INTRODUCTION

The Nature Conservancy (TNC) has hired Ecology and Environment, Inc. (E & E) and its team member, MRD Associates, Inc., to review alternatives to address problems associated with scour around the Black Lake Bridge. The project site is located in Escambia County, Florida (Figure 1) near the Perdido River. Prior to bridge construction in 2011, access across the creek was provided by a low earthen roadway. Water was conveyed under the roadway through large 60-inch-diameter corrugated metal culverts, with the roadway overtopped during infrequent high flow events. The roadway and culverts were removed, and a 24-foot-long bridge was constructed along with wooden abutment wing walls. The wing walls extended just below the mud line and did not have any protective armoring in front. Since the construction of the bridge, the wing walls have been undermined at least twice. The most recent event occurred during an exceptionally extreme event in late April 2014 which completely eroded the bank behind the south wing wall (Figure 2) and caused piping of material on the north wing wall (Figure 3). There is an approximate 24-foot gap between the bridge on its south end and the top of bank of the south approach. The National Weather Service (NWS) called the event “historic.” The official rain gauge at Pensacola’s airport measured 5.68 inches in a single hour. An analysis by the NWS office in Mobile, Alabama, estimated that single hour to be a 1-in-200- to 1-in-500-year event. The total rain over the three-day period was over 21 inches (NASA 2014). Per Florida Department of Transportation (FDOT) Precipitation Intensity Duration Frequency Curves (FDOT 2001), the event over the three-day period was in excess of a 100-year rainfall event. TNC reported that the maximum water level during the multiple-day storm event in April 2014 reached the bottom girders of the bridge and the bridge was not overtopped.

It is our understanding that the budget TNC has to address the problem is somewhat limited, thus the need for a cost-effective solution that does not require frequent replacement. Initially, TNC had proposed a solution that included deeper sheet piles (40 feet long); however, E & E expressed concerns about the cost and the likely need for much longer wing walls extending landward for the south abutment. Thus, E & E proposed to conduct a planning level review of various alternatives, identify what, if anything, can be done within TNC’s funding limits, and make a recommendation.

SITE CONDITIONS

As shown on Figure 1, the Black Lake Bridge is located along the outside bend of the creek channel and the southern bank is exposed to the highest water velocities. Topographic maps show a natural constriction in cross section near the bridge crossing (Figure 4) based on higher elevations of at least 3 to 6 feet North American Vertical Datum of 1988 (NAVD 88). These higher elevations do not allow overland sheet flow. Based on observations made during the April 2014 storm event (discussed further below), out-of-bank water elevations may not get much higher than several feet. The natural topographic constriction funnels out-of-bank flows and results in higher velocities near the bridge in comparison to other areas upstream and downstream where there is a wider flood plain. Based on aerial photos, the natural width of the creek channel at the bridge location is approximately 75 to 80 feet; however, with the bridge construction and abutments that extend into the creek, the opening is only 24 feet at the bridge. As a result, the cross sectional opening has been reduced over one-third, which effectively increases the water velocity through the bridge opening by three times. During high rainfall events, the flow increases, thereby substantially increasing the potential for scour. Further discussion is provided in the Hydrologic Assessment Section.

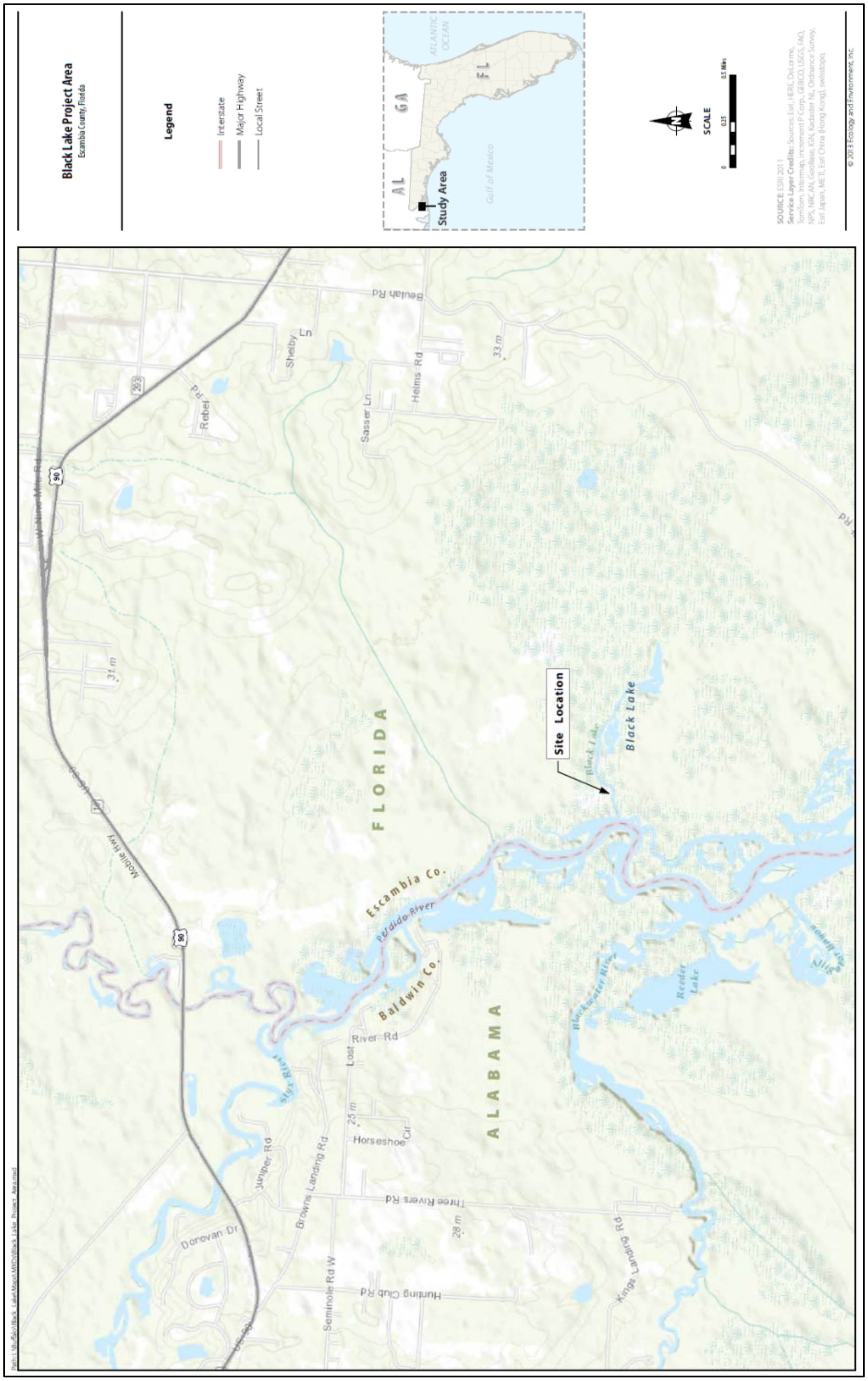

Figure 1. The Black Lake Bridge project site is located in Escambia County, Florida, near the Perdido River.

Figure 2. Complete erosion of the bank behind the south wing wall occurred during an extreme event in April 2014.

Figure 3. The extreme event of April 2014 caused piping of material on the north wing wall.

