature, slotted holes on one side of the bridge was deemed necessary. The connections between the beams and abutments on the north end will be bolted in place while the south end will have slotted holes to allow for thermal expansion and contraction. Using AASHTO's Pedestrian Bridge Guidelines, the thermal expansion was estimated to be around 0.76 inches. To allow for this expansion, the slotted holes are designed to be 2.5 inches long with a 1.125 inch width to accomodate the 1 inch anchor bolt. The 2.5 inch slot size allows for an expansion or contraction of 1.25 inches, allowing for a large clearance of our estimated expansion length.

To connect the W24x68 beams to the abutment, a bearing plate, a neoprene pad, and anchor bolts are utilized, as visualized in Figure 5. The beams are attached with a fillet weld to the bearing plate, which lays on top of the neoprene pad. The base plates on the south end of the bridge will have the aforementioned slot hole to allow for thermal expansion and contraction. The neoprene pad, however, will not have a slotted hole and will remain stationary.

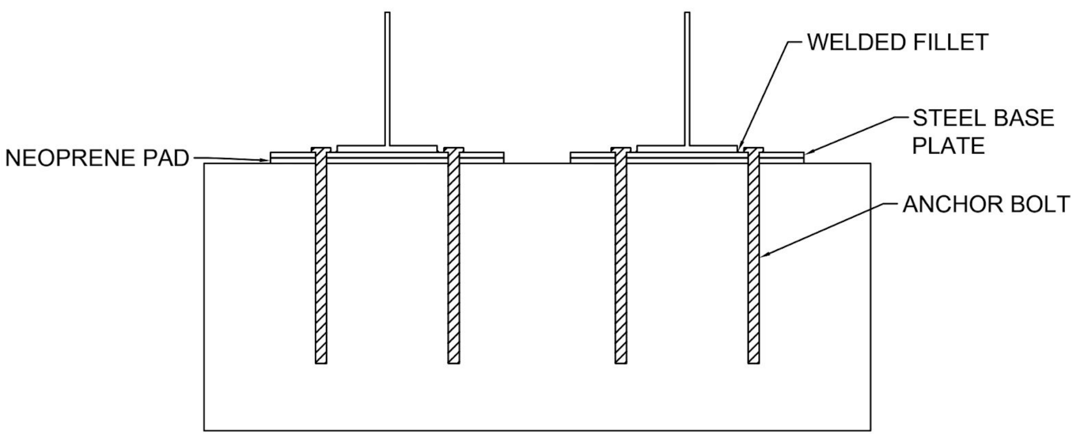

Figure 5: Beam-to-abutment connection geometry.

Constructability

A differentiating factor of this bridge design is the simplicity in construction. The small bridge footprint fits perfectly between the existing houses. For a bridge of this span, and without the help of machinery, it is anticipated that shoring will be required for construction. This will be true for a truss or a stringer bridge, eliminating potential additional costs. This simple and intuitive design consists of two beamlines. The designed field splices allow for each individual piece that could be positioned by hand without the need for construction equipment. Eliminating the need for construction equipment further increases the cost savings of this design. With few members, this design offers straightforward construction and avoids the confusion of many structural components. Volunteers will be helping with the construction and safety is integral. A simple design is safer for all those involved.

Potential challenges may arise during the transportation of the large members to the rural community and moving the heavy members from the bank onto the falsework. Transportation of such large members has been considered. The longest members are about 28 feet in length and large trucks are necessary. During the site visit, trucks of this size were seen near the proposed bridge site, indicating it is possible to get the materials to site. The heaviest members weigh approximately 1,900 pounds. Once onsite, members can be transferred one of two ways or a combination of both. By rolling members on top of logs or carrying them using straps, the beams can be safely moved towards the falsework. Rolling members is a cheap method to move the beams without necessarily having to completely lift them. Using straps, volunteers will stand on each side of the bridge and lift together to move the bridge. The simplicity of construction minimizes risks to both the bridge's integrity and those who construct it.

IV. CLOSING ASSESSMENTS

Community Assessment

The community assessment is attached in Appendix F**.** The assessment was led to confirm that the addition of a bridge to the community would be an effective solution. While on the site visit, team members were able to interact with community leaders and with the community at large. Afterward, it was determined that the clinic is incredibly well run. Patients attend their appointments and the staff members consistently update knowledge on the medical conditions of the communities they serve, meaning that a bridge would solve the last issue tampering with the effectiveness of the clinic: being able to physically cross the river. Furthermore, community members are engaged in community projects and are willing to lend a hand for the construction and upkeep of the bridge. This is proven in their meeting engagement, as seen in Figure 6, as well as with a past bridge-build to a soccer field. Finally, the needs of the community had to be taken into account in the design process. While the team initially chose a suspended/suspension bridge for their low cost and ease of construction, there was potential for the tiebacks from such a bridge to infringe on private property. As such, it was not the right choice for the community and a stringer bridge was utilized instead.

Figure 6: A community-wide meeting during the site visit.

Environmental Assessment

The environmental assessment is attached in Appendix G. This assessment was conducted to examine both the carbon impacts of this project and the impact on the local ecosystem. A goal during the design process was to remain environmentally conscious of this impact and minimize the impact when able. Total steel was reduced as much as possible in the bridge design. Carbon emissions were further minimized by the usage of local resources. To maintain the local ecosystem, there was a focus on managing erosion, water runoff, and sedimentation throughout the construction process.

Cost Estimate

The cost estimate spreadsheet is attached in Appendix H. This spreadsheet details each bridge component's total cost, total quantity, and unit cost. Table 1 below provides a summary of the design's most significant cost categories.

Table 1: Cost Estimate Summary

	Cost Category	Quantity	Cost/Unit ($)	Total Cost ($)	Total Cost (HNL)
SUMMARY	**Overall**			**30,770.69**	**762,082.29**
	Structural Steel	13,110.62 lbs	$1.25/lb	16,388.30	405,880.83
	Decking Panels	280 square-ft	$20.00/square-ft	5,600.00	138,692.40
	Concrete	596.5 cubic-ft	$4.82/cubic-ft	2,875.13	71,206.91
	Bolts and Screws			1,721.00	42,623.15
	Rebar	776.08 lbs	$1.00/lb	776.08	19,220.79
	Masonry	650 blocks	$1.00/block	650.00	16,098.23
	Superstructure			**26,062.25**	**645,470.71**
	Girders			14,280.00	353,665.62
	Splices			1,365.88	33,828.07
	Cross-Framing			700.27	17,343.24
	Railing			1,820.14	45,078.50
	Decking			5,884.40	145,735.99
	Ramps			2,011.56	49,819.30
	Substructure			**4,708.44**	**116,611.58**
	Primary Foundations			2,991.15	74,080.32
	Ramp Foundations			1,053.51	26,091.76
	Excavation		$1.11/cubic-ft	663.78	16,439.51

Throughout the bridge's design, cost was always minimized without compromising safety or design quality. Some of the ways in which costs were reduced include:

- The use of long, continuous spans to lessen the number of expansion joints and bearings required, consequently limiting the future maintenance costs accompanied by these design aspects (*Steel Bridge Design Handbook*)
- Minimizing the bridge span while complying with freeboard requirements
- Choosing the most economical and effective beam shape
- Maximizing the use of locally sourced materials, such as in the design of railing components

The overall bridge construction is expected to cost $30,770. This total cost includes the transportation fees associated with all the materials needed. Note that all values listed are educated estimations informed by industry professionals' pricing knowledge and reliable online resources and therefore do not guarantee the price of any specific material or bridge component. For this reason, a 10% contingency of $3,077 has been included in the cost assessment.

V. LESSONS LEARNED

During the fall semester, a professor from the Peabody school came to the senior design seminar to deliver a lecture on small group dynamics. He talked about the four stages of group development, one of which is "norming." During norming, group members are finding their place in the group, how they will contribute, and who the leaders are. One thing a group can do to help during this time is to create structures that promote interpersonal relationships that are not project-based and make members feel included. The Honduras Bridge Design Team attempted to accomplish this through monthly social events. Group members would take turns hosting a meal at their home, and other group members would often bring ingredients and help cook. The team found these events helped tremendously to boost morale and develop friendships.

On the site visit to Honduras, it was very evident the bridge would not be designed and built in an ideal environment. One example of this was the difficulty of collecting data. With no GPS or standardized coordinate system to tie into, the engineering survey was done using relative coordinates based on benchmarks set near the site. Not all necessary geotechnical equipment was able to be transported to the site from the United States, so data collection was limited for the geotechnical report. Some of the soil samples were confiscated when reentering the United States because they contained trace amounts of organic material. The team was unable to obtain local precipitation data, so data from a singular storm was used to run the hydrologic model. All of these demonstrate that, while ideal data sets may not be available or possible, an engineering team must be resourceful in obtaining the information required to deliver a robust, safe design. This may necessitate using a conservative estimate on some design aspects, which will increase costs. However, the extra expense is worth the assurance of safety.

The site visit also helped contribute to the team's understanding of the needs of the Bacadillas community. An engineering solution that does not account for these needs is at best useless and at worst harmful to the people it serves. Care was taken to interview community leaders so as to ensure a design that benefited the community and would protect their safety.

Working on a project in another country poses unique challenges. The site visit helped with many of these challenges, but the team still had to work around many of them, such as a language barrier in many reports, working in both metric and english units, and ensuring the bridge could be constructed with minimal equipment and volunteer labor. These challenges helped give the team a deeper experience and a greater appreciation for the necessary codes and standards in the United States.

There were many entities that contributed to this project, and lines of communication needed to be open between all of them. The design team needed to communicate with Predisan staff, community members, the Lipscomb Design Team and their faculty, and the professional mentors. The team also needed to communicate among its own members. Communication was typically communicated via email outside the group, and via Slack and team meetings within. The members of the team learned to communicate clearly and succinctly over the course of the year, which increased the efficiency of work. One example of how this impacted the project was the bridge type. Initially, the team planned to design a suspended or suspension bridge. However, after presenting this idea to Predisan, they expressed concerns over obtaining steel cables. This allowed the team to switch to a stringer design, which will be of better service to Predisan and avoid impinging on private property near the clinic, serving the needs of the community better.

Each aspect of the bridge needed to be designed with attention to detail. Unfortunately, it was a learning curve to reach the appropriate level of detail needed for the final plan set. Each aspect of the bridge took several iterations of design, team discussion, and mentor review before it reached an appropriate level of detail. In retrospect, a better understanding of the amount of specifications required for each component of the bridge would have streamlined design.

The team was made up of five members, all with different experiences, strengths, and interests. Each member did a good job of articulating which aspects of the project they were interested in and being proactive about contributing to those aspects. This allowed the project to run more efficiently and gave each team member incentives to work hard on things they were interested in.

VI. CONCLUSION AND FUTURE WORK

Attached to this report are construction-like documents that can aid the Predisan Health Ministry in deciding on a final bridge design. Construction of the substructure was originally planned to begin during the summer while the superstructure was to go in August. Due to COVID-19, the construction schedule is now uncertain. Lipscomb University has a hopeful estimate that the substructure will now go in around August 2020, and the superstructure will follow a few months after. The attached design documents will need review by professional engineers if this design is chosen. They are not construction-ready. After this review and updated cost assessment, a bill of materials can be compiled and construction may begin. The Vanderbilt Design team has included all calculations used in the design process to this point for optimal clarity, but the team will gladly work with any engineers with questions as the process moves forward.

Bridge Maintenance

In order to be a successful long-term solution, the bridge will have to be regularly maintained. The members of the community will play a large role in this. During the site visit, several community leaders testified that the community is well versed with forming effective committees to complete tasks. The community will have to organize groups that will repaint the bridge regularly, as well as perform structural checks to ensure no deformations are forming in the bridge. If any decking or railing components decay or are damaged, they will need to be replaced promptly for safety purposes.

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- *E.4: Foundation*
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Appendix I: Design Documents

Appendix A: References

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Appendix B: Schedule

Final Schedule

	Deadlines shown in bold on chart	September				October				November					December				January					February				March				April				
Phase	Item	2	9	16	23	1	7	14	21	1	4	11	18	25	2	9	16	23	1	6	13	20	27	3	10	17	24	2	9	16	23	1	6	13	20	27
Start	Teams Announced		**9/9**																																	
Preliminary Project Work	Initial Literary Review																																			
	Specific Literary Review																																			
	Initial Mentor Meeting																																			
	Planning trip to Bacadilla																																			
	Researching standards and ratings																																			
	Prepare First Semester Presentation																																			
	First Semester Report																																			
Site Visit and Assessment	Engineering data collection																																			
	Community data collection																																			
Pre-Design	Finalize design standards																																			
	Brainstorm bridge type, materials, logistics																																			
	Determine bridge type, materials, logistics																																			
Engineering Design	Geotechnical Assessment																						**2/2**										**4/12**			
	Hydrological Assessment																						**2/2**													
	Design Iteration 1																									**2/23**										
	Design Iteration 2																													**3/22**						
	Final Engineering Design																															**4/3**				**4/29**
	Perform environmental assessment																																			
Final Deliverables	Draft 1 of Report																															**4/3**				
	Final Draft of Report Complete																																**4/12**			**4/29**
	Practice Presentation																															**4/2**				
	Final Presentation																																**4/15**			
End	Design Day																																		**4/20**	

Actual Schedule
Planned Schedule
Missed Original Deadline
Met Original Deadline

Appendix C: Hydrologic Assessment

Vanderbilt University Hydrologic Assessment of Bacadillas Watershed

Watershed Description

The watershed boundary was delineated by hand using a topographical map (attached). The watershed boundary was then drawn onto the topographical map in AutoCAD Civil 3D, then delineated into 9 separate subbasins. There are four stream reaches in the watershed located in subbasin 1, subbasin 2, subbasin 6, and subbasin 8. The delineations are outlined in Figure 1 below. The X is the approximate location of the bridge site.

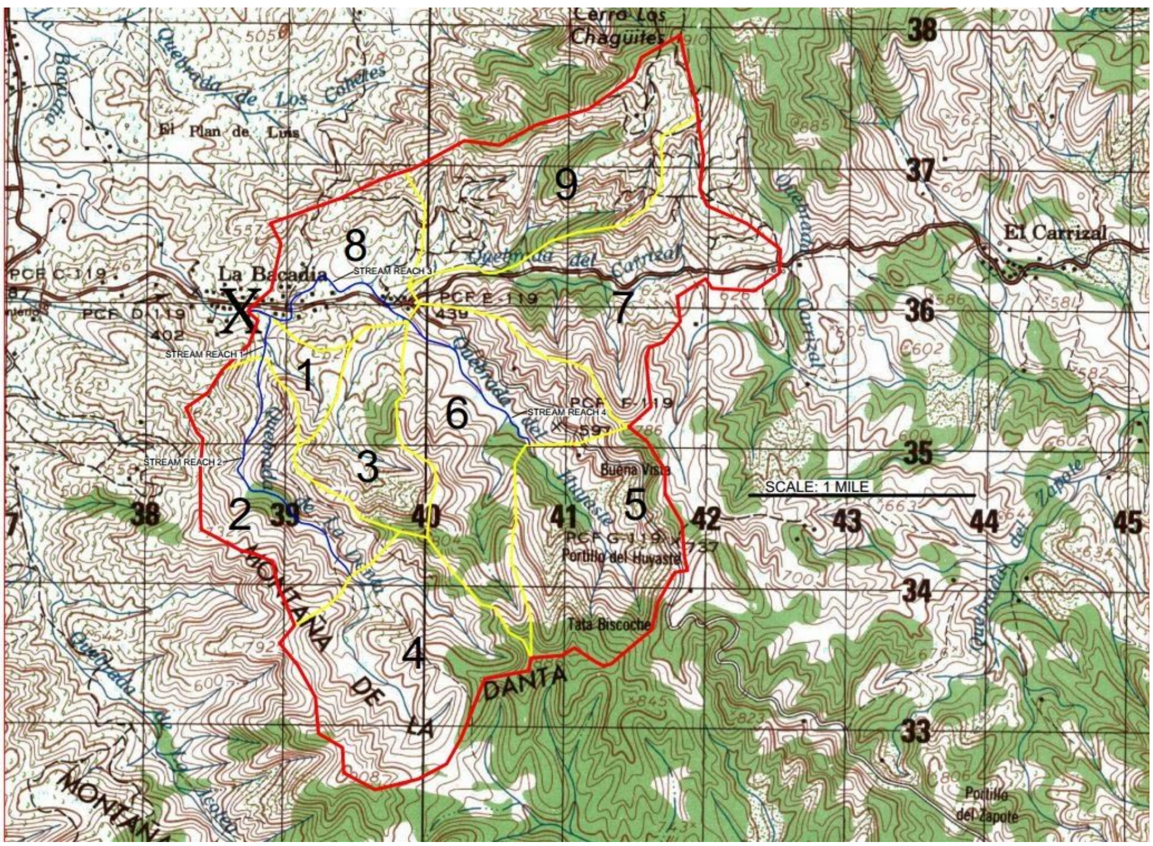

Figure 1: Watershed and Subbasin Overview

The SCS Curve Number Loss Method was used to estimate runoff. A curve number for each subbasin had to be estimated. The curve number relates the land use to infiltration rates. Based on the soil map in Figure 2, the predominant soil surrounding our area of interest were alluvial soils. We used both this map and our observations from the auger samples to determine the soil to be class D. We assumed a wet antecedent moisture Based on these properties and the land use we estimated the curve number to fall between 65-90. Using google maps, we estimated the impervious areas for each subbasin.

Figure 2: Honduras Soils Map

Synder's Unit Hydrograph Method is the transform method used. The time to peak and peaking coefficient were used in the model. The time to peak was determined by estimating L_{ca}, the distance along the main stream from the base to a point near the center of gravity of the basin and L, the length of the main stream channel. The C_t values inputted into the model were determined from typical values found for foothills areas (0.7) and mountainous watersheds (1.2) [2]. Using google maps, the site visit, and the topographical map, we estimated the basin roughness to determine the C_t. The peaking values (C_p) values were determined from the common range of 0.4-0.8 [2]. The higher values correspond to more mountainous areas, while the smaller values correspond to flatter areas.

The Muskingum Routing method was used to route the reaches in the watershed. This storage routing method depends on two parameters, X and K. The K value is the travel time through the reach determined by finding the time of concentration using estimates from the USBR Designs of Small Dams equation. It is dependent on the change in elevation between the outlet and divide and the channel length. We deemed this was applicable to our watershed and the error from estimating the parameters is minimal compared to other time of concentrations equations. The X value is a storage constant. Based on literature, the X is typically 0.2. This X value corresponds with the roughness of the stream routing channels observed near Radnor Lake, which would disseminate the peak flow entering the stream reach before it reaches the lake.

