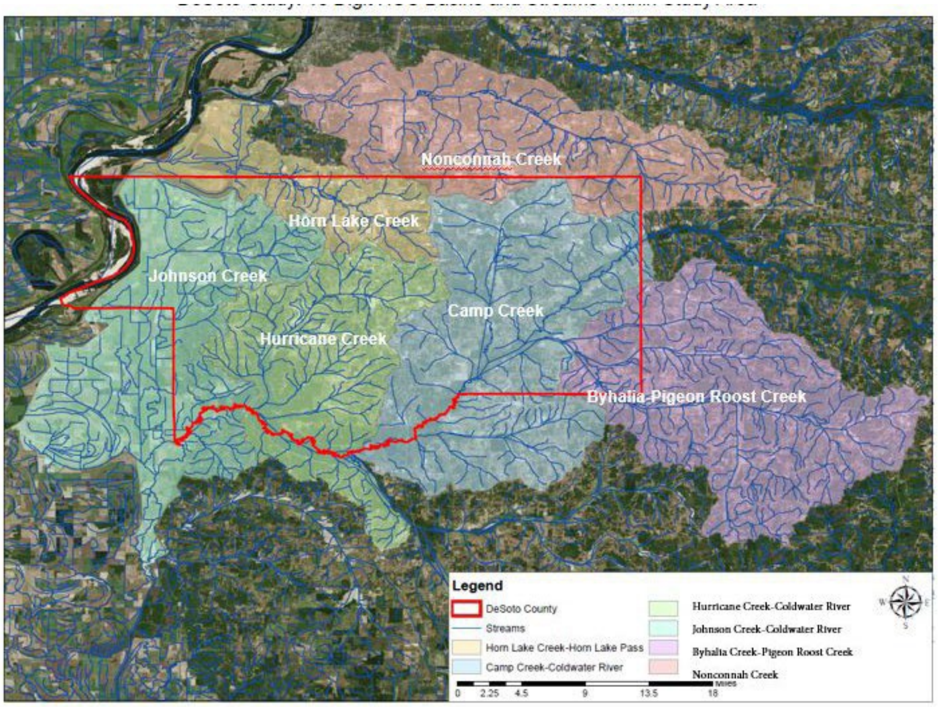
Mississippi Valley Division,
Regional Planning and Environment Division South

Memphis Metropolitan Stormwater – North DeSoto County Feasibility Study, DeSoto County, Mississippi



Revised Draft Feasibility Study with Integrated Supplemental Environmental Impact Statement Appendix I - Design

June 2023

Section 1
General Overview

1.1 BASIN DESCRIPTION

The study area includes the watersheds of Horn Lake Creek, Hurricane Creek, Johnson Creek, and Coldwater River in DeSoto County, MS. DeSoto County is in northern Mississippi and is a part of the Memphis, TN, Metropolitan Statistical Area. The most significant flooding impacts occur within Horn Lake and Camp Creek. Horn Lake Creek is a tributary to the Mississippi River. The creek drains 54 square miles and flows generally northwest for 26 miles before joining the Mississippi River at Horn Lake in Memphis.

The terrain consists of gently rolling forested hills, with relief generally less than 50 vertical feet. The area is highly developed – the county is the most populous in the state of Mississippi – with two interstate highways and three rail lines running through the study area.

The study area is densely developed with a mix of suburban-scale residential, commercial, and light industrial development.

1.2 ENGINEERING PROBLEM STATEMENT

Mitigate the impact of short-duration high-volume headwater flooding within DeSoto County.

1.3 HYDRAULIC DESIGN CONSIDERATIONS

Historically, the forested watersheds conveyed runoff through flat alluvial creeks to larger waterways – Johnson and Hurricane creeks to the Coldwater River, and Horn Lake to the Mississippi River. As the region has developed, many tributaries were channelized first to drain land for agriculture, then to dry land for development. Land was developed for residential and commercial uses right up to the edge of the floodplain, and sometimes on fill extending into the floodplain. Changed land use coverages increased runoff, and channelized creeks brought tributary water together faster. This is most noticeable at “Bullfrog Corner” in Horn Lake. As a result, flooding in DeSoto County tends to develop rapidly from headwater events, but durations of flooding tend to only last 12-36 hours.

1.4 CIVIL DESIGN CONSIDERATIONS

The gently sloping streams theoretically offer the opportunity to detain water above areas where damages are occurring. However, the narrow streams and close proximity to development restricts the size of any detention structure.

Existing infrastructure further constrained design. Sewer lines run through the creek valleys and provide additional challenges to siting measures. Initial cost screening determined that large-scale relocations of sewer lines would be unaffordable given the benefits offered by the proposed detention sites. Roadways and the Canadian Northern (CN) railroad bridge restrict flows as well and measures were templated to avoid relocating these items. Electrical and water delivery infrastructure are also widespread but do not impact proposed measures to the same scale.

1.5 DATA SOURCES

The design is based on commercially available topographic maps, aerial imagery, and LiDAR-derived digital elevation models in 2015 and 2018-2019. The data set was provided by the Sponsor.

1.6 DATA QUALITY AND INTERPRETATION

Engineering analysis for this report is based on the most comprehensive data that can be acquired within the defined, relatively compressed schedule and budget. Some of these sources have known limits to accuracy or precision that require more attentive interpretation. Limits to data quality are derived from two sources: 1) Uncertainty, where tools or calculations are unable to precisely match known values (i.e. a lack of significant digits or lack of sufficient observed data); and 2) error, where derived values precisely deviate from known actual values.

This study considered water surface elevation from model results usably accurate to ± 0.5 feet. LiDAR-derived elevation data is considered accurate to ± 0.5 feet. Model results inside a range of ± 0.5 feet and all point elevations used to justify high-risk implementation actions should be further investigated at Preconstruction Engineering and Design (PED). These additional investigations include, but are not limited to, refinement of hydraulic models and acquisition of topographic elevation data to validate the LiDAR-derived elevations used in the study. Quality by source or calculation step is discussed below.

1.6.1 Elevations Derived From LiDAR Data

Elevations were taken from a Digital Elevation Model (DEM) derived from LiDAR data. The error in this class of data averages over a wide area, making it appropriate for modeling watersheds but can be insufficient for analysis of any one discreet point. The study team deemed the dataset accurate to an average elevation of ± 0.5 feet on a 10-foot horizontal grid. Algorithms that process LiDAR point clouds into DEMs remove building and vegetation data points but can produce local error in the dataset based on how it interprets the adjacent ground level. The high number of trees, buildings, water, and utilities in the study area increase the possibility of local error. Spot checks with topographic ground shots indicated LiDAR elevations varied from -0.75 to +2.4 feet in the study area. Thus, any elevation taken at one specific point should use an accuracy of ± 2.4 feet until verified by a topographical survey.

1.6.2 Elevations Derived from Topographical Survey

Accurate to approx. ± 0.05' when properly calibrated to local US Geological Survey monuments.

1.6.3 Finished Floor Elevations Estimate

The finished flood elevation (FFE) is the top of the slab or first floor and is used to determine structural damages. Lacking sufficient field data, these were calculated by the Economics Team by adding estimates of step heights derived from images or field observations to the LiDAR elevation. Step height estimates are accurate to approx. ± 0.2 feet. Noting that any elevation errors in the LiDAR will propagate through calculations, FFE estimates should be considered accurate to only ± 2.6 feet (see discussion above). FFE that are field verified with topographic surveys will have an accuracy of ± 0.05 feet. This survey will be done prior to PED.

1.6.4 Quantity calculations

Volume calculations are based on LiDAR elevations and are estimated to be ± 20% of actual values. During PED, topographical surveys will allow increased accuracy to less than ± 10% of actual values.

1.6.5 Water Surface Elevations from HEC-RAS Modeling

Modeling produces water surface elevations in summarized in a 100-foot grid or computational mesh. This spacing generates error at small discrete points that can be interpreted through manual investigation at PED. Post-processing hydraulic model error was assessed to have a standard deviation of ± 0.5 feet, meaning a 65% probability (more likely than not) that the actual data point is within this range. This was determined to adequately support the water surface elevations and broad conclusions presented in this study (see detailed discussion in Appx. G).

Section 2

Measures and Alternatives

2.1 OVERVIEW

Several previous studies have generated alternatives. These are discussed in detail in Section 4 of the Main Report. Of note are the two alternatives from the 2005 GRR. The Team reevaluated measures from the 2005 GRR. The 25-year extended channel enlargement performed well enough to carry forward to the 2021 array. However, channel improvement was eliminated after more refined modeling indicated a low BCR. This prompted a reformulation and included a variation on the 2005 berm. The berm was strengthened to a levee and floodwall, shortened, and the diversion weir was dropped.

Table 1. Alternatives from the 2005 General Reevaluation Report.

Alternative No.	Design Plan	Location Channel Reach	Bottom Width (feet)	Side Slope	Type of Improvement
1	10-year Plan	Horn Lake Creek 18.86 - 19.39	30	1:3	Channel Enlargement with Riprap Toe Protection
		19.39 – 19.42			Transition Structure
		19.42 - 19.82	30	Vertical	Concrete U-Frame
		19.82 – 19.84			Transition Structure
		19.84 – 19.93	30	1:3	Channel Enlargement with Riprap Toe Protection
		Rocky Creek	20	1:2 to 1:3	Concrete lined in lower 50'and riprap lined upstream to 120'
		Diversion Ditch east of Hwy 51, and south of Goodman	20	1:2.5	Diversion Channel
		West Bank of Horn Lake Creek SM 18.80 – 19.91	Crown width of 10'	1:4	Berm with a Diversion Weir
		Abandoned lagoon upstream of ICRR	25 acres		Detention Basin and Environmental Enhancement
2	25-year	Horn Lake Creek 18.86 - 19.39	40	1:3	Channel Enlargement with Riprap Toe Protection

		19.39 – 19.42			Transition Structure
		19.42 - 19.82	40	Vertical	Concrete U-Frame
		19.82 – 19.84			Transition Structure
		19.84 – 19.93	40	1:3	Channel Enlargement with Riprap Toe Protection
		Rocky Creek	20	1:2 to 1:3	Concrete lined in lower 50’and riprap lined upstream to 120’
		Diversion Ditch east of Hwy 51, and south of Goodman	20	1:2.5	Diversion Channel
		West Bank of Horn Lake Creek SM 18.80 – 19.91	Crown width of 10’	1:4	Berm with a Diversion Weir
		Abandoned lagoon upstream of ICRR			Detention Basin and Environmental Enhancement

2.2 METHODOLOGY

The technical team first reviewed previous studies prepared for USACE, the sponsor, local jurisdictions, and non-governmental organizations. Measures were reviewed to verify completeness of previous analysis. New measures were developed in a brainstorming session with the project sponsor conducted in 2019, and again during a reformulation in 2021. The focus of new measures was to a) reduce damages at the 0.04 AEP (25-year) flow by attenuating or diverting flow; or b) localized protection of existing developed land. This approach led to the development of the new arrays described in the Main Report.

Large-scale dams, levees, and similar measures were considered and eliminated. Dams were eliminated for multiple reasons: 1) there is no one-structure solution for the entire basin due to multiple sources of flood water; 2) the land area required to provide meaningful benefits was larger than what could be reasonably acquired; 3) the expense of large embankment and outlet works was not justified by the anticipated benefits; and 4) the additional risk of raising WSE was not justified by the anticipated benefits. Large-scale levees were similarly eliminated because 1) development up to the top bank means there is little to no space available to place a levee without demolishing structures, and 2) the additional risk of raising the WSE was not justified by the anticipated benefits. A bypass waterway or floodway was also eliminated from consideration because it would require demolition of existing structures, significantly increasing costs.

Smaller measures such as on-site detention appeared effective but would be large in number and provide benefits on a smaller scale. This scale of solution is better implemented by local entities or other federal programs. Thus, the PDT focused initial efforts on identifying multiple channel improvement and/or detention measures that could be combined to provide a 0.04 AEP level of protection.

Sites were screened and selected for further analysis based on potential to reduce damages. Site design focused on using the existing terrain as much as possible, minimizing impacts to existing infrastructure, and balancing benefits and risks of improvements. For instance, detention basin footprints were laid out to take advantage of existing high ground and avoid the cost and risk of an above-ground embankment. Channel improvements, detention, and NER measures were laid out in a manner that would avoid costly relocations of utilities, impacts to existing development, or adverse effects to existing infrastructure. Detention basins were kept mostly in ground to avoid cost of constructing to levee or dam design standards around the entire perimeter, and additional risk from elevating the water surface elevation above the 0.01 AEP line.

The team considered similar projects in the region. The Memphis District maintains a large inventory of drainage ditches and had many examples of enlargement and revetment in similar contexts from which to draw. For detention, smaller dry and wet facilities are found within the region. In particular, the Purple Creek detention facility in Ridgeland, MS, was seen as a close proxy to what could reduce flood in the study area. It is similar in its urban context, terrain, and soils.

Environmental features were analyzed separately by ERDC with support from the Team and are included in Appendixes B and C.

2.3 LIMITATIONS

Existing development is the most significant limitation to structural measures in the study area. Development crowds the floodplain and limits the ability to store floodwater either inline or offline. It also means there is no physical space to divert floodwater around development in a designated floodway.

The team concluded early in the study that large-scale levee protection was less desirable due to higher maintenance costs, higher risk compared to subgrade measures, and continued flood insurance costs for residents. This focused the Team on exploring opportunities to detain water upstream of damage areas.

2.4 ASSUMPTIONS

Key assumptions made by the technical team for this report include

- Soils will be suitable for detention and berms with minimal improvement. A typical slope of 3H:1V is typical in the region and was used pending more detailed geotechnical analysis.
- Disposal of excess fill can be made either within the project footprint, or a haul distance less than 5 miles.
- HEC-RAS model is suitable for screening new alternatives. During feasibility, modeling was refined from 1D to 2D, provide confidence for the conclusions in this report, but not necessarily for implementation. Supplemental data, particularly topographic data, will be needed to support implementation decisions.
- LiDAR is acceptable for reconnaissance-level design.
- Existing utilities should be avoided; relocations of sewer and gas lines are prohibitively expensive and shall be minimized or avoided.
- Demolition of existing structures is typically not justified for the anticipated benefits and should be avoided.

2.5 MEASURES CONSIDERED, NOT CARRIED FORWARD

Brainstorming produced numerous potential measures. These were evaluated based on potential performance, costs, environmental considerations, and suitability to the project sponsor. Many proposals dropped out due to site limitations, poor performance, or utility conflicts. At the end of this exercise, the Team identified seven discrete measures that would be screened for more detailed design and evaluation.

