APPENDIX A4: GEOTECHNICAL ENGINEERING

WACCAMAW RIVER,

HORRY COUNTY, SOUTH CAROLINA

FLOOD RISK MANAGEMENT STUDY INTEGRATED FEASIBILITY REPORT AND ENVIRONMENTAL ASSESSMENT

MAY 2026

MAIN REPORT SUMMARY

The Integrated Feasibility Report and Environmental Assessment (FR/EA), that this appendix addresses, details a collaborative study by the U.S. Army Corps of Engineers (USACE) and Horry County, South Carolina. It is aimed at reducing existing and future flood risks to communities and transportation infrastructure within the Waccamaw River Basin, with a focus on Horry County. The study identifies four key flood impact areas: Longs & Red Bluff, Conway, Bucksport, and Socastee.

The flood impacts in each of these areas were independent of each other, so solutions could be evaluated self-reliantly, making any proposed alternative plans separable. The study considered a range of structural, non-structural, and nature-based solutions while incorporating public feedback gathered during meetings. An environmental analysis was completed, and a Finding of No Significant Impact is included within the main report. The document completed a public review and comment period while also undergoing internal agency reviews and adapted to those concerns and suggestions. In addition to historical flooding, the report acknowledges the flooding event caused by Hurricane Debby in August 2024 during this study, and its impact was assessed to further inform the study's conclusions.

The Recommended Plan, based on an evaluation of alternatives, includes two separable elements that are incrementally justified: Relief Bridges (cross drains) in the Conway flood impact area and Barrier Removal in the Socastee flood impact area. The Recommended Plan is classified as the National Economic Development Plan and is also the plan that maximizes net comprehensive benefits. No alternatives were justified for Federal investment for the Longs & Red Bluff and Bucksport flood impact areas. This Appendix provides detailed Geotechnical Engineering to support these recommendations.

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A INTRODUCTION

The study area covers the Waccamaw River and its tributaries from the South Carolina state line to its confluence with the Pee Dee River. Horry County (the non-federal sponsor) is situated within South Carolina's coastal plain and is bordered by North Carolina to the north and the Atlantic Ocean to the east. Detailed descriptions of the measures can be found in the main report, as well as the rationale for the selection of the recommended plan.

A.1 Purpose

The purpose of this Appendix is to provide a geological description in the general vicinity of the structural and nonstructural measures that were considered in each focus area. Design phase considerations and general construction recommendations are discussed.

B REGIONAL GEOLOGY

B.1 Waccamaw River Basin

In South Carolina the Piedmont Unit is separated from the Coastal Plain Unit by a "Fall Line" that begins near the Edgefield-Aiken County line and traverses to the northeast through Lancaster County. The Fall Line is an unconformity that marks the boundary between an upland region (bed rock) and a coastal plain region (sediment). The Waccamaw River Basin lies within the Coastal Plain Unit.

The Coastal Plain is underlain by Mesozoic/Paleozoic basement rock. This wedge of sediment is comprised of numerous geologic formations that range in age from the late Cretaceous Period to Recent. The sedimentary soils of these formations consist of unconsolidated sand, clay, gravel, marl, cemented sands, and limestone that were deposited over the basement rock. The basement rock consists of granite, schist, and gneiss similar to the rocks of the Piedmont Unit. Predominantly, sediments lie in nearly horizontal layers; however, erosional episodes occurring between depositions of successive layers are often expressed by undulations in the contacts between the formations.

The vertical stratigraphic sequence overlying the basement rock consists of unconsolidated Cretaceous, Paleogene, Neogene, and Quaternary sedimentary deposits. The surface deposits of the Lower Coastal Plain were formed during the Quaternary Period that began approximately 1.6 MYA and extends to present day. The Quaternary Period can be further subdivided into the Pleistocene Epoch (1.6 MYA to 10 thousand years ago) and the Holocene Epoch (10 thousand years ago to present day). The Pleistocene Epoch is marked by the deposition of the surficial soils, the formation of the Carolina Bays and the scarps found throughout the East Coast due to sea level rise and fall. Barrier islands and flood plains along the major rivers were formed during the Holocene Epoch. The sections below show a geologic map for each focus area of the Waccamaw River Basin.

Source: SCDOT Design Manual, January 2019

B.2 Socastee

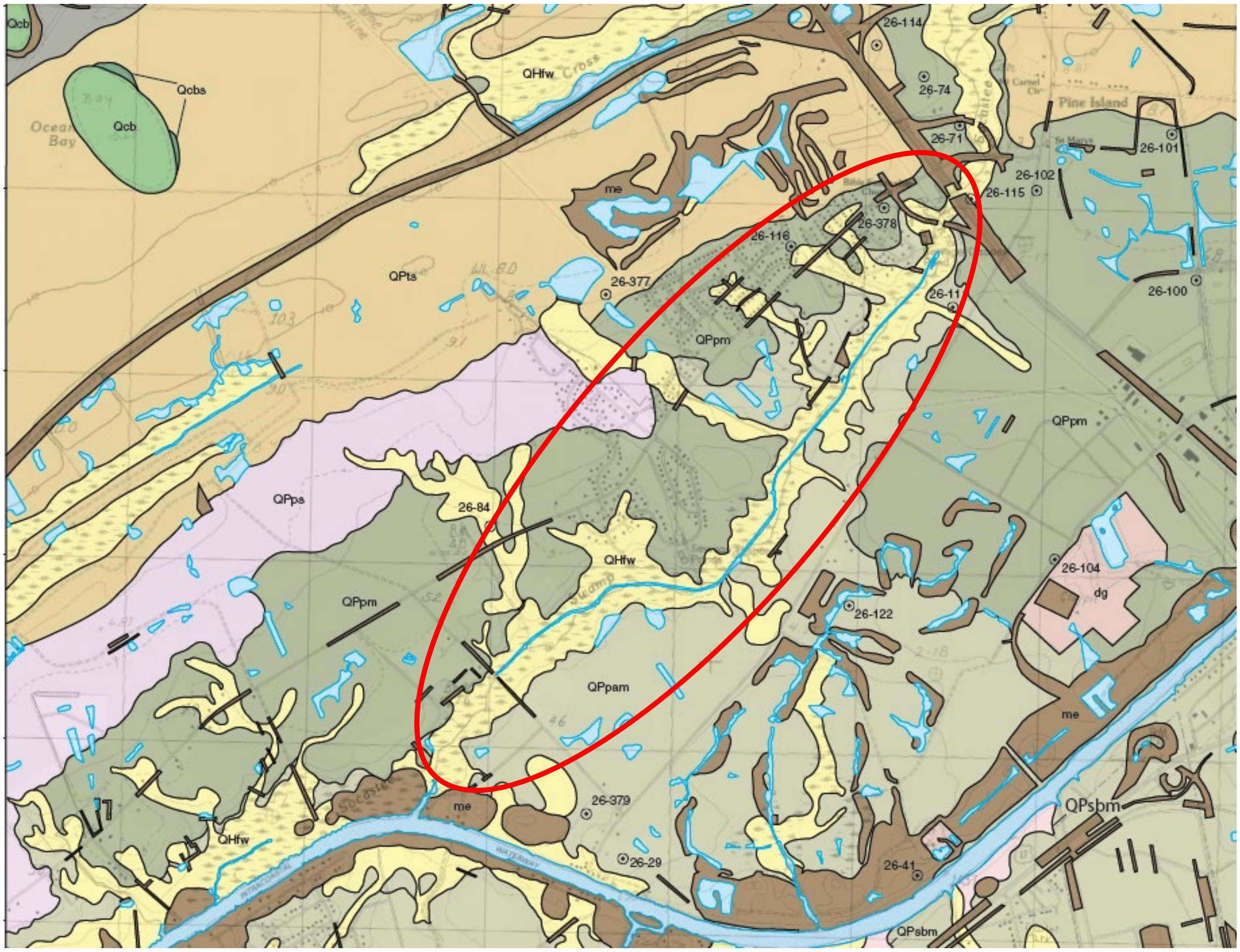


Figure B-1: Socastee Focus Area Geologic Map. Location of proposed alternatives circled.

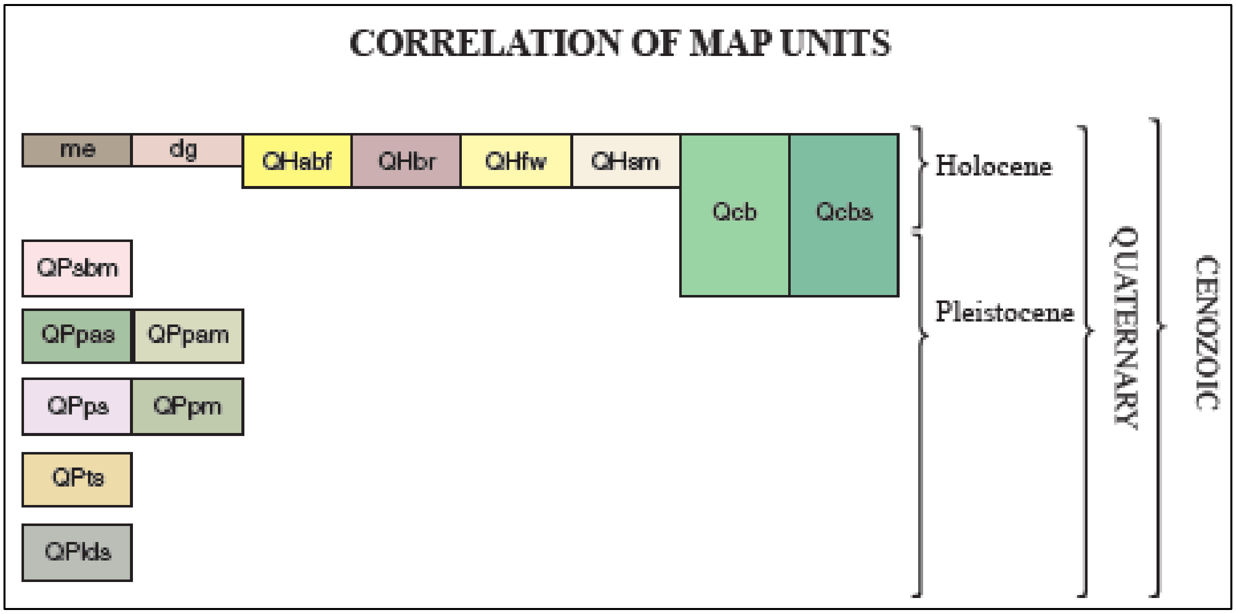
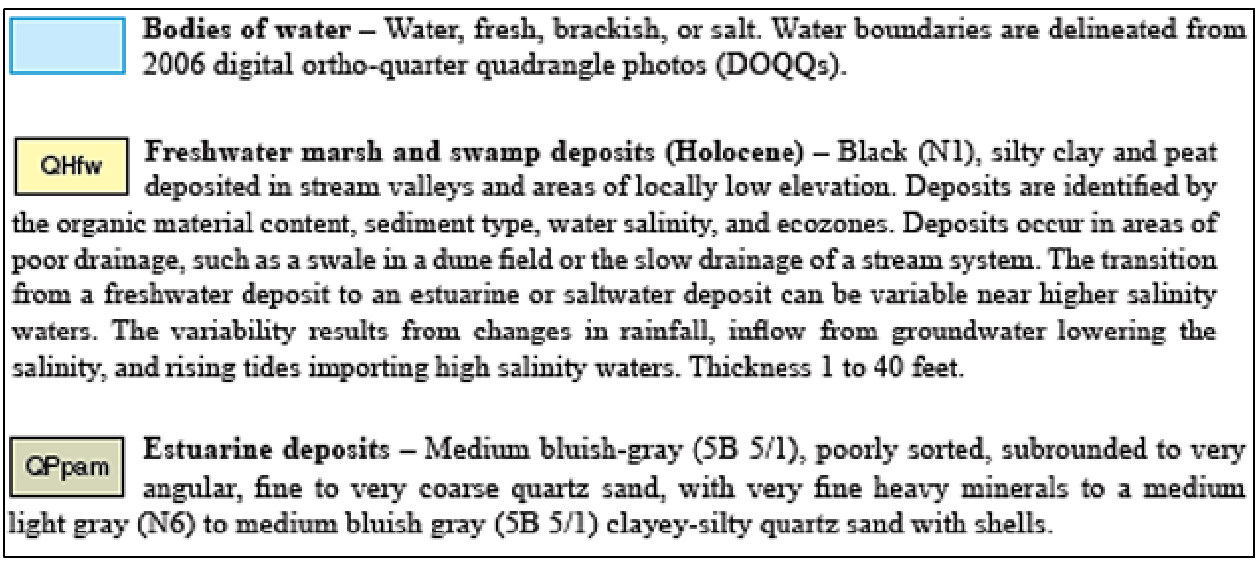


Figure B-2: Map Units and Descriptions

The proposed structural alternatives are in the Socastee Swamp area. Below is the description of main map units for this area.