Computer Model

HEC-HMS is the computer system used to analyze the watershed. The data outlined in Appendix A was inputted into the model. The basin model for Bacadillas is seen in *Figure* 3. The precipitation data used for this analysis was from a 72-hour storm from Hurricane Mitch. This storm was estimated to be a 1 in 500/600 year storm event.

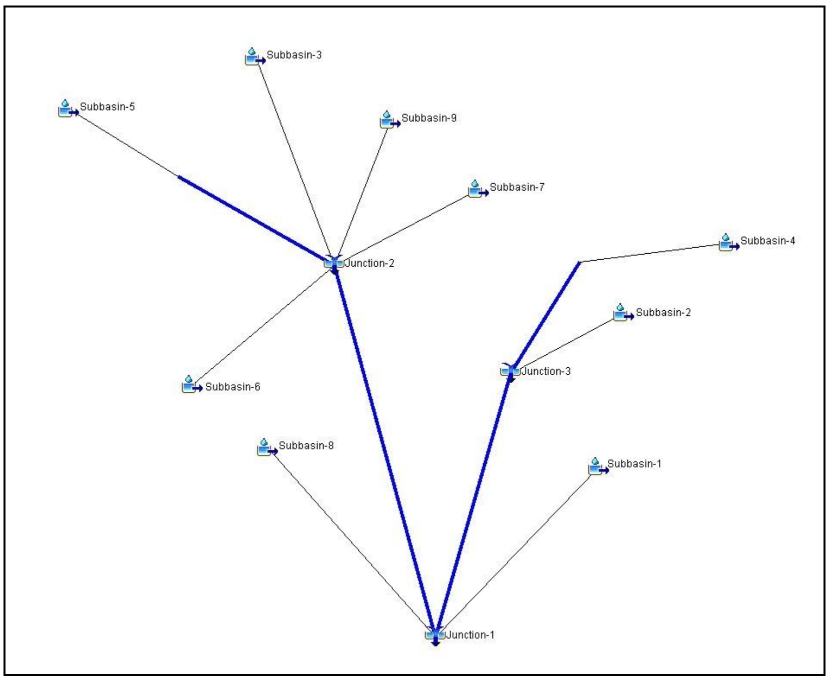

Figure 3. HEC-HMS Basin Model

After inputting all necessary information, the model computed the maximum flow downstream at Junction 1. This flow was then used to compute the maximum water height using Manning's equation for open channel flow based on an estimated cross section seen in Figure 4.

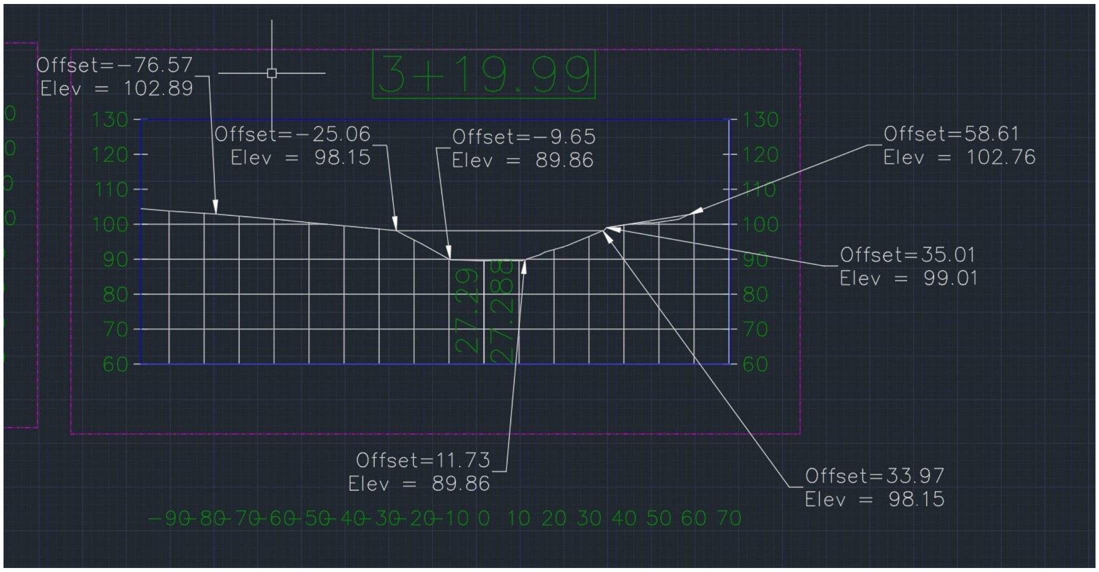

Figure 4: Stream Crossing at Proposed Bridge Location

Table 1: Curve Numbers, Discharge, and Max Elevation

CN	Q (cfs)	H (ft)
65	1321.4	8.079
70	1500.0	8.867
75	1566.2	9.139
79	1651.5	9.446
85	1758.5	9.786
90	1825.4	9.980

This process was repeated at the varying range of curve numbers to determine how our estimations affect the flow and consequently the maximum water height.

A roughness coefficient (n) of 0.15 was estimated for the stream channel based on the high amount of trees, shrubs, rocks, etc. found in the stream.

Results

Based on our model, the maximum flood height using the most conservative curve number estimate is approximately 10-ft. The measured high water marks are about 11-ft above the bottom of the river. Therefore we recommend the lowest point of the bridge should be 11 feet above the centerline of the stream, which is an elevation of 101.4’ at the proposed bridge site. This elevation corresponds to the elevation of the back porch of one of the community member’s homes, which was presented as a high water mark by the local residents. However, the bridge could be lowered if it is determined that a 500-year design is too cautious.

Many assumptions were made using this model and a lot of the data was older. The precipitation storm data was gathered from Tegucigalpa during Hurricane Mitch in 1998, which is considered a 1 in 500/600 year storm. The topographic map used to delineate the subbasins was from 1998. Using Manning's equation, the bottom of the riverbed was assumed to be flat and the slope was extended linearly to counter lack of data.

Sources

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Data

Table 2: Stream Reach Data

Reach No.	L, Length of course, feet	L, Length of longest water course, mi	H, elevation of diff. b/w divide and outlet, m	H, elevation of diff. b/w divide and outlet, ft	time of concentration, (minutes)	K (hrs)	X
1 (sub 1)	1526	0.289	10	32.8	9.68	0.161	0.2
2 (sub 2)	6664	1.262	165	541.2	18.06	0.301	0.2
3 (sub 8)	4987	0.945	15	49.2	32.52	0.542	0.2
4 (sub 6)	4376	0.829	60	196.8	16.40	0.273	0.2

Table 3: SCS Curve Number Data

SCS Curve Number Method				
Subbasin No.	Area (sqft)	Area (sq mi)	Curve Number	Impervious (%)
1	3342486	0.120	65-90	0
2	14514673	0.521	65-90	0
3	8981561	0.322	65-90	0
4	20030828	0.719	65-90	0
5	17372249	0.623	65-90	0
6	19915403	0.714	65-90	0
7	20709632	0.743	65-90	0

8	11198907	0.402	65-90	1
9	18577955	0.666	65-90	0

Table 4: Snyder's Method Data

Snyder's UH Method					
Subbasin No.	Ct	Lca (mi)	L (mi)	tl (hr)	Cp
1	0.7	0.271	0.3258	0.338	0.8
2	0.9	0.709	0.9517	0.800	0.7
3	1.0	0.754	0.9614	0.908	0.6
4	0.9	0.676	1.1566	0.836	0.7
5	1.2	0.799	0.9883	1.118	0.4
6	1.0	0.756	0.8470	0.875	0.6
7	1.1	0.977	1.2566	1.170	0.5
8	0.7	0.549	0.9182	0.570	0.8
9	1.1	0.795	1.0735	1.049	0.5

Table 5. Precipitation Data, Hurricane Mitch 1 in 500/600 year storm (Incremental Inches)

Hour	Rainfall (in)
0	0.000
1	0.011
2	0.000
3	0.008
4	0.017
5	0.026
6	0.157
7	0.219
8	0.050
9	0.000
10	0.000
11	0.000
12	0.035
13	0.039

14	0.000
15	0.005
16	0.000
17	0.000
18	0.048
19	0.000
20	0.000
21	0.000
22	0.000
23	0.000
24	0.017
25	0.005
26	0.092
27	0.037
28	0.164
29	0.153
30	0.127
31	0.109
32	0.087
33	0.131
34	0.119
35	0.066
36	0.535
37	0.394
38	0.234
39	0.149
40	0.087
41	0.085
42	0.015
43	0.151
44	0.151
45	0.234
46	0.297
47	0.814

48	0.525
49	0.755
50	0.569
51	0.510
52	0.503
53	0.477
54	0.433
55	0.212
56	0.235
57	0.026
58	0.026
59	0.168
60	0.096
61	0.328
62	0.262
63	0.042
64	0.009
65	0.011
66	0.007
67	0.079
68	0.014
69	0.000
70	0.007
71	0.000
72	0.007

Appendix D: Geological Assessment

Bacadilla Pedestrian Bridge Geotechnical Engineering Report

April 16, 2020

NECT Solutions

Noah Kimbrough, Emily Morgan, Chris Schneider, Trent Beacham

One University Park Drive

Nashville, TN 37204

Table of Contents

3 NECT Solutions: Bacadilla Bridge 19001

Introduction

This report documents the findings of our surveying and geotechnical investigation for the proposed bridge for the Predisan clinic located in Bacadilla, Honduras. This report will include information on existing site conditions, foundation design, and more.

Information regarding testing of the soil samples and results of the testing will be in the Exploration and Testing Procedures and Exploration Results sections, respectively. In addition the Boring Logs will also be located in the Exploration Results section.

4 NECT Solutions: Bacadilla Bridge 19001

Site Conditions

The following description of the existing site conditions is from our site visit in November 2019 in collaboration with the Vanderbilt Team.

Item	Description
Parcel Information	Proposed site is located in Bacadilla, Honduras. N14°47.746’ W085°42.574’
Existing Improvements	Some houses with wall
Current Ground Cover	Vegetation in and around river
Existing Topography	Site slopes down from southeast to northwest about 96 feet to 90 feet, MSL, into the river bed. Then it slopes back up from southeast to northwest about 90 feet to 106 feet, MSL, to top of river bank.

Project Description

Our scope of project can be located in our initial proposal. A brief overview of the project will be stated below:

Item	Description
Project Description	Predisan plans to build a permanent bridge to replace the temporary wooden bridge that allows the villagers to cross the river in order to access the clinic. They have procured a new site for the proposed bridge.
Proposed Structure	The proposed structure consists of a steel truss bridge with carbon fiber decking. The bridge is approximately 100’ in length. In addition to the bridge, there will be a retaining wall added to the site as well.
Bridge Construction	The means of construction are undecided at this time.
Finished Deck Elevation	The finished deck elevation will be approximately El. 108 feet, MSL.
Maximum Loads	Reactions= 27.7 kips per abutment
Grading/Slopes	A 10% approach has been proposed on both the clinic and the road sides of the bridge.
Below Grade Structures	Both foundations and the retaining wall will be partially below grade.
Free-Standing Retaining Walls	One retaining wall will be built on the road side of the site.
Pavements	There are no proposed pavements included in this project.

Construction was planned to start in May of 2020 but due to the unforeseen circumstances of Covid-19, the new construction date is TBD.

Geotechnical Characterization

Much of the geotechnical information for this site is unknown as some of the soil samples were confiscated when traveling back to the United States. Only two samples made it back. Both of these samples were disturbed, therefore we have no samples to test for unconfined compressive strength.

Subsurface Profile

We have a basic idea of the subsurface profile of the site based on the site visit and research of previous projects in Honduras. Borings B-1 to B-4 revealed around 1 foot to 4 feet of sandy soil before auger refusal. The area around the site has many large boulders which was the cause of the auger refusal.

Two penetration tests were executed but due to a missing cone, a modified version had to be created. Unfortunately, there was no opportunity to correctly calibrate the dynamic cone penetration test values to standard penetration test values due to the Covid-19 outbreak. One calibration was done but due to incorrect testing procedures, the results are inaccurate. Under the observation that the standard cone tip will be more resistant than the modified cone bit leading to an increased number of blows with the standard tip. Therefore, our values will be considered a conservative estimate due to the decreased number of blows.

Finally, due to the limited geotechnical samples obtained, we assumed a maximum allowable bearing capacity of 2000 psf for the foundations and retaining wall. For the complete boring logs, please see the Exploration Results section. Below is a table giving a brief overview of our borings:

Boring No.	Approximate Surface Elevation (feet)	Auger Refusal Depth (feet)	Approximate Auger Refusal Elevation (feet)
B-1	100.23	2	98.23
B-2	100.23	4	96.23
B-3	100.23	1.83	98.4
B-4	100.23	1	99.23

Groundwater Conditions

No groundwater was observed within the boreholes. We are assuming a deep groundwater table and therefore neglecting it.

Geotechnical Overview

Based on the information from our investigation, the site can be developed for the proposed bridge and retaining wall. Although, the following geotechnical considerations were identified:

- **Rock Excavation-** Due to the nature of our borings, there is sure to be rock that we run into when constructing the foundations and retaining walls. It is unsure whether this rock is bedrock or not but for now it is assumed to be boulders. These boulders will need to be excavated before placement of the foundations and the retaining wall.

Earthwork

The fill material that will be used for the approach on both sides of the bridge will be local river soil that will allow for optimum drainage in high water conditions. The roadside approach requires 275 square feet of fill to produce a 10% slope. The clinic side approach will require 279 square feet of fill to produce a 10% slope. These fill estimates were generated in NECT Solutions Civil 3D site design model.

Shallow Foundations

The proposed bridge design will include two strip footings, one on each side of the bridge. The base of each footing will be 5’ x 6’ x 1’ .The entire height of the foundation will be 10’ tall and approximately 3’ will be below ground surface. The top of the foundation will be a 2’ x 6’ x 6’ wall.

Some considerations for the foundation and retaining wall construction is the rock within the soil at the site. There were large boulders discovered in the boreholes which will need to be excavated prior to construction.

Deep Foundations

There is no need for any deep foundations for this project.

Seismic Considerations

There are no seismic considerations for this project.

General Comments

This report was based upon research of past projects, site visits, and educated assumptions due to missing data. Let it be known that site conditions could change due to natural causes such as weather or due to the construction. Not included in our scope is environmental or biological investigations of the site. Please note that this report is solely for design purposes.

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Attachments

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Exploration and Testing Procedures

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We had access to two samples from the bridge site in Bacadilla, Honduras. Both samples were taken from the clinic side of the river. We started our soil testing by pulverizing each sample to remove any large clumps. This step was needed for both the sieve analysis and the Atterberg limits tests. We then weighed out a decent portion of the soil. This allowed us to have enough for our tests but also enough left over to use in case something went wrong with the tests. These samples were classified using the Unified Soil Classification System (USCS) which can be seen in figure 6.

Sample 1

We classified soil sample 1 by doing a sieve analysis and calculating the percent passing each sieve. The data gathered from sample 1 is included in figure 1. Once we exhausted the percent passing information, we looked at the uniformity coefficient and the coefficient of curvature. We plotted the grain size distribution which is included in figure 2. Using this graph, we were able to calculate the coefficients we needed to classify the soil. The uniformity coefficient is 12 but the coefficient of curvature is 0.6. Using the USCS, sample 1 is a poorly graded sand.

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Mass of dry sample: (g)	655.88				
Sieve no.	**Diam. (mm)**	**Mass Retained**	**% Retained**	**% Retained Cumulative**	**% Passing**
3/8	9.5	93.8	14.3013966	14.3013966	85.6986034
4	4.75	54.24	8.269805452	22.57120205	77.42879795
8	2.36	70.88	10.80685491	33.37805696	66.62194304
16	1.18	93.16	14.20381777	47.58187473	52.41812527
30	0.6	82.81	12.6257852	60.20765994	39.79234006
50	0.3	113.24	17.26535342	77.47301336	22.52698664
100	0.15	81.95	12.49466366	89.96767701	10.03232299
200	0.075	48.45	7.387022016	97.35469903	2.64530097
Pan		16.25	2.477587364	99.83228639	

Figure 1: Soil Sample 1 Data

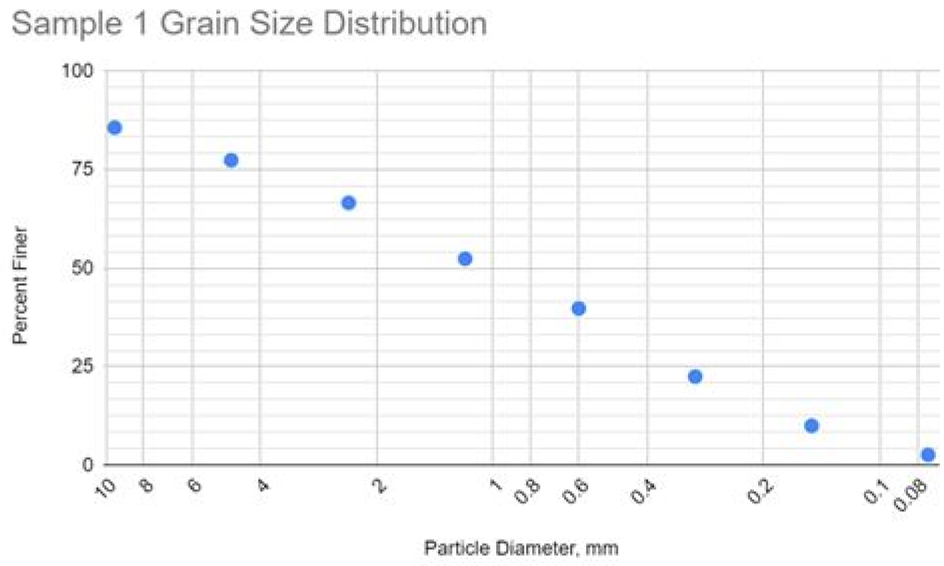

Figure 2: Soil Sample 1 Grain Size Distribution

Sample 2

We classified soil sample 2 by doing a sieve analysis and Atterberg limits test. Once we exhausted the percent passing information, we looked at the uniformity coefficient and the coefficient of curvature. Using figure 4, we were able to calculate the coefficients we needed to classify the soil. The uniformity coefficient is 10.67 which satisfies the requirements but, the coefficient of curvature is 0.91. This information helped us determine the sample is poorly graded. After following the USCS chart with the sieve analysis data, we used the Atterberg limits data to finish classifying the sample. Using the values calculated for liquid limit and plasticity index, we plotted the sample of the plasticity chart, shown in figure 8, and gathered that the sample is a poorly graded sand with silt.