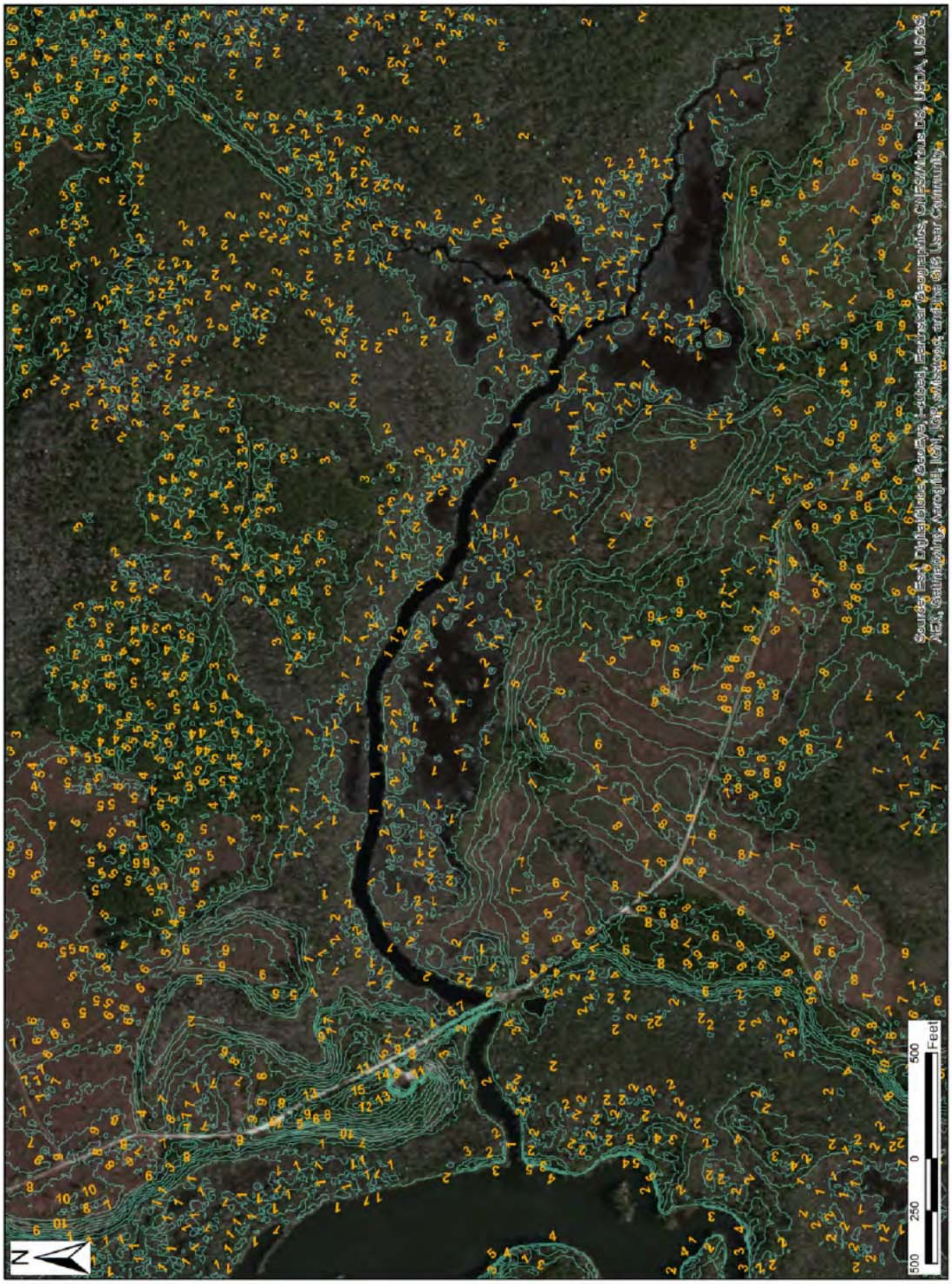

Figure 4. Topographic maps show a natural constriction in elevations at the bridge crossing, which created increased velocities in the area.

Access across the bridge is not possible due to extensive scour, particularly at the south abutment. Both abutments have inadequate levels of protection for scour. The south abutment is most vulnerable to scour since it is located on the outside bend of the creek channel where the highest water velocities occur.

During the team's site visits on November 6 and November 20, 2014, elevations were recorded at select cross sections (see Figures 5 - 7). The water level at the time of the survey was -1.65 feet NAVD 88 and was recorded during a low tide. The high tide water mark on the bridge pilings was approximately 1 foot higher. The cross sections indicate that the bottom of the channel under the bridge (Sections B-B' and C-C' in Figures 5 and 6, respectively) is about 3 feet deeper than the channel depths several hundred feet upstream and downstream of the bridge (Sections D-D' and A-A') and show substantial scour behind the south wing wall. Just after bridge construction in 2011, TNC reported that the water depths immediately under the bridge were only a few feet since the road fill was not completely removed during construction resulting in a hump in the creek channel. This hump was eroded away and the channel in the immediate area of the bridge currently reflects a somewhat more natural condition.

The engineering design drawings for the existing bridge and approaches show a lower area on the northern upland approach (approximately 75 to 150 feet north of the bridge), which is a low water crossing. The elevations were checked during our field visit, and the lowest elevations at the low water crossing were found to be +1.4 feet NAVD 88. There was visual evidence that this low water crossing was overtopped during the April 2014 storm event based on the presence of high water debris piles.

The current conditions and observations provide valuable insight into what can occur during a major event, such as the April 2014 storm event, and the resulting cross section that was generated in the area of the bridge to pass a significant storm flow.

HYDROLOGIC ASSESSMENT

TNC had initially requested input on design options for various storm events. Without extensive modeling and the lack of flow and stage data for Black Lake Creek, we are not able to reliably determine peak flows and associated velocities for various storm events. However, we are able to use flow estimates from a U.S. Geological Survey (USGS) online tool, called "StreamStats," for this watershed to aid in the determining the potential extent of flow for various storm events and how that might impact review of alternatives to address problems associated with scour around the Black Lake Bridge.

StreamStats is a Web-based Geographic Information Systems (GIS) application for use in water resources planning, management, and engineering design. The tool was developed through a cooperative effort of the USGS and ESRI, Inc. StreamStats makes the process of computing stream flow statistics for ungauged sites much faster, more accurate, and more consistent than previously used manual methods (USGS 2014).

Though StreamStats is not available for the State of Florida, the data for Black Lake creek are available through Alabama State StreamStats program and were used to support this hydrologic assessment.

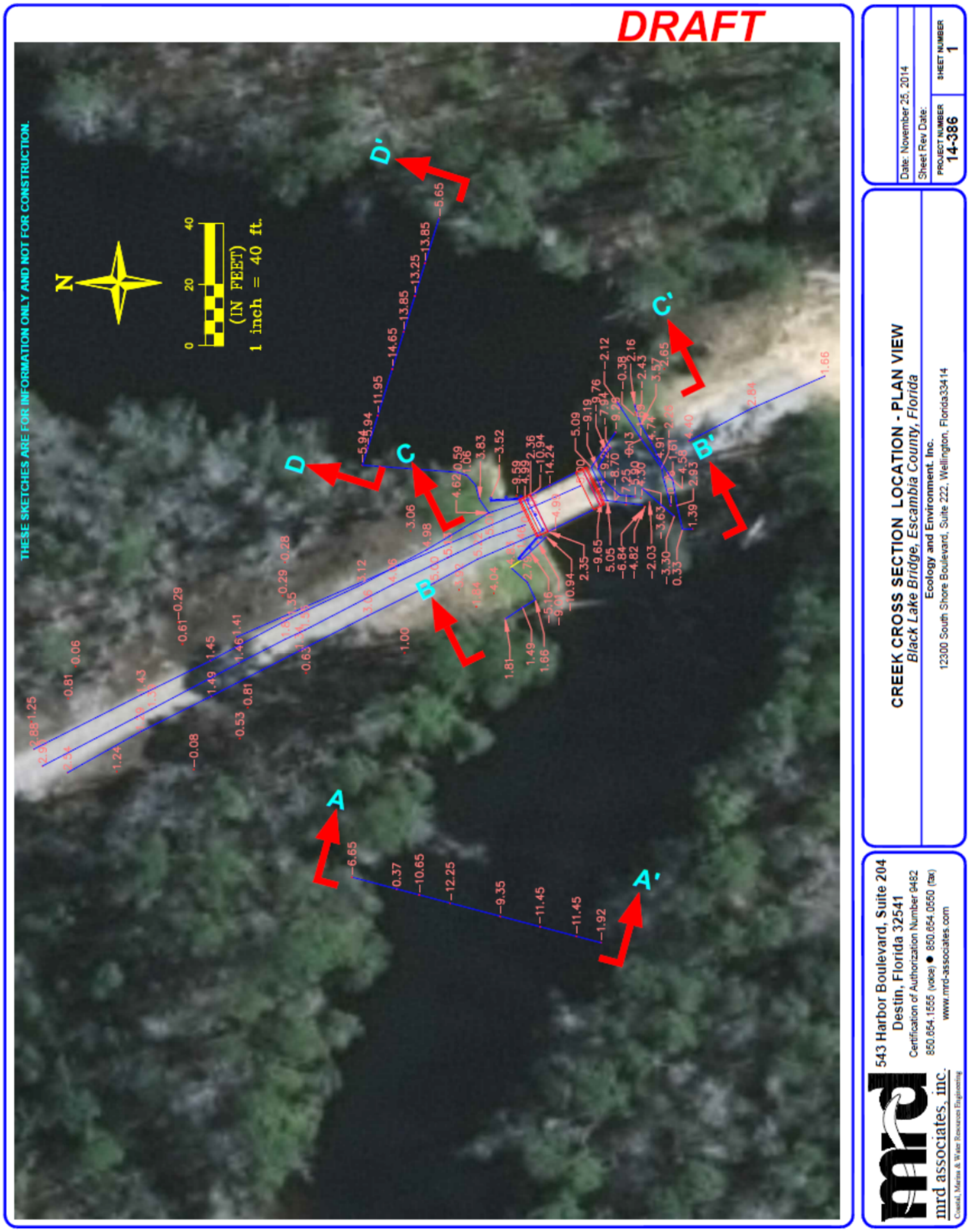

Figure 5. Cross Section Locations.