Bodies of water – Water, fresh, brackish, or salt. Water boundaries are delineated from 2006 digital ortho-quarter quadrangle photos (DOQQs).

QHfw **Freshwater marsh and swamp deposits (Holocene)** – Black (N1), silty clay and peat deposited in stream valleys and areas of locally low elevation. Deposits are identified by the organic material content, sediment type, water salinity, and ecozones. Deposits occur in areas of poor drainage, such as a swale in a dune field or the slow drainage of a stream system. The transition from a freshwater deposit to an estuarine or saltwater deposit can be variable near higher salinity waters. The variability results from changes in rainfall, inflow from groundwater lowering the salinity, and rising tides importing high salinity waters. Thickness 1 to 40 feet.

QPpam **Estuarine deposits** – Medium bluish-gray (5B 5/1), poorly sorted, subrounded to very angular, fine to very coarse quartz sand, with very fine heavy minerals to a medium light gray (N6) to medium bluish gray (5B 5/1) clayey-silty quartz sand with shells.

Figure B-3: Map Units

Source: SCDNR Geological Survey, Geologic Map of the Myrtle Beach Quadrangle, Horry County, South Carolina. W.R. Doar, III. 2014.

B.3 Longs/Red Bluff

The location of the proposed flood walls in Longs are along Buck Creek adjacent to the Aberdeen Country Club. Below is the description of main map units for this area.

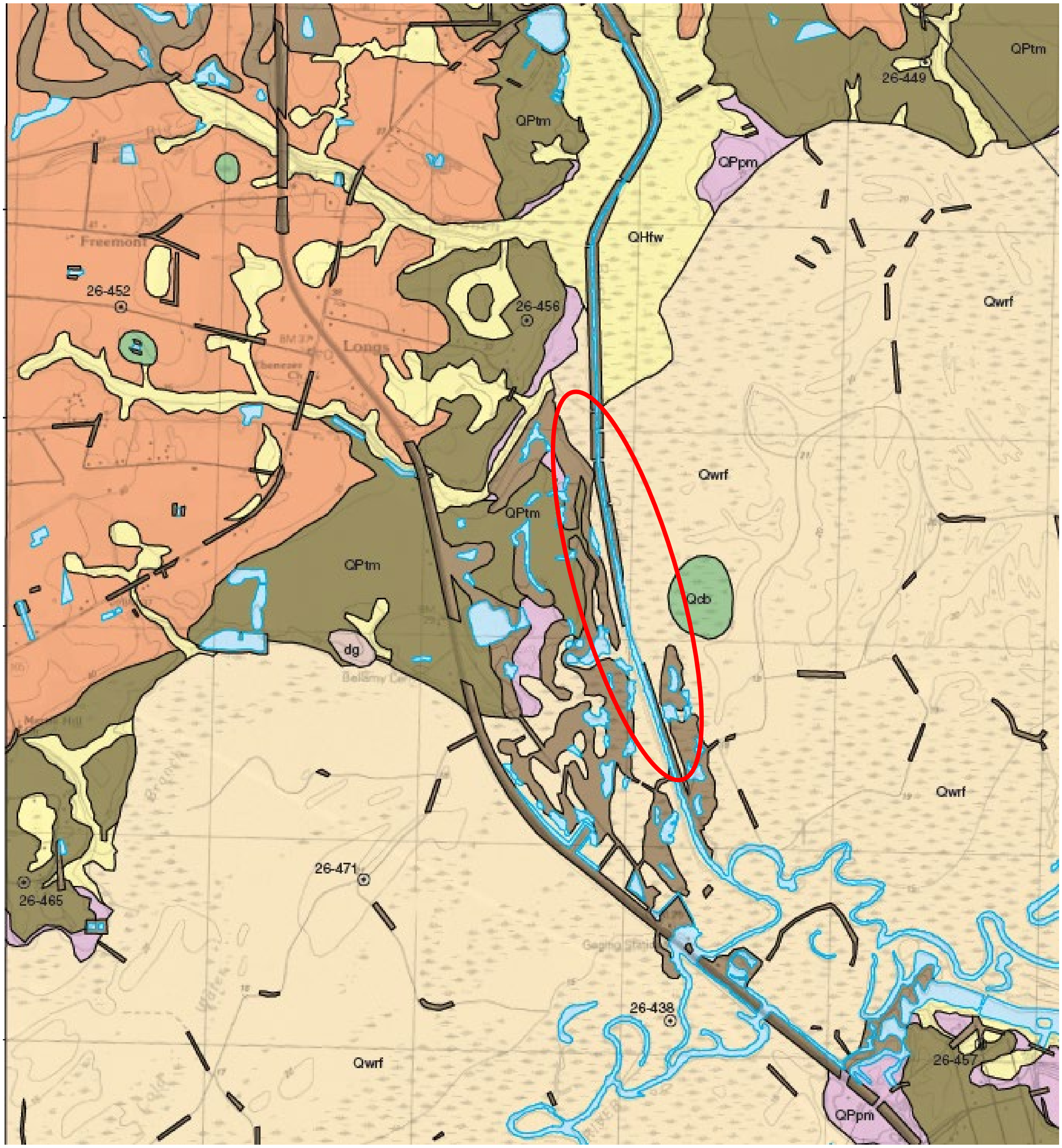
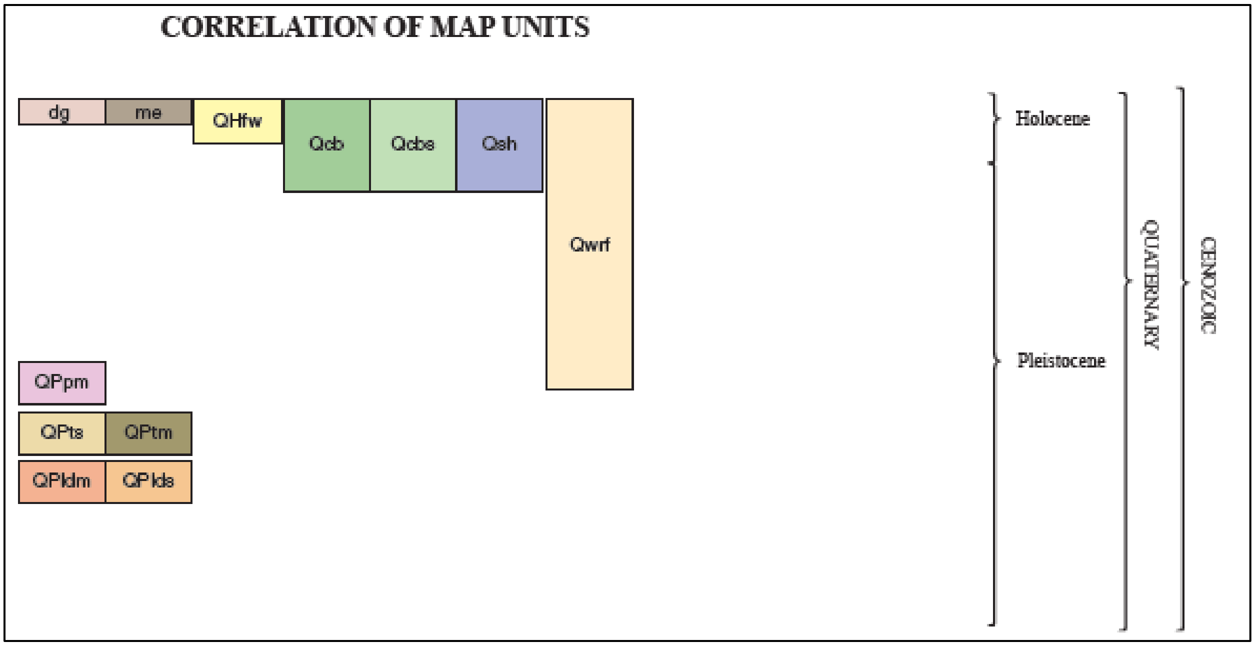


Figure B-4: Longs Focus Area Geologic Map. Location of Proposed alternatives circled.



Bodies of water – Water, fresh, brackish, or salt. Water boundaries are delineated from 2006 digital ortho-quarter quadrangle photos (DOQQs).

Waccamaw River fluvial system (Holocene to Pleistocene)

Qwrf **Waccamaw River floodplain sediments** – Clay to gravel, gray (N4-N9), medium greenish-white (5GY 4/1), brownish-white (5Y 9/1), pale brown (2.5Y 8/2-8/3), brown (5YR 4/6), and yellowish-orange (10YR 6/6-8/6), clay, silt, woody peat, and sand with granules or pebbles. The sand is poorly to very well sorted, very angular to well-rounded with occasional blocky, very fine to very coarse, quartz sand; with minor amounts of coarse blue quartz, medium jasper, iron-stained quartz, very fine- to medium garnet, rose quartz, fine olivine, very fine rutile, and opaque minerals. Comprised of non-marine sediments deposited in the Waccamaw River floodplain. These sediments vary from channel, to bar, to floodplain, to swamp facies deposits in a historically meandering river system. Thickness is 21 to 65 ft.

Pleistocene Sediments

Pleistocene stratigraphic units are interpreted to be alloformations because the surfaces bounding and separating them from other units are unconformities. The North American Commission on Stratigraphic Nomenclature (2005) defines an allostratigraphic unit as "…a mappable body of rock that is defined and identified on the basis of its bounding discontinuities."

Pamlico alloformation (Pleistocene)

Sediments of the Pamlico alloformation are generally above the elevation of 17 feet at their seaward margin where overlapped by sediments of the Princess Anne alloformation. At their landward margin, Pamlico sediments generally are below the elevation of 25 feet where the deposits overlap, overlie, or abut sediments of the Ten Mile Hill alloformation.

QPpm **Estuarine deposits (Pleistocene)** – Silt and clay, medium bluish-gray (5B 5/1), soft, silt and clay; with minor amounts of very fine quartz and phosphate sand. Thickness is 1 to 15 feet.

Ten Mile Hill alloformation (Pleistocene)

Sediments of the Ten Mile Hill alloformation are generally above the elevation of 25 feet at their seaward margin where overlapped by sediments of the Pamlico alloformation. At their landward margin Ten Mile Hill sediments generally are below the elevation of 35 feet where the deposits overlap, overlie, or abut the Ladson alloformation.

QPts **Strand deposits** – Quartz sand, light gray (N7) to dark gray (N3), sub- to well-rounded, moderately sorted, fine- to medium quartz sand; with common fine-grained heavy minerals and shell hash. Forms subdued ridges on this map. Thickness varies from 2 to 40 feet.

QPtm **Estuarine deposits** – Clay to quartz sand, yellowish-orange (10YR 6/6-7/6), pale brown (10YR 8/2), gray (N6-8), black (N-1), brown (10YR 5/4), yellow (10YR 8/4), and medium bluish-gray (5B 5/1), clay; quartz sandy clay; silty clay; silty sand; and sand. The sand is well sorted, sub- to well-rounded, very fine- to medium quartz sand; with minor amounts of coarse blue quartz, fine- to medium amethyst and epidote, and very fine opaque minerals. Forms a gently riverward-sloping plain along the Waccamaw River. Thickness is 12 to 25 ft.