Mass of dry sample:	370.5				
Sieve no.	**Diam. (mm)**	**Mass Retained (g)**	**% Retained**	**% Retained Cumulative**	**% Passing**
3/8	9.5	86.55	23.36032389	23.36032389	76.63967611
4	4.75	10.36	2.796221323	26.15654521	73.84345479
8	2.36	17.49	4.720647773	30.87719298	69.12280702
16	1.18	38.5	10.39136302	41.26855601	58.73144399
30	0.6	46.39	12.52091768	53.78947368	46.21052632
50	0.3	52.42	14.14844804	67.93792173	32.06207827
100	0.15	59.52	16.06477733	84.00269906	15.99730094
200	0.075	40.7	10.9851552	94.98785425	5.012145749
Pan		15.47	4.175438596	99.16329285	

Figure 3: Soil Sample 2 Data

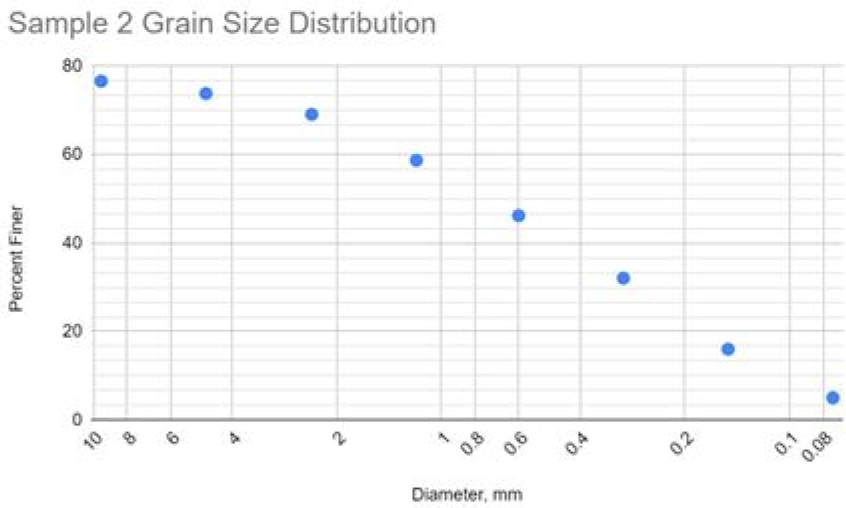

Figure 4: Soil Sample 2 Grain Size Distribution

Can no.	Mass of Wet Soil + Can (g)	Mass of Dry Soil + Can (g)	Mass of Can (g)	Mass of Dry Soil (g)	Mass of Moisture (g)	Water Content (%)	No. of Blows
55	43.59	40.49	28.31	12.18	3.1	25.45155993	21
79	36.33	34.58	28.36	6.22	1.75	28.13504823	26
71	39.05	36.85	28.19	8.66	2.2	25.40415704	31
Can no.	Mass of Wet Soil + Can (g)	Mass of Dry Soil + Can (g)	Mass of Can (g)	Mass of Dry Soil (g)	Mass of Moisture (g)	Water Content (%)	
56	38.06	36.16	28.18	7.98	1.9	23.80952381	
69	36.33	34.78	28.2	6.58	1.55	23.556231	
Liquid Limit:	26.25						
Plastic Limit:	23.68287741						
Plasticity Index:	2.567122594						

Figure 5: Sample 2 Atterberg Limit Test Data

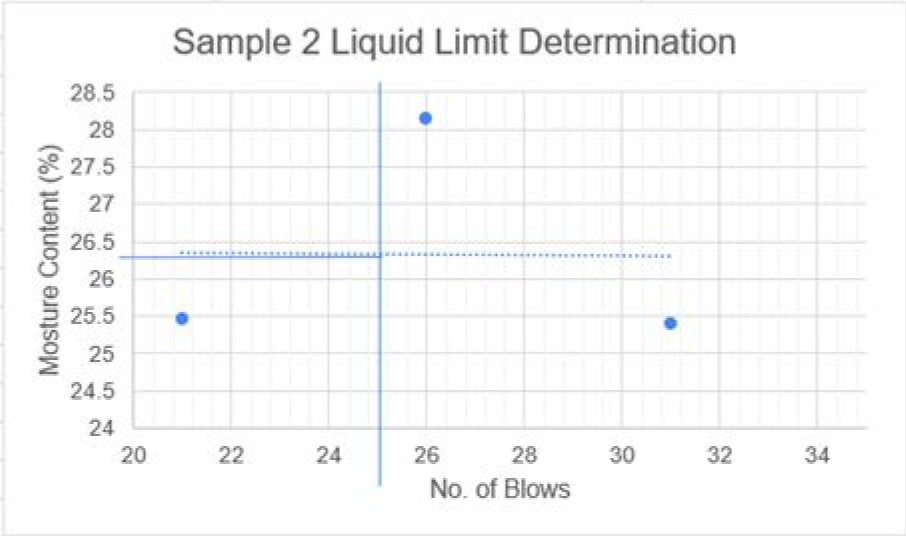

Figure 6: Soil Sample 2 Liquid Limit Determination

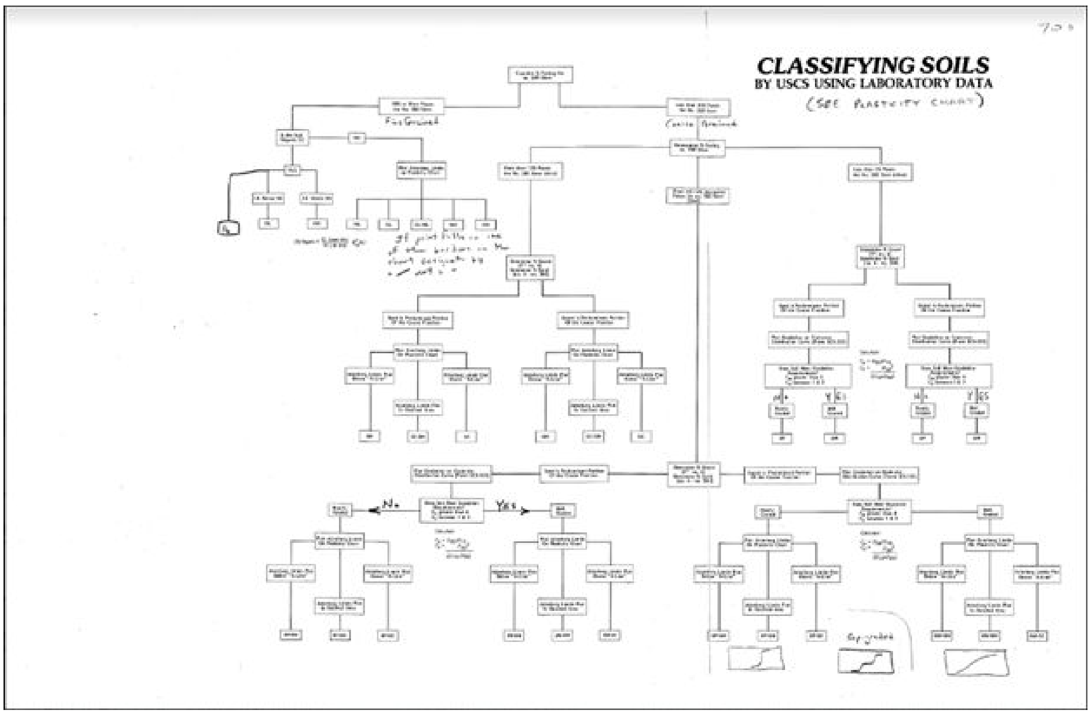

Figure 7: USCS Chart

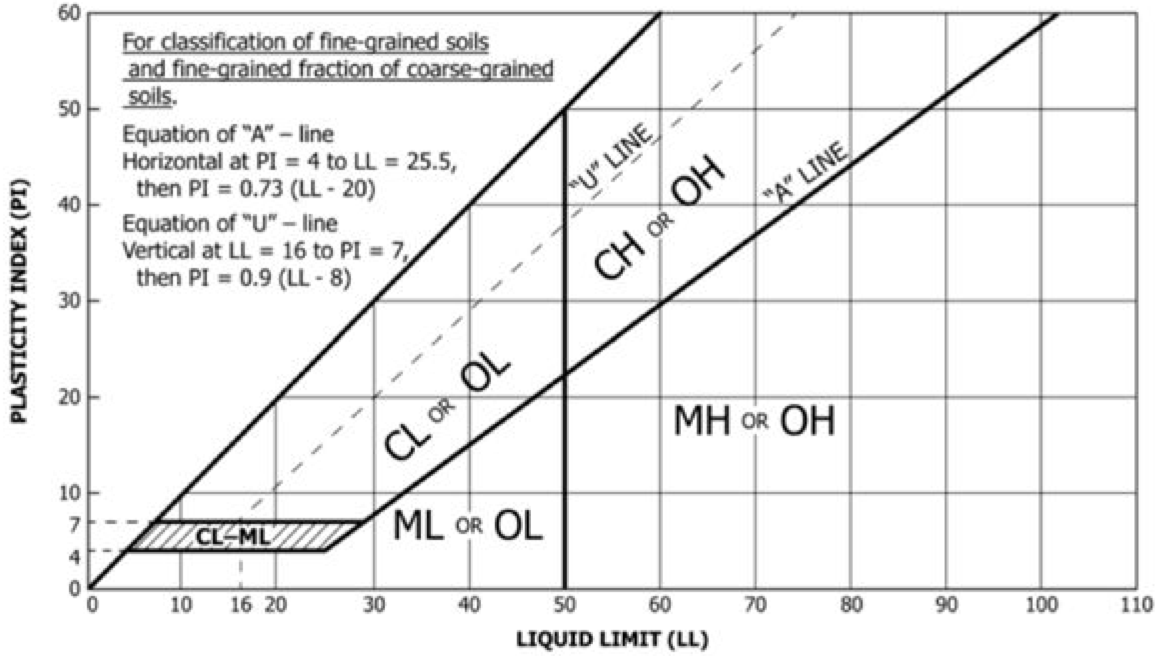

Figure 8: Casagrande Plasticity Chart (Source: https://www.nzgs.org/library/nzgs20_hind/)

Site Location and Exploration Plans

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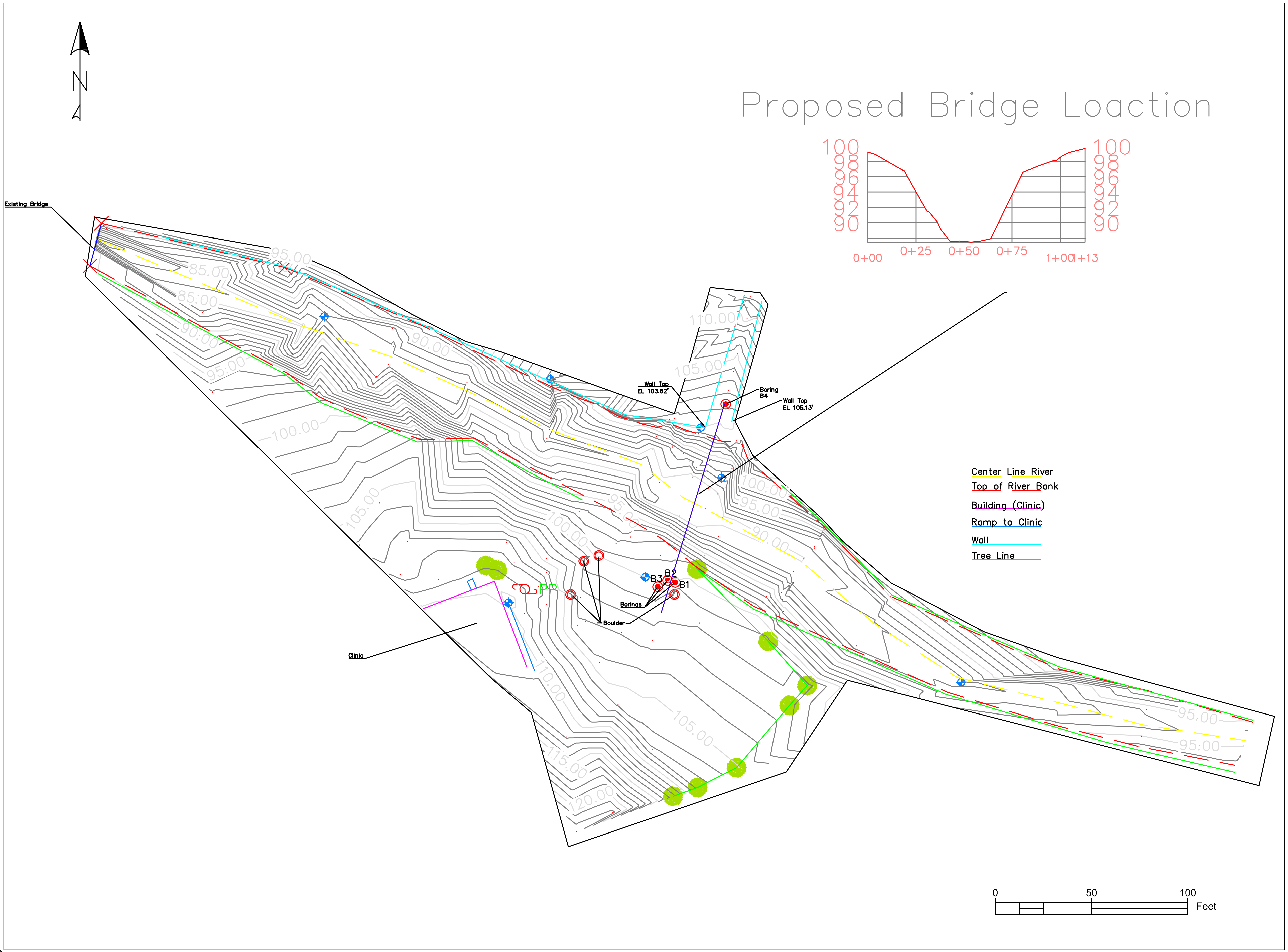

General Notes

Survey Team:
Trent Beacham, Noah Kimbrough,
Chris Schneider, Juan Carrera
Nathan Miller, Miranda Mangahas,
Caroline Janssen

No.	Revision/Issue	Date

Firm Name and Address
Joint Team
Lipscomb University
Vanderbilt University

Nashville, TN

Project Name and Address
Bacadilla Pedestrian Bridge
Bacadilla, Honduras

Project: Pedestrian Bridge
Date: 1/14/20
Scale: 1" = 30'
Sheet: 0001

PRODUCED BY AN AUTODESK STUDENT VERSION
PRODUCED BY AN AUTODESK STUDENT VERSION
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Exploration Results

LIPSCOMB UNIVERSITY | RAYMOND B. JONES COLLEGE OF ENGINEERING

SUBSURFACE LOG

Page 1 of 1

Project Number	101	Location	Bacadilla, Honduras
Project Name	Bacadilla Pedestrian Bridge	Boring No.	1 Total Depth SP @ 2'
Country	Honduras	Surface Elevation	100.23'
Project Type	Geotechnical Investigation	Date Started	11/23/19 Completed
Supervisor	Chris Gwaltney Driller Noah Kimbrough	Depth to Water	N/A Date/Time
Logged By	Trent Beacham	Depth to Water	N/A Date/Time

Lithology		Description	Sample #	Depth	Remarks
Elevation	Depth			Run	
100.23'	0'	Top of Ground	1	0'	
98.23'	2'	Poorly Graded Sand (SP)		1.75'	98.48'

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SUBSURFACE LOG

Page ____ of ___

Project Number	101	Location	Bacadilla, Honduras
Project Name	Bacadilla Pedestrian Bridge	Boring No.	2 — Total Depth **SP** @ 3' 1.5"
Country	Honduras	Surface Elevation	100.23'
Project Type	Geotechnical Investigation	Date Started	11/23/19 — Completed
Supervisor	Chris Gwaltney — Driller Noah Kimbrough	Depth to Water	N/A — Date/Time
Logged By	Trent Beacham	Depth to Water	N/A — Date/Time

Lithology		Description	Sample #	Depth	Remarks
Elevation	Depth			Run	
100.23'	0'	Top of Ground -	2		
98.23'	2'	Poorly Graded Sand (SP)		1.75'	98.48'
97.1'	3' 1.5"	Poorly Graded Sand (SP)		4'	
96.23'	4'	Refusal			

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SUBSURFACE LOG

Page ____ of ___

Project Number 101
Project Name Bacadilla Pedestrian Bridge
Country Honduras
Project Type Geotechnical Investigation
Supervisor Chris Gwaltney Driller Noah Kimbrough
Logged By Trent Beacham

Location Bacadilla, Honduras
Boring No. 3 Total Depth 1’10”
Surface Elevation 100.23’
Date Started 11/23/19 Completed
Depth to Water N/A Date/Time
Depth to Water N/A Date/Time

Lithology		Description	Sample #	Depth	Remarks
Elevation	Depth			Run	
100.23’	0’	Top of Ground	3	0’	
98.4’	1’10”	Soil		1’10”	

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SUBSURFACE LOG

Project Number	101	Location	Bacadilla, Honduras (Road Side)
Project Name	Bacadilla Pedestrian Bridge	Boring No.	4 Total Depth 1’
Country	Honduras	Surface Elevation	100.23’
Project Type	Geotechnical Investigation	Date Started	11/23/19 Completed
Supervisor	Chris Gwaltney Driller Noah Kimbrough	Depth to Water	N/A Date/Time
Logged By	Trent Beacham	Depth to Water	N/A Date/Time

Lithology		Description	Sample #	Depth	Remarks
Elevation	Depth			Run	
100.23’	0’	Top of Ground	4	0’	
99.23’	1’	Soil		1’	

Appendix E.1: Beam Choice

BEAM SELECTION CALCULATIONS

Loadings

$deck := 30 \; plf$

$railing := 40 \; plf$

$steel := 70 \; plf$

$Dead\_Loads := deck + railing + steel = 140 plf$

$pedestrians\_and\_motors := 255 \; plf$

$Live\_Loads\_strength := 255 \; plf$

$ped\_load := 25 \cdot \dfrac{200}{160 \cdot 1000} = 0.0312 \; klf$

$$Factored\_Load := \frac{(1.25 \cdot Dead\_Loads) + (1.75 \cdot Live\_Loads\_strength)}{1000 \cdot 2} = 0.3106 \; \frac{kip}{ft \cdot beam}$$