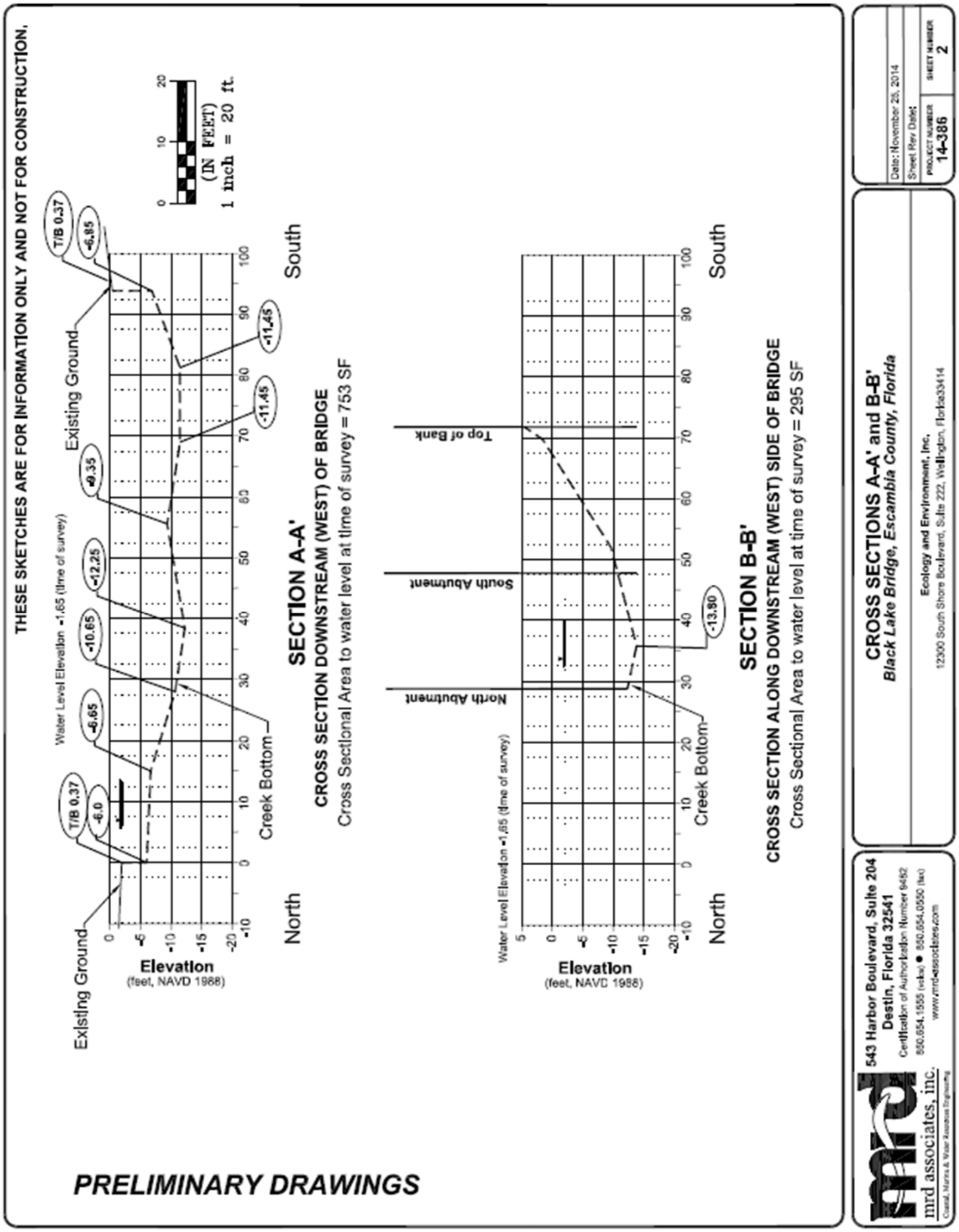

Figure 6. Existing Cross Sections A-A and B-B.

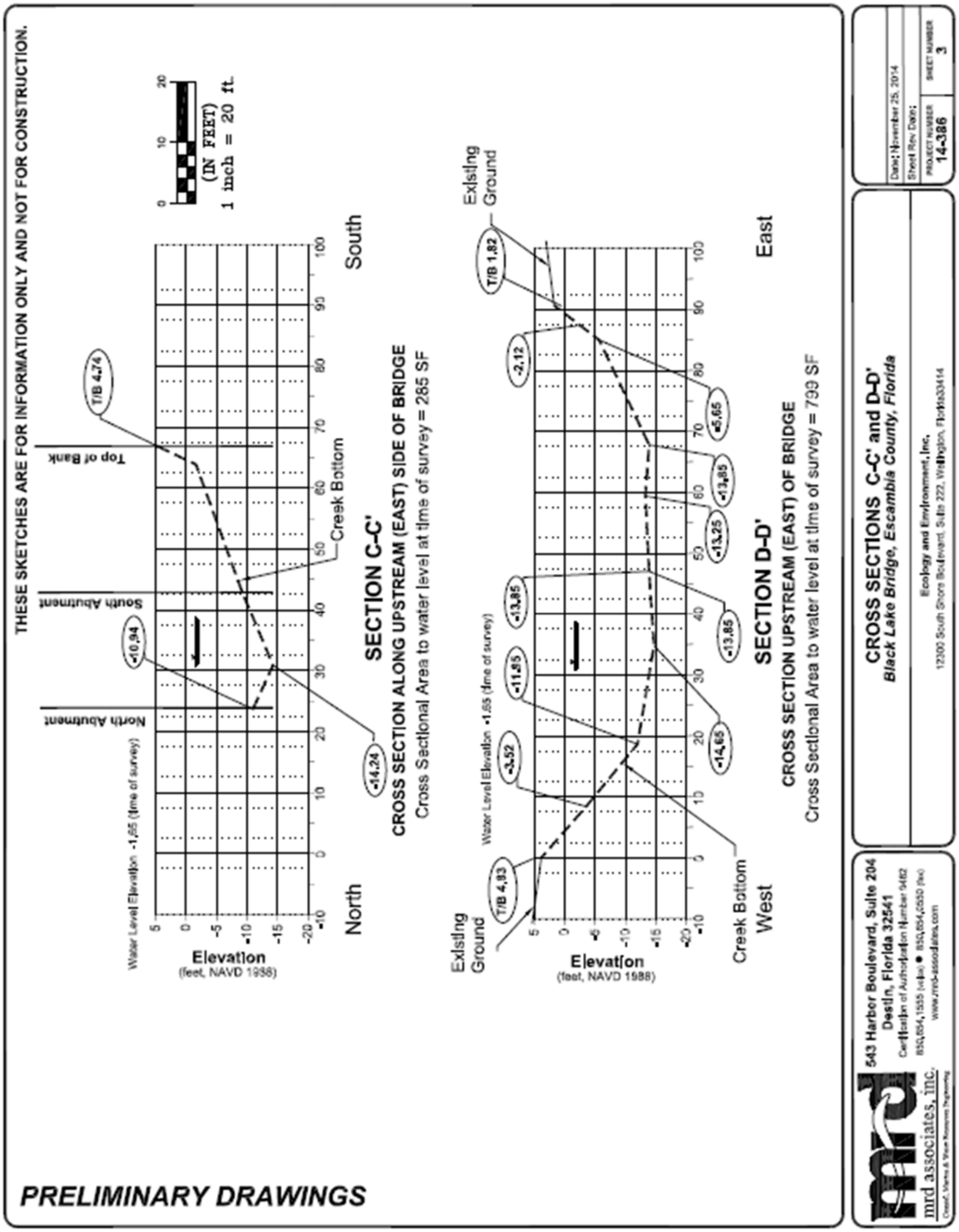

Figure 7. Existing Cross Sections C-C and D-D.

StreamStats estimates drainage basin boundaries by use of digital elevation data. These data are usually derived from digital elevation data from the National Elevation Dataset (NED) that have been specially processed so that the elevation data conform to the digital stream channels depicted in the high-resolution version of the National Hydrography Dataset (NHD), and to the drainage basin boundaries of the Watershed Boundary Dataset. Figure 8 shows the drainage delineated for Black Lake Creek produced by StreamStats. The basin has a total area of approximately 4.3 square miles (sq mi), which is a small basin (less than 5 sq mi). The estimated peak flows for 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, and 200-year storm events from the StreamStats program for Black Lake Creek are presented in Table 1.