QP ldm **Estuarine deposits** – Silty-clayey quartz sand, sandy clay, clay, moderate yellowish brown (10YR 6/6-8/6), pale brown (2.5Y 8/4), medium brown (5YR 6/6), moderate brown (10YR 4/2), pink (10R 8/4), yellow (2.5Y 7/6-8/6), gray (N3-N7), bluish-gray (5B 5/1-7/1), and medium greenish-gray (5G 6/1-7/1), silty clay matrix supported, well sorted, but can be poorly sorted, subangular to subrounded, very fine quartz sand with medium-to-very coarse quartz sand in the poorly sorted layers; with minor very fine opaque minerals and rare fine- to medium amethyst sand; stiff clay; sandy clay. Forms a flat plain with incised creek channels on this map. Thickness is 16-26 ft.

Source: SCDNR Geological Survey, Geologic Map of the Longs Quadrangle, Horry County, South Carolina. W.R. Doar, III. 2016.

The location of the proposed benching and culverts in the Red Bluff focus area are along Simpson Creek. Below is the description of main map units for this area.

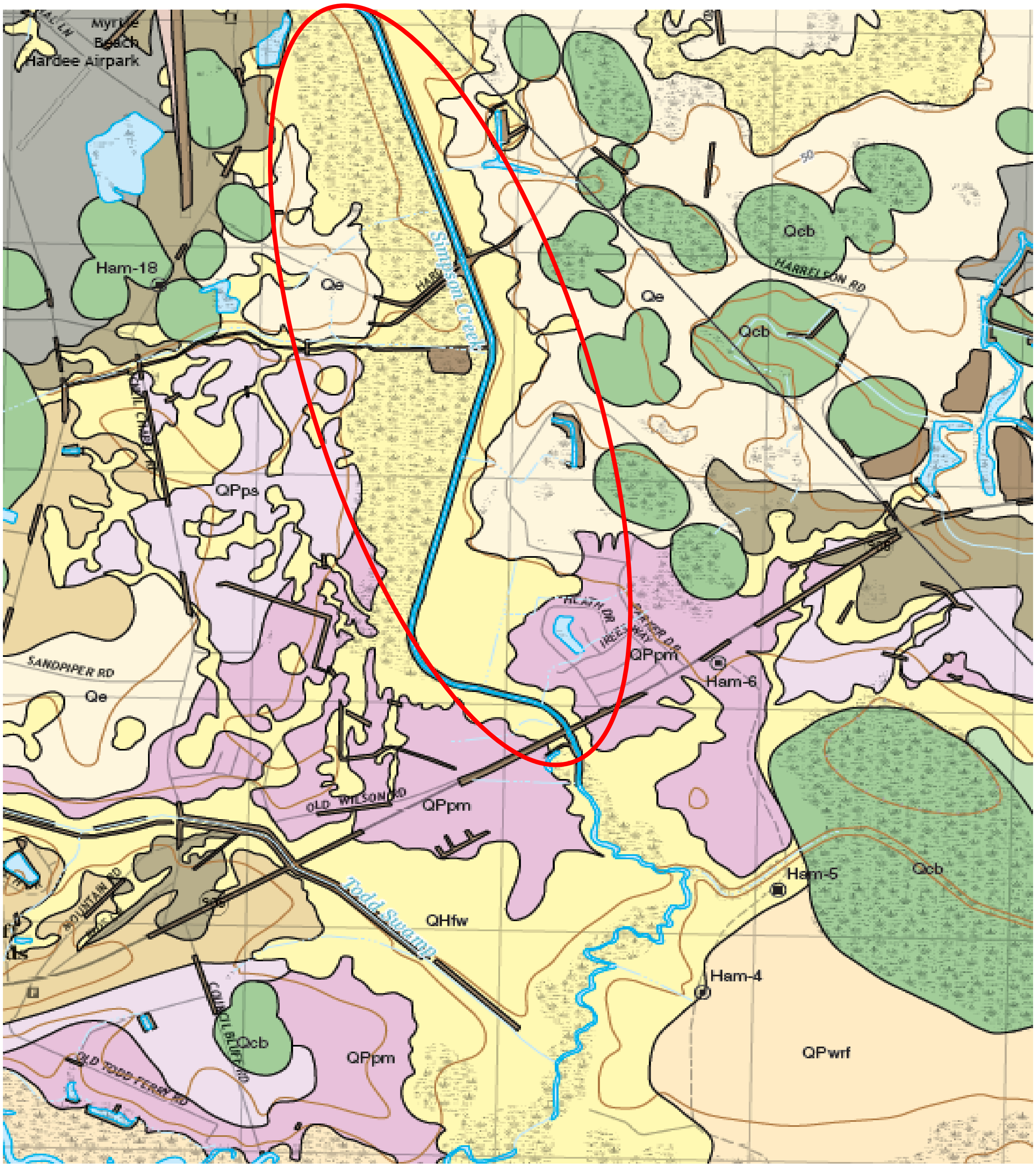
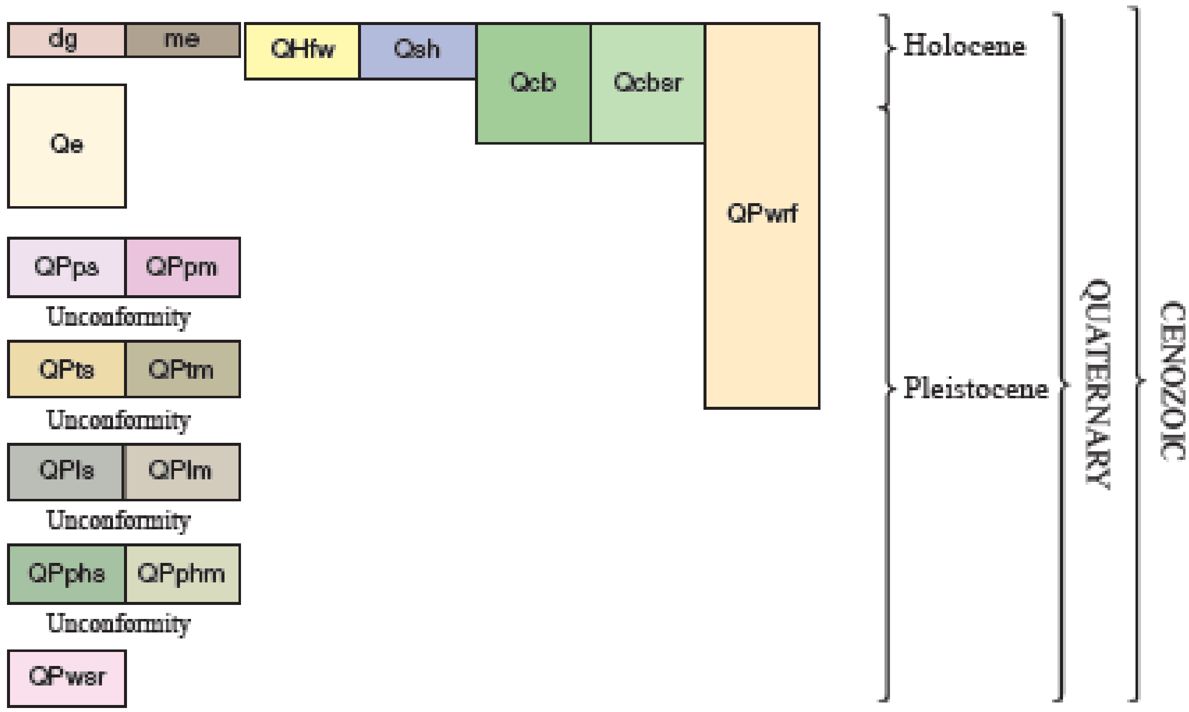


Figure B-5: Red Bluff Focus Area Geologic Map. Location of alternatives circled.

CORRELATION OF MAP UNITS



Water – Water, fresh, brackish, or salt. This designation includes altered shorelines (usually shoreline retreat or stream meanders) or flooded lands (manmade ponds) covered by water after publication of the base map. Water boundaries are delineated from 2006 digital ortho-quarter quadrangle photos (DOQQs).

QHfw **Freshwater marsh and swamp deposits (Holocene)** – Silty clay and peat, black (N1), silty clay and peat deposited in stream valleys and areas of locally low elevation. Deposits are identified by the organic material content, sediment type, and ecozones. Deposits occur in areas of poor drainage, such as a swale in a dune field or the slow drainage of a stream system. Thickness 1 to 12 feet.

Pamlico alloformation (Pleistocene)
Sediments of the Pamlico alloformation are generally above the elevation of 17 feet at their seaward margin where overlapped by sediments of the Princess Anne alloformation. At their landward margin, Pamlico sediments generally are below the elevation of 25 feet where the deposits overlap, overlie, or abut sediments of the Ten Mile Hill alloformation.

QPpm Estuarine deposits – Silt and clay, medium bluish-gray (5B 5/1) to medium gray (N5) to light red (5R 7/6), soft, silt and clay; with minor amounts of very fine quartz and phosphate sand. Thickness is 1 to 15 feet.

Figure B-6:

Source: SCDNR Geological Survey, Geologic Map of the Hammond Quadrangle, Horry County, South Carolina. W.R. Doar, III. 2017.

B.4 Conway and Bucksport

The SCDNR Geologic Quadrangle maps were not yet available for the Bucksport and Conway focus areas at the time the report was written.

C STRUCTURAL MEASURES

C.1 Flood Walls

Preliminary analysis was performed on the proposed flood walls for the Longs and Socastee focus areas to determine what type of wall would be appropriate to estimate construction costs for the TSP milestone. A conceptual analysis was performed on a sheet pile wall using USACE Computer Aided Structural Engineering (CASE) Program CWALSHT. The analysis is not complete, and the results were used for cost estimation purposes only. The current analysis showed that deflections at the top of the wall and at the ground surface were nearing the limit to make an I-Wall feasible. Further refinement in the analysis would be needed, possibly a full numerical analysis, along with site specific geotechnical field investigation to determine if a more robust structure is needed in some locations. Alternatives containing flood walls were not carried forward through the TSP.

Geotechnical reports in the vicinity of the proposed flood wall in the Socastee area were obtained from the non-federal sponsor. The boring locations are not in the exact location of the flood wall, but due to the conceptual nature of these measures, they were used to represent the soil conditions of the area. No geotechnical reports were obtained for the Longs area, so the analysis for Socastee was used to estimate costs for the Longs flood wall. While these locations are not geographically located in close proximity to each other, using the SCDNR Geological Survey maps shown above, both flood wall locations are assumed to be in similar Geologic Units, with the Socastee flood wall location having less desirable soil conditions. The Socastee flood wall was assumed to be in the Freshwater marsh and swamp deposits (Holocene) unit, and the Longs flood wall was assumed to be in the Waccamaw River floodplain sediments (Holocene to Pleistocene) unit.

C.1.1 Assumptions for Soils Data:

Using the Geotech Report for the New Forestbrook Fire Station at Burcale Rd:

The results from the Cone Penetration Test (CPT) were used to estimate a tip resistance and friction ratio. An average value from the three soundings for each stratum were then used to estimate a unit weight using the reference from “Estimating soil unit weight from CPT”, P.K. Robertson and K.L. Cabal, Gregg Drilling and Testing Inc., Signal Hill, California, USA. The same unit weight was used for both moist and saturated unit weight in the analysis input. The groundwater table in this area was observed to be around 3ft below the water surface, so majority of the soil is saturated in situ.