Assume lateral bracing at 1/3 points (20 ft)

Dimensions

$L\_b := 20 \; ft$

$L := 80 \; ft$

Design Moments

$$M\_u := \frac{Factored\_Load \cdot L^2}{8} = 248.5 \; kip \; ft$$

From Table 3-10 in AISC 15th Edition

$W24x68$

$\phi M\_n := 362 \; kip \; ft$

$W21x62$

$\phi M\_n := 282 \; kip \; ft$

Deflection Calculations

$W24x68$

$I\_xx := 1830 \; in^4$

$E := 29000 \; ksi$

$$\Delta\_LL := \left(\frac{5 \cdot ped\_load \cdot L^4 \cdot 12^3}{384 \cdot E \cdot I\_xx} \right) = 0.5427 \; in \qquad GOOD$$

$W21x62$

$I\_xx := 1330 \; in^4$

$$\Delta\_LL := \left(\frac{5 \cdot ped\_load \cdot L^4 \cdot 12^3}{384 \cdot E \cdot I\_xx} \right) = 0.7467 \; in \qquad GOOD$$

Choose W24x68 for greater moment capacity

SPLICE DESIGN CALCULATIONS

Loadings

$DC\_M := 100$ kip ft $DC\_V := 1.82$ kip

$LL\_M := 182.5$ kip ft $LL\_V := 3.32$ kip

Girder Properties

All Grade 50 Steel

$Flange\_t := .5625$ in

$Flange\_w := 12.91$ in

$Web\_t := 0.4375$ in

$Web\_d := 22.53$ in

Miscellaneous

$Stiffner\_spacing := 20$ ft

Splice Plates

All Plate Material Grade 50 Steel

$PFlange\_t := 0.5$ in

$IFlange\_w := 3$ in

$OFlange\_w := 8.97$ in

$Inner\_Ag := 3$ in^2

$Outer\_Ag := 4.485$ in^2

$PWeb\_t := 0.5$ in

Bolt Properties

A325

$Dia := 0.875$ in

All Threads Excluded

Surface Condition B

Standard Hole Size

$Top\_N := 2$

$Bottom\_N := 2$

$Web\_N := 2$

$Spacing := 3$ in

$Flange\_edge := 1.5$ in

$Flange\_end := 1.5$ in

$Web\_edge := 1.75$ in

$Web\_end := 1.75$ in

$Web\_ws := .75$ in

$Web\_wc := .5$ in

$Web\_gap := .5$ in

$Enter\_clr := 3$ in

Assumed Number of Bolts

$Top\_Flange := 6$

$Bottom\_Flange := 6$

$Web := 10$

I. Factored Loadings

$M\_u\_Positive := (1.25 \cdot DC\_M) + (1.75 \cdot LL\_M) = 444.375$ kip ft

$M\_u\_Negative := (0.9 \cdot DC\_M) = 90$ kip ft

$M\_service\_positive := (DC\_M) + (1.3 \cdot LL\_M) = 337.25$ kip ft

$M\_service\_negative := DC\_M = 100$ kip ft

$V\_service\_positive := DC\_V + (1.3 \cdot LL\_V) = 6.136$ kip

$V\_service\_negative := DC\_V$ kip

II. Factored Yield Resistance

Flange Outer Plates ---

$A\_e := 4.36$ in^2

$P\_Fy := 50 \cdot A\_e = 218$ kip

$Design\_Strength\_O := 0.6 \cdot P\_Fy = 130.8$ kip

$\varphi\_y := 0.95$

$A\_g\_O := 4.49$ in^2

$Plate\_Strength\_O := \varphi\_y \cdot A\_g\_O \cdot 50 = 213.275$ kip

GOOD

$A\_n := A\_e = 4.36$ in^2 $0.85 \cdot A\_g\_O = 3.8165$ in^2 $A\_n < A\_g\_O$ GOOD

Flange Inner Plates ---

$Design\_Strength\_I := 0.4 \cdot P\_Fy = 87.2$ kip

$A\_g\_I := 3.0$ in^2

$Plate\_Strength\_I := \varphi\_y \cdot A\_g\_I \cdot 50 = 142.5$ kip

GOOD

Web Plate ---

$F\_y := 50$ ksi

$A\_gross\_web := 9.86$ in^2

$V\_n := 0.58 \cdot F\_y \cdot A\_gross\_web = 285.94$ kip

$A\_vg := 19$ in^2

$Web\_Strength := A\_vg \cdot F\_y \cdot 0.58 = 551$ kip

GOOD

III. Net Section Fracture

Flange Outer Plates ---

$Design\_Strength\_O = 130.8$ kip

$\varphi\_u := 0.8$

$F\_u := 65$ ksi

$A\_control\_O := 3.55$ in^2

$O\_Flange\_Strength := \varphi\_u \cdot F\_u \cdot A\_control\_O = 184.6$ kip

GOOD

Flange Inner Plates ---

$Design\_Strength\_I = 87.2$ kip

$A\_control\_I := 2.06$ in^2

$I\_Flange\_Strength := \varphi\_u \cdot F\_u \cdot A\_control\_I = 107.12$ kip

GOOD

IV. Block Shear

Flange Plates ---

$Flange\_Plate\_DS := 130.8$ kip

$A\_vn := 5.16$ in^2

$A\_tn := 1.03$ in^2

$R\_r := ((0.58 \cdot F\_u \cdot A\_vn) + (F\_u \cdot A\_tn)) \cdot 0.8 = 209.1856$ kip

$R\_r > Flange\_Plate\_DS$

GOOD

Web Plate ---

$A\_vn\_w := 13.03$ in^2

$V\_r\_w := 285.85$ kip

$A\_tn\_w := 3.34$ in^2

$R\_r\_w := ((0.58 \cdot F\_u \cdot A\_vn\_w) + (F\_u \cdot A\_tn\_w)) \cdot 0.8 = 566.6648$ kip

$R\_r\_w > V\_r\_w$

GOOD

V. Slip Resistance

Flange Check ---

$Service\_Moment := 337.25$ kip ft

$Moment\_Arm := 23.09$ in

$bolt\_tension := 234$ kip

$Resistance\_Strength\_f := bolt\_tension \cdot \frac{Moment\_Arm}{12} = 450.255$ kip ft

$Resistance\_Strength\_f > Service\_Moment$

GOOD

Web Check ---

Positive Shear Check

$Bolts := 10$

$Service\_shear\_p := 6.14$ kip

$Shear\_per\_bolt := 39$ kip

$Total\_Strength := Bolts \cdot Shear\_per\_bolt = 390$ kip

$Total\_Strength > Service\_shear\_p$

GOOD

Negative Shear Check

$Service\_Shear\_n := 1.82$ kip

$Total\_Strength > Service\_Shear\_n$

GOOD

VI. Bearing Resistance

Outer Flanges ---

$End\_resistance := 64.35$ kip

$Interior\_resistance := 129.31$ kip

$Outer\_resistance := End\_resistance + Interior\_resistance = 193.66$ kip

$Outer\_Design\_Force := 130.57$ kip

$Outer\_resistance > Outer\_Design\_Force$

GOOD

Single Outer Plate

$end\_res := 72.39$ kip

$int\_res := 245.70$ kip

$single\_resistance := end\_res + int\_res = 318.09$ kip

$single\_design\_force := 217.91$ kip

$single\_resistance > single\_design\_force$

GOOD

Inner Plate

$end\_res\_i := 64.35$ kip

$int\_res\_i := 129.31$ kip

$inner\_resistance := end\_res\_i + int\_res\_i = 193.66$ kip

$inner\_design\_force := 87.34$ kip

$inner\_resistance > inner\_design\_force$

GOOD

Web

$end\_web\_res := 69.96$ kip

$interior\_web\_res := 279.83$ kip

$web\_resistance := end\_web\_res + interior\_web\_res = 349.79$ kip

$web\_design\_strength := 290.79$ kip

$web\_resistance > web\_design\_strength$

GOOD

Appendix E.3: Railing

RAILING STRENGTH DESIGN CALCULATIONS

$L := 8$ ft

Live Load on Post. Extra 0.015 Term for Chain Link

$P\_LL := 0.2 + (0.05 \cdot L) + \left(0.015 \cdot 8 \cdot \frac{3.5}{2}\right) = 0.81$ kip

$Bolt\_Strength := 39 \; \frac{kip}{bolt}$

Using 2 Bolts

Bolt Strength > Design Loading

GOOD

Induced moments from the Live Loading are negligable compared to both strength and the steel sectional strengths.

Appendix E.4: Foundation

$L := 72$ in $\quad C\_L := 60$ in

$B := 48$ in $\quad C\_B := 36$ in

$f'c := 3000$ psi

$qagross := \left(\frac{2000}{144}\right) \qquad qagross = 13.8889$ psi

$\gamma conc := \frac{150}{1728} \qquad \gamma conc = 0.0868$ pci

$T := 12$ in

$qanet := qagross - (\gamma conc \cdot T) \qquad qanet = 12.8472$ psi

$factor := 1.4$

$q\_u := qanet \cdot factor \qquad q\_u = 17.9861$

Column Bearing

$A\_1 := C\_B \cdot C\_L \qquad A\_1 = 2160 \text{ in}^2$

$A\_2 := B \cdot L$

$\phi\_c := 0.65$

bearing_on_footing

$\phi Bn\_1 := \phi\_c \cdot 0.85 \cdot f'c \cdot A\_1 \cdot \sqrt{\frac{A\_2}{A\_1}} \qquad \phi Bn\_1 = 4.5286 \cdot 10^6$ lb

$\phi Bn\_2 := \phi\_c \cdot 2 \cdot 0.85 \cdot f'c \cdot A\_1 \qquad \phi Bn\_2 = 7.1604 \cdot 10^6$ lb

bearing_on_column

$\phi Bn\_3 := \phi\_c \cdot 0.85 \cdot f'c \cdot A\_1 \qquad \phi Bn\_3 = 3.5802 \cdot 10^6$ lb

$\phi Bn\_4 := \phi\_c \cdot 2 \cdot 0.85 \cdot f'c \cdot A\_1 \qquad \phi Bn\_4 = 7.1604 \cdot 10^6$ lb

$B\_u := q\_u \cdot A\_2 \qquad B\_u = 62160$ lb

Confirm that B_u is <= ϕBn

One Way Shear

$\phi\_s := 0.75$

$\alpha\_s1 := \frac{q\_u}{4 \cdot \phi\_s \cdot \sqrt{f'c}} \qquad \alpha\_s1 = 0.1095$

$Lc\_s1 := L - C\_L \qquad Lc\_s1 = 12$ in

$Lc\_s2 := B - C\_B \qquad Lc\_s2 = 12$ in

$Lc\_s := 12$ Set Lc_s equal to the larger of Lc_s1 and Lc_s2

$d\_s1 := \frac{Lc\_s}{2 + \left(\frac{1}{\alpha\_s1}\right)} \qquad d\_s1 = 1.0776$ in

Two Way Shear

$d\_temp := 6$ in   Set equal to d_s1 or something larger

$\alpha\_s := 40$

$\beta := \frac{C\_L}{C\_B}$ $\beta = 1.6667$

$b\_0 := 2 \cdot (C\_B + C\_L + 2 \cdot d\_temp)$ $b\_0 = 216$

$v\_c1 := 4 \cdot \sqrt{f'c}$ $v\_c1 = 219.089$

$v\_c2 := \left(2 + \frac{4}{\beta}\right) \cdot \sqrt{f'c}$ $v\_c2 = 240.9979$

$v\_c3 := \left(2 + \alpha\_s \cdot \frac{d\_temp}{b\_0}\right) \cdot \sqrt{f'c}$ $v\_c3 = 170.4026$

$v\_c := v\_c3$ $v\_c = 170.4026$ Set equal to the least of the three v_c's

$\alpha\_s2 := \frac{q\_u}{\phi\_s \cdot v\_c}$ $\alpha\_s2 = 0.1407$

$a := \alpha\_s2 + 4$ $a = 4.1407$

$b := (\alpha\_s2 + 2) \cdot (C\_B + C\_L)$ $b = 205.5105$

$c := \alpha\_s2 \cdot ((C\_B \cdot C\_L) - (B \cdot L))$ $c = -182.3916$

$d\_s2 := \frac{\left(-b + \sqrt{b^2 - 4 \cdot a \cdot c}\right)}{2 \cdot a}$ $d\_s2 = 0.8722$ in

Flexure

$d\_temp2 := 6$ in   Set >= previous d's

longer side, L=72 in

$Lc\_f1 := \frac{(L - C\_L)}{2}$ $Lc\_f1 = 6$ in

$Mu\_1 := \frac{\left(\frac{q\_u \cdot B \cdot Lc\_f1^2}{2}\right)}{12000}$ $Mu\_1 = 1.295$ k ft

$As\_1 := \frac{Mu\_1}{4 \cdot d\_temp2}$ $As\_1 = 0.054$ in

Shorter Side, L=48 in

$Lc\_f2 := \frac{(B - C\_B)}{2}$ $Lc\_f2 = 6$ in

$Mu\_2 := \frac{\left(\frac{q\_u \cdot L \cdot Lc\_f2^2}{2}\right)}{12000}$ $Mu\_2 = 1.9425$ k ft

$As\_2 := \frac{Mu\_2}{4 \cdot d\_temp2}$ $As\_2 = 0.0809$ in

$d\_b := 0.75$ in   Set based on size needed to meet As

$T\_2 := 12$ in   Set T_2 >= to d+1.5(d_b)+3

$As\_min\_1 := 0.0018 \cdot L \cdot T\_2$ $\quad As\_min\_1 = 1.5552$ in $\quad As\_min\_2 := 0.0018 \cdot B \cdot T\_2$ $\quad As\_min\_2 = 1.0368$ in

Development_Length

$ld\_1 := 44 \cdot d\_b$ $\quad ld\_1 = 33$ in $\quad ld\_2 := 44 \cdot d\_b$ $\quad ld\_2 = 33$ in

Ensure that ld_1 <= (Lc_f1-3")

Ensure that ld_2 <= (Lc_f2-3")

Currently, our ld_1 and ld_2 are too big

$As\_1semi := As\_min\_1$ $\quad As\_1semi = 1.5552$ in

Set equal to the larger of As_min_1 and As_1

$As\_2final := As\_min\_2$ $\quad As\_2final = 1.0368$ in

Set equal to the larger of As_min_2 and As_2

Distribution_of_Short_Bars

$\beta\_2 := \frac{L}{B}$ $\quad \beta\_2 = 1.5$

$\gamma s := \frac{2}{(\beta\_2 + 1)}$ $\quad \gamma s = 0.8$

$As\_short := \beta\_2 \cdot \gamma s \cdot As\_1semi$ $\quad As\_short = 1.8662$

$As\_1final := As\_short$ $\quad As\_1final = 1.8662$ in

Appendix E.5: Thermal Expansion

THERMAL DESIGN CALCULATIONS

Per AASHTO 3.12.3.3-1

Procedure A:

$\alpha := 6.6 \cdot 10^{-6}$ in /in / degrees F
$L := 80 \cdot 12 = 960$ in
$T\_max := 120$ degrees F
$T\_min := 0$ degrees F

(World Record Temperatures)

$\Delta\_T := \alpha \cdot L \cdot (T\_max - T\_min) = 0.7603$ in

Procedure B:

$T\_min := 45$ degrees F
$T\_max := 103$ degrees F

(June Climate History for Catacamas)

$\Delta\_T := \alpha \cdot L \cdot (T\_max - T\_min) = 0.3675$ in

Oversized / Slotted Hole
Table J3.3
small slot -- 0.25 inches
long slot -- 0.75 inches

No matter the procedure (A or B), we use a long slot.

Appendix E.6: Beam-to-Abutment Connections

Tensile and Shear Strength of Bolts and Threaded Parts

$\phi R\_n1s := \phi\_1 \cdot F\_ns \cdot A\_b$

$\phi\_1 := 0.75$

$F\_ns := 27$ ksi

$d\_1 := 1$ in

$A\_b := \frac{\pi \cdot d\_1^2}{4}$ in^2

$\phi R\_n1s = 15.9043$ kip

$\phi R\_n1t := \phi\_1 \cdot F\_nt \cdot A\_b$

$F\_nt := 45$ ksi

$\phi R\_n1t = 26.5072$ kip

Minimum Bolt Strength

$\phi R\_n := 0.25 \cdot d\_l$

$d\_l := 33$ kip

$\phi R\_n = 8.25$ kip

High Strength Bolts in Slip-Critical Connections

$\phi R\_n2 := \phi\_2 \cdot \mu \cdot D\_u \cdot h\_f \cdot T\_b \cdot n\_s$

$\phi\_2 := 0.70$

$\mu := 0.50$

$D\_u := 1.13$

$h\_f := 1.0$

$T\_b := 51$ kip

$n\_s := 1$

$\phi R\_n2 = 20.1705$ kip

Bearing Strength at Bolt Holes

$\phi R\_n3 := \phi\_3 \cdot 2.4 \cdot d \cdot t \cdot F\_u$

$\phi\_3 := 0.75$

$d := 1.0$ in

$t := 0.585$ in

$F\_u := 36$ ksi

$\phi R\_n3 = 37.908$ kip

Tearout Strength at Bolt Holes

$\phi R\_n4 := \phi\_4 \cdot 1.2 \cdot l\_c \cdot t \cdot F\_u$

$\phi\_4 := 0.75$

$l\_c := 2.4375$ in

$t := 0.585$ in

$F\_u := 36$ ksi

$\phi R\_n4 = 46.2004$ kip

FALSEWORK SUPPORT DESIGN CALCULATIONS

$Dead\_Load := 70 \; \frac{lb}{ft \cdot beam}$

$Construction\_Load := 50 \; \frac{lb}{ft^2}$

$DL\_safety\_factor := 1.4$

$bridge\_width := 3.3 \; ft$

$tributary\_width := 27 \; ft$

Calculations

$factored\_DL := Dead\_Load \cdot 2 \cdot DL\_safety\_factor = 196 \; \frac{lb}{ft}$

$total\_construction\_load := Construction\_Load \cdot bridge\_width = 165 \; \frac{lb}{ft}$

$Support\_Load := \frac{tributary\_width}{1000} \cdot (factored\_DL + total\_construction\_load) = 9.747 \; kip$

Use 9.75 kip per falsework support

Appendix F: Community Assessment

BACKGROUND

The community of Bacadillas, Honduras has requested assistance in the form of a bridge built to connect the community (along with 11 other nearby communities) to a medical clinic across a river. The clinic serves 3,165 people in 12 districts, taking care of medication, check-ups, prenatal care, and more. The design team, along with designing said bridge, also wants to ensure that a bridge is the right solution for the community, and that the community is able take care of the bridge after it is built. As such, the team performed a community assessment to consolidate community information gathered on-site and discuss what the right solution is for the community.