2.6 MEASURES CARRIED FORWARD, NOT SELECTED

2.6.1 Expanded Channel Enlargement (M18.6-19.4)

This measure was adopted from the 2005 GRR. The measure was reevaluated and found to perform well enough to carry forward for further analysis. The channel engagement improved hydraulic efficiency downstream of Goodman Rd. and better conveys floodwater away from development, reducing damages. Extending the channel enlargement to the railroad overpass further improved suitability compared to the 2005 plan. The Horn Lake Creek channel enlargement will increase the bottom width to 40 feet for approximately 4,500 linear feet from Mile 18.6 to Mile 19.4, downstream of Goodman Rd. in Horn Lake, MS. The banks of the improved channel will be flattened to a 3H to 1V slope for stability. Though the 2005 proposal had continuous stone under the entire channel, hydraulic analysis determined that revetment was only warranted on the banks and toe, not across the entire bottom, and that a smaller stone gradation was acceptable. The revised enlargement and slope flattening will require 68,200 cubic yards of excavation, all of which will be disposed off-site. Approximately 21,200 tons of riprap will be placed at the slope toe to prevent scour damage. The riprap will be placed 2-foot deep at the toe and 5 feet up both banks. Only at the downstream transition will stone protection extend across the entire bottom width. The riprap will be placed over approximately 4,300 tons of filter material. The upper banks will be protected with 22,800 square yards of turf reinforcing mat.

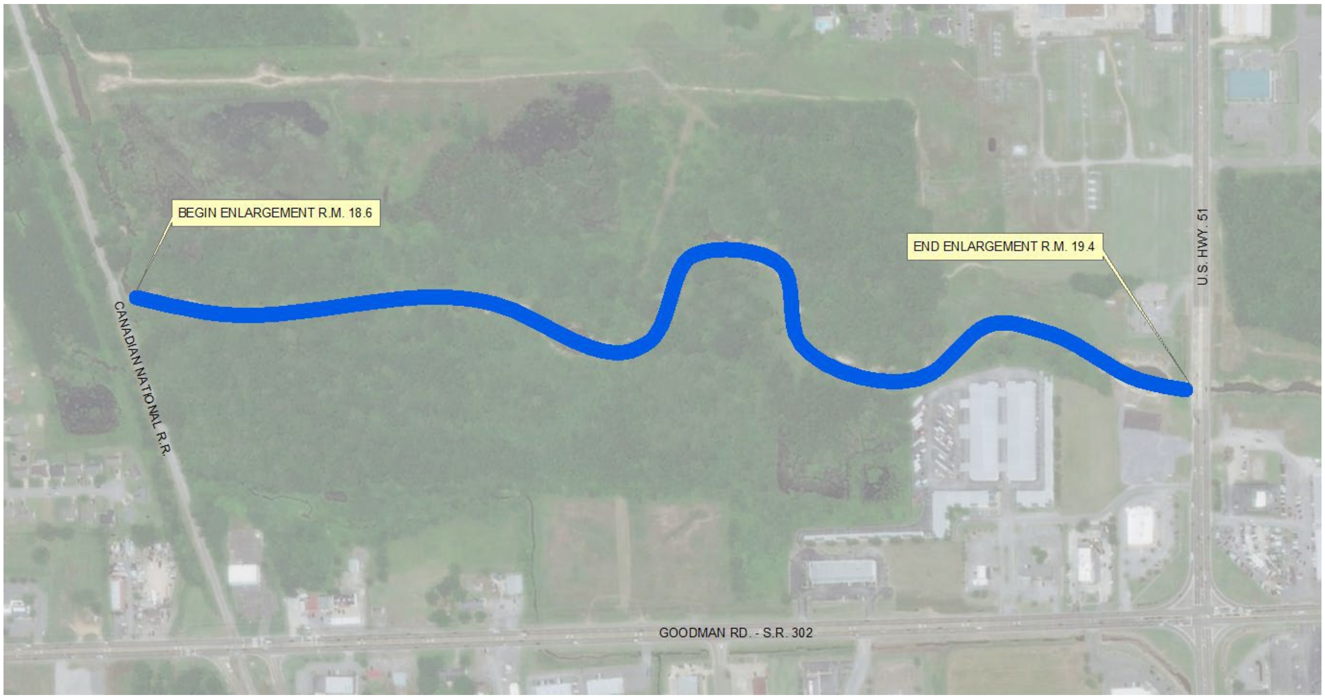


Figure 1. Expanded channel enlargement, R.M. 18.6 to 19.4.

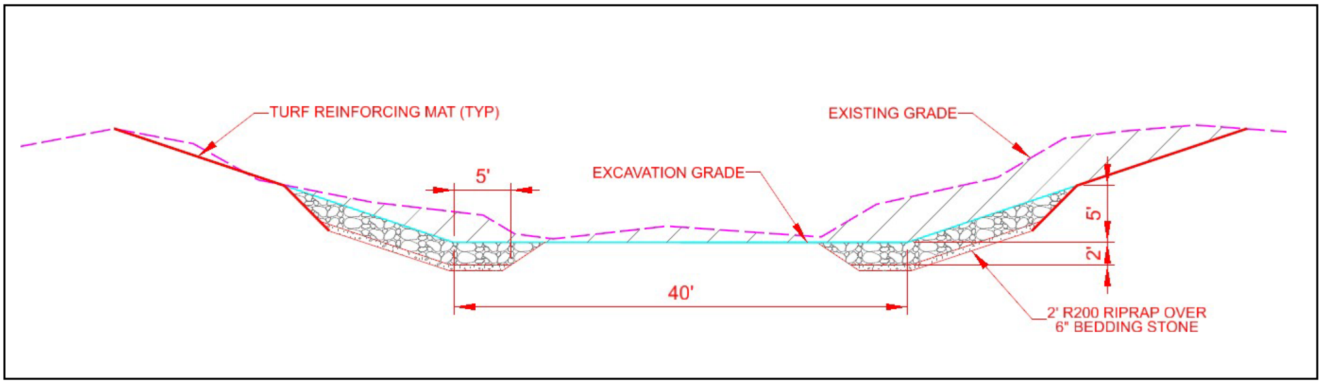


Figure 2. Channel improvement typical section.

2.6.2 Cowpen Cr. Detention - South

Two detention basins were sited within Cowpen Cr. Both basins will operate passively to attenuate the flow by storing floodwater. This will lengthen the crest but reduce the peak flows that cause damages along Cowpen Cr.

The upstream basin is a 12-acre inline detention basin south of Nail Rd. in Horn Lake, MS. The dry detention basin will have a bottom elevation of 262.0, bottom area of 10 acres, and shall be sloped back up to grade at 3H to 1V. A 500-foot-long outlet embankment will include a 48 in. dia. Reinforced Concrete Pipe outlet and 100-foot-wide overflow spillway armored with approx. 2,000 tons of riprap on the downstream side. The riprap will be placed over approximately 500 tons of filter material. A gravel-surfaced access road and security fence will be installed along the perimeter of the basin. The basin will be turfed and may include limited tree and shrub plantings at the edge of a low-flow channel. The 100-foot-wide spillway will operate at elevation 272.0, approx. at the 0.50 AEP event. The maximum storage of 108 acre-feet requires approx. 175,000 cubic yards of excavation.

Figure 3. Cowpen Cr. Detention – South.

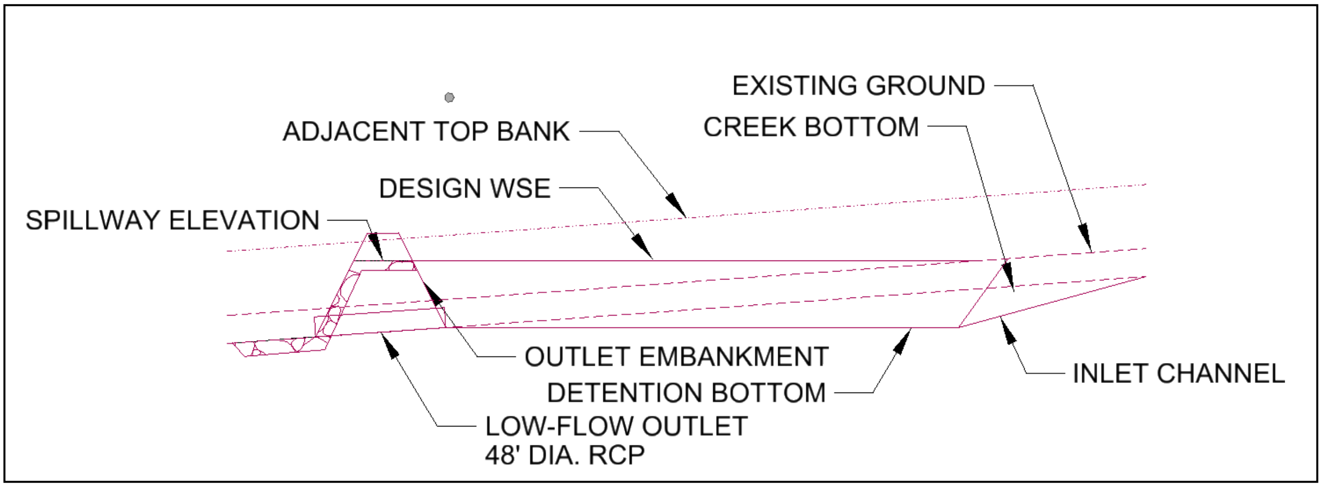


Figure 4. Typical detention profile.

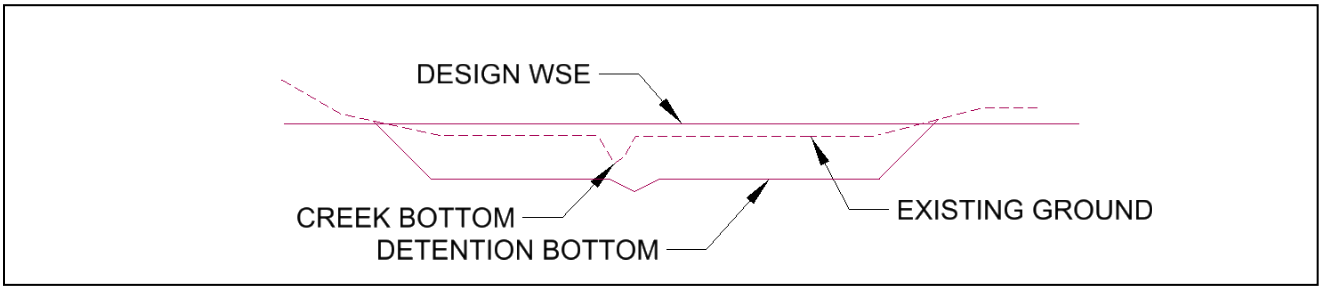


Figure 5. Typical detention cross section.

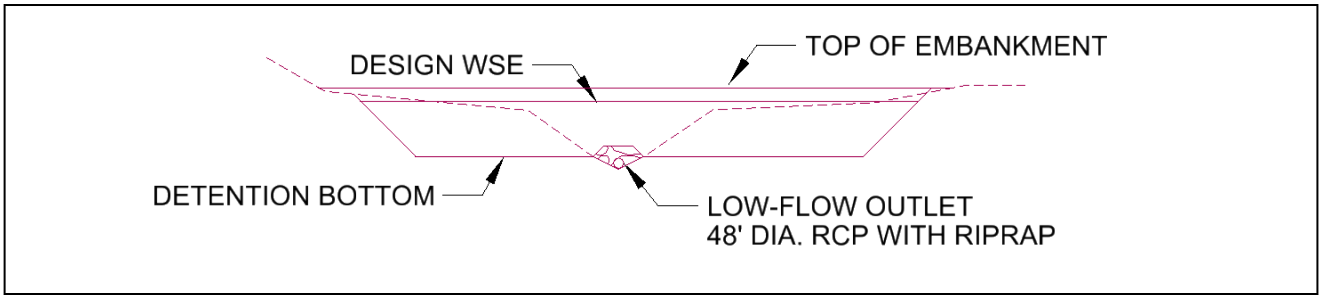


Figure 6. Typical detention outlet.

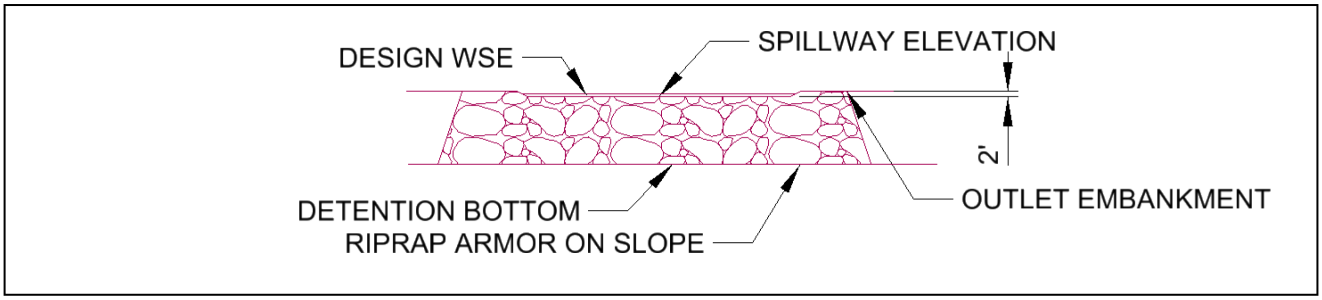


Figure 7. Typical detention spillway.

2.6.3 Cowpen Cr. Detention - North

The second basin in Cowpen Cr. will be located north of Nail Rd. at an existing ballfield complex in Horn Lake, MS. The dry detention basin will have a bottom elevation of 258.0, bottom area of 6 acres, and shall be sloped back up to grade at 3H to 1V. A 500-foot-long outlet embankment will include a 48 in. dia. Reinforced Concrete Pipe outlet and 100-foot-wide overflow spillway armored with approx. 2,000 tons of riprap on the downstream side. An inlet sill will require an additional 800 tons of riprap. The riprap will be placed over approximately 680 tons of filter material. A gravel-surfaced access road and security fence will be installed along the perimeter of the basin. The basin will be turfed and may include limited tree and shrub plantings at the edge of a low-flow channel. The 100-foot-wide spillway will operate at elevation 268.0, approx. at the 0.50 AEP event. The maximum storage of 68 acre-feet requires approx. 115,000 cubic yards of excavation.

Figure 8. Cowpen Cr. Detention – North.

2.6.4 Lateral D Detention

A 22-acre inline detention basin will be located on Lateral D south of Church Rd in Southaven, MS. This dry detention basin will have a bottom elevation of 290.0, bottom area of 16 acres, and shall be sloped back up to grade at 3H to 1V. A 500-foot-long outlet embankment will include a 48 in. dia. Reinforced Concrete Pipe outlet and 100-foot-wide overflow spillway armored with approx. 2,000 tons riprap on the downstream side. The riprap will be placed over approximately 500 tons of filter material. A gravel-surfaced access road and security fence will be installed along the perimeter of the basin. The basin will be turfed and may include limited tree and shrub plantings at the edge of a low-flow channel. The 100-foot-wide spillway will operate at elevation 300.0, at the 0.50 AEP event. The maximum storage of 177 acre-feet requires approx. 350,000 CY of excavation.