Figure C-1: Sounding ID C-1 Data

Layer	Depth	Tip Resistance	Friction Ratio
1	0 to -7ft	8	7
2	-7 to -10ft	150	0.5
3	-10 to -18	25	0.5
4	-18 to -20	25	7
5	-20 to -23	25	2
6	-23 to -25	125	1

Figure C-2: Sounding ID C-2 Data

Layer	Depth	Tip Resistance	Friction Ratio
1	0 to -7ft	8	7
2	-7 to -12ft	125	0.5
3	-12 to -16	25	1
4	-16 to -20	25	4
5	-20 to -25	25	2
6	-25 to -26.8	250	2

Figure C-3: Sounding ID C-3 Data

Layer	Depth	Tip Resistance	Friction Ratio
1	0 to -8ft	8	7
2	-8 to -17ft	50	0.5
3	-17 to -19ft	25	4
4	-19 to -22ft	70	1
5	-22 to -25ft	25	2
6	-25 to -26	150	1

Figure C-4: Averaged Vales from CPT Data

Stratum	Depth	Tip Resistance (tsf)	Friction Ratio	γv/γw	γw	γ
1	0 to -7ft	8	7	1.8	62.4	112.3
2	-7 to -13ft	100	0.5	1.89	62.4	117.9
3	-13 to -25ft	45	2	1.94	62.4	121.1
4	-25 and below	200	1.5	2.15	62.4	134.2

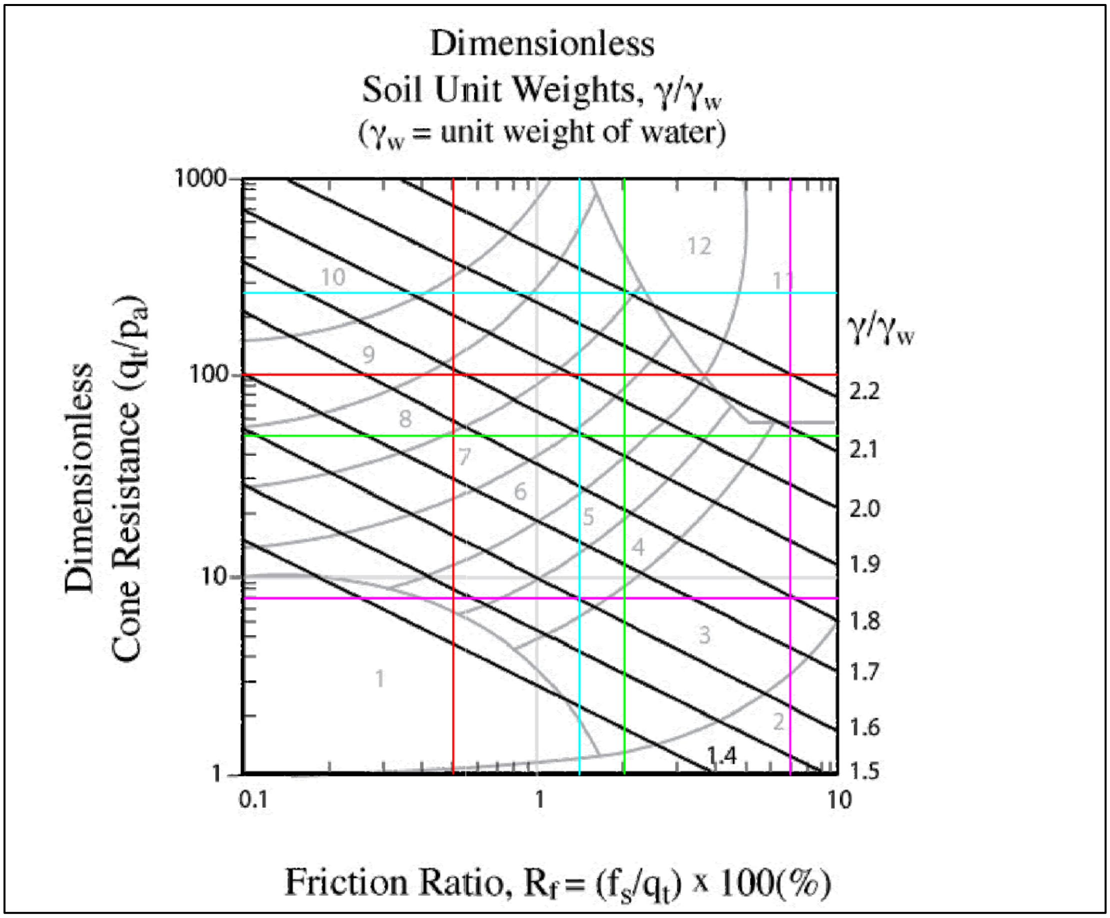


Figure C-5: Estimated Unit Weight Ratio, from "Estimating soil unit weight from CPT", P.K. Robertson and K.L. Cabal, Gregg Drilling and Testing Inc., Signal Hill, California, USA.

Stratum I: Upper Soft to Firm Fat Clays
Ground Surface to 7 feet below surface
Geotech Report from US 501 had ϕ values of 18.4-18.8 for Fat Clay (CH) recorded. 18.5 was selected.
Geotech Report from US 501 had cohesion of 180 psf recorded for Fat Clay (CH). This value was selected.

Stratum II: Intermediate Medium Dense to Dense Sands
Depth of 7ft to 13ft below surface
The soils of Stratum II typically exhibited an N60 value of about 5-20, with majority being in the range of 10-30.
UFC 3-220-10 tables 8-3, 8-4 were used to estimate the ϕ at 35 based on M. Dense sand and the N_{60} values.

Stratum III: Interbedded Silts, Clays, and Sands
Depth of 13ft to 25ft below surface
The soils of Stratum III typically exhibited an N60 value of about 5-20, with majority being in the range of 0-5.
UFC 3-220-10 tables 8-3, 8-4 were used to estimate the ϕ at 30 based on loose sand and the N_{60} values.

Stratum IV: Lower Medium Dense to Very Dense Sands

Depth of 25ft to maximum depth of 26.8 of test soundings.
The soils of Stratum II typically exhibited an N_{60} value of about 5-20, with majority being in the range of 10-30.
UFC 3-220-10 tables 8-3, 8-4 were used to estimate the ϕ at 40 based on dense to very dense sand and the N_{60} values.

C.1.2 Assumptions for Structural Inputs:

An analysis was performed at the tallest wall height at Station 13012.815. The water surface elevation for the 100yr 2075 is 11.281, the terrain elevation is at 2.71ft.

Adding two feet of freeboard to the WSE and subtracting the terrain elevation, a max wall height of 10.571ft was calculated with the top elevation at 13.3ft. It was assumed that the ground elevations on either side of the wall were equal. A debris impact load of 500lb/ft at top of the wall was included. The calculations were performed assuming δ′ = 0 and ca = 0. This should be conservative and require greater required sheet pile depth and higher design forces in the sheet pile. A maximum head differential was used for the analysis with the flood side water elevation to top of wall and groundwater elevation at ground surface.

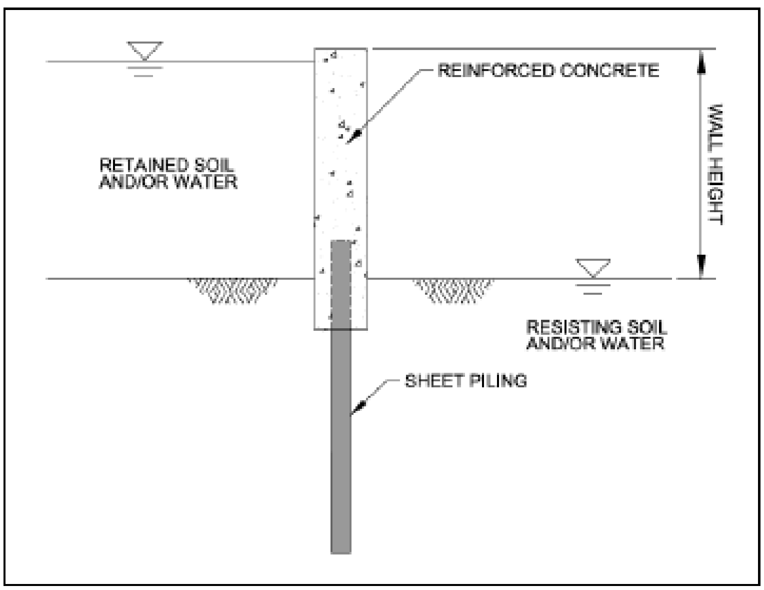


Figure C-6: Example Sheet Pile Wall with Concrete Cap

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS
DATE: 2-AUGUST-2024 TIME: 13:57:11

****
* INPUT DATA*
****

I.--HEADING
'SOCASTEE CREEK SHEET PILE WALL DESIGN

II.--CONTROL
CANTILEVER WALL DESIGN
FACTOR OF SAFETY FOR ACTIVE PRESSURES = 1.00
FACTOR OF SAFETY FOR PASSIVE PRESSURES = 1.50

III.--WALL DATA
ELEVATION AT TOP OF WALL = 13.30 FT.

IV.--SURFACE POINT DATA

IV.A.--RIGHTSIDE
DIST. FROM WALL (FT) ELEVATION (FT)
50.00 2.71

IV.B.—LEFTSIDE
DIST. FROM WALL (FT) ELEVATION (FT)
50.00 2.71

V.--SOIL LAYER DATA

V.A.--RIGHTSIDE
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURE= 1.00
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURE = 1.50

Table C-1:

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COHESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADHE-SION (PSF)	BOTTOM ELEV. (FT)	BOTTOM SLOPE (FT/FT)	SAFETY FACTOR ACT.	SAFETY FACTOR PASS.
112	112	18.5	180	0	0	-4.79	0	1	1.5
118	118	35	0	0	0	-10.79	0	1	1.5
121	121	30	0	0	0	-22.79	0	1	1.5
134	134	40	0	0	0			1	1.5

V.B.--LEFTSIDE
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURE= 1.00
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURE = 1.50

Table C-2:

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COHESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADHE-SION (PSF)	BOTTOM ELEV. (FT)	BOTTOM SLOPE (FT/FT)	SAFETY FACTOR ACT.	SAFETY FACTOR PASS.
112	112	18.5	180	0	0	-4.79	0	1	1.5
118	118	35	0	0	0	-10.79	0	1	1.5
121	121	30	0	0	0	-22.79	0	1	1.5
134	134	40	0	0	0			1	1.5

VI.--WATER DATA

UNIT WEIGHT = 62.40 (PCF)
RIGHTSIDE ELEVATION = 13.30 (FT)
LEFTSIDE ELEVATION = 2.71 (FT)
SEEPAGE ELEVATION = 2.71 (FT)
SEEPAGE GRADIENT = AUTOMATIC

VII.--VERTICAL SURCHARGE LOADS

NONE

VIII.--HORIZONTAL LOADS

VIII.A.--HORIZONTAL LINE LOADS

ELEVATION (FT)	LINE LOAD (PLF)
13.30	500.00

VIII.B.--HORIZONTAL DISTRIBUTED LOADS

NONE

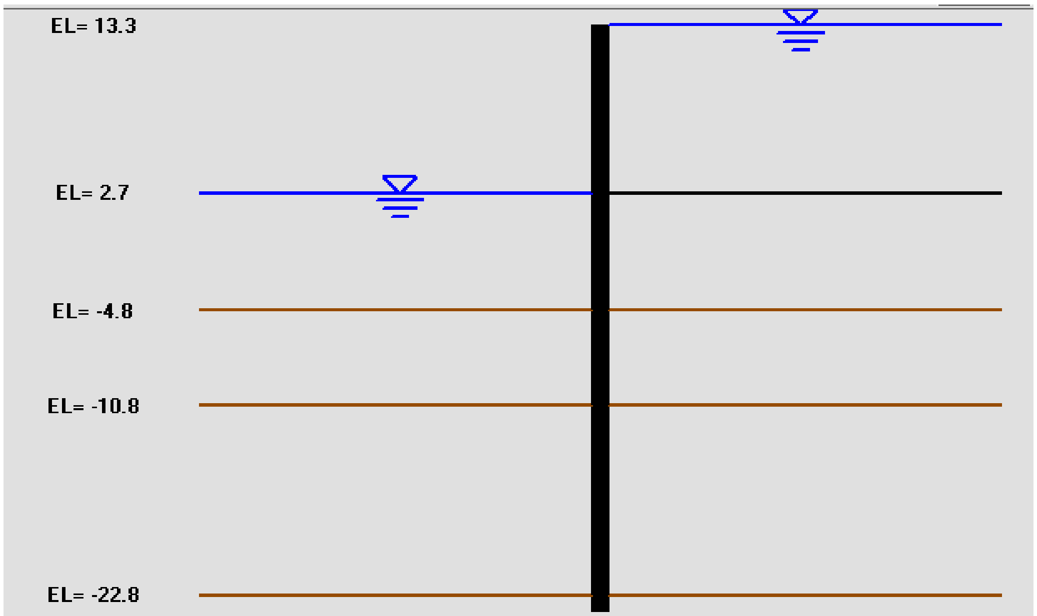


Figure C-7: Sheet Pile Wall Input Plot

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS
DATE: 2-AUGUST-2024 TIME: 14:01:01

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*************************
*   SOIL PRESSURES FOR   *
* CANTILEVER WALL DESIGN *
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I.--HEADING
'SOCASTEE CREEK SHEET PILE WALL DESIGN

II.--SOIL PRESSURES
RIGHTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS
AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.
LEFTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS
AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.
SOIL PRESSURES ARE REPORTED FOR A SEEPAGE GRADIENT = 0.0001
AND MAY CHANGE WITH AUTOMATIC ADJUSTMENT OF THE GRADIENT.