SITE VISIT

In order to assess the community, one must interact with the community. Three members of the design team travelled to Bacadillas, Honduras in November 2019 with students from Lipscomb University. While there, they performed tests and collected measurements to design the bridge. Significant time, however, was also dedicated to connecting with residents and leaders, to understand the capabilities and dynamic of the community. This was accomplished through casual interactions while working on data collection, as well as several meetings with various groups.

Welcome Reception

The design team was kindly welcomed upon arrival to the community. Children from the kindergarten school in the community joined in the reception, dancing to music and creating their own music, visible in Figure 1. Louisa, the head nurse at the clinic who owns the clinic land and has built a new wooden bridge every year after it is washed away, lead the reception. She shared information about the clinic, the districts it serves, and introduced different community leaders.

Figure 1: A warm welcome

This reception also emphasized the importance of having a strong foundation of culture, as emphasized in the Bridges to Prosperity Bridge Builder Manual. Visitors must be aware of local culture, language, sociopolitical history, and economics (*Bridges to Prosperity*). A member of the design team joined in the school children who were dancing during the reception. The team member was not aware that in the religious culture observed by the community pastor, however, it is not considered proper for adults to dance in such circumstances.

Community Leaders

The second meeting the team got to take part in was with the community leaders, including the pastor, two of the clinic's health promoters, the president of the committee of neighbors, a nurse from the clinic, and Louisa. This meeting allowed for a dive into the inner workings of the clinic. The leaders shared that the clinic serves around 30 patients per day, mostly seniors for diabetes and hypertension, young children, and pregnant women. The furthest district is 28 kilometers away, and yet people from that district still reliably attend their appointments. This is ensured by neighbors who lend car rides and the health promoters who make house visits to check on the medical status of all community residents.

After the meeting, the design team got a tour of the clinic itself. The clinic is well-outfitted to handle most minor medical events, as well as reliably provide medication. Any major medical event or surgery, however, is taken into Catacamas, the city about 30 minutes away from the clinic by car. On the walls of the clinic, the staff has a map of all 12 communities and the houses within each, as seen in Figure 2. There are pins in each house to indicate the medical conditions of any occupants. Detailed information such as this map show that the clinic is serving its constituents well.

Figure 2: A map displaying the medical conditions of residents in all 12 districts

Neighbors
Next, the team met with those who live nearby the proposed build site. Several neighbors offered to store materials in their house during construction. This could help preserve the materials and minimize environmental impact of construction. The group of neighbors also told of a bridge they built in the past across the river to connect the community to a soccer field. The group then walked around and pointed out several high water marks from past flood events, to flesh out the hydrologic analysis. The two neighbors living directly adjacent to the bridge site were not able to make the meeting.

Community at Large
Each meeting was structured to gather information on the community and its structure while also spending time asking about the desires and needs of the community. The large community meeting allowed this on the biggest scale. The design team got to see the united nature of the community, as many residents showed up and voiced their opinions, as seen in Figure 3. When asked about their bond, residents said that they are "very united" and that it has "always been this way."

The team also learned the importance of inquiring into the community's desires and needs while also not offering the world. When asked if they wanted a roof to the bridge, they said "yes!". When asked if they wanted motorcycles to be able to traverse the bridge, they said "yes!". When asked how wide they wanted the bridge, they said "2 meters!". They were incredibly engaged, which is something to be very grateful for, but they naturally wanted the bridge to be as amazing as it could possibly be.

Figure 3: A community member speaking up during the large community meeting

Casual Interactions
Outside of the structured meetings, the design team also had many casual interactions with the community. The temporary bridge installed by Louisa had worn out for the year, so residents were navigating the riverbed to cross the river. The sides were often steep and unstable, visible in Figure 4, and older residents were helped across by other members of the community. It was already precarious during the dry season, putting into perspective how dangerous it could become during the wet season.

Team members got to talk with residents who would come watch the surveying, getting to know several of the families of the community. At the end of the trip, the team played a soccer game with residents, seeing firsthand how the community comes together and spends time amongst one another.

Figure 4: Community members crossing the river

DISCUSSION

After getting to know the community on the site visit, the design team took those interactions and applied them to three questions concerning the bridge and its design.

Is this solution right for the community?

If the clinic was not serving its constituents well, or if residents were not attending appointments, a bridge would not suddenly create the perfect circumstances for the community. On the contrary however, the clinic is quite well run, particularly since Predisan partnered with the clinic several years ago. Patients regularly attend their appointments and take part in social groups to connect with others who have similar medical conditions. The health promoters ensure that members from far away districts have the transportation needed to get to the clinic. Communication between the clinic and patients is strong, with the wall map painting a detailed picture of the health of all patients. The very last problem is that, once residents reach the clinic, they simply cannot cross the river safely in order to attend their appointment. As such, a bridge is the right solution for the community.

Is the community invested?

The bridge will not be successful unless the community has a stake in the project. Our many meetings set out to understand the bond of the community and its ability to take on such a task. Sure enough, the community shared their desire and preparedness. They want the bridge, meaning that it is not an attempt by some outside entity to throw in a solution that was not requested. The community members are able to help with the bridge, shown in their work on a similar bridge over to a soccer field in the past. Finally, there must be accountability for upkeep of the bridge in the future. Not only was the community willing to provide this upkeep, but they put immense trust in their community leaders (such as Louisa and the president of the committee of neighbors), who will lead the charge.

How will the design change to adapt to community needs?

Finally, the team had to be sure to make sure their design fully fit the community's needs. The original design plan was a suspended or suspension bridge, as they are cheaper and easy to build given the site conditions and unknown construction timeline (it is difficult to build shoring during the wet season in a fast-flowing river). However, there was potential for the tiebacks of the cables to infringe upon the private property of the two families living directly adjacent to the bridge. As the team was not able to speak to these families while in Bacadillas, the team contacted Predisan asking if it was okay. Predisan asked the team to move away from a suspension design. They did not want to infringe on private property, and they were also concerned able acquiring the high-quality cables necessary for such a bridge. The design team eventually settled on a stringer bridge, which best considered the needs of the community, as well as constructability.

CLOSING

To ensure an effective bridge, the design team completed a community assessment. This allowed the team to take the community's desires and needs into consideration at every step. As a result of the assessment, the bridge is deemed an effective solution, one that the community has a stake in.

Appendix G: Environmental Assessment

This environmental impact assessment is an overview of carbon costs associated with this project and the project's influence on the surrounding ecosystem. Our goal was to create a design that would have the minimum carbon output and have little local environmental impact. Based on data from the World Steel Association, there is approximately 1.85-tonnes of CO2 emitted for every tonnes of steel produced (*Position Paper on Climate Change*). However, recycling scrap steel there is an approximate 0.464-tonnes of CO2 emissions per tonne of steel produced. ("The Carbon Footprint of Steel") The design consists of two 80-feet W24x68, resulting in 10,880-pounds of steel. There's approximately 12-tonnes of steel within the design, this includes all structural components, plates, and bolts. The steel used in the stringer design results in approximately 22.2-tonnes of carbon. This bridge design is more conservative with respect to total steel usage seen by the usage of wood railings rather than fully steel railings. Steel is also one of the most recycled materials in the world (*Steel is the World's Most Recycle Material*). Using the recycled steel scrap CO2 conversion rate, the carbon emissions is approximately 5.57-tonnes. Using recycled material and minimizing total steel will reduce the carbon impact.

Other design considerations taken to minimize carbon emissions is the usage of as many in-country and local resources. This not only minimizes transportation emissions, but also supports the local economy. During the site visit in November, local hardware shops were visited to determine available resources. Minimizing the haul distances reduces gas carbon emissions and other international transport impacts.

This bridge is constructed across a river in rural Honduras in Bacadilas. The impacts on the surrounding ecosystem is an important consideration for bridge location, design, and construction methods. The Hondurand Emerald Hummingbird is the only recorded endangered species in the region. This species preference for arid climates indicates the lack of likelihood that the species will be near the bridge location ("Endangered and Threatened Wildlife and Plants). Site access remains a critical factor in determining the feasibility of the project. Due to the limited space on the roadside of the bridge location, it is necessary to have a bridge design that minimizes impact to the surrounding properties.

A major consideration for construction is to manage erosion, water runoff, and sedimentation. Some construction methods to reduce impacts on the environment are to utilize perimeter control barriers if applicable, minimize the total disturbed area, and apply erosion controls.It is already planned to store and stockpile materials in local community member's houses. This may reduce potential water pollution into the river. All of these aspects will minimize the direct impact on the ecosystem during the bridge's construction process.

Appendix H: Cost Assessment

Categorization					Description	Quantity	Unit	Total Unit Quantity	Cost/Unit	Total Cost ($)	Sources
Superstructure	Girders				W24x68 steel beams, Grade 50, 28' in length, 80' span length	6	1 lb	11,424 lbs	$1.25/lb	14,280.00	
	Splices	Bolts	Top Flange		A325 bolts for splice connections	176	1 bolt	176 bolts	$4.00/bolt	704.00	*(Buy A325 & A490)*
		Plates	Top Flange	Inner	Grade 50 steel plates, 4.25"x9"x0.5", used for both splice connections, 0.283 lb/cubic-in	8	1 lb	43.30 lbs	$1.25/lb	54.13	
				Outer	Grade 50 steel plates, 9"x18.5"x0.5", used for both splice connections, 0.283 lb/cubic-in	4	1 lb	94.24 lbs	$1.25/lb	117.80	
			Bottom Flange	Inner	Grade 50 steel plates, 4.25"x9"x0.5", used for both splice connections, 0.283 lb/cubic-in	8	1 lb	43.30 lbs	$1.25/lb	54.13	
				Outer	Grade 50 steel plates, 9"x18.5"x0.5", used for both splice connections, 0.283 lb/cubic-in	4	1 lb	94.24 lbs	$1.25/lb	117.80	
			Web		Grade 50 steel plates, 15.5"x14.5"x0.5", used for both splice connections, 0.283 lb/cubic-in	8	1 lb	254.42 lbs	$1.25/lb	318.02	
	Cross-Framing	Members	Cross-Frames		L4x4x3/8 steel angles, 1.5' length, 9.8 lb/ft	8	1 lb	117.6 lbs	$1.25/lb	147.00	*(ASTM A36 Structural Steel)*
			Diaphragm		C12x20.7 steel C-shapes, 33" length	3	1 lb	170.78 lbs	$1.25/lb	213.48	*(Structural A36 Steel Channel)*
		Plates	Beam Connection		4"x4"x1/2" steel plates to connect cross-frames to beams, 0.283 lb/cubic-in	8	1 lb	18.11 lbs	$1.25/lb	22.64	
			Cross-Frame Connection		12"x12"x0.5" steel plates to connect cross-frames to eachother, 0.283 lb/cubic-in	2	1 lb	40.75 lbs	$1.25/lb	50.94	
		Bolts			A325 bolts connecting cross-framing members to steel beams and plates	44	1 bolt	44 bolts	$4.00/bolt	176.00	*(Buy A325 & A490)*
		Stiffeners			20"x4.25"x0.5" steel stiffeners, 0.283 lb/cubic-in	6	1 lb	72.17 lbs	$1.25/lb	90.21	
	Railing	Posts			4x4x69" wooden posts oriented vertically, locally sourced materials, located along each side of the bridge for its entire span including the ramps, spaced every 8' along the bridge, 127' in total	22	1 post	22 posts	$7.50/post	165.00	*(Lumber & Composites)*
		Longitudinal Members			2x4" wooden boards, used for hand railing and toe boards	320	1 ft	320 ft	$0.75/ft	240.00	*(Lumber & Composites)*
		Wood Screws			Used for railing and decking connections	2,300	1 screw	2,300 screws	$7.00/100 screws	161.00	*(Wood Screws)*
		Chainlink Fence			Chainlink fence connected to the posts and longitudinal members on each side of the bridge and for the bridge's entire length including approach ramps	640	1 square-ft	640 square-ft	$0.375/square-ft	240.00	*(Chain-Link Fence)*
		C-shape			C7x9.8 C-shapes connecting railing posts to beam webs, 9" length	22	1 lb	161.7 lbs	$1.25/lb	202.13	*(Structural A36 Steel Channel)*
		Bolts			A325 bolts for decking and railing connections	88	1 bolt	88 bolts	$4.00/bolt	352.00	*(Buy A325 & A490)*
		Brackets			L3x3x1/2 brackets connecting posts to composite deck panels, 3" length, 9.4 lb/ft	44	1 lb	103.4 lbs	$1.25/lb	129.25	*(ASTM A36 Structural Steel)*
		Stiffeners			20"x4.25"x0.5" steel stiffeners, 0.283 lb/cubic-in	22	1 lb	264.61 lbs	$1.25/lb	330.76	
	Decking	Panels			Composite decking panels spanning the entirety of the bridge's length excluding the ramps	280	1 square-ft	280 square-ft	$20.00/square-ft	5,600.00	*(FRP Profiles & Products)*
		Counter Sunk Bolts			Counter sunk bolts connect nailer boards to beams	42	1 bolt	42 bolts	$4.00/bolt	168.00	*(Buy A325 & A490)*
		Nailer Board			Nailer boards placed between composite decking and steel beams	194	1 square-ft	194 square-ft	$0.60/square-ft	116.40	*(Lowe's)*
		Wood Screws			Included in railing calculations	See Above					*(Wood Screws)*
	Ramps	Walls			Comprised of stone and masonry, 8" masonry blocks	650	8" block	650 blocks	$1.00/block	650.00	
		Fill			60 cubic yards of ramp fill consisting of locally sourced materials, starting with large stones followed by small stones and finally gravel and sand	60	1 cubic-yd	60 cubic-yds	$10.00/cubic-yd	600.00	*(2020 Topsoil)*
		Approach Slabs			79 cubic feet concrete per slab, will be placed on top of ramp walls	2	1 cubic-ft	158 cubic-ft	$4.82/cubic-ft	761.56	

Categorization				Description	Quantity	Unit	Total Unit Quantity	Cost/Unit	Total Cost ($)	Sources
Substructure	Primary Foundations	Bearing Plates	North End	21"x18"x0.5" steel bearing plates, 0.283 lb/cubic-in	2	1 lb	106.97 lbs	$1.25/lb	133.72	
			South End	21"x17"x0.5" steel bearing plates, 0.283 lb/cubic-in	2	1 lb	101.03 lbs	$1.25/lb	126.29	
		Neoprene Pads	North End	21"x18"x0.5" neoprene pads	2	1 cubic-in	378 cubic-in	$1.00/cubic-in	378.00	*(AASHTO Rubber)*
			South End	21"x17"x0.5" neoprene pads	2	1 cubic-in	357 cubic-in	$1.00/cubic-in	357.00	*(AASHTO Rubber)*
		Anchor Bolts		1" diameter, 18" length anchor bolts	8	1 bolt	8 bolts	$20.00/bolt	160.00	*(1"x18" w/ 4" Thread)*
		Concrete	North End	115.5 cubic-ft concrete for footings, piers, and end walls	1	1 cubic-ft	115.5 cubic-ft	$4.82/cubic-ft	556.71	
			South End	123.0 cubic-ft concrete for footings, piers, and end walls	1	1 cubic-ft	123.0 cubic-ft	$4.82/cubic-ft	592.86	
		Rebar	North End	217.8' of #6 rebar for footings, piers, and end walls, 1.502 lb/ft	1	1 lb	327.14 lbs	$1.00/lb	327.14	
			South End	239.3' of #6 rebar for footings, piers, and end walls, 1.502 lb/ft	1	1 lb	359.43 lbs	$1.00/lb	359.43	
	Ramp Foundations	Rebar		134' of #4 rebar, 0.668 lb/ft	1	1 lb	89.51 lbs	$1.00/lb	89.51	
		Concrete		200 cubic-ft concrete	1	1 cubic-ft	200 cubic-ft	$4.82/cubic-ft	964.00	
	Excavation			Excavation for substructure	1	1 cubic-ft	598 cubic-ft	$1.11/cubic-ft	663.78	
SUMMARY				**Overall**					30,770.69	
				Structural Steel			13,110.62 lbs	$1.25/lb	16,388.30	
				Decking Panels			280 square-ft	$20.00/square-ft	5,600.00	
				Concrete			596.5 cubic-ft	$4.82/cubic-ft	2,875.13	
				Bolts and Screws					1,721.00	
				Rebar			776.08 lbs	$1.00/lb	776.08	
				Masonry			650 blocks	$1.00/block	650.00	
				Superstructure					26,062.25	
				Girders					14,280.00	
				Splices					1,365.88	
				Cross-Framing					700.27	
				Railing					1,820.14	
				Decking					5,884.40	
				Ramps					2,011.56	
				Substructure					4,708.44	
				Primary Foundations					2,991.15	
				Ramp Foundations					1,053.51	
				Excavation				$1.11/cubic-ft	663.78	

Appendix I: Design Documents

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LAS BACADILLAS STRINGER BRIDGE

GPS COORDINATES	14.795767,-85.709567
COUNTRY	HONDURAS
STATE	OLANCHO
MUNICIPALITY	BACADILLAS
SPAN	80.0 FEET

DESIGN DATA:

DESIGN LOADS
DEAD LOAD = 140 plf
LIVE LOAD = 255 plf
WIND LOAD = 0 plf

GENERAL NOTES:

1. BRIDGE CONSTRCTION SHALL BE IN ACCORDANCE WITH THE BRIDGES TO PROSPERITY BRIDGE BUILDER MANUAL, 5TH EDITION
2. DRAWINGS NOT TO SCALE. REFER TO DIMENSIONS GIVEN.