Figure 9. Lateral D detention site.

2.6.5 Rocky Creek

A nine-acre inline detention basin will be located on Rocky Creek east of Swinnea Rd. in Southaven, MS. The dry detention basin will have a single pool elevation 302.0. The pool bottom area is six acres. All slopes back up to grade shall be 3H to 1V. Downstream embankment is 500 linear feet and will include a 48 in. dia. Reinforced Concrete Pipe outlet and 100-foot-wide overflow spillway armored with approx. 6,000 tons riprap on the downstream side. The riprap will be placed over approximately 1,500 tons of filter material. A gravel-surfaced access road and security fence will be installed along the perimeter of the basin. The basin will be turfed and may include limited tree and shrub plantings at the edge of a low-flow channel. The 100-foot-wide spillway will operate at elevation 312.0 at the 0.50 AEP event. The maximum storage of 72 acre-feet requires approx. 115,000 cubic yards of excavation.

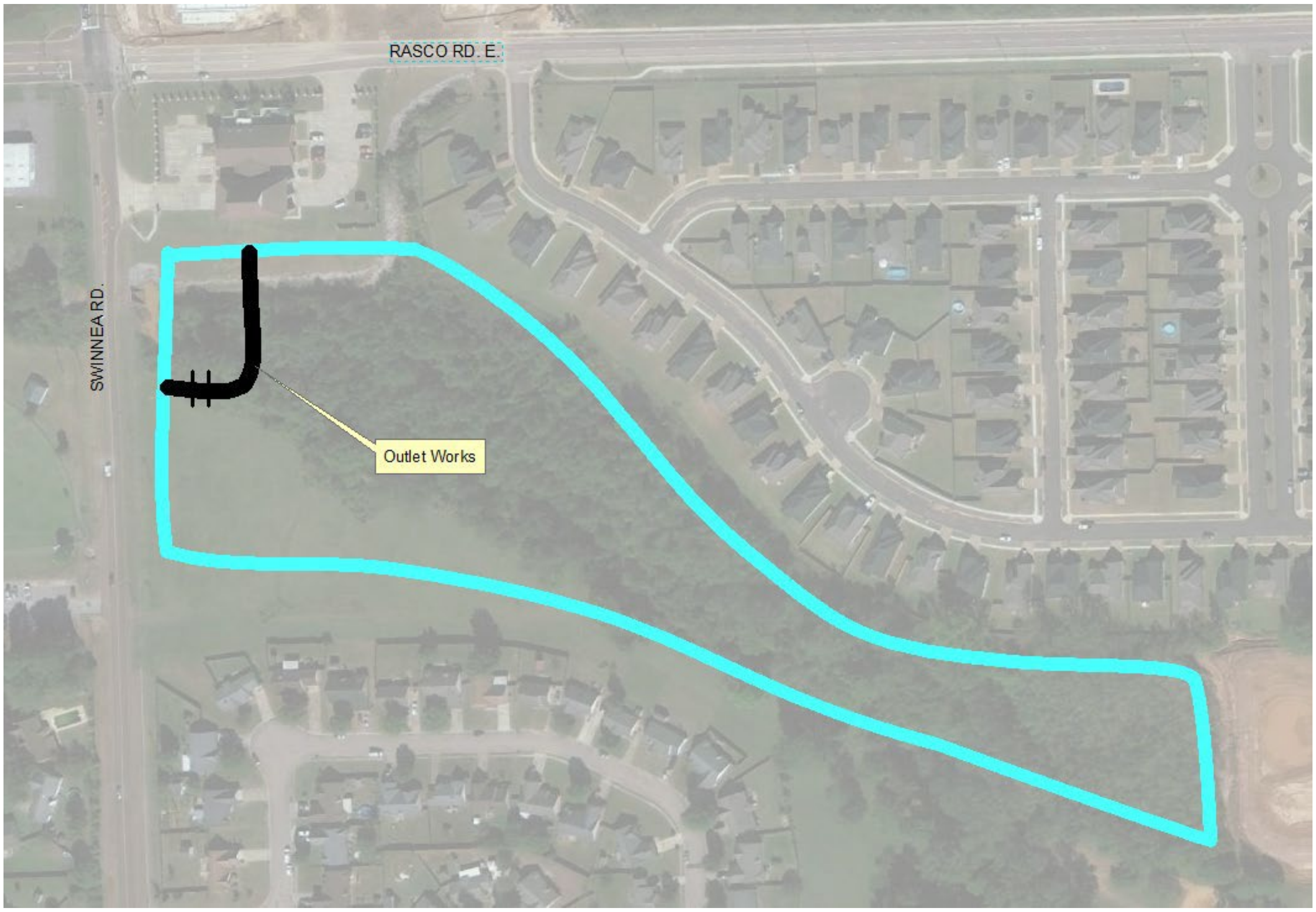


Figure 10. Rocky Cr. detention site.

2.7.3 Flood Egress Walking Path

Sutton Pl. was found to be impassable during extreme high water in both the future without project and the future with project conditions. A walking path is proposed to allow residents of the apartments on Sutton Pl. complex an emergency route out of the neighborhood via the retail shopping lot on the hill. The path starts at the eastern end of the apartment complex. The slope was restricted to a maximum of 20:1 (horizontal:vertical) to allow for ease of wheelchair, wagon, and bicycle traffic. The route requires a switchback due to the 25' elevation gain. The path then continues level past the water tower to connect to the retail parking area. See route in Figure 14, and section in Figure 15.

Table 4. Basis of design for flood egress walking path.

DESIGN CRITERIA	
Elevation	275 to 300 (+25' vertical)
Width	6'
Length	700'
Fence	420 LF, 42" heigh
Handrail	(3)2x6 boards with (2)ea. Carriage bolts
Connections	1/2" carriage bolts through post
Posts	4x4 treated wood, concr anchor 2' deep
Surface	2" asphalt
Base Course	8" crushed stone

Table 5. Flood egress walking path quantities.

Item	Quantity	Unit	Notes
Clearing	0.5	AC	
Turf	0.25	AC	
ROW	1	AC	
Asphalt	54	TN	
Crushed stone base	185	TN	
Grading	60	CY	Balanced on site
Posts	27	EA	4x4x10'
Boards	162	EA	2x6x8'
Conc.	27	Bags	2 holes per bag
Carriage bolts	324	EA	1/2"x5" with washers and nut
Erosion Control	700	LF	Silt Fence

Figure 14. Flood egress walking path plan view.

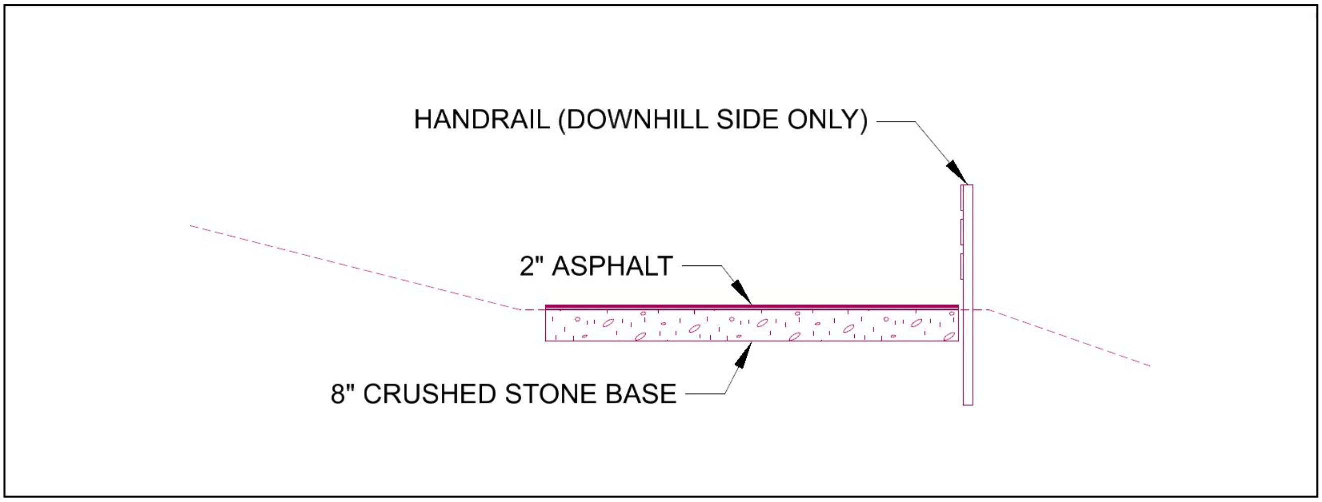


Figure 15. Flood egress walking path typical section.

2.7 MEASURES SELECTED

2.7.1 Horn Lake Creek Levee & Floodwall

After detention was found to have a poor Benefit Cost Ratio (BCR), the team was asked to reconsider alternatives previously discarded considering current costs. Large-scale levees had been screened out early, but the team targeted a cluster of flood damages west of US Hwy. 51 in Horn Lake, Miss., for localized protection. Though limited in length, the resulting barrier performed well during hydraulic modeling. Though refinement, 525 LF was designated as floodwall as the area was too narrow to construct a levee without demolishing a large commercial building.

A new 3,000 linear foot levee and floodwall system will protect structures on the left-bank of Horn Lake Creek upstream of Goodman Rd. The levee will be constructed with 3-foot horizontal to 1-foot vertical (3H:1V) side slopes and a 12-foot-wide crown. These dimensions were selected because they are consistent with similar sized levees constructed with similar soils in the region.

The levee will run approx. 2,475 linear feet adjacent to US Hwy. 51 with an average height between 5-7'. A 600-linear-foot ditch will drain a depression on the riverside of the levee. Where development makes a levee infeasible, protection will transition to a 525-linear-foot floodwall. The floodwall be 18" thick with an eight-foot-wide foundation. The wall will be 5 feet high and protrude 3.5 feet above ground level. The levee will require approx. 13,500 cubic yards of fill, and the floodwall will require 300 cubic yards of reinforced concrete. This alternative will require relocation of several utility poles and signs, removal and replacement of asphalt, and demolition of an existing vacant structure. Removal of the structure and setting back the levee will also support additional environmental habitat.

The system will tie into high ground at the upstream end. The ground rises steadily to an elevation exceeding 275.0 at the south end, and the levee tie-in will be designed such that water will be unable to flank the levee. The downstream (north) end of the floodwall will tie into the highway embankment of Goodman Rd. (S.R. 302) where it rises above 274.0. The design team will coordinate with state and local transportation officials during PED to ensure the wall is properly embedded into the embankment.

The Team initially targeted 0.04 AEP level of protection; however, the small height of the levee allowed greater protection to be provided without much additional cost. The height of protection proposed at TSP provides protection exceeding the 0.01 AEP event.

2.7.1.1 Levee Height Optimization

The levee height was run through an optimization process in accordance with ECB 2019-8 *Managed Overtopping*. The team sought to determine if there was a more cost-effective elevation that would lower risk to the levee than the NED design (considered the "baseline" condition for levee optimization). In accordance with the Bulletin, the PDT selected a location of managed overtopping. The selected overtopping reach straddled an existing ditch that would convey overtopping water through the study area. For Optimization Run #1, The levee was lowered for 300' long and the controlling elevation was set to overtop at 273.3, which corresponds to the 0.02 AEP (50-year) event. This event was selected because it still provided meaningful protection to the left bank but might reduce water surface elevations on the right bank.

The results of hydraulic and economic modeling for Optimization Run #1 determined that reducing protection on the left bank resulted in less annual benefits compared to the baseline levee ($1,383,000 vs. $1,248,000 for the baseline levee). Optimization Run #1 also did not lead to reduced water surface elevations on the right bank or reduced costs of construction. Overall, optimizing the protection to the 0.02 AEP resulted in a lower Benefit Cost Ratio (3.01 vs. 3.34 for the baseline BCR, as defined at the time of analysis).

Table 1. Comparison of results between baseline levee and optimization run #1.

LEVEE OPTIMIZATION COMPARISON		
	Baseline Levee	Optimization Run #1
Elevation	273.5-274.5	Same as baseline with a 300' fuse plug at Elev. 273.3
Level of protection	0.002 AEP (500-year)	0.02 AEP (50-year)
Total Costs	$11,586,000	$11,581,000
Annual Costs	$414,000	$414,000
Annual Benefits	$1,383,000	$1,248,000
Net Annual Benefits	$969,000	$834,000
Benefit Cost Ratio	3.34	3.01

The results also indicated that a modeling optimization run at the 0.04 AEP (25-year) event would yield far fewer benefits, while a 0.01 AEP (100-year) run would still not reduce the WSE on the right bank or reduce project costs over the baseline run. The optimization effort was terminated because it was clear none of these runs could exceed the baseline BCR. The PDT selected the baseline design elevation, which provides protection in excess of the 0.002 AEP (500-year) event. See additional discussion in Appx. G (Hydraulics) and Appx. L (Economics).

2.7.1.2 Basis of Design

Planning goals and engineering parameters that formed the basis of design for flood control features are summarized in Table 1. Quantities that result from this design are summarized in Table 2.

Table 2. Basis of design for flood risk reduction features.

BASIS OF DESIGN FOR FLOOD RISK REDUCTION	
Engineering parameters	**Planning goals**
Levee	
12' crown width (MVM best practice)	Tie into high ground both sides
3H:1H side slopes (MVM; best practice, typical for similar soils)	Gradual transitions through curves
Design elevation of 273.5 to 274.5 (MVM: developed to provide protection in excess of the 0.002 AEP (500-year) event; validated through levee optimization analysis)	Avoid utilities and limit relocations
Floodwall	
Reinforced concrete cantilver wall (MVM; based on similar sized walls)	Limit application to only locations where a levee will not fit
18" cantilever wall (MVM; bsed on simiar sized walls)	Tie into high ground both sides
12" slab 8' width (MVM; bsed on simiar sized walls)	Gradual transitions through curves
	Avoid utilities and limit relocations

Table 3. Horn Lake Cr. levee and floodwall quantities.