Table C-3:

ELEV. FT	NET WATER (PSF)	LEFTSIDE PASSIVE (PSF)	LEFTSIDE ACTIVE (PSF)	NET SOIL+ WATER ACTIVE (PSF)	NET SOIL+ WATER PASSIVE (PSF)	RIGHTSIDE ACTIVE (PSF)	RIGHTSIDE PASSIVE (PSF)
0	0	0	0	0	0	0	0
12.3	62.4	0	0	62.4	62.4	0	0

ELEV. FT	NET WATER (PSF)	LEFTSIDE PASSIVE (PSF)	LEFTSIDE ACTIVE (PSF)	NET SOIL+ WATER ACTIVE (PSF)	NET SOIL+ WATER PASSIVE (PSF)	RIGHTSIDE ACTIVE (PSF)	RIGHTSIDE PASSIVE (PSF)
11.3	124.8	0	0	124.8	124.8	0	0
10.3	187.2	0	0	187.2	187.2	0	0
9.3	249.6	0	0	249.6	249.6	0	0
8.3	312	0	0	312	312	0	0
7.3	374.4	0	0	374.4	374.4	0	0
6.3	436.8	0	0	436.8	436.8	0	0
5.3	499.2	0	0	499.2	499.2	0	0
4.3	561.6	0	0	561.6	561.6	0	0
3.3	624	0	0	624	624	0	0
2.7+	660.8	0	0	660.8	660.8	0	0
2.7-	660.8	299.4	0	361.4	960.2	0	299.4
2.3	660.8	331.1	0	329.7	991.9	0	331.1
1.7	660.8	376.6	0	284.2	1037.5	0	376.7
1.3	660.8	408.3	0	252.5	1069.1	0	408.3
0.3	660.8	485.5	0	175.3	1146.3	0	485.5
-0.7	660.8	562.7	0	98.1	1223.5	0	562.7
-1.7	660.8	639.9	0	20.9	1300.7	0	640
-2.0	660.8	660.8	0	0	1321.6	0	660.8
-2.7	660.7	717.1	0	-56.3	1377.9	0	717.2
-3.7	660.7	794.3	0	-133.5	1455.1	0	794.4
-4.7	660.7	871.5	0	-210.7	1532.3	0	871.6
-4.8+	660.7	878.4	0	-186.7	1508.4	0	878.6
-4.8-	660.7	917.3	100.8	-186.7	1508.4	100.8	917.5
-5.7	660.7	1042	114.5	-266.8	1588.5	114.5	1042.3
-6.7	660.7	1179.2	129.6	-388.9	1710.6	129.6	1179.4
-7.7	660.7	1316.3	144.6	-510.9	1832.6	144.7	1316.6
-8.7	660.7	1453.4	159.7	-632.9	1954.7	159.7	1453.7
-9.7	660.7	1590.5	174.8	-755	2076.7	174.8	1590.8
-10.7	660.6	1727.6	189.8	-877	2198.8	189.9	1728
-10.8+	660.6	1739.9	191.2	-744.3	2066.1	191.2	1740.3
-10.8-	660.6	1496.5	235.2	-744.3	2066.1	235.2	1496.9
-11.7	660.6	1609.6	252.9	-696	2017.7	253	1610
-12.7	660.6	1733.9	272.5	-800.7	2122.4	272.5	1734.3
-13.7	660.6	1858.2	292	-905.5	2227.2	292.1	1858.6
-14.7	660.6	1982.5	311.5	-1010.3	2332	311.6	1982.9
-15.7	660.6	2106.8	331.1	-1115	2436.8	331.1	2107.2
-16.7	660.6	2231	350.6	-1219.8	2541.5	350.7	2231.6

ELEV. FT	NET WATER (PSF)	LEFTSIDE PASSIVE (PSF)	LEFTSIDE ACTIVE (PSF)	NET SOIL+ WATER ACTIVE (PSF)	NET SOIL+ WATER PASSIVE (PSF)	RIGHTSIDE ACTIVE (PSF)	RIGHTSIDE PASSIVE (PSF)
-17.7	660.6	2355.3	370.1	-1324.5	2646.3	370.2	2355.9
-18.7	660.5	2479.6	389.7	-1429.3	2751.1	389.8	2480.2
-19.7	660.5	2603.9	409.2	-1534.1	2855.8	409.3	2604.5
-20.7	660.5	2728.2	428.7	-1638.8	2960.6	428.8	2728.8
-21.7	660.5	2852.5	448.3	-1743.6	3065.4	448.4	2853.1
-22.7	660.5	2976.8	467.8	-1848.4	3170.1	467.9	2977.4
-22.8+	660.5	2987.9	469.5	-2493.5	3815.4	469.7	2988.6
-22.8-	660.5	4096.1	306.3	-2493.5	3815.4	306.4	4097
-23.7	660.5	4285.5	320.5	-3304.5	4626.5	320.5	4286.5
-24.7	660.5	4493.7	336	-3497.1	4819.1	336.1	4494.7
-25.7	660.5	4701.9	351.6	-3689.7	5011.8	351.7	4702.9
-26.7	660.4	4910	367.2	-3882.4	5204.4	367.2	4911.1
-27.7	660.4	5118.2	382.7	-4075	5397	382.8	5119.3
-28.7	660.4	5326.4	398.3	-4267.6	5589.7	398.4	5327.6
-29.7	660.4	5534.6	413.9	-4460.2	5782.3	414	5535.8

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS
DATE: 2-AUGUST-2024 TIME: 14:01:11

****
* SUMMARY OF RESULTS FOR *
* CANTILEVER WALL DESIGN *
****

I.--HEADING
'SOCASTEE CREEK SHEET PILE WALL DESIGN

II.--SUMMARY
RIGHTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS AND THEORY OF ELLASTICITY EQUATIONS FOR SURCHARGE LOADS.
LEFTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS AND THEORY OF ELLASTICITY EQUATIONS FOR SURCHARGE LOADS.

WALL BOTTOM ELEV. (FT): -26.55
PENETRATION (FT): 29.26

MAX. BEND. MOMENT (LB-FT): 7.4808E+04
AT ELEVATION (FT): -13.37

MAX. SCALED DEFL. (LB-IN^3): 6.2180E+10

AT ELEVATION (FT): 13.30

SEEPAGE GRADIENT: 0.1808

NOTE: DIVIDE SCALED DEFLECTION MODULUS OF ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN^4 TO OBTAIN DEFLECTION IN INCHES.

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS
DATE: 2-AUGUST-2024 TIME: 14:01:11

****
* COMPLETE OF RESULTS FOR*
* CANTILEVER WALL DESIGN*

****

I.--HEADING
'SOCASTEE CREEK SHEET PILE WALL DESIGN

II.—RESULTS

Table C-4:

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	DEFLECTION (LB-IN^3)	PRESSURE (PSF)
13.3	0	500	62180000000	0
12.3	510.4	531	59603000000	62.4
11.3	1083.2	625	57026000000	124.8
10.3	1780.8	781	54451000000	187.2
9.3	2665.6	999	51879000000	249.6
8.3	3800	1280	49312000000	312
7.3	5246.4	1623	46751000000	374.4
6.3	7067.2	2029	44199000000	436.8
5.3	9324.8	2497	41660000000	499.2
4.3	12082	3027	39137000000	561.6
3.3	15400	3620	36635000000	624
2.71	17647	3999	35171000000	660.82
-2.71	17647	3999	35171000000	361.38
2.3	19316	4140	34160000000	327.68
1.71	21813	4319	32714000000	279.17
1.3	23606	4427	31718000000	245.46
0.3	28142	4631	29316000000	163.25
-0.7	32841	4753	26964000000	81.04
-1.69	37553	4793	24700000000	0

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	DEFLECTION (LB-IN^3)	PRESSURE (PSF)
-1.7	37621	4793	24668000000	-1.17
-2.7	42400	4751	22438000000	-83.38
-3.7	47096	4627	20280000000	-165.59
-4.7	51626	4420	18204000000	-247.8
-4.79	52023	4401	18021000000	-174.3
-5.7	55952	4232	16217000000	-196.89
-6.7	60067	3978	14327000000	-310.62
-7.7	63871	3611	12540000000	-424.35
-8.7	67250	3129	10864000000	-538.08
-9.7	70092	2535	9303900000	-651.81
-10.7	72281	1826	7864900000	-765.54
-10.79	72443	1762	7741400000	-653.6
-11.7	73780	1182	6550600000	-622.1
-12.7	74634	510	5363700000	-721.74
-13.7	74766	-262	4305700000	-821.37
-14.7	74077	-1133	3376800000	-921.01
-15.7	72467	-2104	2575800000	-1020.65
-16.7	69836	-3174	1899800000	-1120.28
-17.7	66085	-4344	1344300000	-1219.92
-18.7	61114	-5614	902890000	-1319.56
-19.7	54823	-6984	566860000	-1419.2
-20.7	47113	-8453	325370000	-1518.83
-21.57	39188	-9809	182040000	-1605.37
-21.7	37885	-10008	165060000	-1416.51
-22.7	27408	-10706	70033000	20.03
-22.79	26445	-10699	64077000	149.32
-23.7	16952	-9968	22368000	1456.58
-24.7	7951.3	-7793	4204000	2893.13
-25.7	1844.3	-4182	196950	4329.68
-26.55	0	0	0	5546.21

NOTE: DIVIDE SCALED DEFLECTION MODULUS OF ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN^4 TO OBTAIN DEFLECTION IN INCHES.

III.--WATER AND SOIL PRESSURES

Table C-5:

ELEVATION (FT)	WATER PRESSURE (PSF)	SOIL PRESSURE LEFTSIDE PASSIVE (PSF)	SOIL PRESSURE LEFTSIDE ACTIVE (PSF)	SOIL PRESSURE RIGHTSIDE ACTIVE (PSF)	SOIL PRESSURE RIGHTSIDE PASSIVE (PSF)
13.30	0	0	0	0	0
12.30	62	0	0	0	0
11.30	125	0	0	0	0
10.30	187	0	0	0	0
9.30	250	0	0	0	0
8.30	312	0	0	0	0
7.30	374	0	0	0	0
6.30	437	0	0	0	0
5.30	499	0	0	0	0
4.30	562	0	0	0	0
3.30	624	0	0	0	0
2.71+	661	0	0	0	0
2.71-	661	299	0	0	299
2.30	652	324	0	0	338
1.71	638	359	0	0	394
1.30	629	384	0	0	433
0.30	606	443	0	0	528
-0.70	584	503	0	0	623
-1.69	562	562	0	0	716
-1.70	561	562	0	0	717
-2.70	539	622	0	0	812
-3.70	516	682	0	0	907
-4.70	494	741	0	0	1002
-4.79+	492	747	0	0	1010
-4.79-	492	709	78	124	1126
-5.70	471	808	89	140	1276
-6.70	448	917	101	158	1441
-7.70	426	1027	113	176	1606
-8.70	403	1136	125	195	1771
-9.70	381	1245	137	213	1936
-10.70	358	1355	149	231	2101
-10.79+	356	1364	150	232	2116
-10.79-	356	1174	184	286	1820
-11.70	336	1265	199	307	1955
-12.70	313	1365	215	330	2103
-13.70	290	1466	230	354	2251
-14.70	268	1566	246	377	2399