SHEET INDEX:

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LAS BACADILLAS STRINGER BRIDGE TITLE SHEET

01

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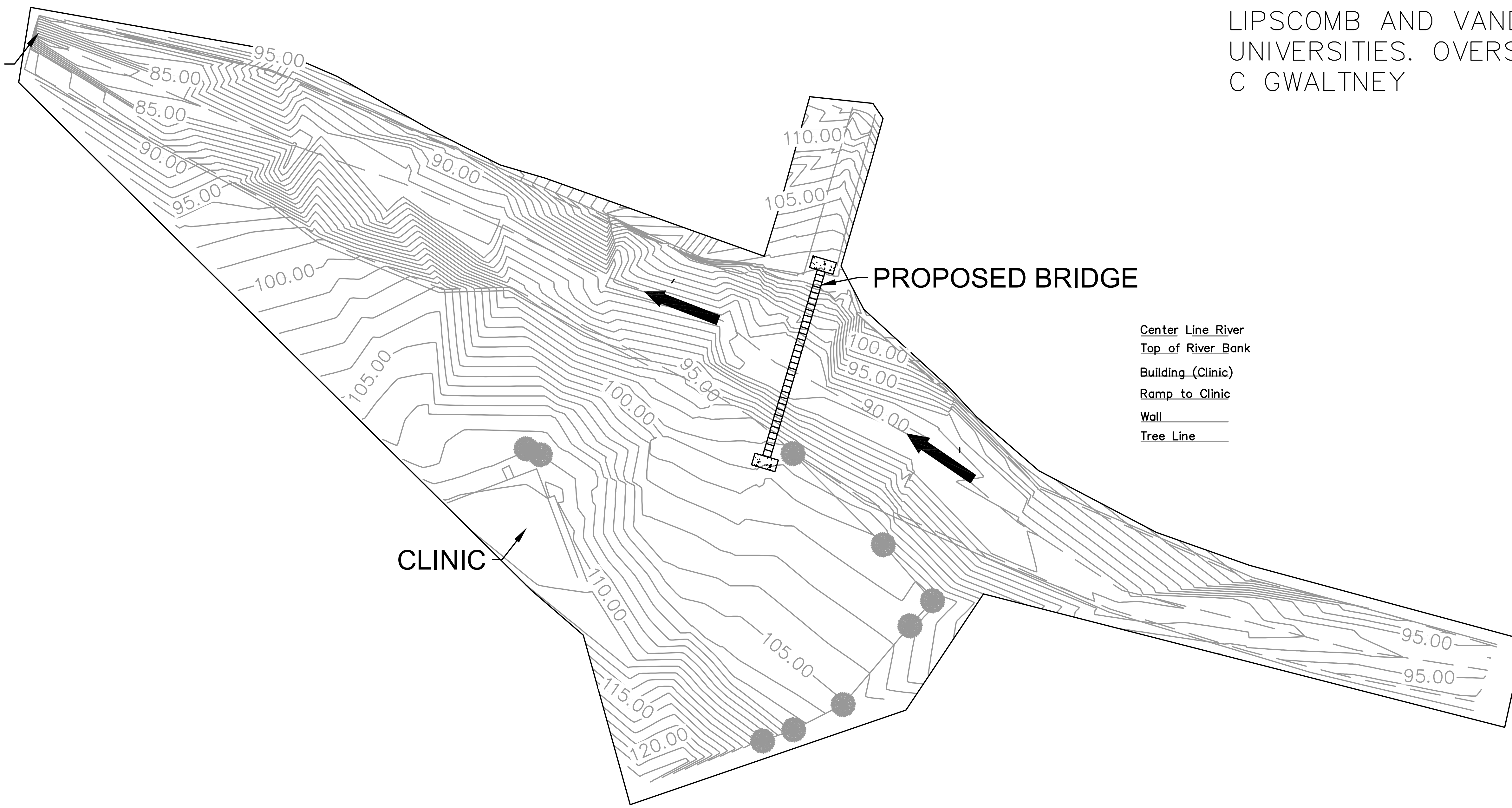
NOTES:

1. LIVE LOAD DEFLECTION: 0.543 INCHES
2. SURVEY INFORMATION PROVIDED BY STUDENT TEAM FROM LIPSCOMB AND VANDERBILT UNIVERSITIES. OVERSEEN BY DR. C GWALTNEY



Center Line River
Top of River Bank
Building (Clinic)
Ramp to Clinic
Wall
Tree Line

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LAS BACADILLAS STRINGER BRIDGE LAYOUT

02

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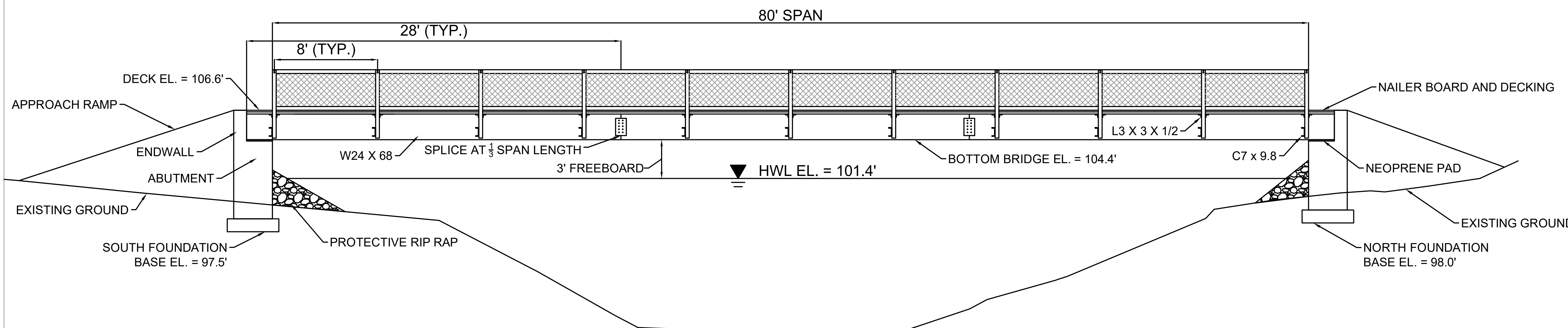

ELEVATION VIEW

VANDERBILT UNIVERSITY

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LAS BACADILLAS
STRINGER BRIDGE
ELEVATION VIEW

03

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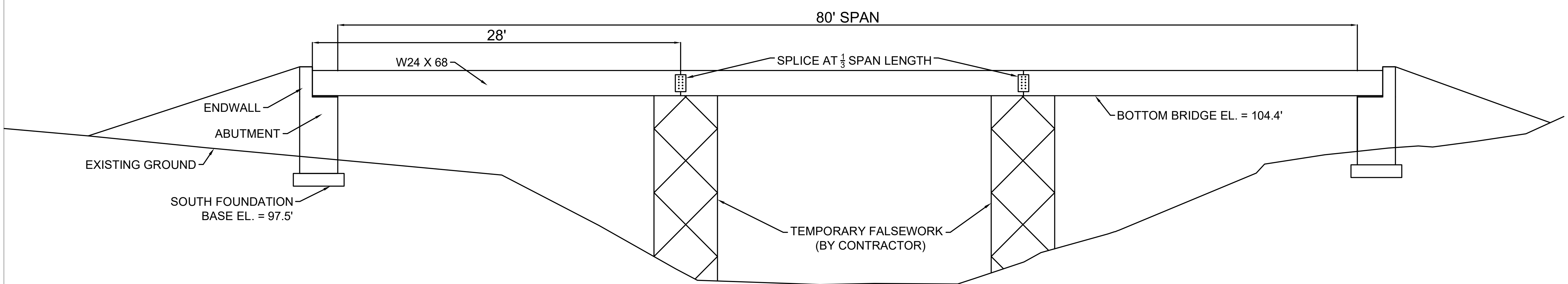
NOTES:

1. FALSEWORK SHALL SUPPORT A FACTORED LOAD OF 9.75 KIPS AT EACH LOCATION



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LAS BACADILLAS
STRINGER BRIDGE
CONSTRUCTION DETAILS

04

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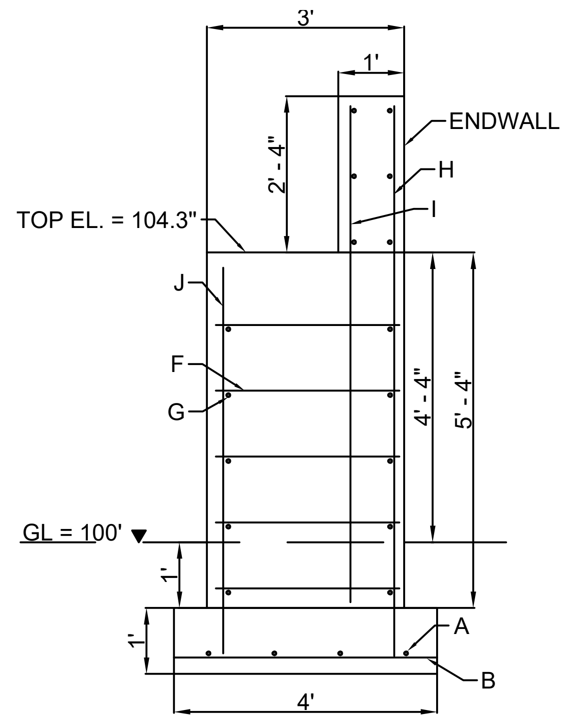
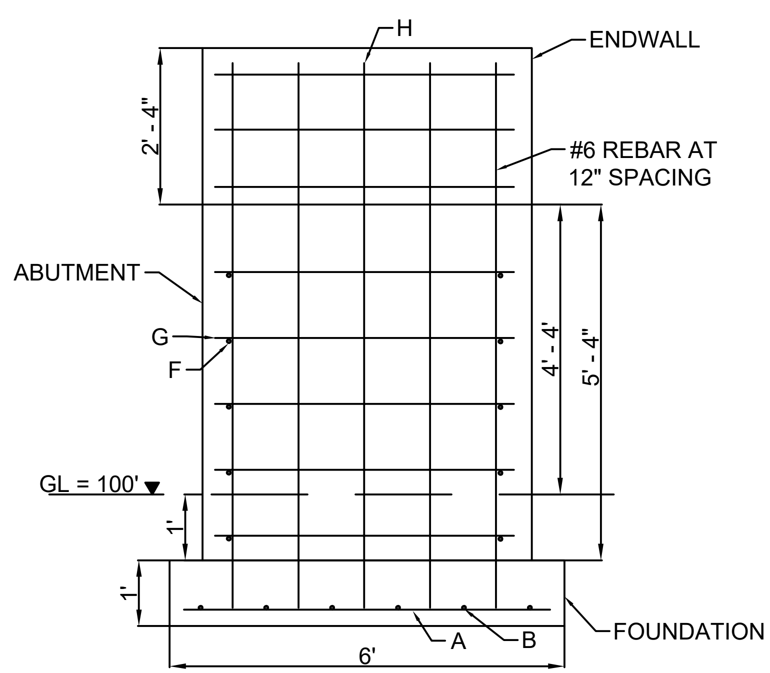
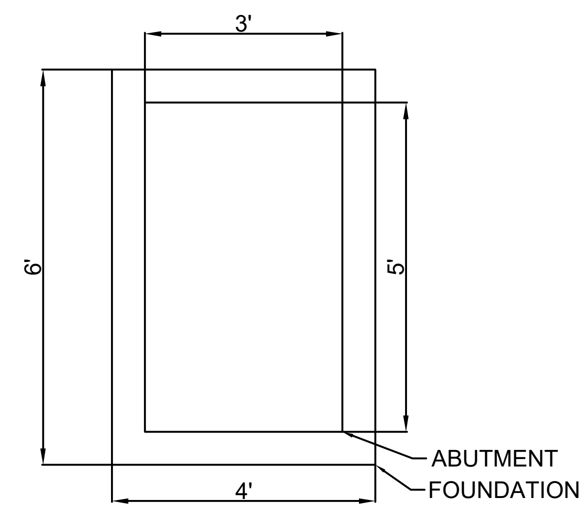


ELEVATION:
NORTH FOUNDATION,
ABUTMENT, ENDWALL

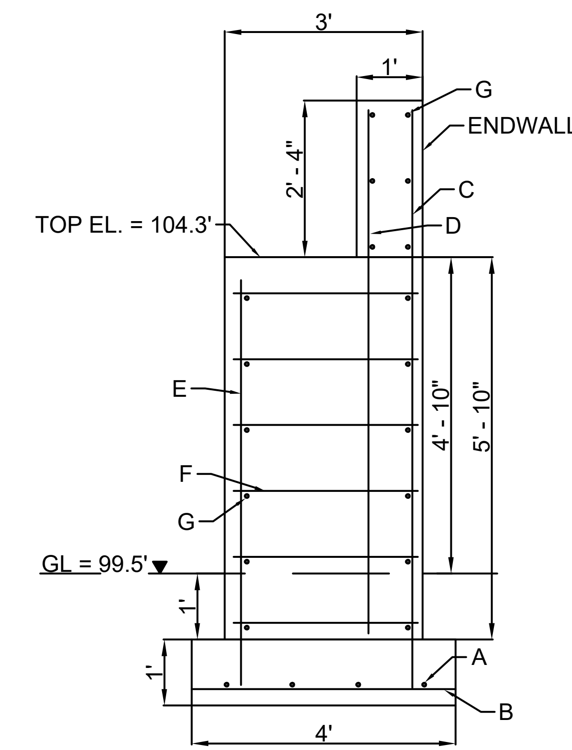


SECTION:
NORTH FOUNDATION,
ABUTMENT, ENDWALL

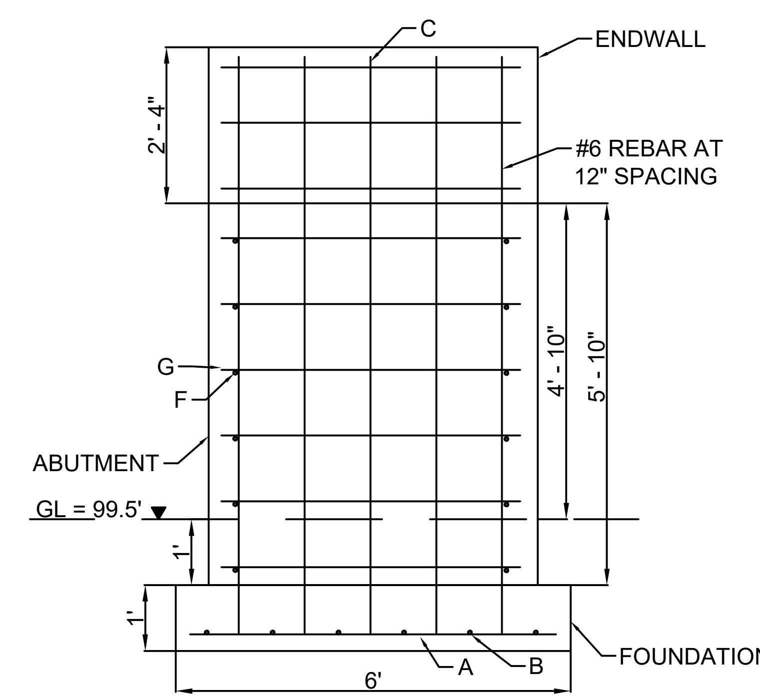
BILL OF MATERIALS				
NAME	BAR SIZE	LENGTH	QUANTITY	TOTAL LENGTH
A	#6	66"	8	528"
B	#6	42"	12	504"
C	#6	104"	5	520"
D	#6	92"	5	460"
E	#6	76"	5	380"
F	#6	30"	14	420"
G	#6	54"	26	1404"
H	#6	98"	5	490"
I	#6	86"	5	430"
J	#6	70"	5	350"
			TOTAL	458 FT
CONCRETE				239 CF



PLAN
(REBAR NOT SHOWN)