QUANTITIES			
Embankment	13500	CY	From InRoads volume measurement
Reinforced Concrete	300	CY	
Excavation	3000	CY	2000CY inspection trench, 400CY floodwall, 600CY ditch
Backfill	2200	CY	2000CY inspection trench, 200CY floodwall
Right-of-way	8	AC	
Temp. Const. Easement	1.5	AC	For structural demolition
Seeding	7.5	AC	
Asphalt removal	12000	SY	N. parking area: 2000 SY, bldg: 10000 SY
Asphalt replacement	2000	SY	N. Parking Lot only
Structural demolition	1	EA	15000 SF steel frame. Remove utilities.
Sign relocation	1	EA	Behind strip mall
Power Pole relocation	4	EA	Behind strip mall

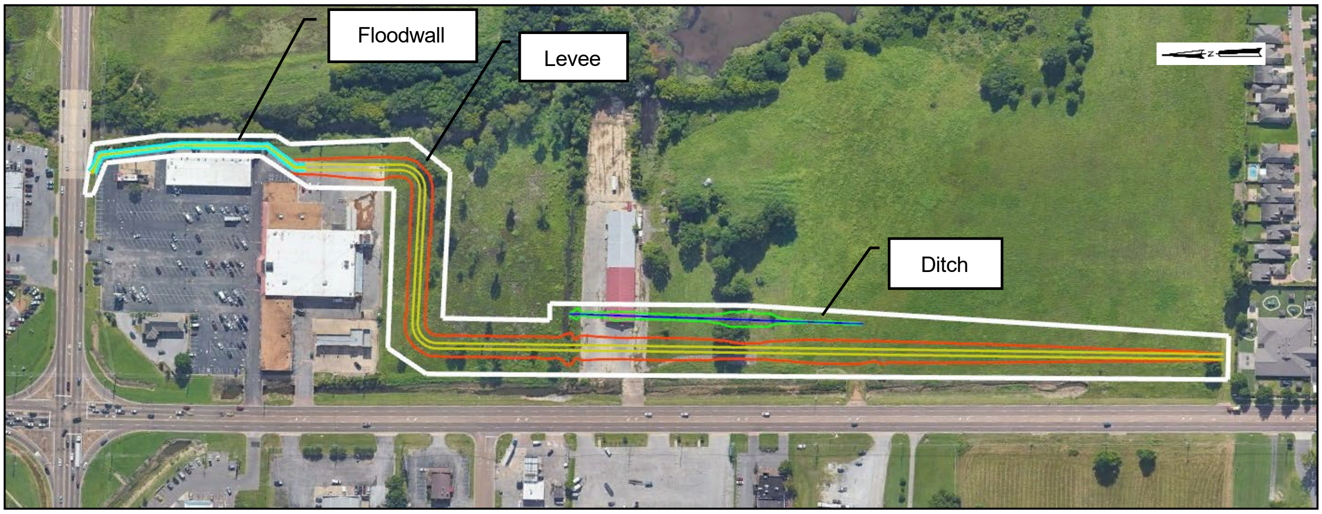


Figure 11. Horn Lake Creek Levee and floodwall, SE of the intersection of US 51 and Goodman Rd.

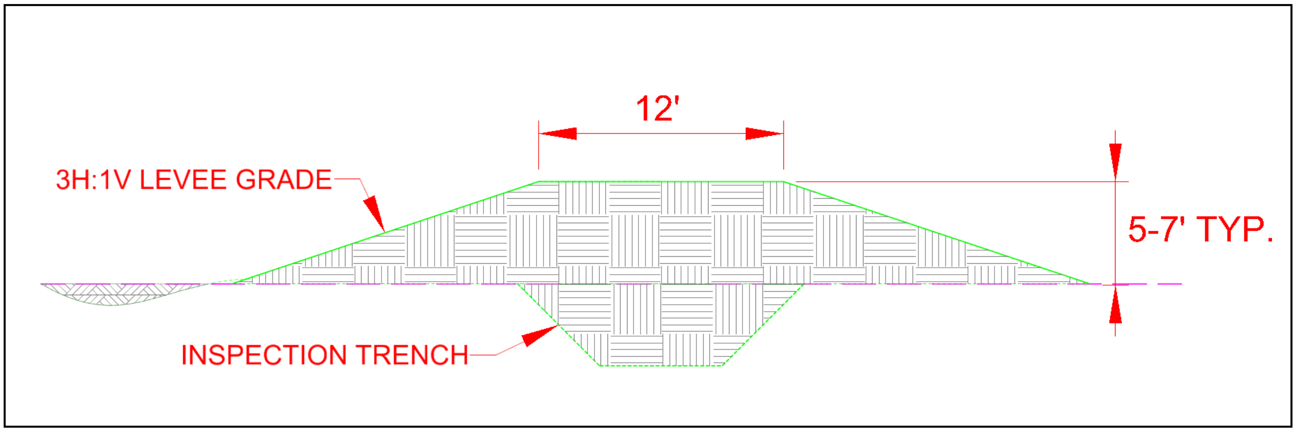


Figure 12. Levee.

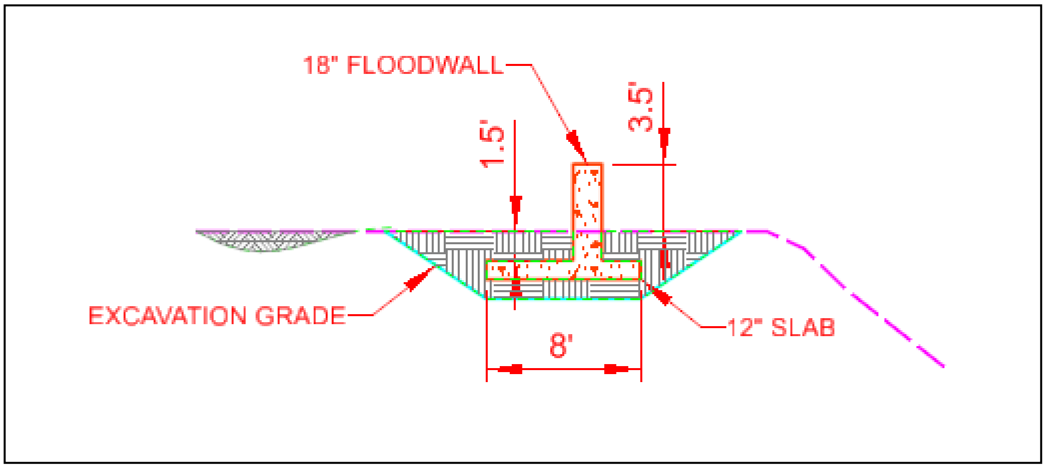


Figure 13. Floodwall.

2.7.2 Dry Floodproofing

Voluntary dry floodproofing was identified for Plans 8 and 8b. During PED, site specific designs will be completed for each structure that will participate in the project. Examples of techniques expected to be used are raising air conditioning compressors, waterproofing electrical switches, raising transformers, installing removable door barriers, and sealing siding to prevent water intrusion. An implementation plan is included in Appx. D Nonstructural Implementation Plan.

2.8 NEW MEASURES NOT CARRIED FORWARD

2.8.1 Elmore-Swinnea Detention

The team explored inline detention options on Horn Lake east of Elmore Rd. The site is large but slopes such that a single detention pool would be limited to about a ¼ of the site or require much more extensive embankment and outlet works to hold 20+ feet of water. The high-embankment option was dropped from consideration due to risk and costs. Adding smaller pools in series increased theoretical capacity from 240 ac-ft to 700 ac-ft at much lower risk. However, the volume of detention after adjusting for utilities were only marginally higher than the storage from the current forested bottomland, and the high costs of excavation made this alternative no longer economically justified.

2.8.2 Horn Lake Detention

The team explored offline detention options on Horn Lake upstream (south) of Goodman Rd. Early hydraulic modeling indicated negligible benefits at the 1.0 AEP (1-year) event, and the site was dropped from further consideration.

2.8.3 Other locations

Other county-owned sites in the Horn Lake basin were investigated for detention but were screened out from lack of benefits. Similarly, potential detention sites in Johnson Cr., Hurricane Cr., and other waterways in the county were investigated but not carried forward for lack of benefits.

2.9 NATIONAL ECOSYSTEM RESTORATION PLAN MEASURES

National Ecosystem Restoration (NER) measures were formulated by ERCD with input from the Team. Measures proposed include grade control, bank armoring, riser pipes, and riparian buffers (non-structural). Improvements were evaluated for 11 streams and 10 were selected into the NER array. Horn Lake Creek was evaluated but not selected. It is included for reference. The evaluated streams are described in detail in Appx. C. These measures provide environmental benefits such as reduced scour and deposition. These measures were not evaluated for flood risk management benefits.

Table 6. Ecosystem restoration engineered features in 11 streams evaluated within the N. DeSoto Co. study area. Note that Horn Lake Cr. was not selected into the final NER array.

Stream	Grade Control Structures (EA)	Longitudinal Peaked Stone Toe Protection (LF)	Riser pipes (EA)
Horn Lake Creek*	14	19900	12
Johnson Creek	11	6300	9
Nolehole Creek	11	5500	8
Hurricane Creek	9	2250	6
Camp Creek	7	2350	9
Nonconnah Creek	7	2000	2
Cane Creek	9	2500	4
Mussacuna Creek	3	1300	1
Lick Creek	3	2000	2
Short Fork Creek	9	3650	5
Red Banks Creek	5	2500	0
Total, all streams	**88**	**50250**	**58**
Total, without Horn Lake Creek*	***74***	***30350***	***46***

2.9.1 Grade Control

Up to 74 grade control structures (GCS) are proposed in the NER Plan. These GCSs counteract headcutting that was observed in these streambeds. Structural improvements are designed to stabilize the streambed and reduce future headcutting, as well as improve aqueous and riverine habitat. The structures will typically be 3.5 feet high from the channel bottom. Larger 650 lb. stone will face upstream, with smaller 200 lb. stone protecting the downstream side. Side slope armoring and keys will reduce the risk of flanking or undercutting the structure. This design was adapted from ERDC loose rock riffle, with additional slope armor and keys to account for the erodibility of local soils. Gradation was selected based on expected velocities. Design parameters, considerations, and examples can be found in *The WES Stream Investigation and Streambank Stabilization Handbook*.

Two cases were identified for GCSs: new grade control, and rehabilitations of existing grade control. New GCSs will be designed according to figures 14 and 15. The design will be field fit based on current topological data collected during design to achieve desired performance. The team identified a total of 20 locations where existing stone protection functions as a grade control structure, with 10 of these residing in Horn Lake Creek. Examples include riprap under bridges or riprap over crossing utilities. In these cases, a factor of 50% of the stone required for a new GCS was used as an estimate to rehabilitate these existing grade control locations. The purpose of this rehabilitation is to establish a hydraulic grade and level of protection consistent with the design described in Appx. C.

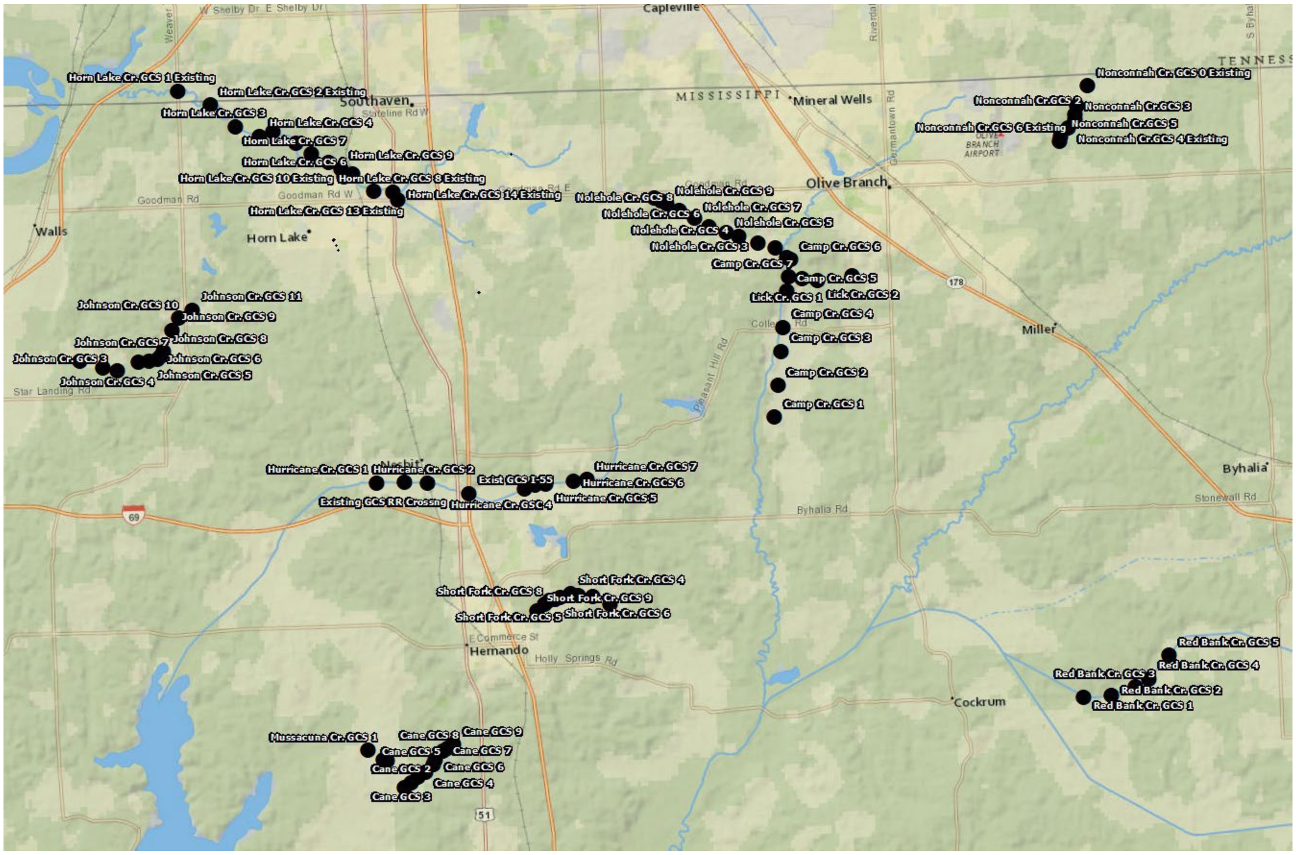


Figure 16. Location of proposed grade control structures.