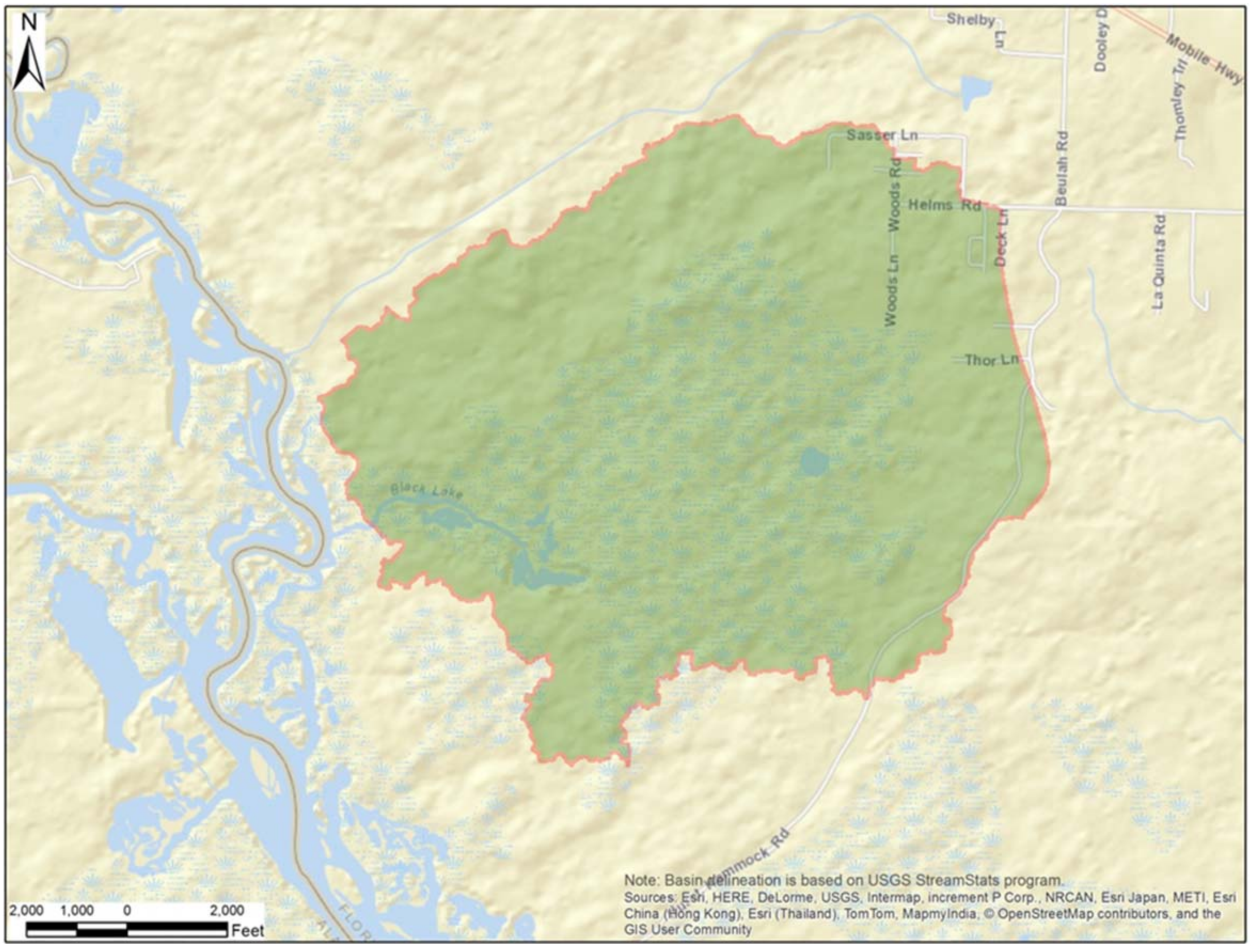

Figure 8. Drainage delineated for Black Lake Creek.

Table 1. Black Lake Basin Peak Flow Statistics

Statistic*	Estimated Flow (cfs)	Prediction Error (Percent)	Equivalent Years of Record
PK2	558	53	2
PK5	964	35	5
PK10	1310	30	9
PK25	1820	29	13
PK50	2260	31	14
PK100	2740	35	14
PK200	3280	40	13

Note:
* *PK2, PK5, PK 10, PK25, PK50, PK100, PK200 are flows for recurrence intervals of 2, 5, 10, 25, 50, 100, and 200 years.*
Key:
cfs = Cubic feet per second.

Flow Velocities

For abutment protection, understanding the magnitude of velocities is important. While cross sectional information has been obtained, and peak storm event flows have been estimated using the USGS StreamStats program, water levels associated with the different peak storm events are not readily known and are needed to calculate cross sectional areas to estimate water velocities. There is no stage data for Black Lake Creek; however, based on field observations, professional judgments can be made to bracket likely water levels and estimate a range of velocities for different storm events. The field observations mentioned above include:

- Approximate typical high water elevation of -0.65 feet NAVD 88; and

- Maximum water elevation during a 150- to 200-year storm reached an elevation of approximately 2.65 feet NAVD 88 (bottom of bridge), which equates to a 3-foot rise over the high tide.

Using the cross section data of the creek, cross-sectional areas were calculated for incremental increases in water levels up to a maximum water level of 3 feet above the approximated mean high tide level. The low water crossing area along the north bridge approach is also treated as part of the cross section area, which provides overflows when the water is above 1.4 feet NAVD 88 in the immediate area around the bridge area.

Flow velocities were estimated for the scenario that the bridge abutments were restored to pre-storm conditions and the bridge opening in the channel remained at 24 feet. Table 2 shows the results for a likely range in water levels for a given storm event and the associated estimated velocity. The estimated velocities were in a range of 7.4 feet per second to 10.7 feet per second for a 50-year, 100-year, and 200-year storm. These velocities will drop if the cross section of the channel at the bridge is increased. For example if the cross section is doubled to 48 feet, the estimated velocities are reduced by about 50 percent and are in a less erosive range of 4.0 to 5.8 feet per second (Table 3). The most important finding is that the flow velocities are somewhat similar for the different storm events. While the flows increase during a larger storm event, the increase in water level results in a larger cross sectional area and thus mitigates velocity increases.

Table 2. Estimated Peak Flows and Velocities – Existing Bridge Design Opening (24 feet)

Elevation (NAVD 1988)	Cumulative Elevation	Total Cross Section Area	Peak Flow	Velocity	Peak Flow	Velocity	Peak Flow	Velocity
			50 Year Storm		100 Year Storm		200 Year Storm	
ft	ft	sf	cfs	ft/s	cfs	ft/s	cfs	ft/s
-0.65	0	217						
0.35	1	236	2260	9.6				
1.35	2	255	2260	8.9	2740	10.7		
1.85	2.5	307	2260	7.4	2740	8.9		
2.35	3	372			2740	7.4	3280	8.8
2.65	3.3	402					3280	8.2

Key:
cfs = Cubic feet per second. *NAVD 1988 = North American Vertical Datum of 1988.*
ft/s = Feet per second. *sf = Square feet.*

Table 3. Estimated Peak Flows and Velocities – Doubling Existing Bridge Opening (48 feet)

Elevation (NAVD 1988)	Cumulative Elevation	Proposed Total Cross Section Area	Peak Flow	Velocity	Peak Flow	Velocity	Peak Flow	Velocity
			50 Year Storm		100 Year Storm		200 Year Storm	
ft	ft	sf	cfs	ft/s	cfs	ft/s	cfs	ft/s
-0.65	0	434						
0.35	1	472	2260	4.8				
1.35	2	510	2260	4.4	2740	5.4		
1.85	2.5	572	2260	4.0	2740	4.8		
2.35	3	646			2740	4.2	3280	5.1
2.65	3.3	681					3280	4.8

Note: Estimated velocity based on cross section with vertical side walls and velocity may vary depending up toe protection or slope in front of sheet pile.

Key:
cfs = Cubic feet per second. *NAVD 1988 = North American Vertical Datum of 1988.*
ft/s = Feet per second. *sf = Square feet.*

DESIGN CONSIDERATIONS

In reviewing design criteria for abutment scour protection, there is no specific standard for small forest/back road bridges from resource agencies, such as the United States Forestry Service. Deferring to the FDOT Drainage Handbook - Bridge Hydraulics, the recommended design scour frequencies for a standard roadway bridge with average annual daily traffic of less than 1,500 vehicles would be a 50-year storm event. Note that the bridge in question is only used by TNC and United States Fish and Wildlife (USFWS) staff on an infrequent basis and an occasional log truck. Bridge access by the logging truck can be prohibited by TNC by limiting access to the property through the northern and southern roads.