SOUTH FOUNDATION,
ABUTMENT, ENDWALL



SOUTH FOUNDATION,
ABUTMENT, ENDWALL

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LAS BACADILLAS
STRINGER BRIDGE
FOUNDATION DETAILS

F01

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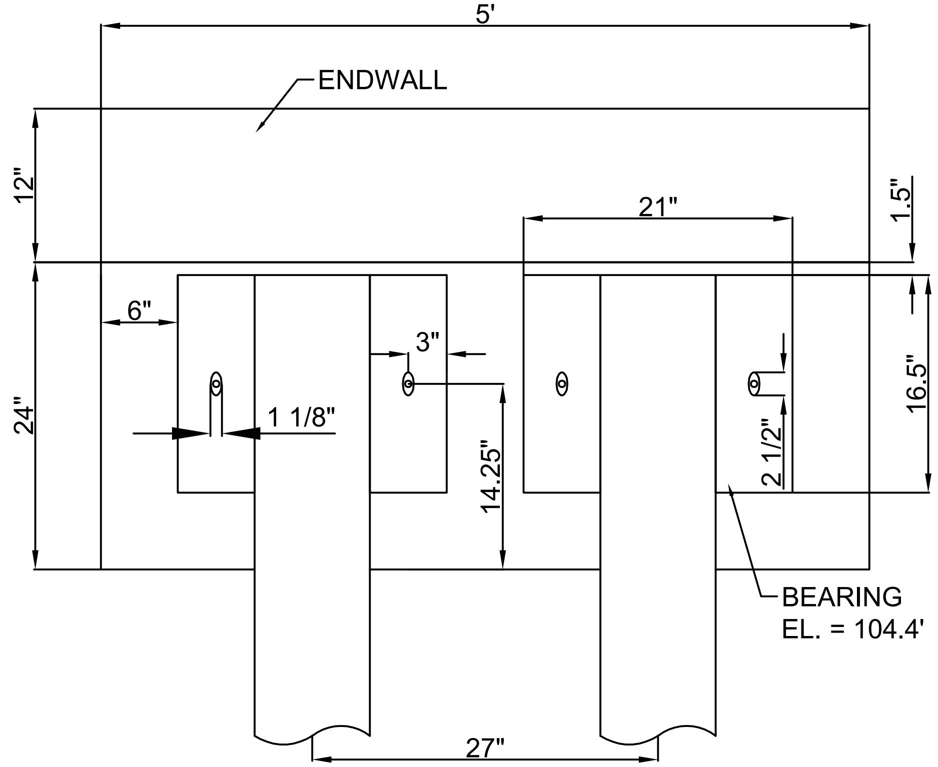
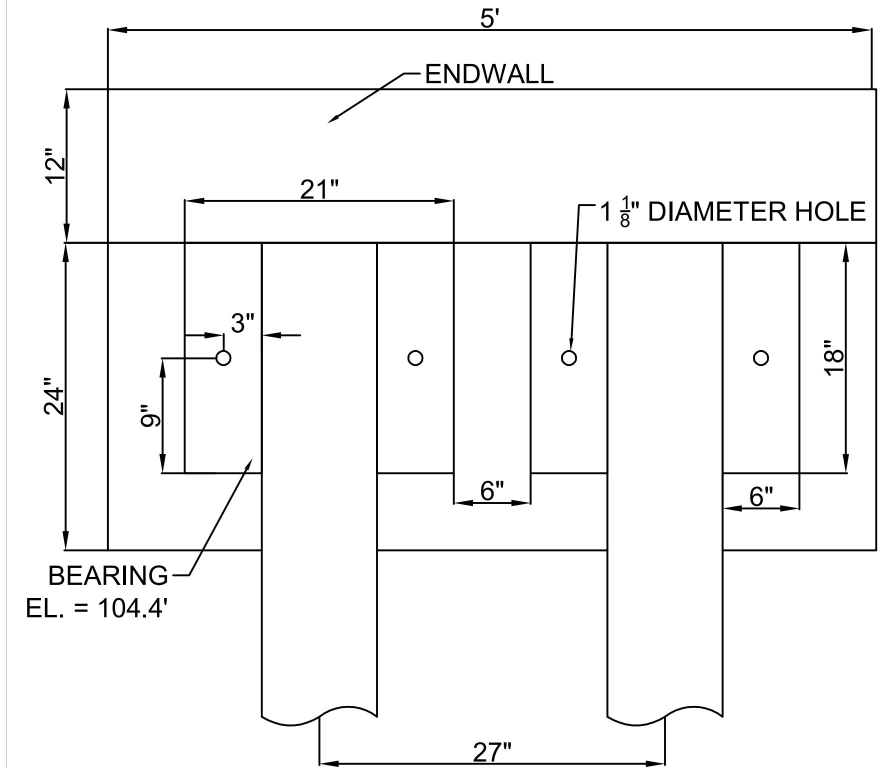
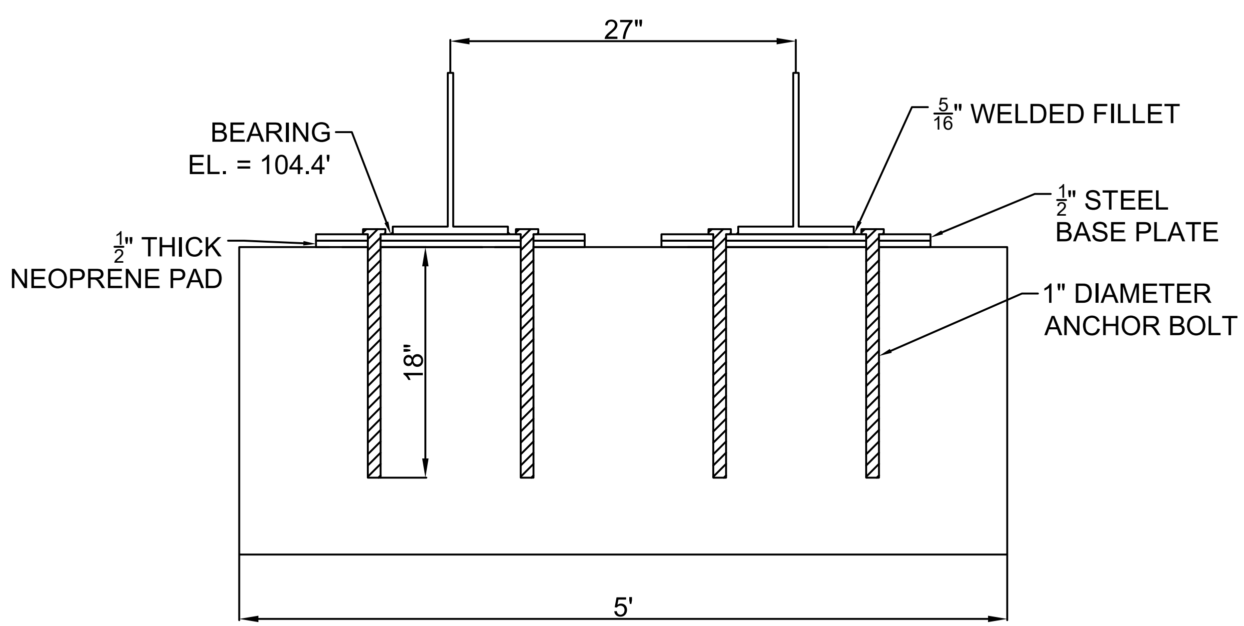


PLAN VIEW: SOUTH ABUTMENT, GIRDER ATTACHMENT



PLAN VIEW: NORTH ABUTMENT, GIRDER ATTACHMENT



SECTION VIEW: GIRDER ATTACHMENT

NOTES:

1. ANCHORS CAN BE DRILLED AND GROUTED.
2. BASE PLATE ON SOUTH END TO HAVE SLOTTED EXPANSION HOLE AS SHOWN. NEOPRENE PAD SHALL NOT BE SLOTTED.

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LAS BACADILLAS STRINGER BRIDGE FOUNDATION DETAILS

F02

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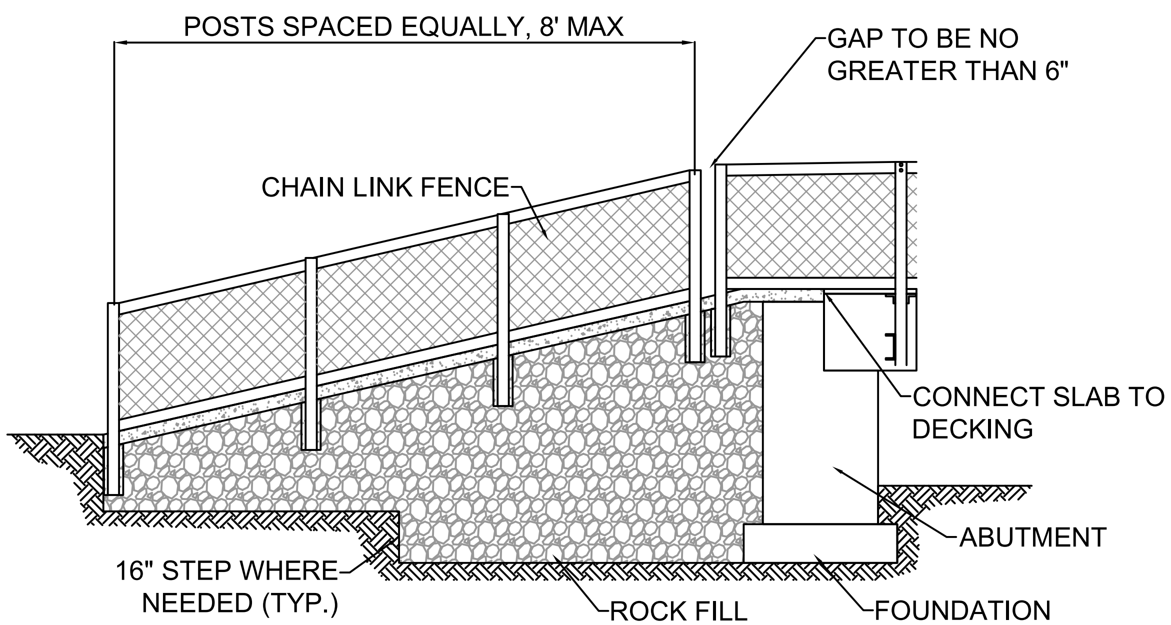
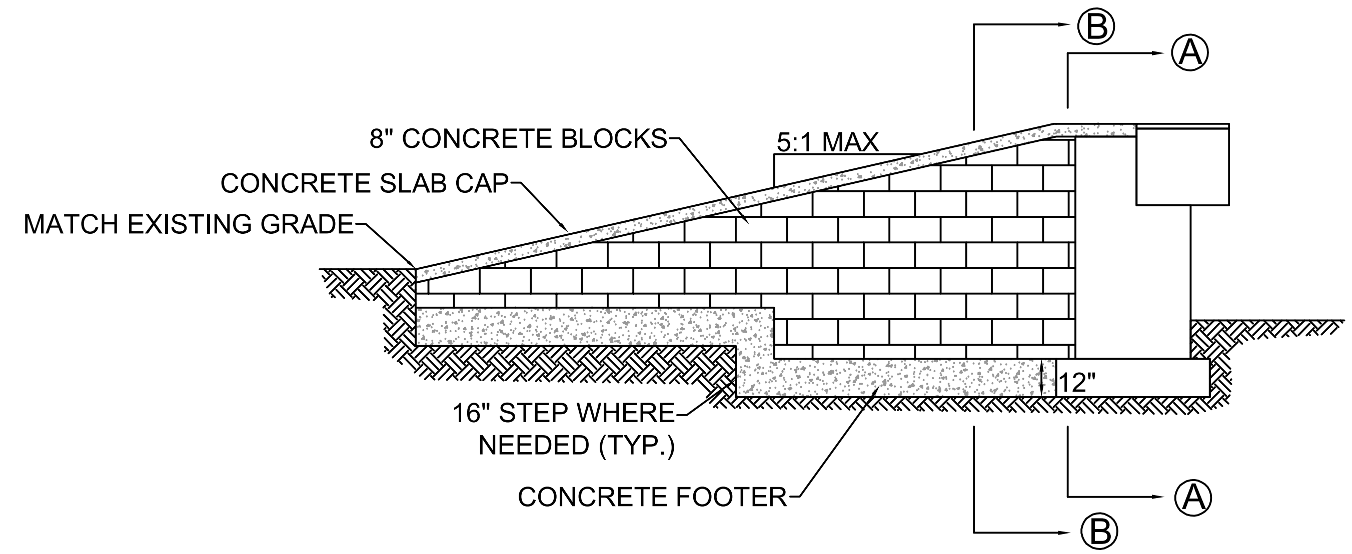
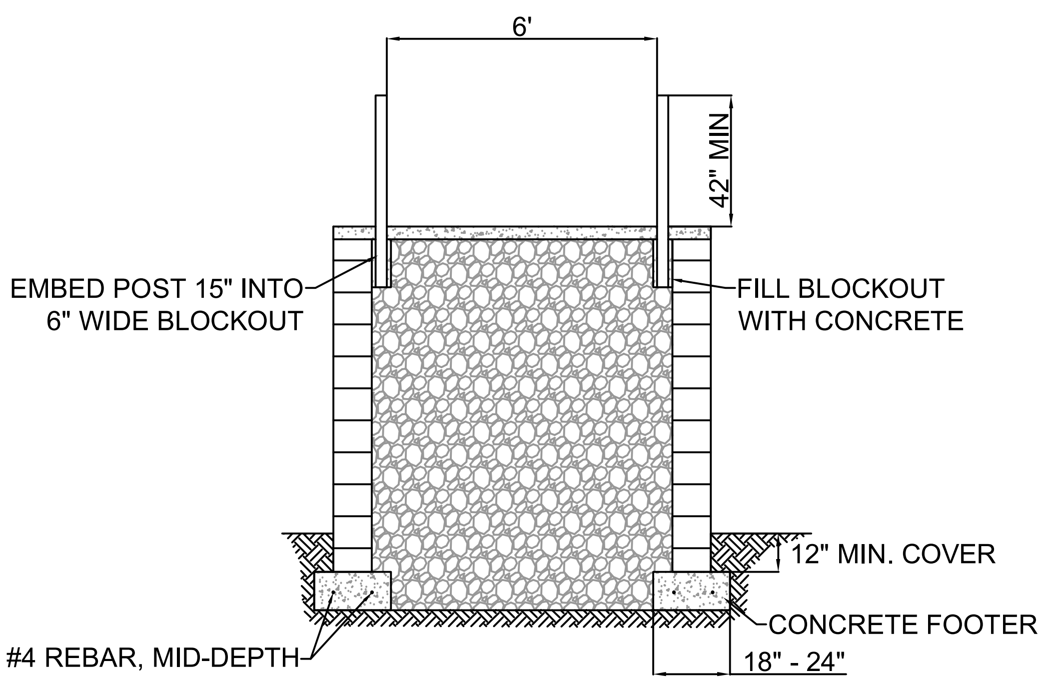
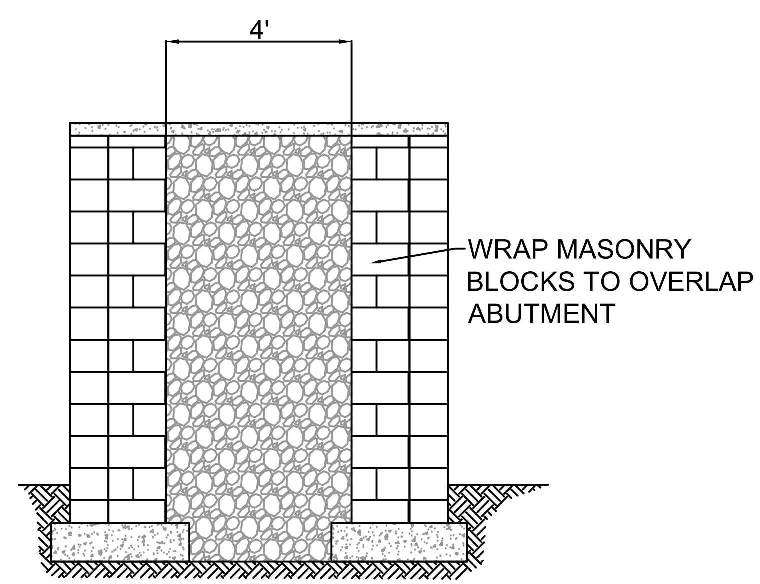






SECTION BB



SECTION AA

NOTES:

1. CONTRACTOR TO DETERMINE RAMP LENGTHS IN FIELD TO FIT ON SITE.
2. RAMP SLOPE NOT TO EXCEED 5:1.

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LAS BACADILLAS STRINGER BRIDGE RAMP DETAILS

R01

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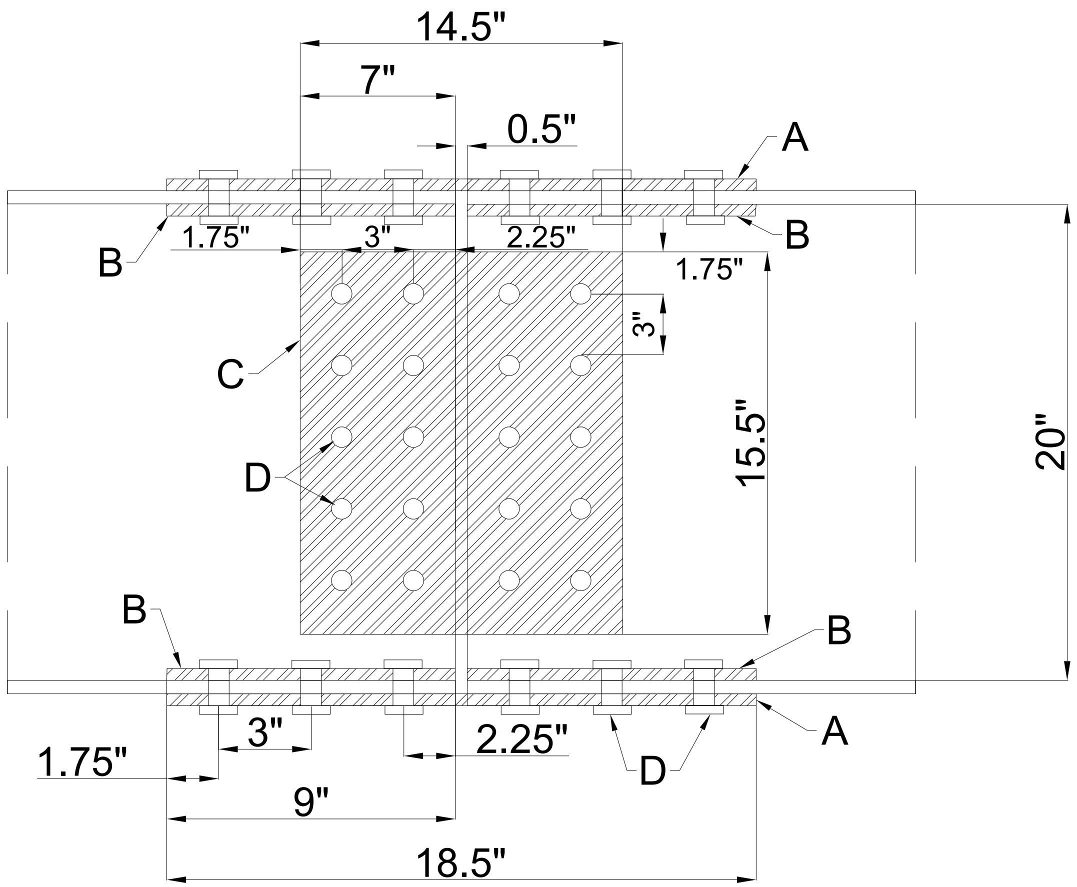
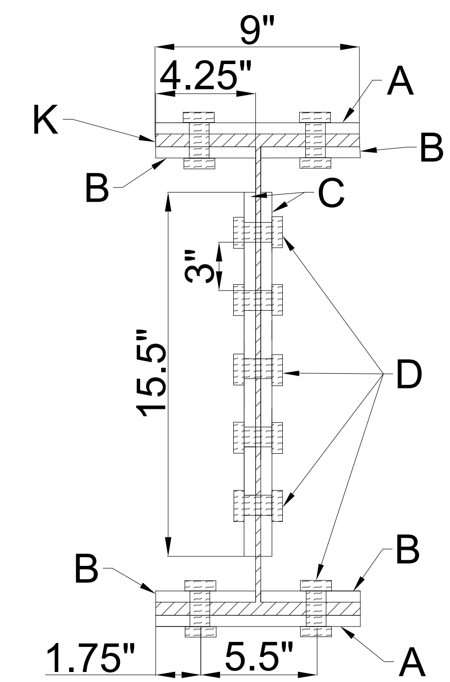


SPLICE
ELEVATION VIEW



SPLICE
CROSS SECTION

NOTES:

1. SPLICE BEAMS AT THIRD POINTS OF BRIDGE, EVERY 28'
2. THERE SHALL BE 2 ROWS OF 3 BOLTS EACH FOR EACH FLANGE SPLICE.
3. THERE SHALL BE 2 ROWS OF 5 BOLTS EACH FOR EACH WEB SPLICE.
4. ALL BOLTS SHALL BE $\frac{7}{8}$" DIAMETER A325 BOLTS.
5. ALL PLATES SHALL BE $\frac{1}{2}$" THICK GRADE 50 STEEL.
6. ALL STEEL SHALL BE WEATHERING STEEL IF AVAILABLE. OTHERWISE PRIME AND PAINT TO PREVENT CORROSION.
7. STIFFENER PLATES TO BE WELDED TO W-SHAPE.