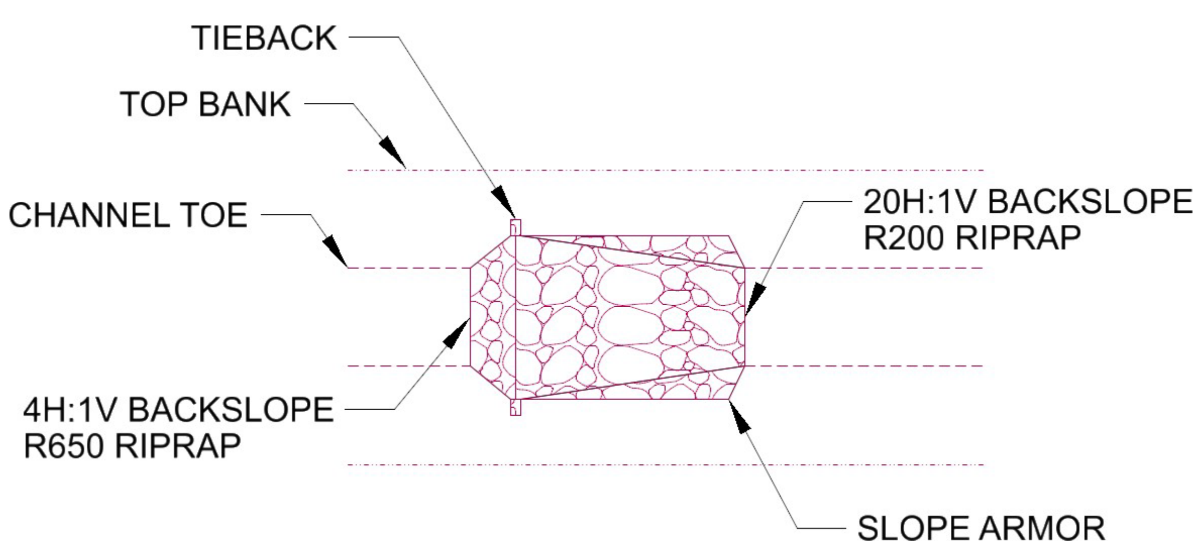


Figure 17. Grade control structure typical plan layout.

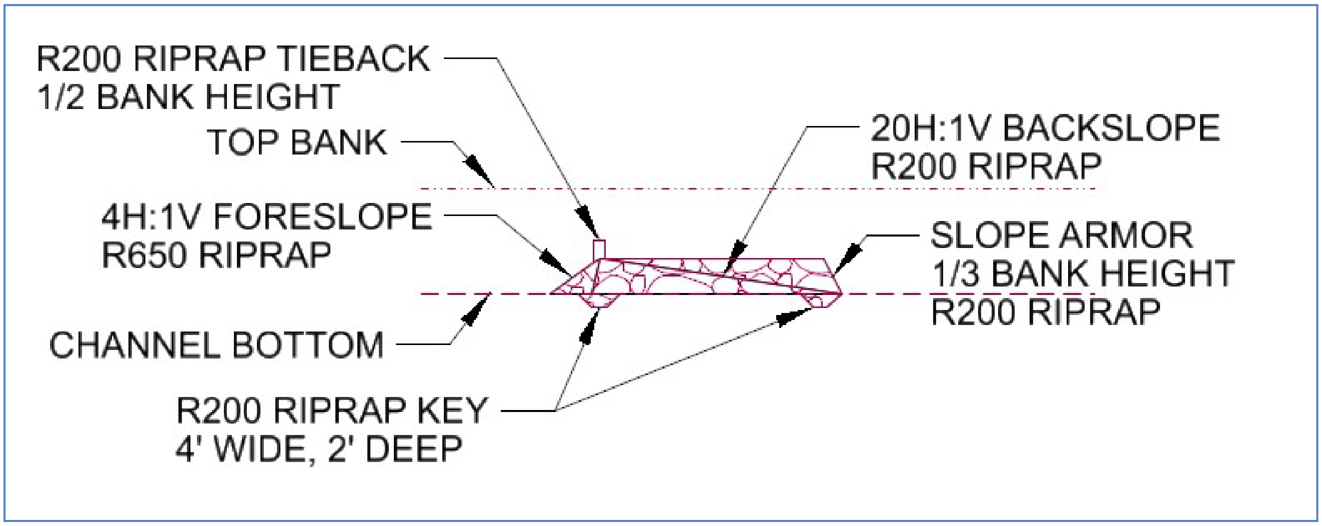


Figure 18. grade control structure typical profile.

2.9.2 Bank Protection

The NER Plan proposes approx. 30,000 LF of Longitudinal Peaked Stone Toe Protection (LPSTP) with tiebacks in the 10 identified streams. These were not located in the field but are to be placed in proximity of identified GCSs. These will reduce damages to banks and protect top bank habitat. It will also reduce the ability of the river to meander and scour into the outside bend of the stream.

The application of LPSTP consists of a windrow of stone placed at the toe of the slope, with tiebacks running up the banks at evenly spaced intervals. The toe protection provides protection against lateral erosion. The tiebacks provide a backstop to bank erosion, limiting it to the cell between tiebacks. Design parameters, considerations, and examples can be found in *The WES Stream Investigation and Streambank Stabilization Handbook*.

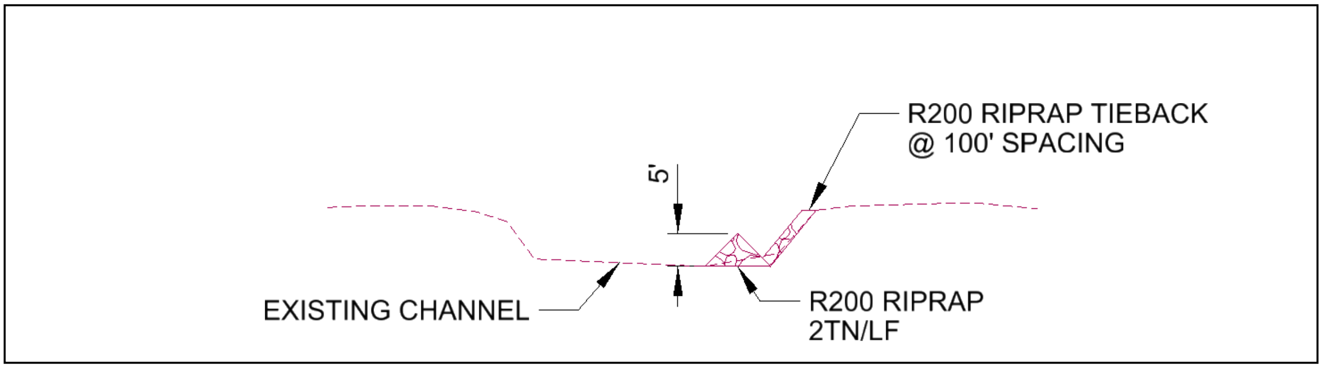


Figure 19. Longitudinal peaked stone toe protection typical section.

2.9.3 Riser Pipes

Concentrated flows can create deep incisions in the bank. Select incisions will be mitigated by installing a riser pipe to handle the grade change without scouring the bank. This will help to retain vegetation and reduce scour at these locations. Riprap at the pipe inlet will be added when warranted, and pipes will outlet onto stone toe protection. A total of 46 riser pipes were estimated, with an average length of 30 LF and diameter of 24”. The riser pipes will be similar in design to National Resource Conservation Service (NRCS) pipe drop inlets. Removable flashboards may be included if warranted. The easements required for construction and maintenance overlap with the easements required for riparian buffer and/or stone toe protection. See Appendix C for additional analysis of riser pipes.

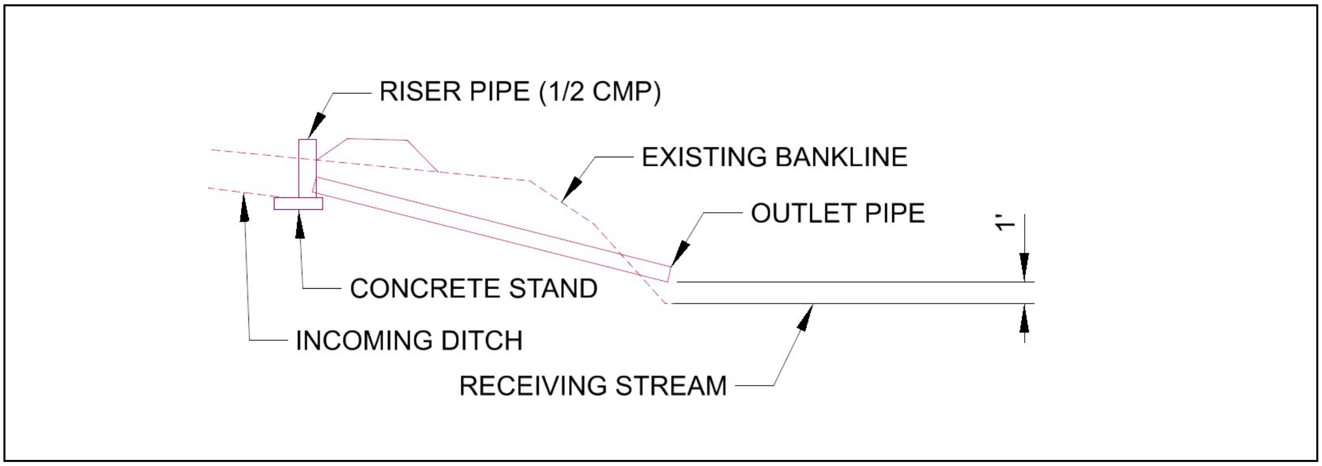


Figure 20. Riser pipe typical section.

2.9.4 Riparian Buffer

Land adjacent to the waterway will be converted to forest to provide a buffer from development and agriculture. There are no structural improvements associated with this measure; however, this could be paired with other measures to mitigate anticipated impacts.

For instance, a parcel prone to flooding may be converted to riparian buffer, reducing the risk of damage to private property. A full description of this feature can be found in Appx. A (Multi-scale Watershed Assessment Model Documentation).

2.9.5 Site Layout

The above features are combined holistically to provide environmental improvements in the watershed. The layout, quantities, and limits included are representative for cost and may be adjusted prior to implementation. LPSTP is typically located downstream of grade control but may be field fit as needed to provide the most benefit to bank stability based on field investigations during PED. Riparian buffers were generally unforested tracts selected within the FEMA floodplain. Likewise, these tracts may be adjusted to maximize environmental goals. Grade control is laid out as a system, with the next higher GCS approximately at the tailwater of the next lower structure. If one structure is field adjusted, it may affect the layout, sill height, or both of the next up-and downstream structure. Proposed layout of the ecosystem restoration features is shown in figures 19 through 29.

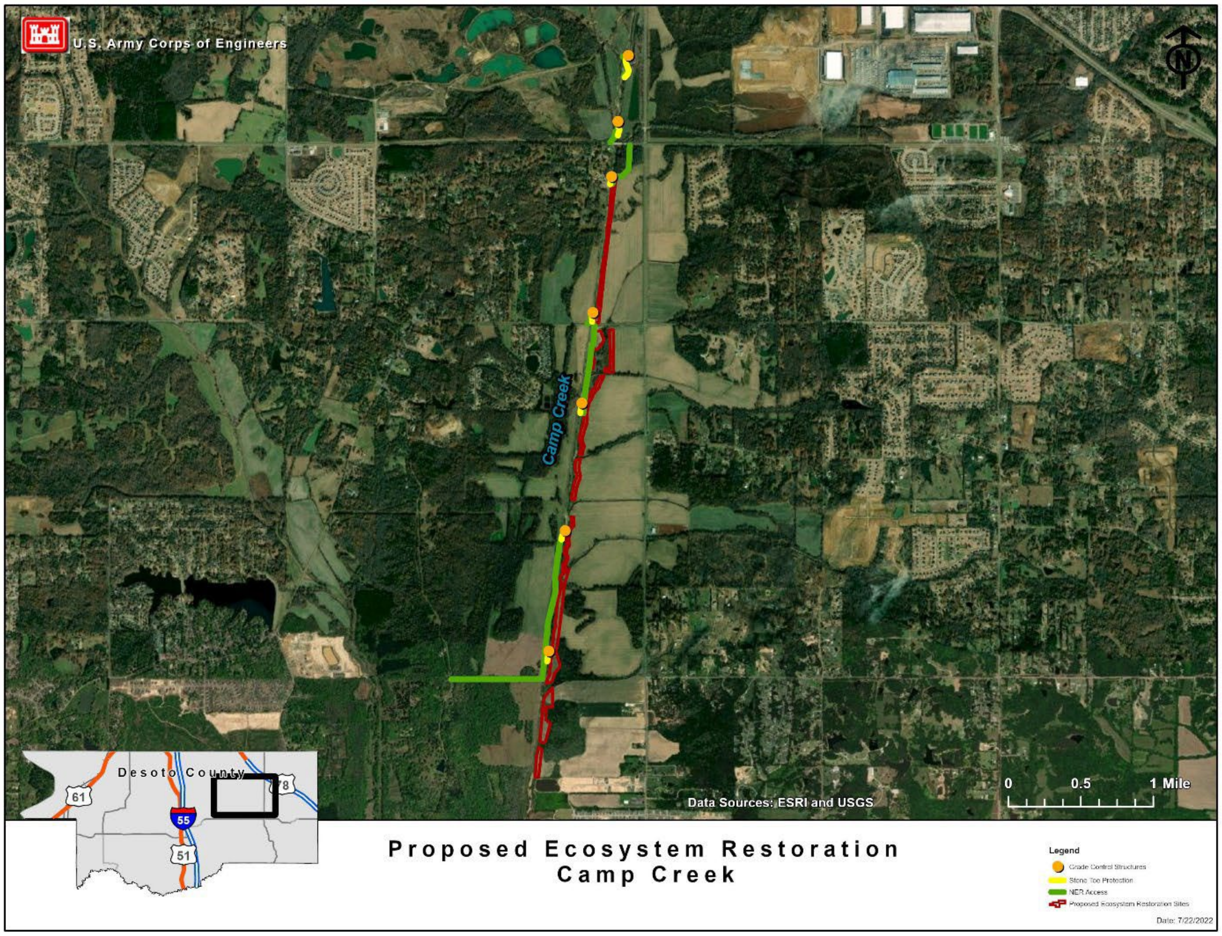


Figure 21. Layout of ecosystem restoration features in Camp Creek.