Addressing the scour that occurred during the April 2014 rainfall event would entail a level of protection in excess of a 50-year storm/rainfall event. This does not include the impacts of storm surges as the entire bridge structure could be overtopped.

For the existing bridge, the design parameters used by CBC Engineers and Associates (CBC) were based on the American Association of State Highway and Transportation Officials (AASTO) H20 Loading utilizing a 32 kip per wheel axle. The Construction Drawings prepared by CBC called for 10-inch-diameter tip timber piles with a 12-inch-diameter butt driven to a minimum depth of 50 feet. Based on the site visit conducted by MRD on November 20, 2014, it appears that a minimum 16-inch butt (12- to 14-inch tip) pile was used. Based on MRD's preliminary analysis to support a 20.1 kips load (as specified in the Construction Plans dated March 5, 2010 by CBC) using a 16-inch-diameter butt, a minimum embedment depth of 40 feet would be required to achieve an equivalent bearing capacity. Thus, it appears the existing piles (assuming they were installed to the specified depth per CBC's plans) would be capable of supporting the CBC design loads with the scour that had occurred in April 2014. In addition, any final design of an abutment, bent, potential bridge expansion, and other key design considerations are related to typical loads. The team will rely on appropriate loads calculated for the existing bridge design such as AASTO H20 loading, lateral wind load, longitudinal seismic loads, hydrodynamic loads, and dead loads.

Additional scour protection is needed to protect the abutments, and some of the high velocities can be abated by increasing the cross sectional flow area at the bridge. This can be accomplished by increasing the bridge length and/or allowing more high water bypass through the low water crossing to the north. Rip rap can be placed along the bank and at the toe of the wall to provide additional scour protection. Sizing rip rap will be based on the U.S. Department of Transportation Hydraulic Engineering Circular No. 11 (HEC 11) and is dependent upon anticipated flow velocities and stone density. For the purpose of assessing the alternatives, the velocities presented in Tables 2 and 3 will be used. Assuming a stone with a density of 165 pounds per cubic foot and a water velocity of 10.7 feet per second, a stone weight between 60 and 70 pounds would likely be needed using the HEC 11 methodology.

ALTERNATIVES

The following alternatives were considered for planning level analysis:

A. Maintain Existing Bridge and Protect Abutment
B. Extend Existing Bridge and Protect Abutment
C. Extend the Existing Bridge with Railroad Car Frame and Protect Abutment
D. Install Large Concrete Box Culverts and Protect Abutment
E. Utilize Bioengineering Techniques

Discussion of each alternative is provided below. Further refinement of the selected alternative will be done in the design phase (under a new proposal).

Alternative A: Maintain Existing Bridge and Protect Abutment

The south abutment was scoured and flanked during the April 2014 storm event. This alternative is shown on Figures 9 and 10 and would include the installation of 38 linear feet of sheet pile along the north side of the bridge and on the inside of the existing wood wall to minimize the piping and loss of sand. In addition, along the southern bank, an additional 77 linear feet (38 feet on the east and 39 feet on the west) of sheet pile would be installed from the existing wood wall into the bank to protect the abutment from flanking during high flow events and contain the fill behind the abutment and existing eroded bluff. The new sheet pile would likely be installed to a minimum tip elevation of -20 feet. This depth considered the existing and additional potential scour below the current creek bottom to allow for an additional factor of safety. The eroded area on the backside of the existing abutment and wall would be backfilled with approximately 209 cubic yards of sand and compacted. It would first be necessary to wrap any gaps or voids between the old and new wall with filter fabric to reduce piping of materials and further loss of sand. Geoweb and gravel stone would be placed on top of the compacted fill to finish the driving surface from the eroded bank to the existing bridge. In addition to the sheet pile to stabilize the shoreline and provide scour protection to the base of the wall and adjacent shoreline, granite riprap sized to protect against a high flow event that generate velocities up to 10 feet per second would be placed at the toe and edges of the wing walls and shoreline. Approximately 1,055 cubic feet of stone with a minimum density of 165 pounds per cubic foot is proposed.

Alternative B: Extend Existing Bridge and Protect Abutment

This alternative, shown on Figure 11, includes extending the existing bridge to the south by approximately 26 feet in length to reach the current top of bank on the south approach. This extension would be based on the previous bridge design with an increased level of abutment protection on both the north and south ends of the bridge. The construction of the new abutment would be located at the waterward face of the top of bank to the south where the shoreline was washed out. It would also be necessary to install a new pile supported concrete bent immediately adjacent to the existing abutment to carry the load of the northern end of the new bridge section. The new bent would be supported by five wood piles driven to a minimum tip elevation of -50 feet in order to achieve the desired bearing capacity. Approximately 44 linear feet of sheet pile would be installed on the landward side of the new abutment and along the shoreline to retain the upland soils and provide a level of scour protection. Instead of "anchoring" the wall together, the sheet pile system in this alternative would be supported by a concrete deadman connected to the wall by galvanized steel rods and waler. As proposed in the previous alternative, approximately 38 linear feet of sheet pile would be installed on the north side to contain the fill and extend the flanking protection to the shoreline and bank. Approximately 1,017 cubic feet of rip rap stone with a minimum density of 165 pounds per cubic foot is proposed to protect against a high flow event that would generate velocities up to 10 feet per second. The rip rap stone would be placed at the toe and edges of the wing walls and shoreline. While the velocities will be lower with a wider bridge, there is not much difference in cost to provide larger stone to account for higher velocities that could occur on the outside bend of the creek channel.

The benefit of this option is that it would increase the area of water conveyance. This can reduce the velocity through the bridge section and risk of damage. A new sheet pile behind the proposed abutment can help to reduce the likelihood of further washout.

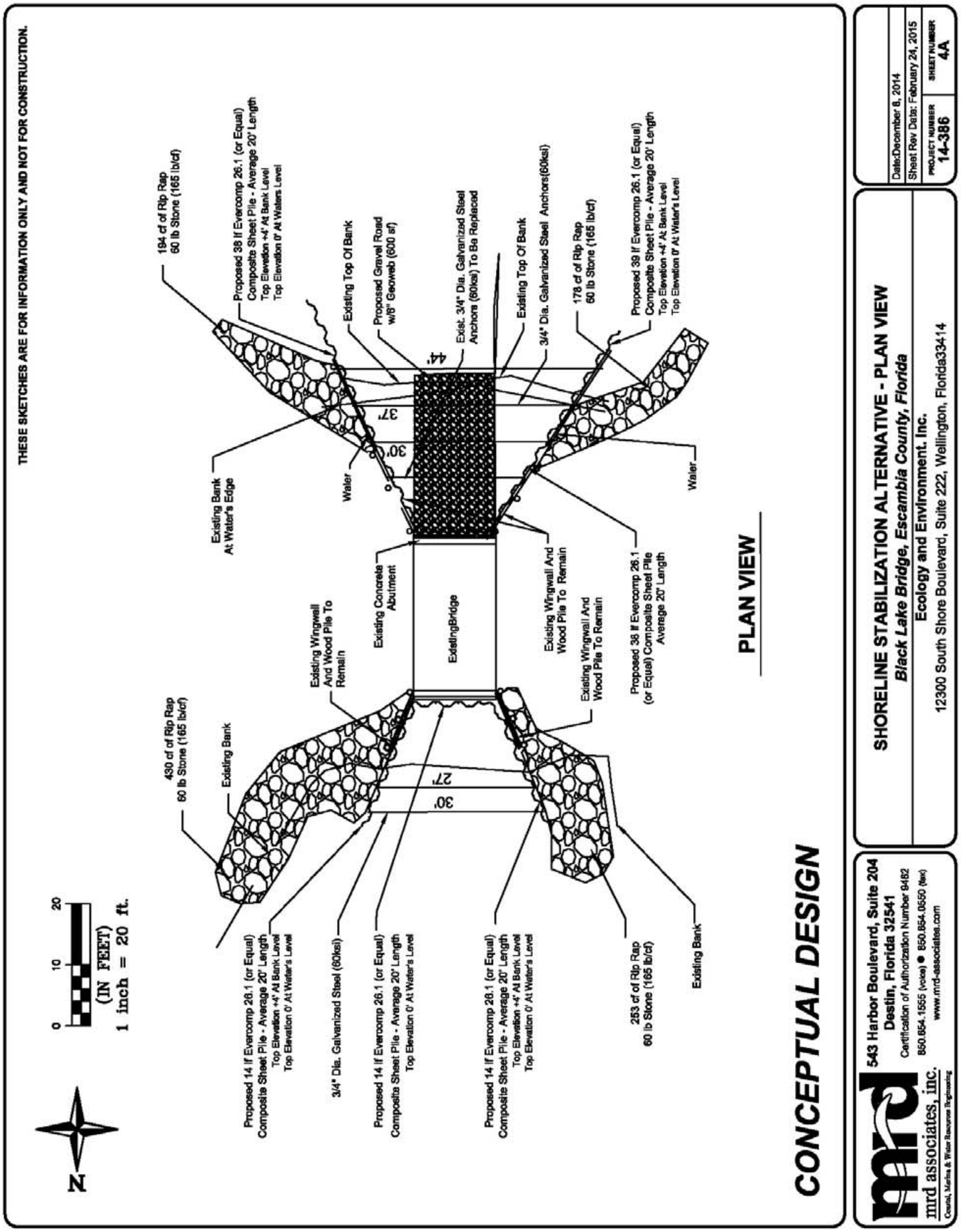

Figure 9. Alternative A – Maintain existing bridge and protect abutment plan view.