BILL OF MATERIALS

NAME	SIZE	QUANTITY PER SPLICE	TOTAL
A	9" X 18.5" X $\frac{1}{2}$"	2	8
B	4.25" X 9" X $\frac{1}{2}$"	4	16
C	15.5" X 14.5" X $\frac{1}{2}$"	2	8
D	A325 BOLT $\frac{7}{8}$" DIA.	44	176
E	20" X 4.25" X $\frac{1}{2}$"	N/A	28
F	A325 BOLT $\frac{7}{8}$" DIA.	N/A	44
G	C 12 X 20.7	N/A	3 @ 33"
H	L 4 X 4 X $\frac{3}{8}$	N/A	8 @ 18"
I	4" X 4" X $\frac{1}{2}$"	N/A	8
J	12" X 12" X $\frac{1}{2}$"	N/A	2
K	W 24 X 68	N/A	6 @ 28'

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LAS BACADILLAS STRINGER BRIDGE STRUCTURAL DETAILS

S01

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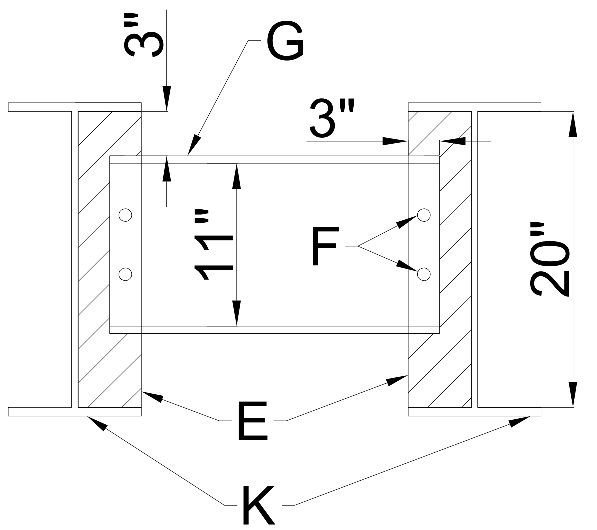
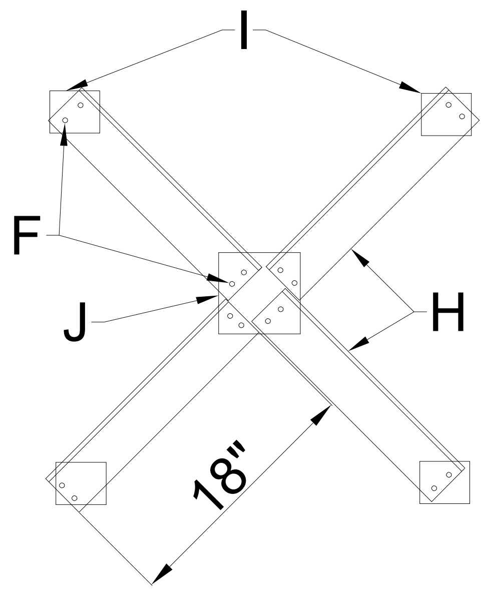
PRODUCED BY AN AUTODESK STUDENT VERSION

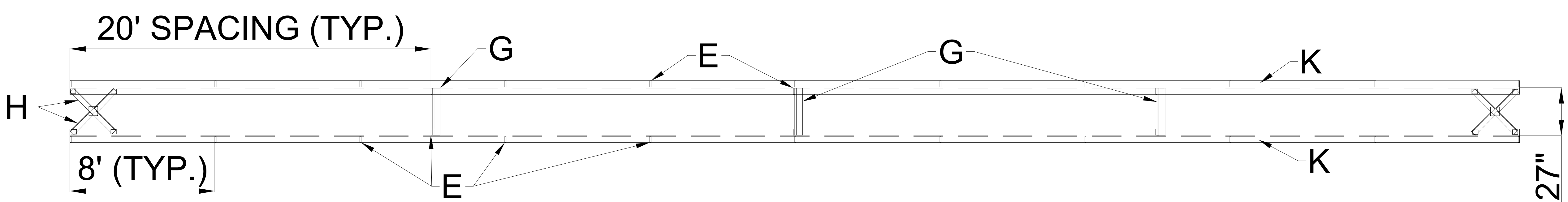


DIAPHRAGM SECTION VIEW



CROSS-FRAMING PLAN VIEW

BILL OF MATERIALS			
NAME	SIZE	QUANTITY PER SPLICE	TOTAL
A	9" X 18.5" X $\frac{1}{2}$"	2	8
B	4.25" X 9" X $\frac{1}{2}$"	4	16
C	15.5" X 14.5" X $\frac{1}{2}$"	2	8
D	A325 BOLT $\frac{7}{8}$" DIA.	44	176
E	20" X 4.25" X $\frac{1}{2}$"	N/A	28
F	A325 BOLT $\frac{7}{8}$" DIA.	N/A	44
G	C 12 X 20.7	N/A	3 @ 33"
H	L 4 X 4 X $\frac{3}{8}$	N/A	8 @ 18"
I	4" X 4" X $\frac{1}{2}$"	N/A	8
J	12" X 12" X $\frac{1}{2}$"	N/A	2
K	W 24 X 68	N/A	6 @ 28'



PLAN VIEW

VANDERBILT UNIVERSITY

REVISIONS

NO.	DATE	BY	BRIEF DESCRIPTION
1	04/29/2020	NSM	FINAL SENIOR DESIGN SUBMISSION

PRELIMINARY NOT FOR CONSTRUCTION

LAS BACADILLAS STRINGER BRIDGE STRUCTURAL DETAILS

S02

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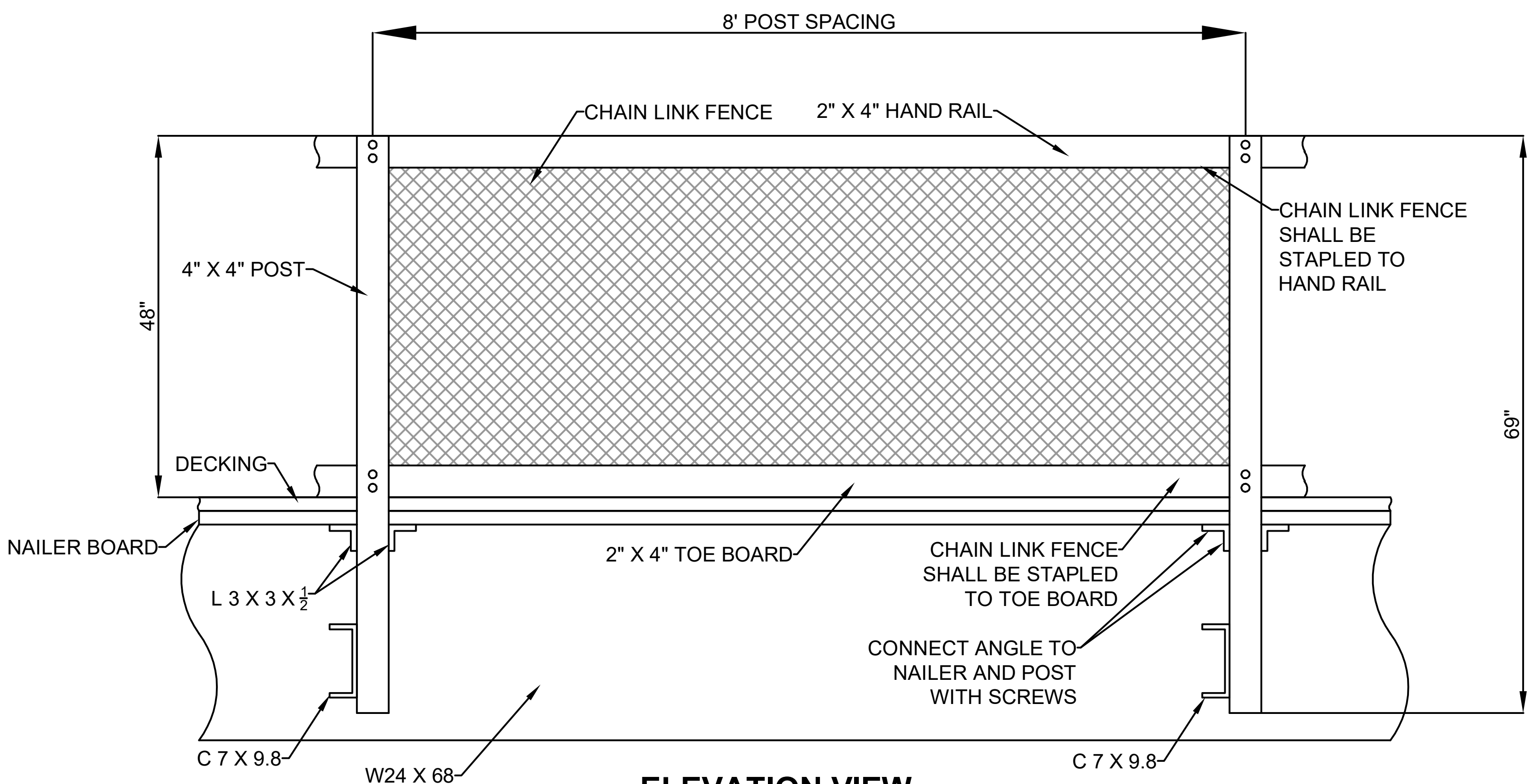

ELEVATION VIEW

BILL OF MATERIALS	
NAME	TOTAL QUANTITY
4" X 4" POSTS	127'
CHAIN LINK FENCING	640 SQFT
2" X 4" BOARDS	320'
DECK AREA	280 SQFT
NAILER BOARD AREA	194 SQFT
COUNTER SUNK BOLTS	42
L 3 X 3 X $\frac{1}{2}$	44 @ 3"
C 7 X 9.8	22 @ 9"
A325 BOLTS	88
WOOD SCREWS	2300

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CONSTRUCTION

LAS BACADILLAS
STRINGER BRIDGE
WALKWAY DETAILS

W01

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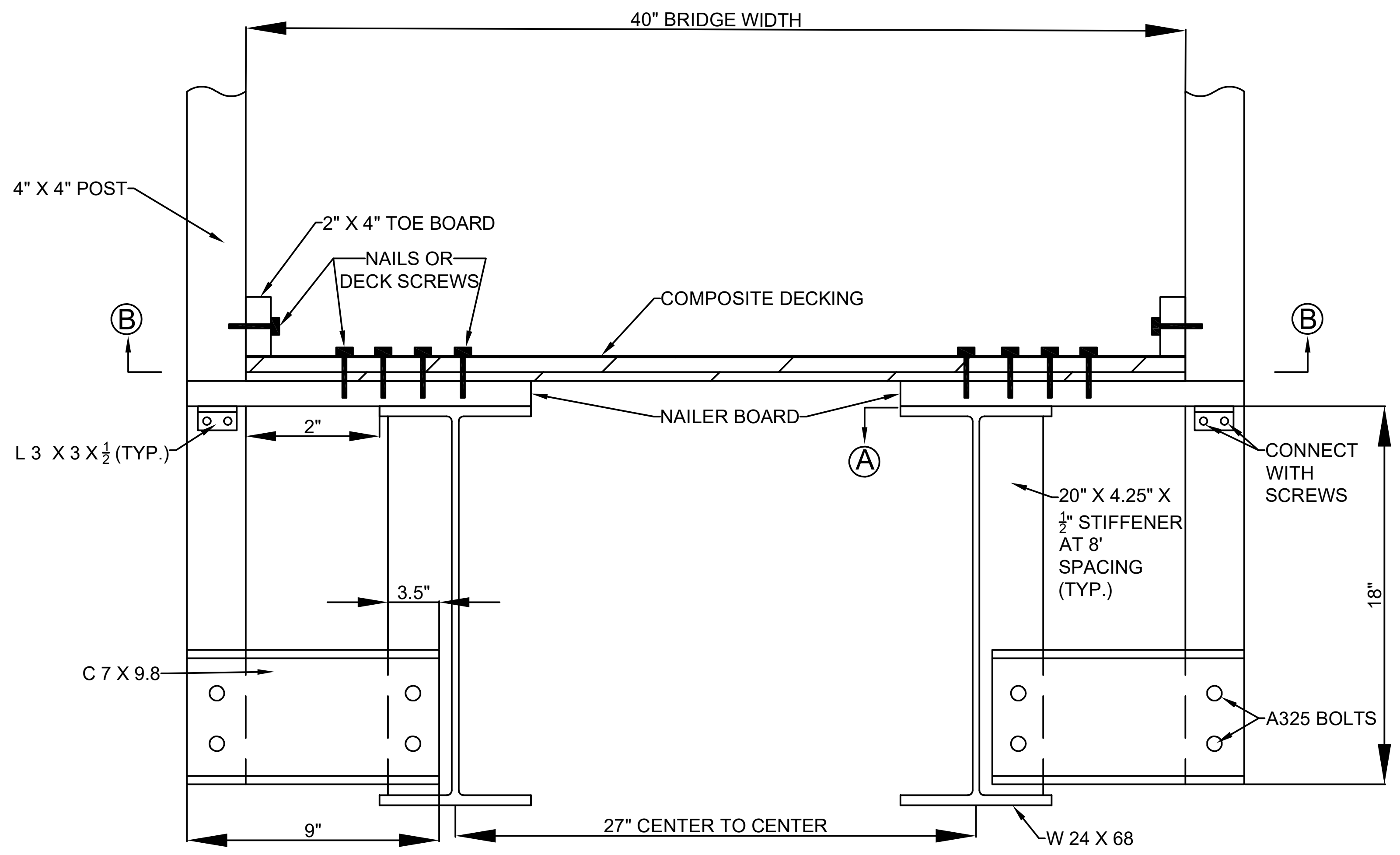
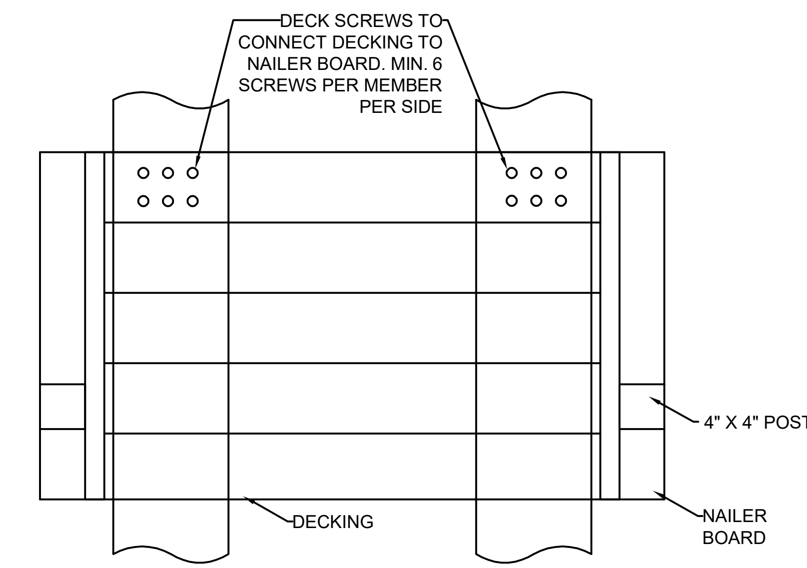
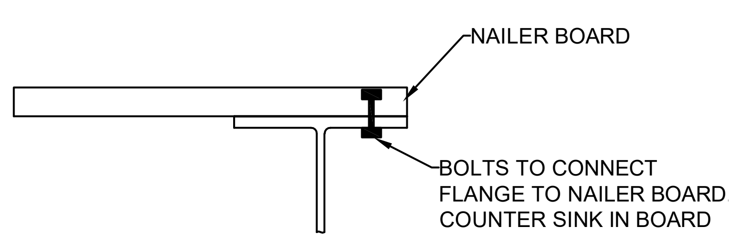


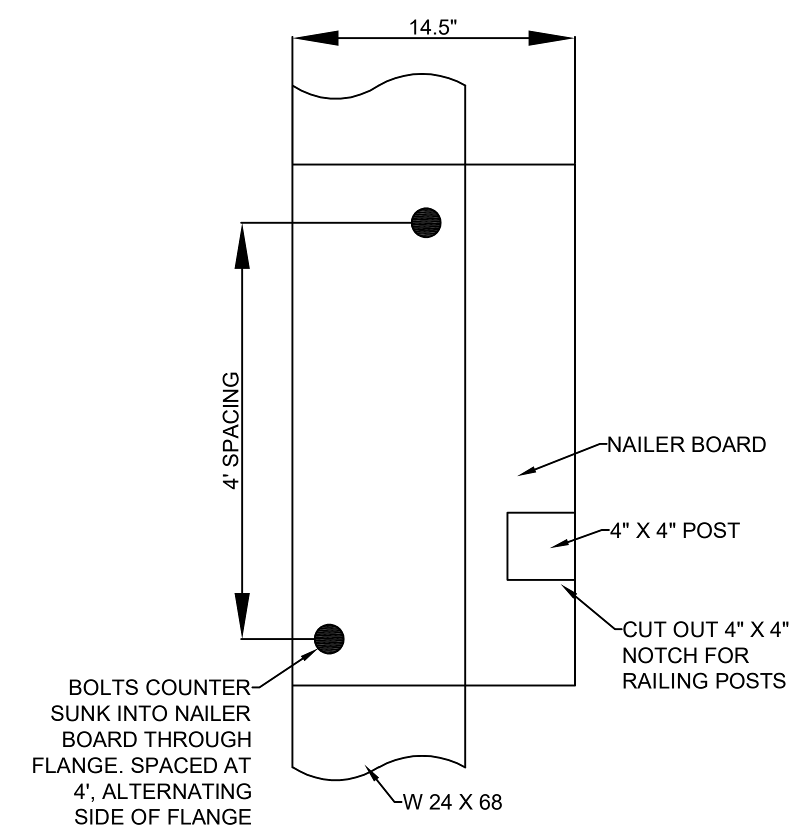
SECTION VIEW



PLAN VIEW BB



SECTION VIEW



PLAN VIEW AA

VANDERBILT UNIVERSITY

REVISIONS

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LAS BACADILLAS STRINGER BRIDGE WALKWAY DETAILS

W02

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