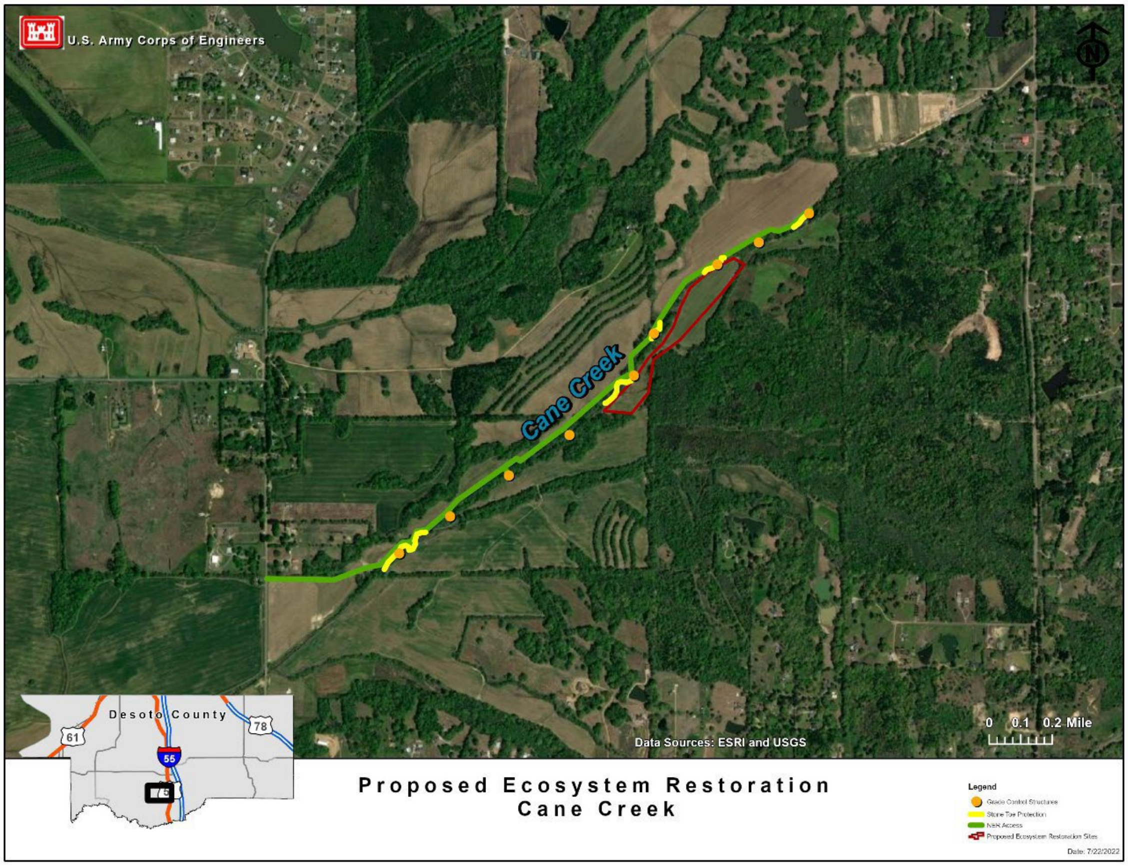


Figure 22. Layout of ecosystem restoration features in Cane Creek.

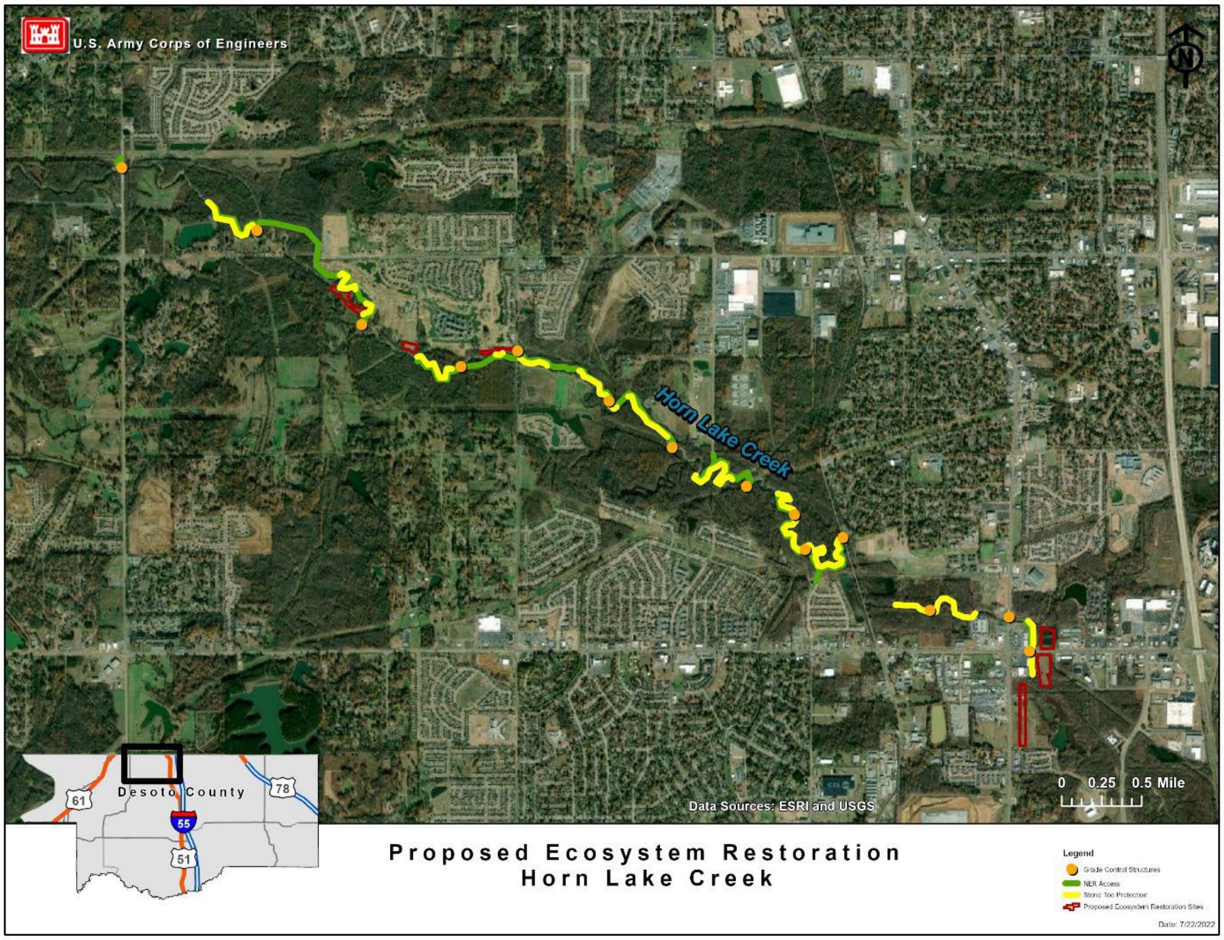


Figure 23. Layout of ecosystem restoration in Horn Lake Creek (evaluated but not carried forward).

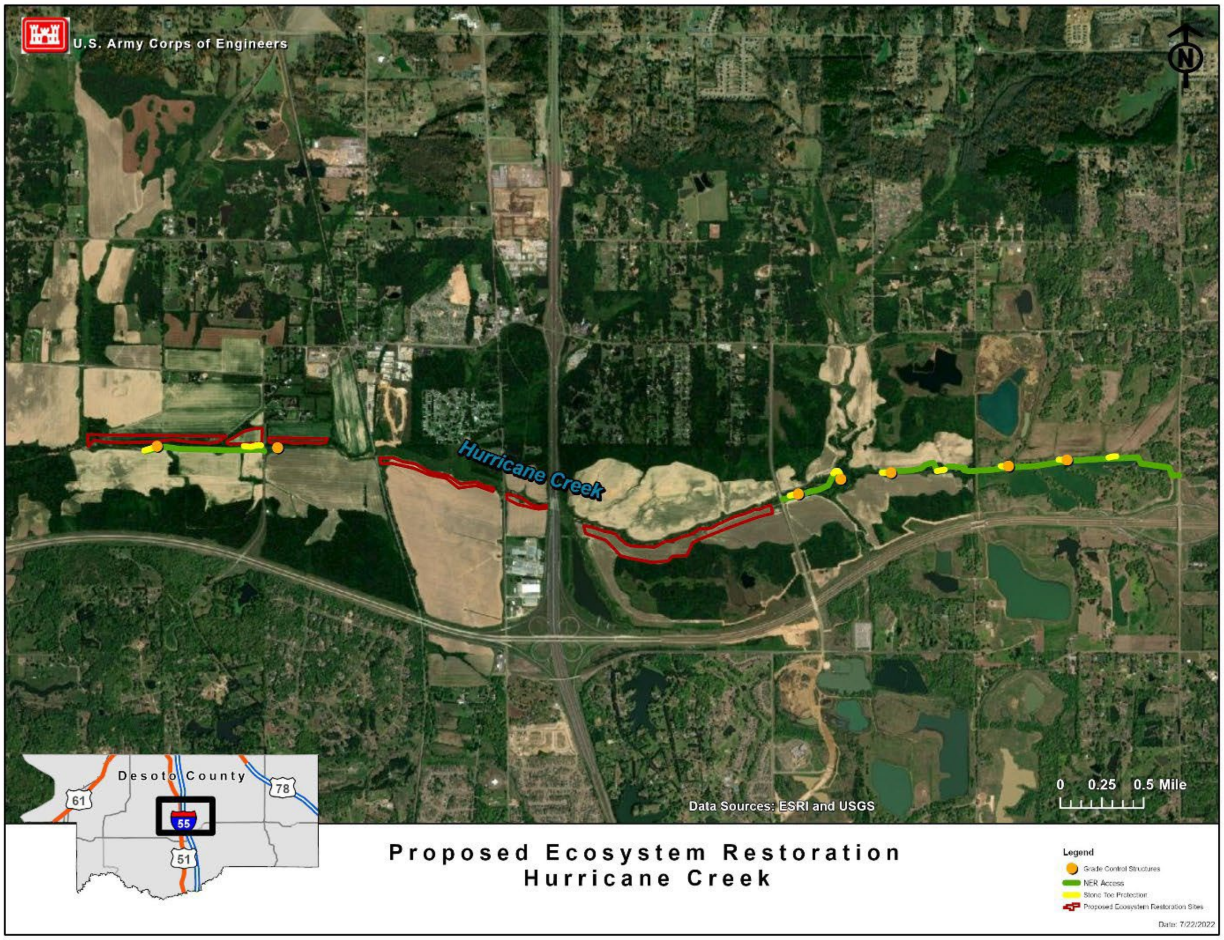


Figure 24. Layout of ecosystem restoration features in Hurricane Creek.

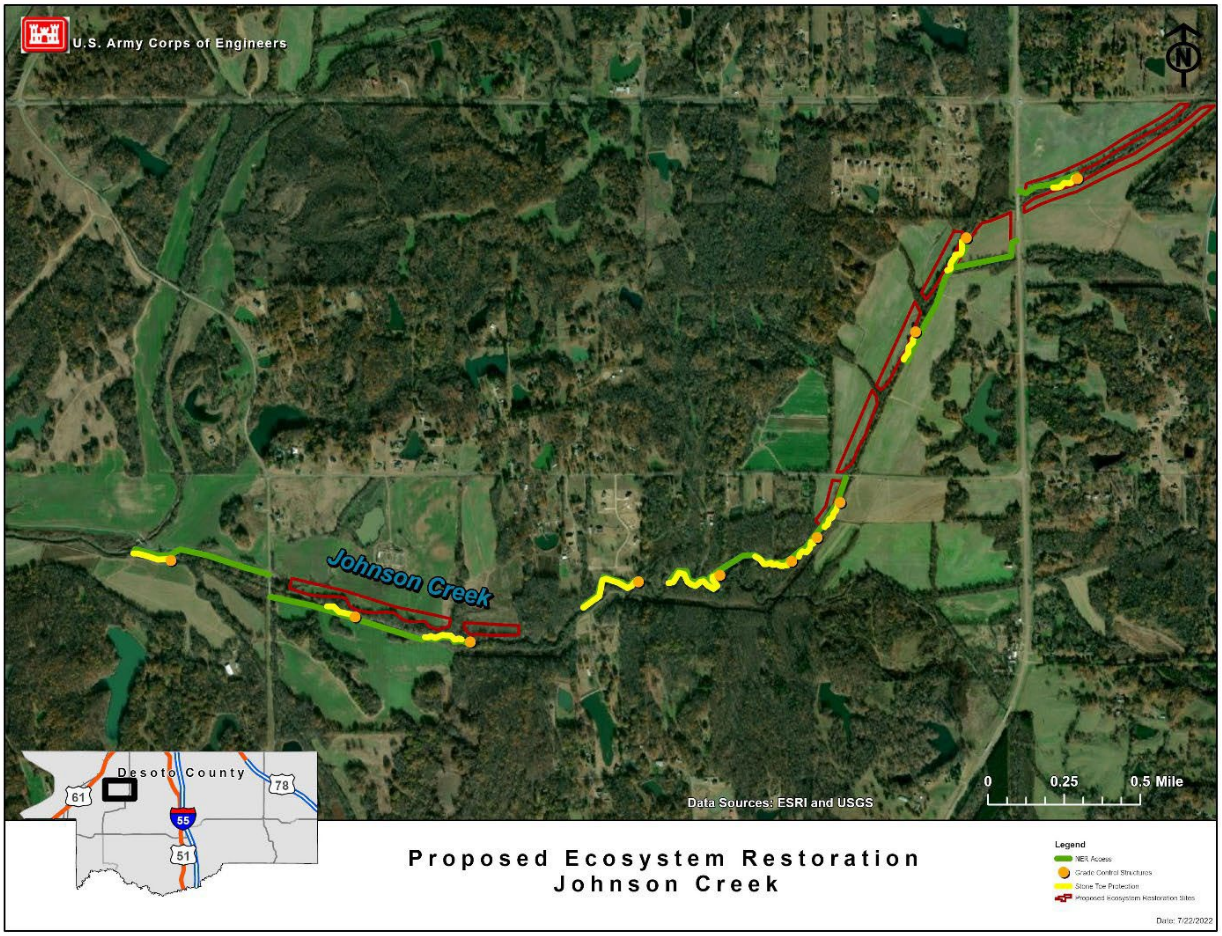


Figure 25.Layout of ecosystem restoration features in Johnson Creek.