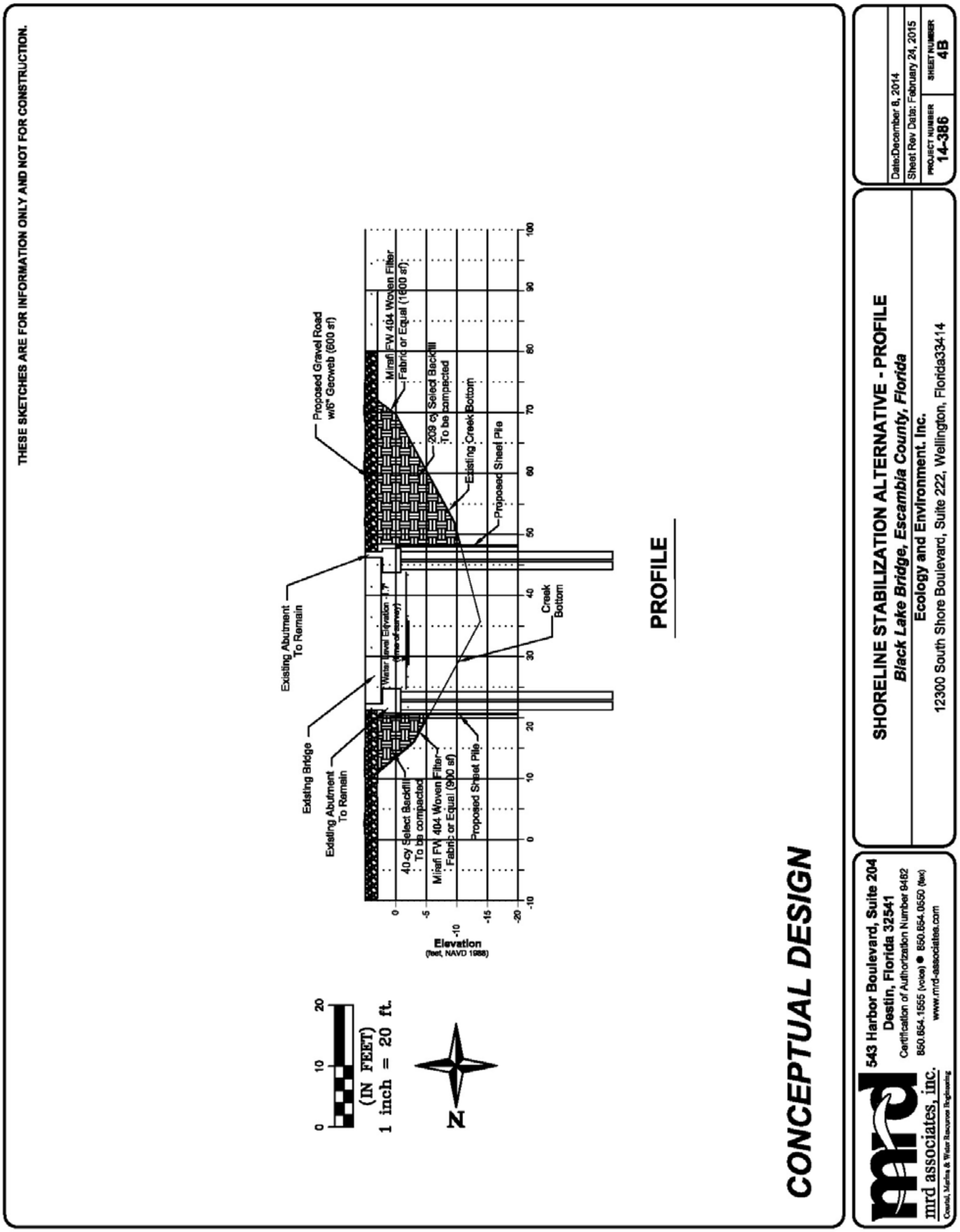

Figure 10. Alternative A – Maintain existing bridge and protect abutment profile view.

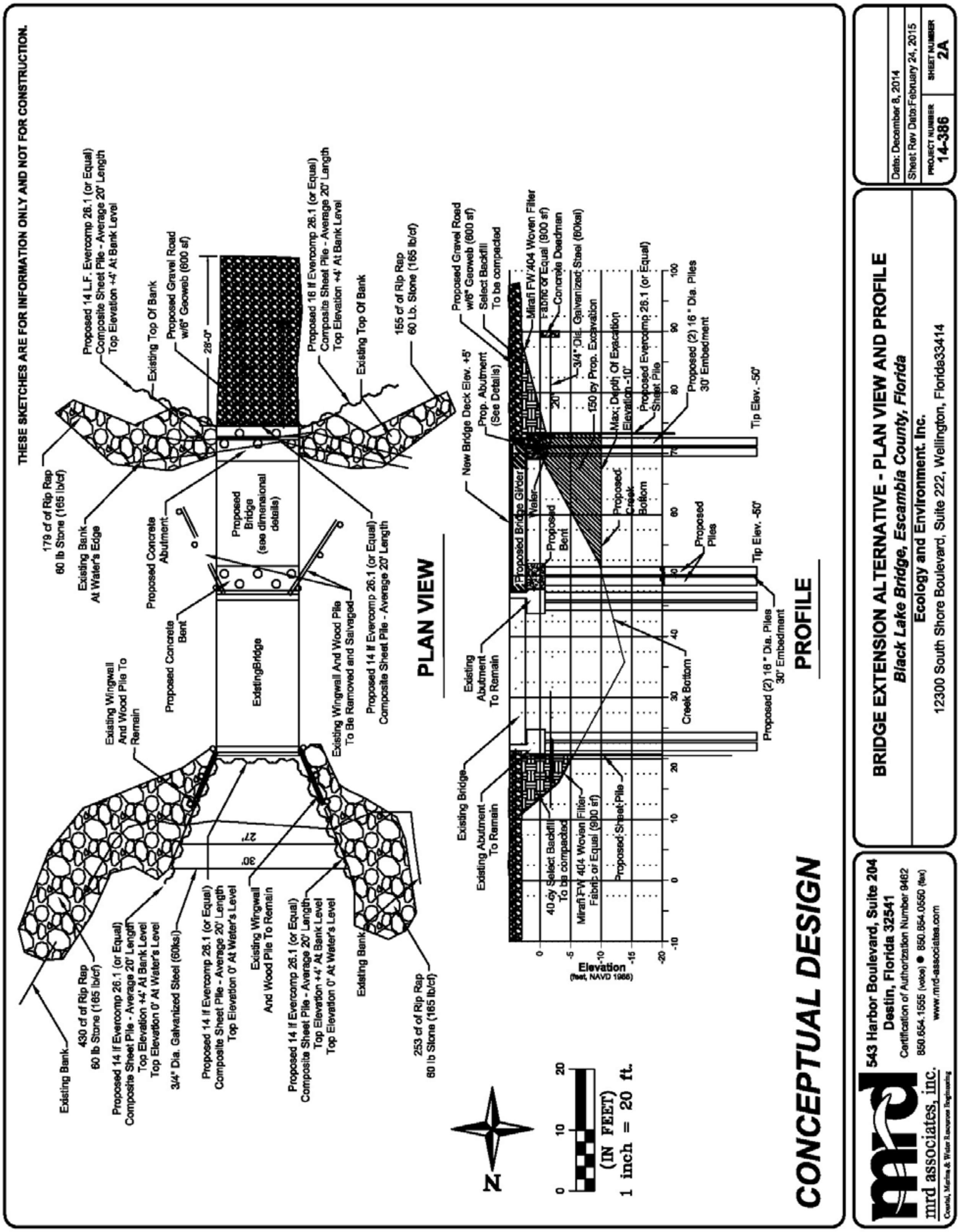

Figure 11. Alternative B – Extend bridge and protect abutment plan and profile view.