ELEVATION (FT)	WATER PRESSURE (PSF)	SOIL PRESSURE LEFTSIDE PASSIVE (PSF)	SOIL PRESSURE LEFTSIDE ACTIVE (PSF)	SOIL PRESSURE RIGHTSIDE ACTIVE (PSF)	SOIL PRESSURE RIGHTSIDE PASSIVE (PSF)
-15.70	245	1666	262	400	2548
-16.70	223	1767	278	424	2696
-17.70	200	1867	293	447	2844
-18.70	178	1967	309	470	2992
-19.70	155	2068	325	494	3141
-20.70	132	2168	341	517	3289
-21.57	113	2255	354	537	3418
-21.70	110	2268	356	540	3437
-22.70	87	2369	372	563	3585
-22.79+	85	2378	374	566	3599
-22.79-	85	3260	244	369	4933
-23.70	65	3419	256	385	5153
-24.70	42	3595	269	403	5394
-25.70	20	3770	282	421	5635
-26.55	0	3922	293	437	5844
-26.70	0	3950	295	439	5871

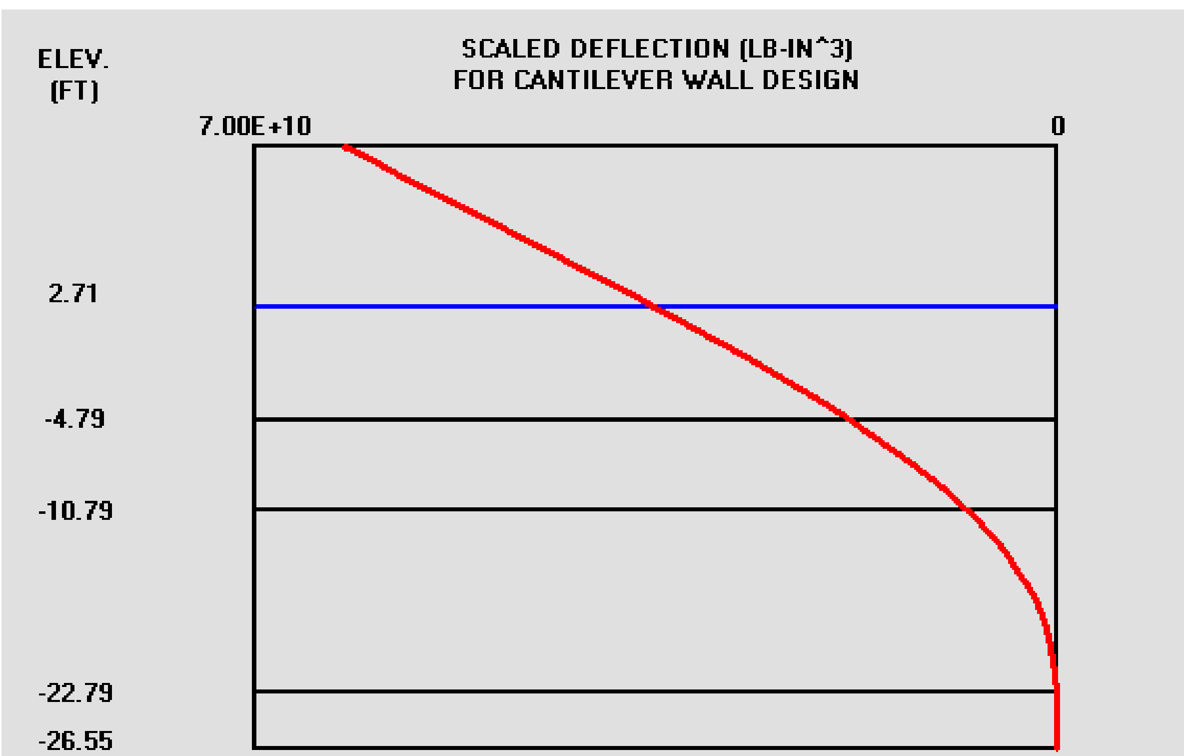


Figure C-8: Sheet Pile Wall Deflection Plot

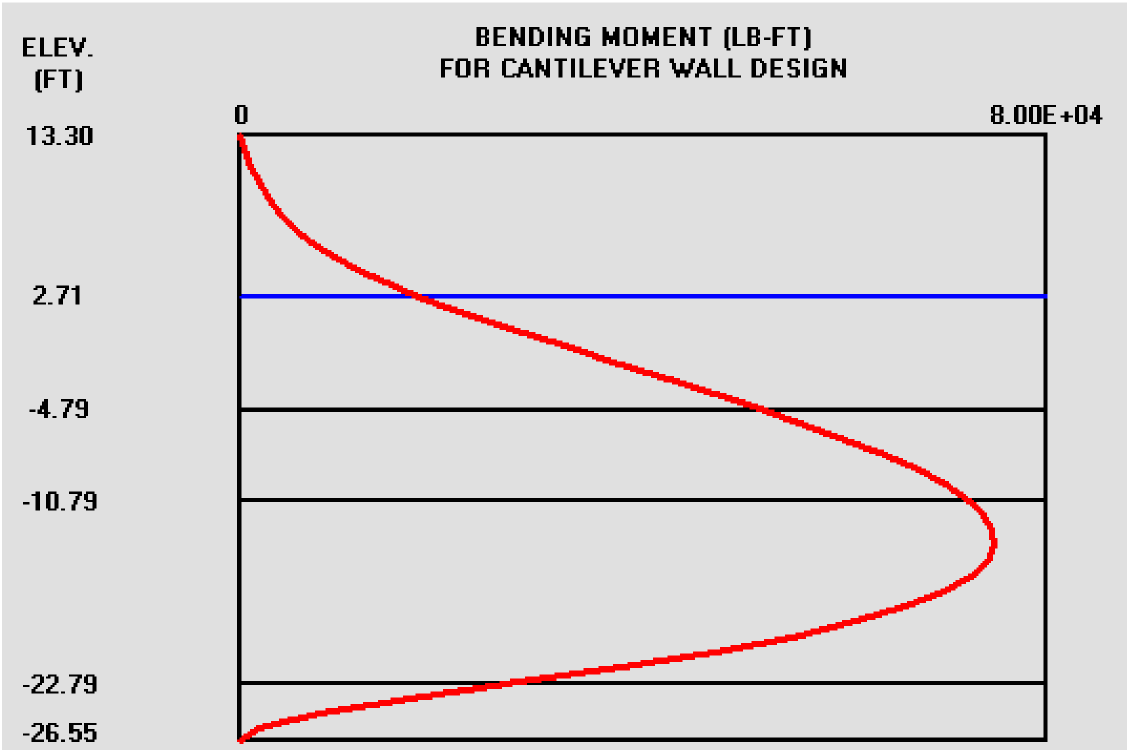


Figure C-9: Sheet Pile Wall Bending Moment Diagram

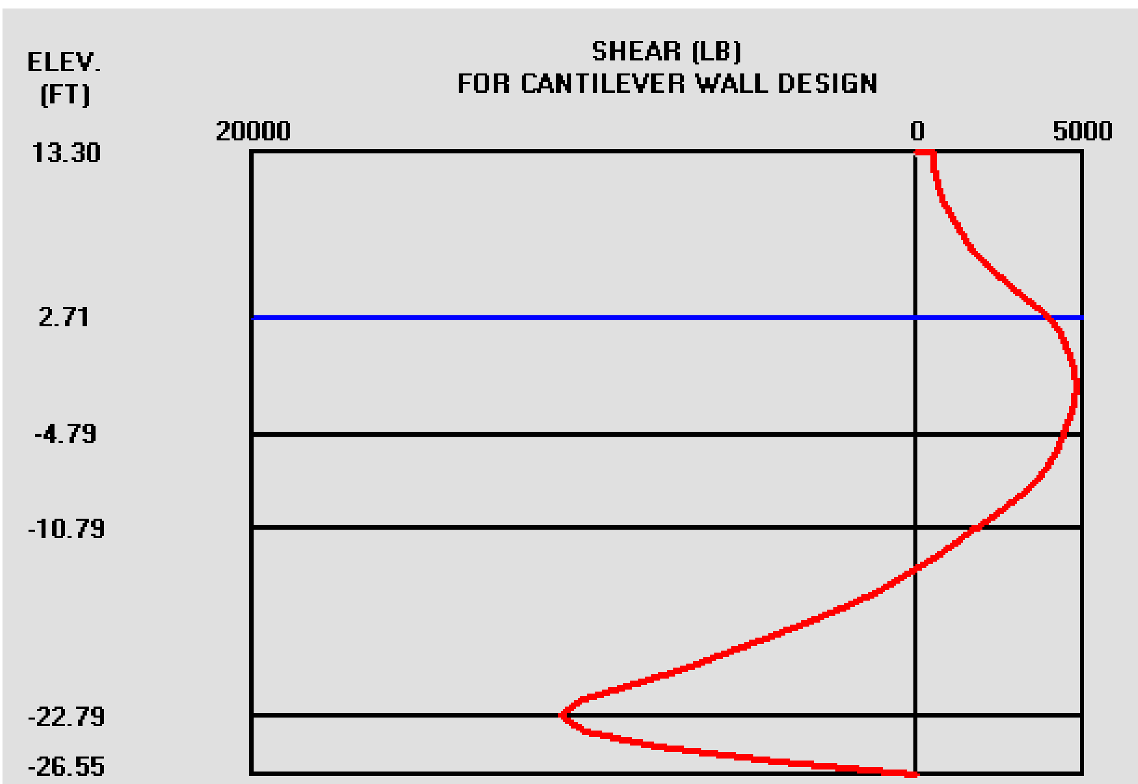


Figure C-10: Sheet Pile Wall Shear Diagram

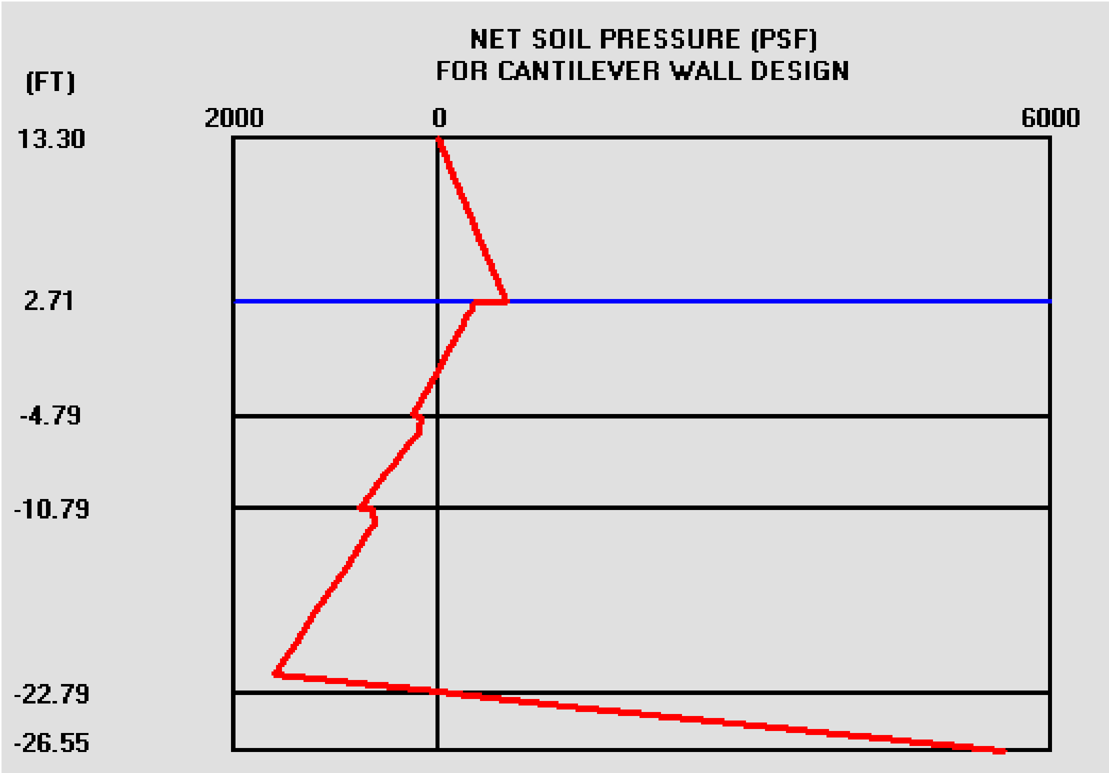


Figure C-11: Net Soil Pressure Plot

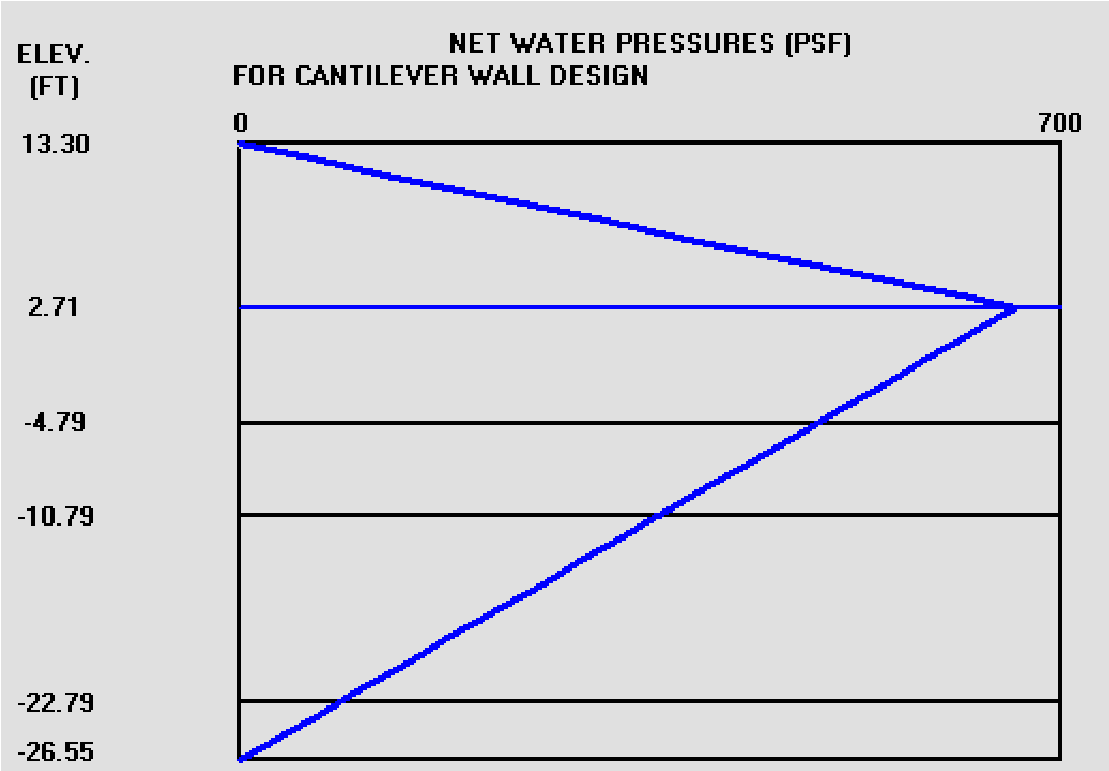


Figure C-12: Net Water Pressure Plot

C.1.3 Steel Sheet Pile Design

Maximum Moment = 74.8 kip-ft/ft = 897.6 k-in/ft

Maximum Shear = 10.7 kip/ft

Mu = 1.4(74.8 kip-ft/ft) = 104.72 kip-ft/ft = 1256.6 kip-in/ft

Vu = 1.4 (10.7 kip/ft) = 15.0 kip/ft

$\phi Mn \geq Mu,\ Mn = Fcr\ Smin$ (from AISC Equation F12.1)

Where:
Fcr – For driven hot rolled sheet pile, the members are restrained against lateral torsional buckling and the pile has sufficient thickness against local buckling; therefore, *Fcr = Fy*.

Smin = Sx

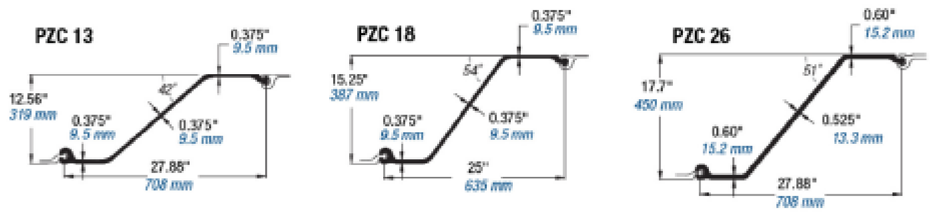
Therefore: *Mn = FySx* where *Fy* is the yield strength and *Sx* is the section modulus of the sheet pile.

$\phi Fy\ Sx \geq Mu$

Where:
(0.9)(50 ksi)Sx ≥ 1256.6 kip-in/ft

Sx-required ≥ 27.9 in^3/ft

A hot rolled steel sheet pile section PZC17 has a section modulus of 31 in^3/ft, which exceeds the required 27.9 in^3/ft. The shear capacity of the chosen sheet pile section must also be checked.



Minimum Grade 60 Standard					Per Single Section						Per Unit of Wall			
GERDAU	Nominal Width	Wall Depth (Height)	Web Thickness	Flange Thickness	Area	Weight	Moment of Inertia	Section Modulus	Total Surface Area	Nominal Coating Area*	Area	Weight	Moment of Inertia	Section Modulus
Section	in. (mm)	in. (mm)	in. (mm)	in. (mm)	in.2 (cm2)	lbs/ft (kg/m)	in.4 (cm4)	in.3 (cm3)	ft2/ft (m2/m)	ft2/ft (m2/m)	in.2/ft (cm2/m)	lbs/ft2 (kg/m2)	in.4/ft (cm4/m)	in.3/ft (cm3/m)
PZC 12	27.88 706	12.52 318	0.335 8.5	0.335 8.5	13.64 88.0	46.6 69.1	324.5 13,510	51.8 850	6.10 1.86	5.60 1.71	5.87 124.3	20.0 97.6	139.7 19,080	22.3 1,200
PZC 13	27.88 708	12.56 319	0.375 9.5	0.375 9.5	14.82 95.6	50.4 75.1	353.0 14,690	56.2 920	6.10 1.86	5.60 1.71	6.38 135.1	21.7 106.0	152.0 20,760	24.2 1,300
PZC 14	27.88 708	12.60 320	0.420 10.7	0.420 10.7	16.15 104.2	55.0 81.8	381.6 15,890	60.5 990	6.10 1.86	5.60 1.71	6.95 147.2	23.7 115.5	164.3 22,440	26.0 1,400
PZC 17	25.0 635	15.21 386	0.335 8.5	0.335 8.5	13.64 88.0	46.4 69.1	491.8 20,470	64.6 1,060	6.10 1.86	5.60 1.71	6.55 138.6	22.3 108.8	236.1 32,235	31.0 1,670
PZC 18	25.00 635	15.25 387	0.375 9.5	0.375 9.5	14.82 95.6	50.4 75.1	532.2 22,150	69.8 1,145	6.10 1.86	5.60 1.71	7.12 150.6	24.2 118.2	255.5 34,890	33.5 1,800
PZC 19	25.00 635	15.30 388	0.420 10.7	0.420 10.7	16.16 104.2	55.0 81.8	576.3 23,990	75.3 1,235	6.10 1.86	5.60 1.71	7.75 164.1	26.4 128.8	276.6 37,780	36.1 1,945
PZC 25	27.88 708	17.66 449	0.485 12.3	0.560 14.2	20.40 131.6	69.4 103.3	938.7 39,070	106.3 1,740	6.65 2.03	6.15 1.87	8.78 185.9	29.9 145.9	404.1 55,190	45.7 2,455