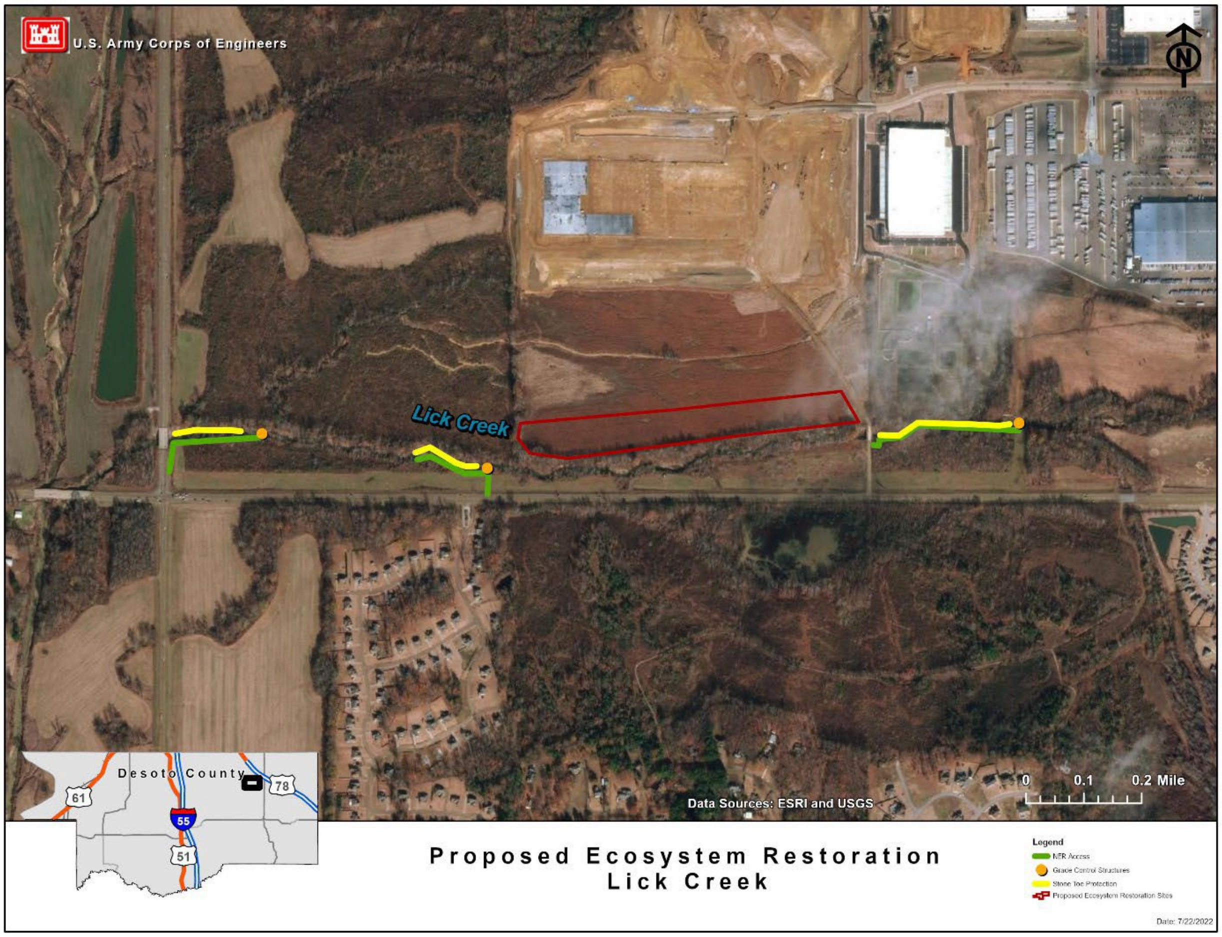


Figure 26. Layout of ecosystem restoration features in Lick Creek.

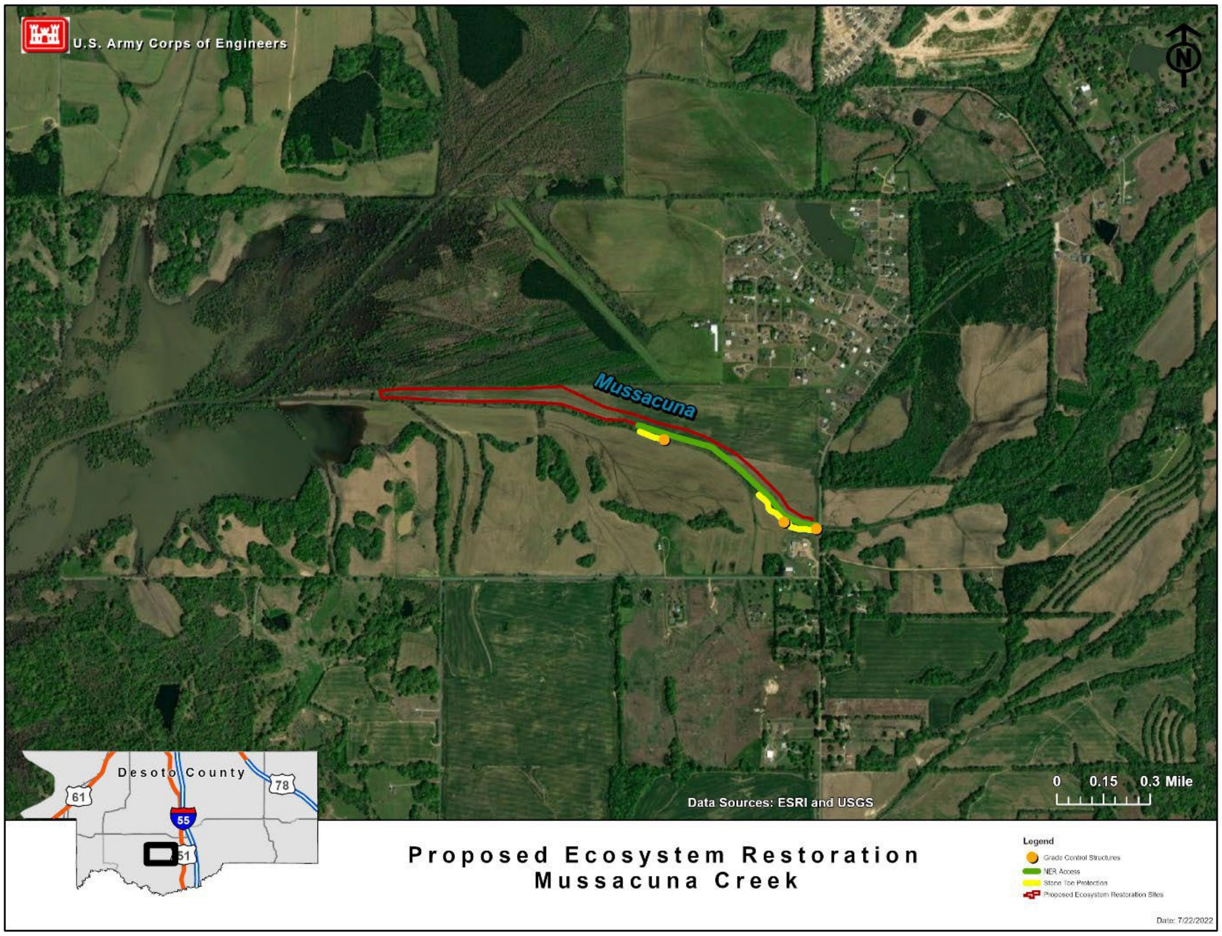


Figure 27. Layout of ecosystem restoration features in Mussacuna Creek.

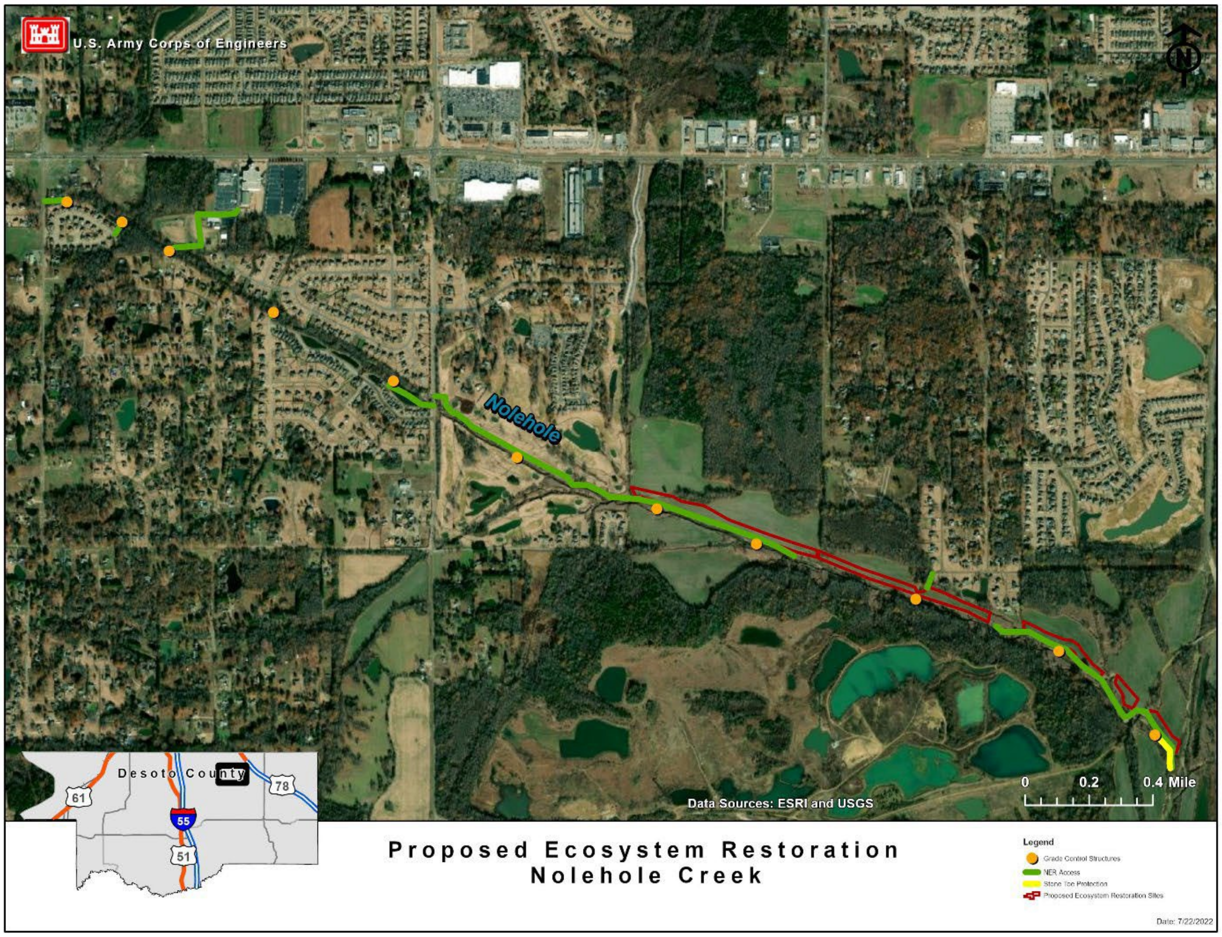


Figure 28. Layout of ecosystem restoration features in Nolehole Creek.

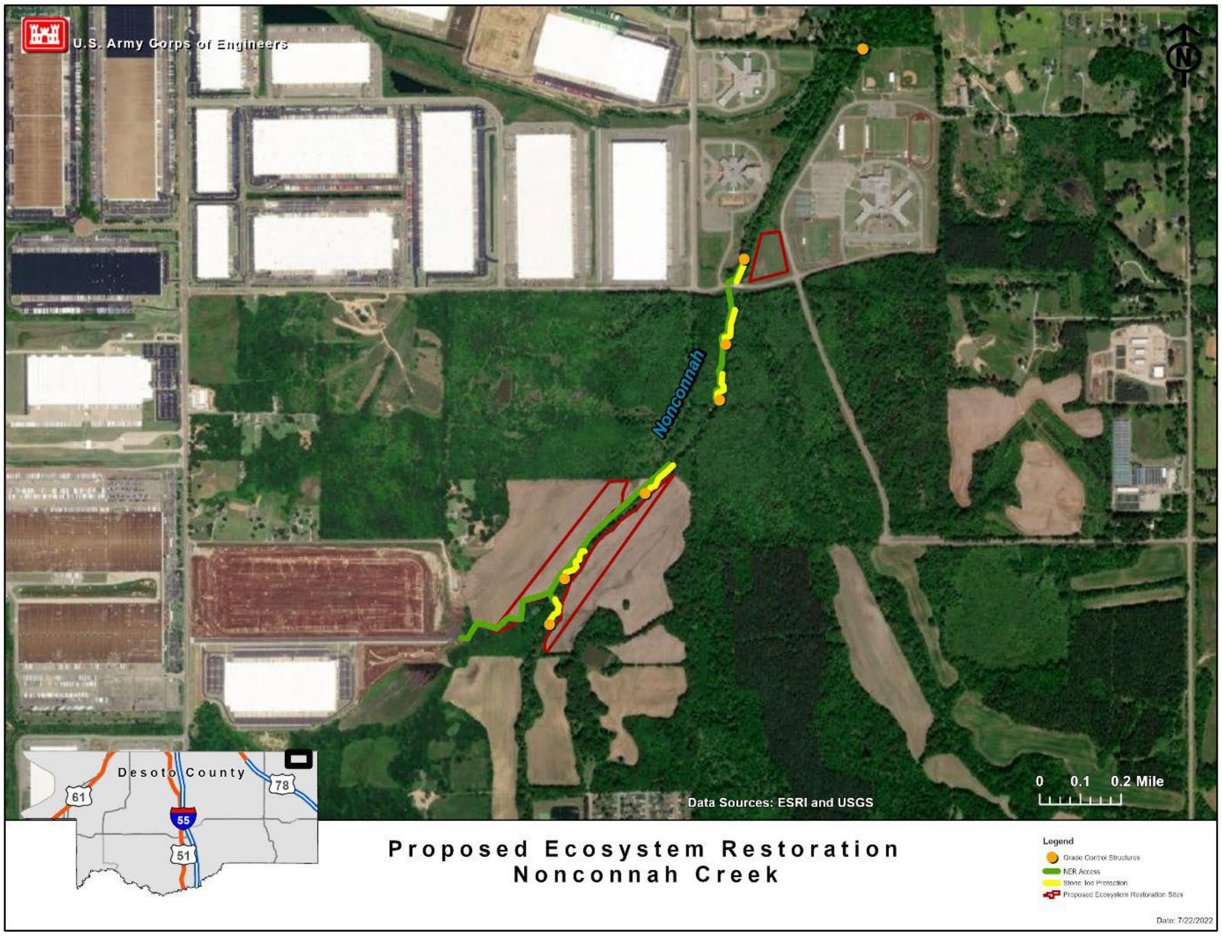


Figure 29. Layout of ecosystem restoration features in Nonconnah Creek.