Alternative C: Extend Existing Bridge with Railroad Car Frame and Protect Abutment

This alternative includes extending the existing bridge with a long span section of a railroad car frame and would provide an increased level of abutment protection on both the north and south ends of the bridge, as proposed in the previous alternative. The proposed purchase of a railroad car has several limitations, such as reliable data and information as to the structural characteristics of the car frame for design, level of corrosion that would have to be treated, and transport costs, etc. If corrosion is found, sand blasting and cathodic protection and painting would have to be completed before use. Photographs can be provided by the supplier, but a visit to the location of the frame is the only method to ensure the condition of the railroad car. The closest available railroad car supplier is in Arkansas and a visit would add additional expense to this alternative. Once acquired and delivered, the railroad car would have to be retrofitted to adhere to existing field conditions and support design. This may require the attachment of additional supports. The width of a railroad car is 8.5 feet, but the existing bridge is 13.5 feet wide. Retrofitting a railroad car to a 13.5-foot width could compromise the structural integrity of the retrofitted frame. If the railroad car option is selected, supports such as a new abutment and bent would still be required to be constructed as proposed in the bridge extension alternative (Alternative B). However, if two railroad cars were used (for a width of 17 feet), the existing bridge (abutment and bent) would have to be modified (widened) at an additional cost. Based on our research, the railroad car alternative would be more costly and take more time to prepare and construct than extending the bridge as proposed in Alternative B.

Alternative D: Install Large Concrete Box Culverts and Protect Abutment

This alternative, shown on Figure 12, includes installing two 12 feet by 12 feet concrete box culverts as part of the south bridge approach within the gap between the existing bridge and the south eroded shoreline. Ten 40-foot-long piles and two concrete caps would be required beneath the structure to support and stabilize the culverts. The southern side of the culvert would retain the soils along the eroded bank, thereby reducing the quantity of sheet pile along the embankment. The eroded southern bank would require excavation to place the culverts. The excavated material would be used to backfill (185 cubic yards between the culvert and excavated bank) and then compacted. Geoweb and gravel stone would be placed on top of the compacted fill to finish the driving surface from the newly excavated bank to the existing bridge. Approximately 30 linear feet of sheet pile would be installed to overlap the culvert and placed along the adjacent shoreline to retain the upland soils and provide a level of scour protection. Instead of “anchoring” the wall, this alternative assumes the sheet pile would be cantilevered, thus eliminating the need for tie-rods or concrete deadman and waler. It would first be necessary to wrap any gaps or voids between the culvert and new wall with filter fabric to protect against the loss of sand. As proposed in the previous alternatives, approximately 38 linear feet of sheet pile would be installed, and 40 cubic yards of fill would be placed on the north side to contain the fill and extend the flanking protection to the shoreline and bank. Approximately 972 cubic feet of rip rap stone with a minimum density of 165 pounds per cubic foot is proposed to protect against a high flow event that would generate velocities up to 10 feet per second. The rip rap stone would be placed at the toe and edges of the wing walls and shoreline.

The benefit of this alternative is that it would provide additional cross sectional area for the conveyance of creek flow and reduce the velocity through the bridge and culvert section during storm events. Having the culverts supported on piles provides structural stability for the culverts. The south end wall of the culvert would act as a retaining wall, thereby eliminating 14 linear feet of sheet pile. Conversely, the culvert would need to be structurally supported and anchored by a pile supported concrete cap. In addition, the contractor would have to install a cofferdam system to construct in a dry environment, which is an additional cost.

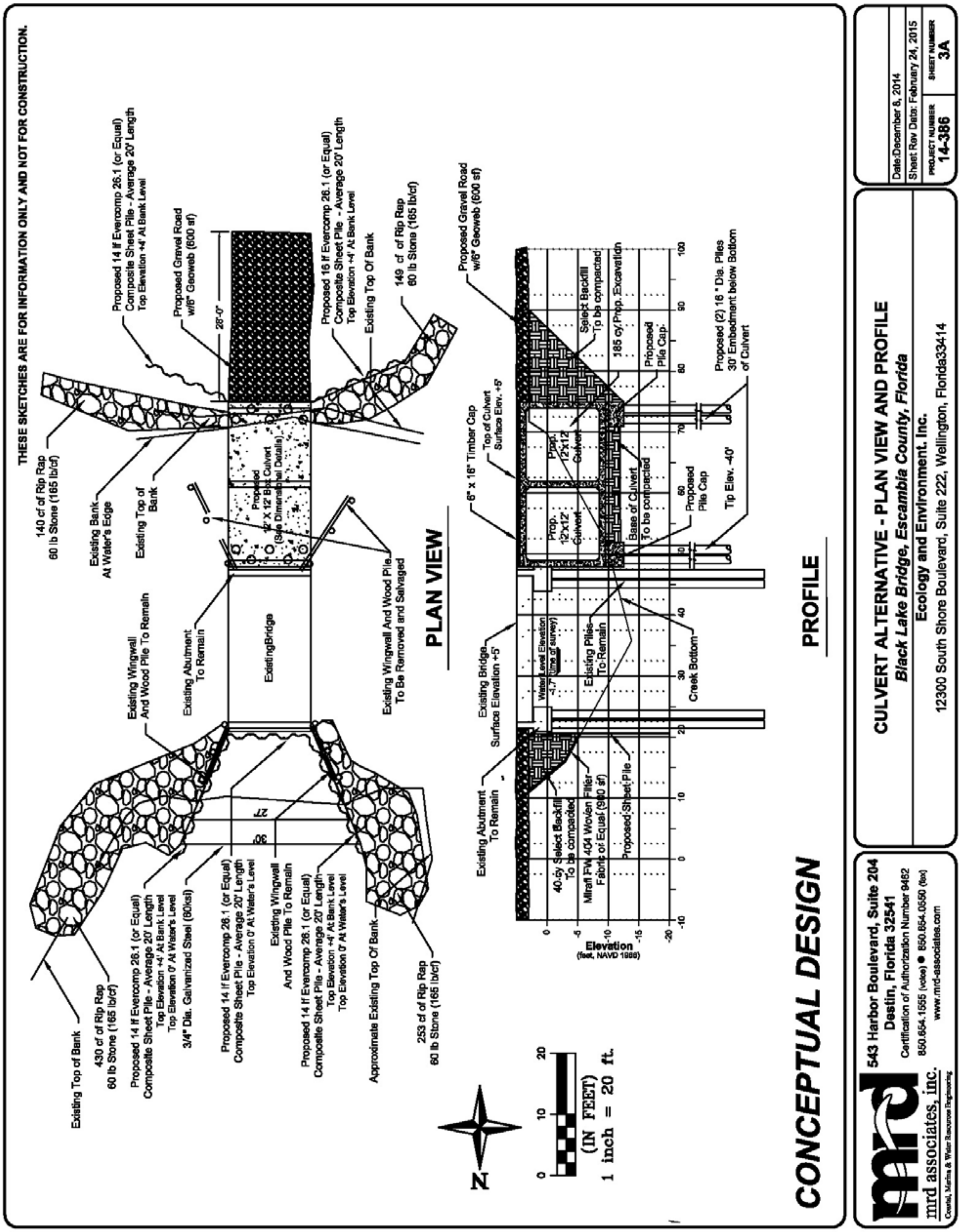

Figure 12. Alternative D – Install large concrete box culverts and protect abutment plan and profile view.

Alternative E: Utilize Bioengineering Techniques

Hydraulic countermeasures using bioengineering techniques fall into two groups: (1) stream/river training; and (2) bank armoring. Stream/river training can involve transverse structures (e.g., spur dikes and bendway wiers), longitudinal structures (e.g., fiber rolls, vegetated gabion baskets, vegetated mechanically stabilized earth, and live brush layering), and channel planform measures (e.g., vegetated floodways and meander restoration). Stream/river training techniques are used to help direct the current away from an area that is eroding, such as the bridge abutments. Bank armoring using bioengineering approach involves groundcovers (e.g., vegetation alone, willow posts, live fascines, and vegetation reinforced mats) and revetments that integrate more natural materials (e.g., root wads, live brush mattress, vegetated articulated blocks, and gabion mattress) compared to traditional engineering design consisting of solely rock revetments or vertical sheet pile.