PZC 26	27.88 708	17.70 450	0.525 13.3	0.600 15.2	21.72 140.1	73.9 110.0	994.3 41,390	112.4 1,840	6.65 2.03	6.15 1.87	9.35 197.9	31.8 155.4	428.1 58,460	48.4 2,600
PZC 28	27.88 708	17.75 451	0.570 14.5	0.645 16.4	23.22 149.8	79.0 117.6	1,057 44,000	119.1 1,950	6.65 2.03	6.15 1.87	10.00 211.6	34.0 166.1	455.1 62,150	51.3 2,755

All dimensions given are nominal. Actual flange and web thicknesses vary due to mill rolling practices; however, permitted variations for such dimensions are not addressed.

Figure C-13: PZC Hot Rolled Sheet Pile Data Sheet. Source: http://www.jdfields.com

$\phi V_n \geq V_u$, where $V_n = 0.6(F_y)(A_w)$ (from AISC Equation G2-1)

and $A_w = A_v = (t_w h)/w$ (from Equation 9.4)

Where: $\phi = 0.9$ (from AISC section G1)

Therefore: $(\phi)0.6(F_y)(A_v) \geq V_u$

(0.9)(0.6)(50 ksi)(0.335 in.)(15.21 in.)/(2.08 ft) = 66.14kip/ft

66.14kip/ft ≥ 15.0 kip/ft. Therefore, shear is OK.

C.1.4 Concrete Cap Design

EM 1110-2-2104 requires design according to ACI 318 but with modifications. The design load case is an unusual load case and therefore reinforced concrete design is performed with single load factor of 1.6. This is the principal load factor for maximum hydrostatic loading with a return period in the unusual category, accounting for serviceability requirements, from EM 1110-2-2104.

Design for Full Section. According to paragraph 9.8.5.5 of EM 1110-2-2502, the top of the connection (top of sheet pile) will be designed for both moment (*Ma*) and shear (*Va*). The sheet pile is extended 36 in. into the concrete cap according to paragraph 9.8.5.2. With the bottom of the concrete set at the frost depth of 6 in. below the ground surface, the top of the sheet pile is one foot above the ground surface elevation of 3.71ft. The forces at the top of the sheet pile from the CWALSHT analysis are:

Ma = 15.4 kip-ft/ft
Va = 3.62 kips/ft

Checking bending moment, $\phi Mn \geq Mu$.
Mu = 1.6 (15.4 kip-ft/ft) = 24.64 kip-ft/ft

Cap Geometry. The concrete cap must provide a minimum of 6 in. (15 cm) of cover over the steel sheet pile but not less than 24 in. (61 cm) in width through the connection.
PZC 17 15.21 wall depth + 6in + 6in = 28in

Minimum cover from EM 1110-2-2104 is 3 in. for this application.

Minimum reinforcement for temperature and shrinkage is 0.0030 of the gross area from EM 1110-2-2104. The required area is 0.0030 (28 ft)(12 ft) = 1.0 in^2 with 0.5 in^2 each face. Try using #7 @ 12 in. with an area (*As*) of 0.6 in^2 per foot.

Calculation of M_n.
$M_n = A_s f_y (d - a/2)$

For this design, *fy* = 60 ksi, *f'c* = 4.0 ksi.
d = 28 in. – 3 in. (cover) – .5 = 24.5 in.

Design for a unit width, b, of 12:
a= $A_s f_y / 0.85 f'_c b$ = 0.6in^2 (60ksi) / 0.85(4.0ksi)(12in) = 0.88 in

M_n = 0.6 in^2 (60ksi) (24.5 in – (0.88/2)) = 866.16 kip-in/ft = 72.2 kip-ft/ft
From ACI 318-19, ϕ = 0.9 for bending.

ϕMn = 0.9(72.2 kip-ft/ft) = 65.0 kip-ft/ft which is greater than M_u = 24.64 kip-ft/ft

Check of reinforcing ratio (ρ) according to EM 1110-2-2104.

$P_{provided} = A_s / bd$ = 0.6 in^2/ft / 12in (24.5in) = 0.002

Check minimum reinforcing requirements. From EM 1110-2-2104 the minimum requirements are:

$$\rho > \frac{3.0\sqrt{f'_c}}{F_y} = \frac{3.0\sqrt{4{,}000\ psi}}{60{,}000\ psi} = 0.0032$$

$$\rho > \frac{200}{F_y} = \frac{200}{60{,}000\ psi} = 0.0033$$

Or that ρ provided is greater than 4/3 of ρ required.

Check that ρ is less than $0.25\rho_b$ as required by EM 1110-2-2104.