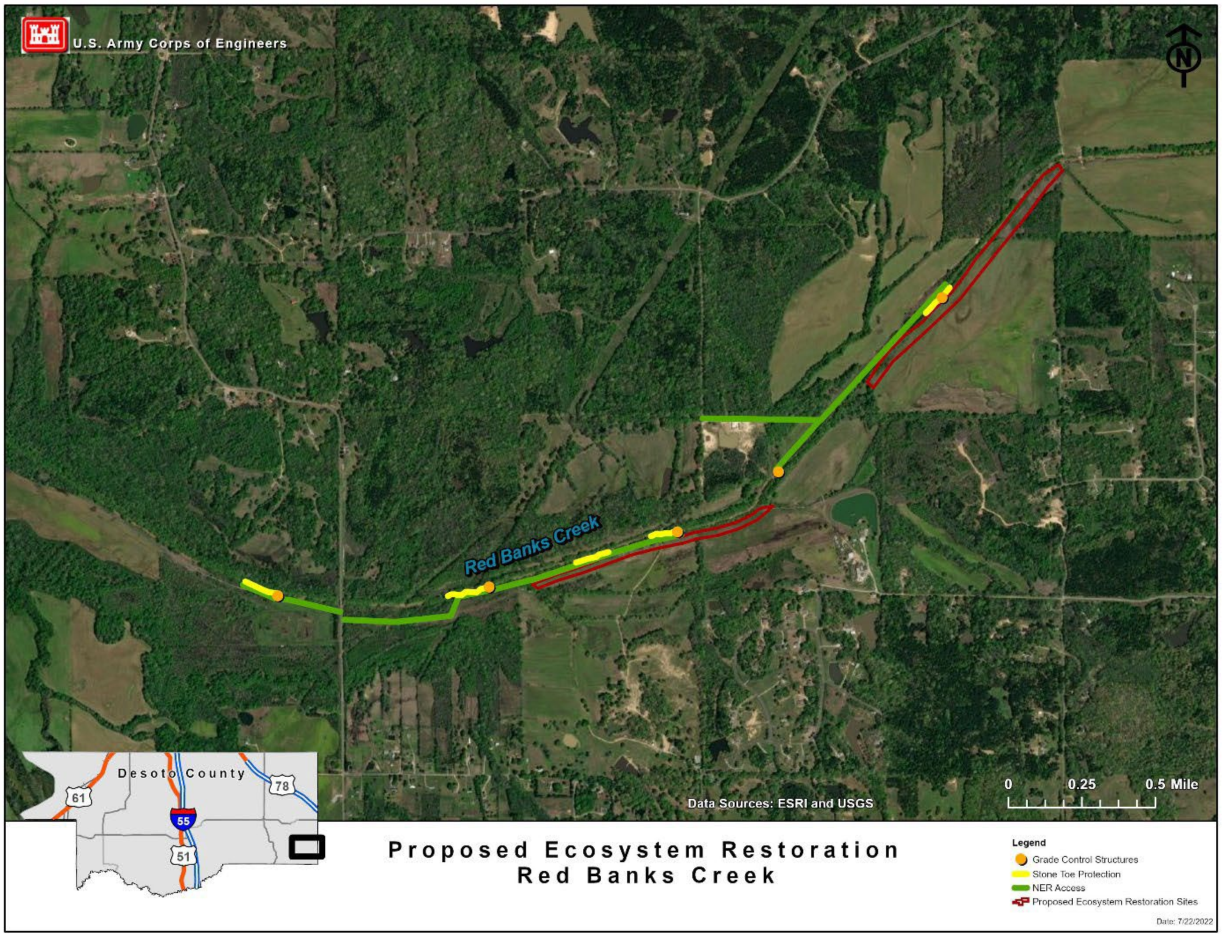


Figure 30. Layout of ecosystem restoration features in Red Banks Creek.

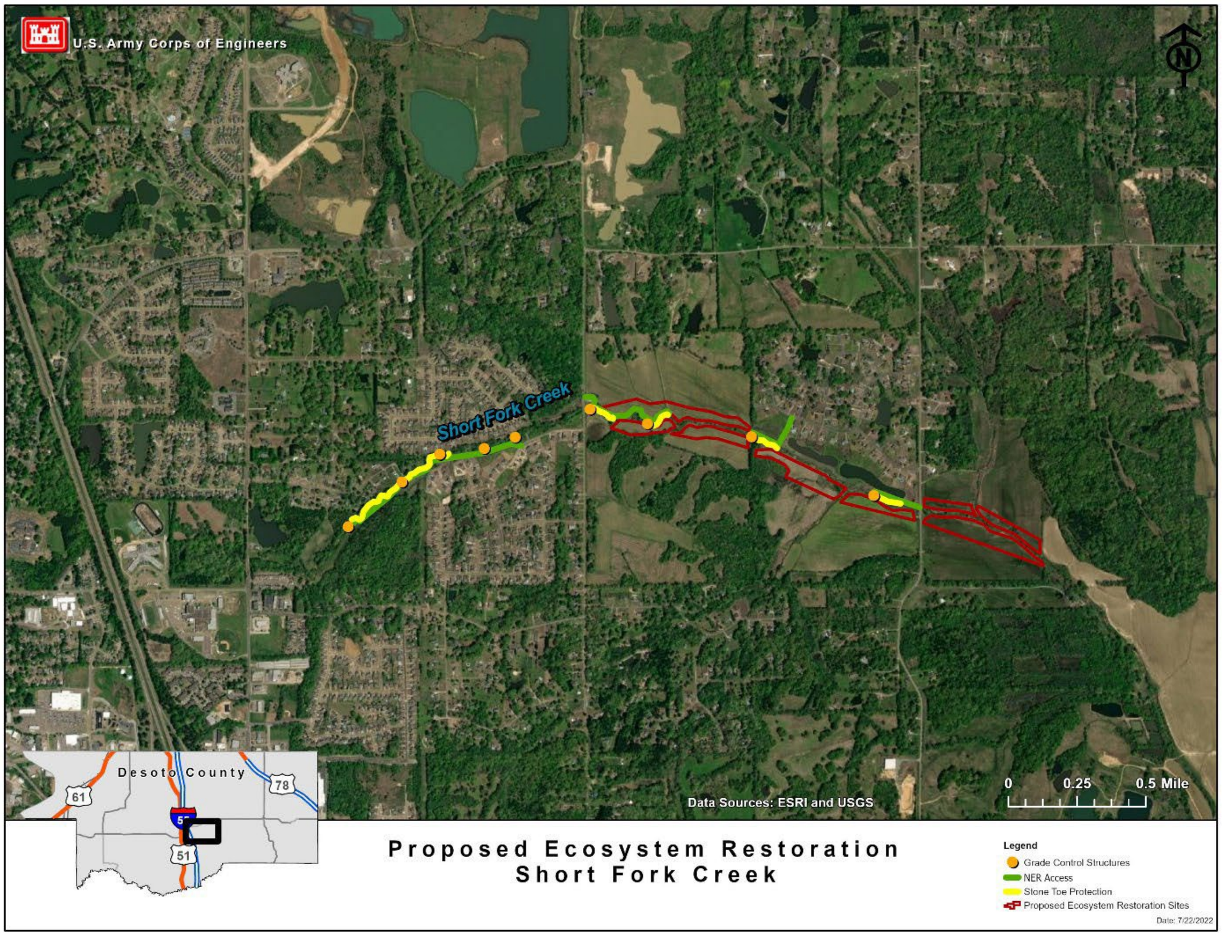


Figure 31. Layout of ecosystem restoration features in Short Fork Creek.

2.9.6 Quantity Calculations for Ecosystem Restoration

2.9.6.1 Basis of Design

Ecosystem restoration quantities were based on best practices and experience with similar structures. Table 7 indicates the source of the most common design goals or parameters that influenced design and layout.

Table 7. Basis of Design for ecosystem restoration features.

BASIS OF DESIGN FOR ECOSYSTEM RESTORATION FEATURES	
Engineering parameters	**Planning goals**
Grade Control Structures	
Width (MVM; based on measurement of LiDAR and aerial imagery)	Locate structures to prevent/slow headcutting where this is occuring
Bank height (MVM; based on measurement of LiDAR and aerial imagery)	Improve water quality by trapping sediement behind structures
3.5' typ height (WES; best practice)	Provide a pool that will benefit aquatic species during dry periods
4H:1V foreslope, 20:1 backslope (WES; best practice)	Do not install where there is a potential to induce flooding
Tiebacks extend up slope to 1/2 of bank height (WES; best bractice)	Make use of existing structures when possible
R650 riprap upstream (MVM; based on expected velocities)	Stabilize the most damaged locations first
R200 riprap downstream (MVM; based on expected velocities)	Designed to work as a system
Aditional keys at center and downstream end (MVM; accounts for local soils)	Next structure upstream is typically located at the tailwater of the last
Additional downstream bank armor (MVM; accounts for local soils)	Final placement requires current field data
Bank armor extends up slope to 1/3 bank height (WES; best practice)	
Rehabilitation of existing structures is based on 50% the stone required for a new structure at that location (WES; best practice)	
LPSTP	
2TN/LF (WES; best practice)	Protects streambank where high velocities occur
5' height (WES; best practice)	Often - but not always - paired with GCS downstream of the structure
Tiebacks every 100' (WES; best practice)	Final placement requires current field data
Tieback stone quantity 15% of toe (WES; best practice)	
Risers	
30 LF length (MVM; average for bank heights in the study area)	Install at incoming ditches that are developing into gullies
24" diameter (MVM; averge for size of ditches in study area)	Locate outfall over LPSTP
	Field fit after PED investigations
Access	
40' width to account for obstacles, grading, and passing areas (MVM; best practice)	Access from closest public roadway
	Link routes to reduce separate access points and number of easements required
	Limit tree clearing from access route
	Avoid crossing ditches as much as possible.

2.9.6.2 Estimated quantities

The basis of design in Table 7 was used to develop the conceptual designs described above. A summary of quantities based on this design is presented in Table 5.

Table 8. Estimated quantities for NER structures.

Stream	Bank Height (FT)	Stream Width (FT)	Grade Control Structures				LPSTP (LF)			Riser pipes		Right of way (ac)			Tree Clearing (ac)
			Number	R650 Riprap (TN)	R200 Riprap (TN)	Bedding (TN)	Number	R200 Riprap (TN)	Bedding (TN)	Number	Length (LF)	GCS	LPSTP	Access	
Camp Creek	10	30-50	7	465	3166	875	2350	5405	979	9	270	3.5	2.2	11.1	0.0
Cane Creek	9	8-12	9	183	1446	389	2500	5750	1042	4	120	4.5	2.3	8.9	0.8
Horn Lake Creek*	15	20-60	14	527	3998	1132	19900	45770	8292	12	360	7.5	18.3	27.5	44.7
Hurricane Creek	11	15-50	9	421	2991	830	2250	5175	938	6	180	4.5	2.1	12.2	0.0
Johnson Creek	15	20-30	11	487	3825	1086	6300	14490	2625	9	270	5.5	5.8	10.2	14.4
Lick Creek	10	40-50	3	161	1098	303	2000	4600	833	2	60	1.5	1.8	2.4	2.5
Mussacuna Creek	10	40-50	3	137	2525	423	1300	2990	542	1	30	1.5	1.2	0.0	0.0
Nolehole Creek	15	30-54	11	635	4736	1340	5500	12650	2292	8	240	5.5	5.1	8.9	3.9
Nonconnah Creek	9	15-25	7	158	1180	320	2000	4600	833	2	60	3.5	1.8	2.4	6.0
Short Fork Creek	6	10-20	9	217	1483	386	3650	8395	1521	5	150	4.5	3.4	4.4	0.0
Red Banks Creek	9	35-50	5	308	2080	572	2500	5750	1042	0	0	2.5	2.3	9.1	0.0
Total, all streams			88	3700	28527	7656	50250	115575	20938	58	1740	44.5	46.1	97.1	72.3
Total, without Horn Lake Creek*			***74***	***3173***	***24529***	***6524***	***30350***	***69805***	***12646***	***46***	***1380***	***37.0***	***27.9***	***69.6***	***27.6***

***Horn Lake Creek not included in final NER array**

2.10 RELOCATIONS AND UTILITIES

One vacant structure was noted for demolition. The structure, foundation, utility connections, and any out structures will be removed and not replaced.

Asphalt parking removed will be replaced during construction. Signs will be stored and reinstalled at the end of construction.

The team sought to avoid relocations as best as possible so to minimize relocation costs. Utilities to be removed and/or replaced are noted above. Remaining utilities at the levee and floodwall are as follows:

Water supply pipeline at Goodman Rd. During feasibility, the Team will investigate methods of protection during PED, such as encasement, flowable fill, or other method to prevent damage to the pipeline.

A gas line runs parallel on the east side of US 51 but outside the proposed levee right-of-way. A sewer line runs on the east side of US 51 but crosses under the highway 330’ from the southernmost (upstream) end of the levee. Additional service connections were noted but those will be removed during the structural demolition noted above.

Other utilities were investigated and found to run along corridors outside the proposed levee and floodwall footprint. Additional gas, sewer, and water run west of US 51, and a sewer line runs east of Horn Lake Cr.

2.11 BORROW AND DISPOSAL

Borrow for the levee is available on site (primary source) and at a city-owned site on Nail Rd. adjacent to Cowpen Creek (continency source). Additional borings are needed during PED to confirm the acceptability of borrow material. Construction debris and any contaminated soil discovered demolition will be disposed of offsite at a commercial landfill.

2.12 OPERATIONS AND MAINTENANCE

Both FRM and NER components are designed to be passively operated. Maintenance activities for the levee include mowing the levee annually and slide repair every 10 years. Maintenance of grade control structures and stone toe protection includes clearing access and replacing up to 10% of the stone every 10 years. This accounts for costs to repair after flood events as well. These estimates are based on maintenance requirements for similar USACE structures in the region. Regular inspection through the District's Inspection of Completed Works program will monitor the structures and allow responsive maintenance to maintain expected performance. Cost estimates are included in Appendix J.

Section 3
Preconstruction Engineering and Design

3.1 TESTS AND DATA COLLECTION

In order to properly analyze and reduce risk, additional data must be acquired early in the PED phase to support final design. A detailed inventory of existing utilities in the project footprint must be obtained in cooperation with the sponsor and utility owners. This is particularly important to subsurface utilities that are not easily located with imagery or site visits. Discussions with owners must determine which conflicting utilities can be relocated, and at what cost. Topographic surveys are required to locate key existing features (including utilities) and gain the necessary fidelity of elevations needed for design. A topographic survey is required to improve the confidence of material quantity estimates, aid in validating hydraulic models, and identify conflicts with existing features. Modeling will also be used to refine the height of flood protection and degree of revetment required. Topographic data for all GCS will be necessary for design and quantity calculations.

3.2 ANALYSIS/OPTIMIZATION STRATEGY

Layout and quantities are based on publicly-available data and limited modeling to develop cost and performance estimates. These are not to be considered final designs for construction. Additional data collected during Feasibility or PED will allow a final analysis for design for both FRM and NER features. This will be particularly important for hydraulic and geotechnical evaluations. The improved fidelity of this data will support the final constructed work items. During PED, the team will first refine models and test FRM and NER features to validate performance anticipated in this report. Next, NER features may be field fit based on current data in order to provide the maximum environmental benefit.

Construction sequence and phasing will be addressed during PED.

3.3 INTEGRATION WITH OTHER AUTHORIZED PROJECTS

Analysis and coordination will be done to ensure this plan integrates into existing structures owned by municipal interests, private entities, and other government agencies. In particular, the team will coordinate with the Vicksburg District as several NER measures overlap improvements authorized by the Delta Headwaters Project (DHP). This coordination was initiated during the study phase and will continue through implementation.

Two DHP structures in Hurricane Creek are proposed for additional riprap. The implementation team will coordinate closely with the Vicksburg District on placement and maintenance of this stone.

3.4 COORDINATION WITH OTHER AGENCIES

The design team will coordinate with other agencies to identify and incorporate regulatory requirements into the design. This includes identifying setback requirements, securing stormwater permits, and complying with all other local, state, and federal requirements. It will also include coordination with utility owners for any relocations, and coordination with MDOT and the City of Horn Lake for improvements adjacent to Goodman Rd. See also Appendix F Interagency Coordination.

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