A decision support tool developed by the National Cooperative Highway Research Program (2005), called “Greenbank,” that allows input of 11 specific environmental attributes of the project site and characterization of the erosion can be used to help classify the spatial extent of the erosion problem. The software decision support tool, along with stream restoration experience, then provides technique recommendations for stream/river training and/or bank armoring.

Based on field observations, further assessment of topographic and bathymetric data, and conceptual design requirements, bioengineering techniques may have limited viability for this site. Aside from the immediate area of the bridge abutments, the creek banks are pristine; therefore, measures that require the staking of features into the bank or on top of the bank would be too disruptive. Within the creek channel, the banks are very steep and the top of the bank is near the high tide mark. Thus, there is no natural shelf to install bioengineering features for either stream training or bank armoring. In order to construct a shelf, fill would have to be placed in the natural creek channel, potentially up to 100 feet upstream of the creek channel and 50 feet downstream of the creek channel to facilitate bank armoring. Also, the storm flow velocities that have been estimated to be over 10 feet per second are likely to be in excess of those that can be withstood (or documented to be withstood) by most bioengineering methods. For example, a combination of fiber roll, root wads, and brush layers have been found to withstand floods with velocities ranging between 1.23 to 3.11 feet per second. Live stakes, brush layers, and root wads have been found to withstand over 5 feet per second. Maximum velocities sustained by bioengineering techniques do not exceed 10 feet per second (Allen and Leech 1997). While the bioengineering methods can be helpful in reducing velocities due to increased drag, the amount of that reduction is not well defined. Increasing the bridge opening will reduce velocities, potentially to below an acceptable threshold; nonetheless, the steep banks and lack of a shelf above the normal mean high water line hinder the installation of potentially effective bioengineering measures.

Options to provide a broad flow way to redirect some of the high water creek flows were reviewed. However, there does not appear to be any readily viable alternative flow way options based on the topographic data other than the existing low- water crossing on the north bridge approach. An option to lower the elevation of the earthen low water crossing was also reviewed. Lowering the crossing would allow more flow through this flow way and enable water at a lower stage to bypass the bridge opening. This, in turn, would reduce velocities in the creek at the bridge opening. Based on a scenario of lowering the grade 0.5 foot to an elevation of approximately 1.0 NAVD 88, the estimated flow velocities for the various storm events were reduced by only 4 percent to 8 percent; thus, there was minimal effect. The grade can be lowered even further, potentially to an elevation of 0.5 NAVD 88; however, we are hesitant to lower it much more without knowing the frequency of the water stages since it may not be desirable to have the crossing regularly overtopped.

PLANNING LEVEL OPINION OF PROBABLE CONSTRUCTION COSTS

HHH Construction (HHH) was contacted as directed by TNC to provide a planning level opinion of probable construction costs (planning level costs) for input for several of the options, specifically:

- Alternative A (Maintain Existing Bridge and Protect Abutment);
- Alternative B (Extend Existing Bridge and Protect Abutment); and
- Alternative D (Install Large Concrete Box Culverts and Protect Abutment).

No planning level costs were to be developed for Alternatives C (Extend the Existing Bridge with Railroad Car Frame and Protect Abutment) and E (Utilize Bioengineering Techniques) due to technical limitations or pre-determined cost implications.

HHH focused on the planning level cost for Alternative A, which was estimated to be $147,000. It is notable that HHH also provided a second planning level cost for a revised Alternative A that included deeper and longer sheet piles similar to what TNC was initially envisioning. The deeper sheet piles resulted in an estimated construction cost in excess of $200,000. HHH indicated that Alternative B and Alternative D would be more expensive than Alternative A; however, a cost breakdown was not developed for these alternatives. Subsequent planning level construction costs for Alternative B, provided by CONTECH Engineered Solutions (CONTECH) indicated that Alternative B might cost only $110,000 to $120,000. CONTECH also looked at different options for the box culvert using a variety of their prefabricated bridge systems, but determined these options were not feasible; thus, no further costing assessment was conducted for this alternative. While there is a discrepancy in costs between HHH (much greater than $147,000) and CONTECH ($110,000 to $120,000) for Alternative B, we believe the total costs for Alternatives A and B may end up being similar. For instance, any savings in reduction in the amount of sheet pile and back fill from Alternative A to B will be offset by the bridge extension (Alternative B). In either case, the costs will exceed the budget that TNC currently has for the abutment repair.

RECOMMENDATIONS

Given the site conditions, project goals, and need for a reasonable degree of abutment protection, Alternative B (Extend Existing Bridge and Protect Abutment) would be the most appropriate option. Increasing the bridge length is more desirable since the constriction at the bridge would be reduced and river current velocities under the bridge would be reduced compared to the pre-April 2014 constricted condition. Extending the bridge to the south would also locate the new abutment further toward the bend of the river, which would be beneficial in flow entering the bridge opening. This and installing sheet pile and rip rap would reduce the potential for flanking. Some value engineering could be conducted in the design phase in an effort to reduce costs. Options to use rip rap in lieu of sheet piling could be considered. However, if TNC just needs a temporary fix, then other less protective measures could be discussed with TNC and implemented.

In addition to the above measure, TNC may want to consider replacing damaged or non-functional culverts that help drain the watershed and reduce discharges into the creek. Also, depending on the extent of logging and where it occurs, there could be some merit to installing check dams at key points in the upland areas to reduce peak discharges into the creek. The benefit of implementing these is not known at this time and is beyond the current scope of work and most pressing need.

REFERENCES

Allen, H. and J. Leech. 1997. Bioengineering for Streambank Erosion Control. Report 1 Guidelines. USACE waterways Experiment Station Technical Report EL-97-8. 105 pages. Available on line at http://www.engr.colostate.edu/~bbledsoe/CIVE413/Bioengineering_for_Streambank_Erosion_Control_report1.pdf.

CBC Engineers & Associates, Ltd. (CBC), 2010. Design of Abutments on Driven Timber Pile Foundations, and Design of Timber Pile and Lagging Retaining Walls for a Proposed Vehicular Bridge; Black Lake, Escambia County, Florida; CBC Report No. 12093D-1-0310-05. March 5, 2010.

CONTECH Bridge Solutions, Inc. (CONTECH). 2009. Report for Job #402679, Beulah, Florida: Dam Bridge for Black Creek, 12'0" x 24'-0" Steel Bridge prepared by M. Watts. October 11, 2009.

Florida Department of Transportation (FDOT), 2001. Precipitation Intensity Duration Frequency Curves.

National Aeronautics and Space Administration (NASA), 2014. Goddard Earth Sciences Data and Information Services Center News. Record-shattering rain inundates Pensacola, Florida in late April 2014. Accessed December 5, 2014. http://disc.sci.gsfc.nasa.gov/gesNews/april_rain_floods_Pensacola

National Cooperative Highway Research Program, 2005. Report 544: Environmentally Sensitive Channel- and Bank-Protection Measures (Greenbank Decision Support Tool). Accessed November 20, 2014. http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_544.pdf

United States Geological Survey (USGS), 2014. The StreamStats Program Application Description. Accessed December 5, 2014. http://water.usgs.gov/osw/streamstats/ssinfo1.html

Perdido River Habitat Restoration

		Task 1	Task 2	Task 3	Task 4	Task 5	**Total**
	Schedule	*Month 1-3*	*Month 4-12*	*Month 12-36*			
Personnel							
	TNC - Planning 110 hours@$32.46	$ 3,571.43					$ 3,571.43
	TNC - Monitoring 220 hours@$32.46			$ 7,142.86			$ 7,142.86
ringe Bene its							
	TNC - Planning	$ 1,428.57					$ 1,428.57
	TNC - Monitoring			$ 2,857.14			$ 2,857.14
Contractual							
	MRD Associates - Planning/Design	$ 12,000.00					$ 12,000.00
	MRD Associates - Permitting	$ 8,000.00					$ 8,000.00
	Contech - Construction		$ 215,000.00				$ 215,000.00
ndirect							
	NICRA @ 21.8%	$ 5,450.00	$ 46,870.00	$ 2,180.00			$ 54,500.00
Total		$ 30,450.00	$ 261,870.00	$ 12,180.00			
Le eraged unds	TNC Alternatives Analysis				$ 13,900.00		
	USFWS Restoration Funds				$ 25,000.00		
	TNC Match to USFWS Funds				$ 25,000.00		
Total Le eraged unds							

1. Describe the scope of each task in more detail.

2. Provide budget details and justification for personnel requests, inlcuding number of labor hours and rate per hour.

3. Provide additional detail to justify travle costs, inluding destination, purpose for the trip, and costs details.

4. Provide additional budget detail to justify materials and supplies, including the purpose and the cost per unit.

5. Provide an estimated schedule for each task (see example above).