$\rho_b = 0.85 f'_c / f_y * \beta_1$ [87,000/(87,000 + 60,000psi)] = 0.0285
0.25 ρ_b = 0.25(0.0285) = 0.0071 > 0.002 OK

C.1.5 Deflections

The above analysis results in a deflection of 4.36 inches at the top of wall, and 2.5 inches at the ground surface. The maximum deflection at the ground surface for floodwalls according to EM 1110-2-2502, Table 9.3 for an unusual loading category is 1.5 inches. A full numerical analysis was not performed, and the deflections calculated using CWALSHT is a lower bound for what a numerical analysis would predict. This analysis was performed with a 1.5 factor of safety for passive pressure, which compounds factors of safety, and a factor of safety of 1.0 is recommend to be used for strength of structural elements per EM 1110-2-2502.

A sensitivity analysis was performed updated the CWALSHT analysis to use a 1.0 passive factor of safety. This resulted in a maximum bending moment of 604.8 k-in/ft, so the required section modulus would be 18.8 in^3/ft. A PZC 12 sheet pile section could be selected, and the ground surface deflection with this sheet pile size would be 1.35in. If a PZC 17 sheet pile that was originally selected was used, the ground surface deflection would be 0.9in using the CWALSHT analysis results. Further analysis to optimize the floodwall design would be required if this alternative was selected.

C.2 Flood Gate

A vertical lift gate structure was proposed in the Bucksport focus area along the Old Pee Dee Road Cowford Swamp Bridge. The exact geometry of the structure is unknown, including the span of the gate structure. Structural loads have not been calculated for the gate structure as the alternative was not carried forward, so the foundation required to support this structure is conceptual. No site-specific geotechnical data for this structure was obtained. Based on structural drawings of the adjacent Cowford Swamp bridge, it is assumed that a prestressed concrete pile deep foundation would be required for the gate structure.

C.3 Relief Bridges (Culverts)

Culverts under existing bridges in the Conway focus area were proposed to help connect the floodplain and improve conveyance by reducing bottle necking. The culvert locations are shown in Figures C-14 through C-16 and are referred to as 1) Site A: 905 Bridge, 2) Site B: 501 Business Bridge, and 3) Site C: 501 Bridge.

C.3.1 Project Description

Precast box culverts appear feasible for open-cut installation of the relief culverts located at Sites A and B. For Site C, both open-cut installation and trenchless methods could be utilized depending on highway department requirements. Trenchless methods are discussed in Section 3.3.4.3. Preliminary box culvert sizing details are indicated below:

Site A: SC 905 Bridge (Relief Culvert) *

72-in (width) x 36-in (clear barrel height) triple box culverts
Length (roadway width) = ~83-feet with variable length wingwalls

Road Surface Elevation = ~ 10 ft NAVD88
Inlet Invert Elevations = 3.6 ft NAVD88
Outlet Invert Elevation = 3.2 ft NAVD88

<u>Site B: Hwy 501 Business Bridge (Relief Culvert)</u> *

96-in (width) x 84-in (clear barrel height) double box culverts
Length (roadway width) = ~83-feet with variable length wingwalls
Road Surface Elevation = ~ 16.7 ft NAVD88
Inlet Invert Elevations = 2.9 ft NAVD88
Outlet Invert Elevation = 2.5 ft NAVD88

<u>Site C: Hwy 501 Bridge (Relief Culvert)</u>

96-in (width) x 84-in (clear barrel height) double box culverts *
10-ft (width) x 4-ft (clear barrel height) triple box culverts **
Length (roadway width) = ~127-feet w/ inlet and outlet headwalls
Road Surface Elevation = ~ 14 ft NAVD88
Inlet Invert Elevations = 3.0 ft NAVD88
Outlet Invert Elevation = 2.5 ft NAVD88

* Open-cut installation within existing roadway embankment.
** Trenchless installation within existing roadway embankment.

Figure C-14: Site A, 905 Relief Culvert Location.

Figure C-15: Site B, Highway 501 Business Relief Culvert Location.

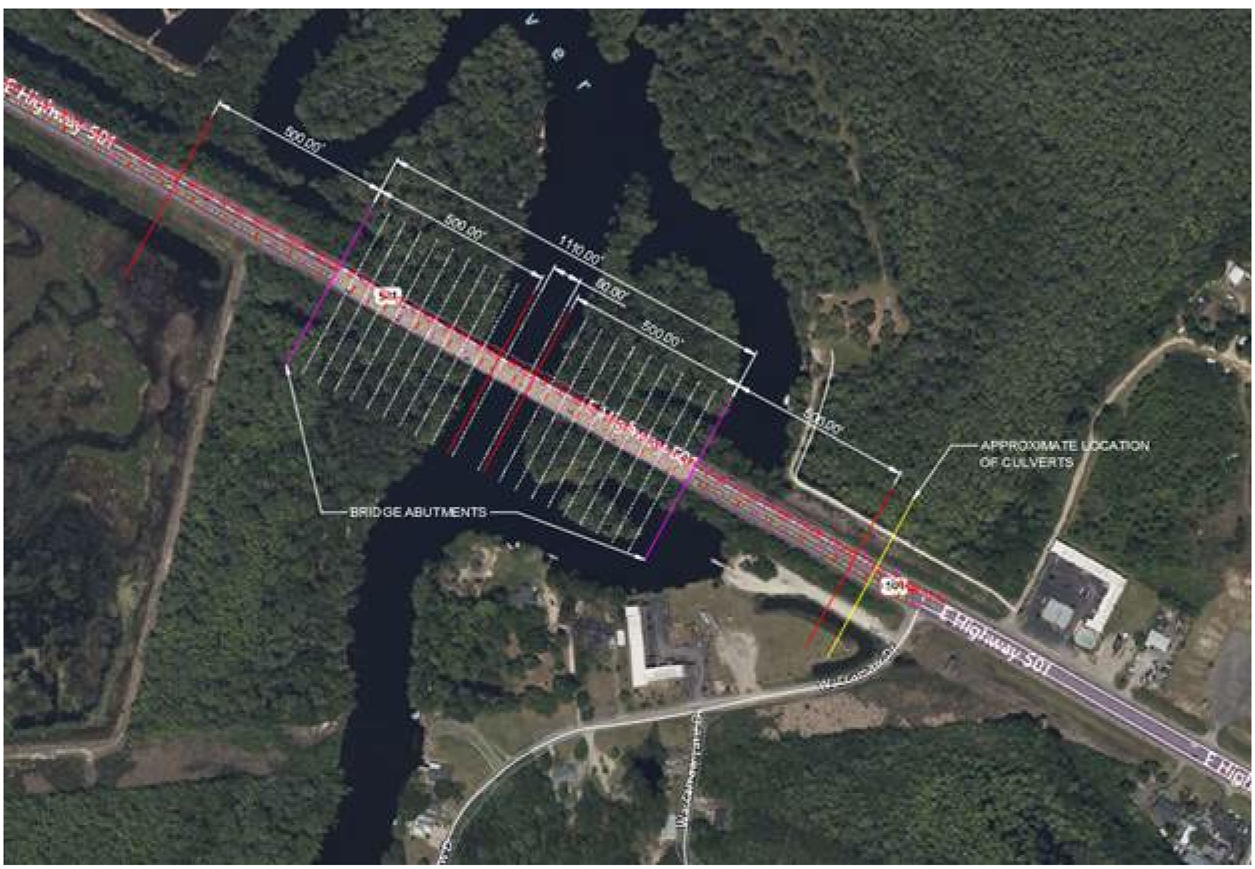


Figure C-16: Site C, Highway 501 Relief Culvert Location.

C.3.2 Preliminary Subsurface Investigation

The purpose of the preliminary subsurface investigation was to collect sufficient basic information to assist in the feasibility study. SCDOT subsurface investigation guidelines were generally adhered to for preliminary subsurface investigations for culverts/pipes that cross an alignment in a transverse direction, a current Average Daily Traffic greater than 5,000 vehicles per day, having a diameter greater than or equal to 48-in, and will be founded at or below the original grade. Our investigation did not include the following:

- Electro-chemical testing (pH, resistivity, chloride, and sulfate testing) to determine the potential impacts of the soils, groundwater, and surface water on the structural components.

C3.2.1 Site Reconnaissance

The proposed project sites and surrounding areas were visually inspected by a geotechnical engineer. The observations were used in planning the exploration, in determining areas of special interest, and in relating site conditions to known geologic conditions in the area.

C3.2.2 Field Exploration

The subsurface exploration consisted of two (2) standard penetration test (SPT) borings at each relief culvert location. The boring locations are shown in Figures C-17 through C-19. Ground surface elevations at each boring location were surveyed by the Savannah District. No historic borings were available within the project vicinity. In addition, surface shear wave velocity measurements were obtained using Refraction Microtremor (ReMi) methods.

Table C-6: Boring Locations and Depths

Site	Borings	Depth [1]
Site A, 905 Bridge	B-1 & B-2	44-ft (B-1), 15-ft (B-2)
Site B, 501 Business Bridge	B-3 & B-4	15-ft (B-3), 50-ft (B-4)
Site C, 501 Bridge	B-5 & B-6	15-ft (B-5), 32.5-ft (B-6)

1. Below existing ground surface (bgs)

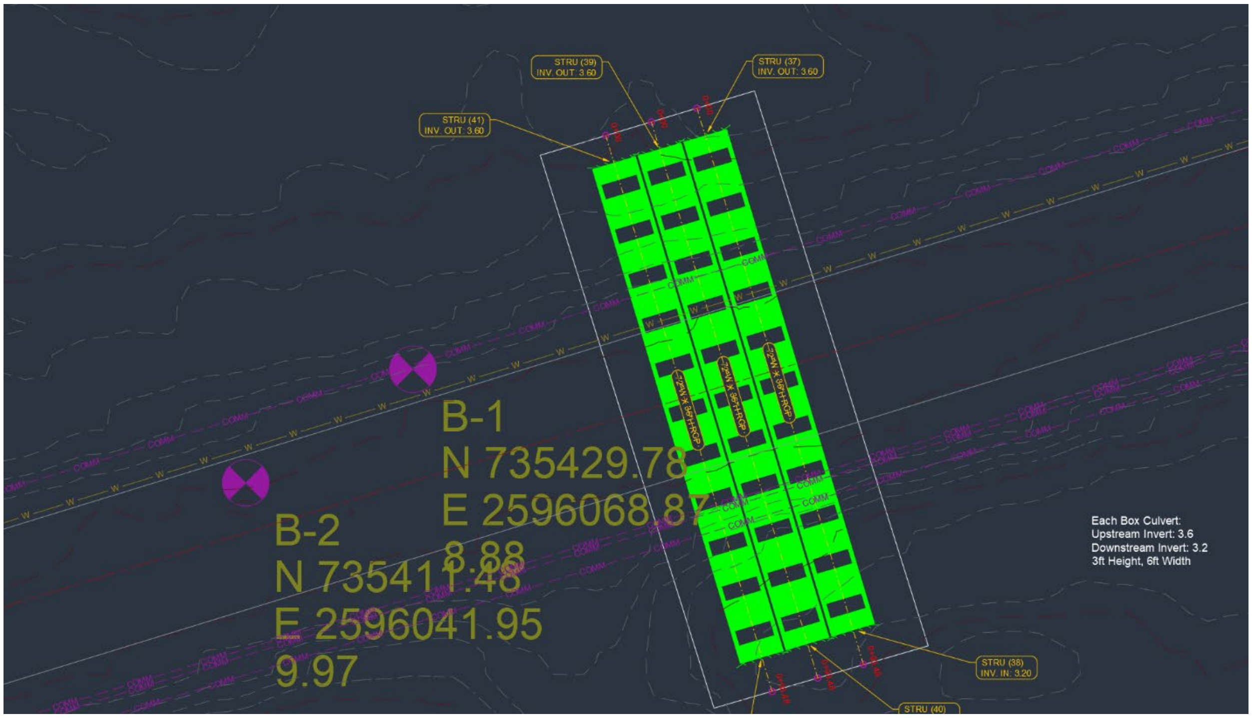


Figure C-17: Site A, 905 Bridge boring locations.

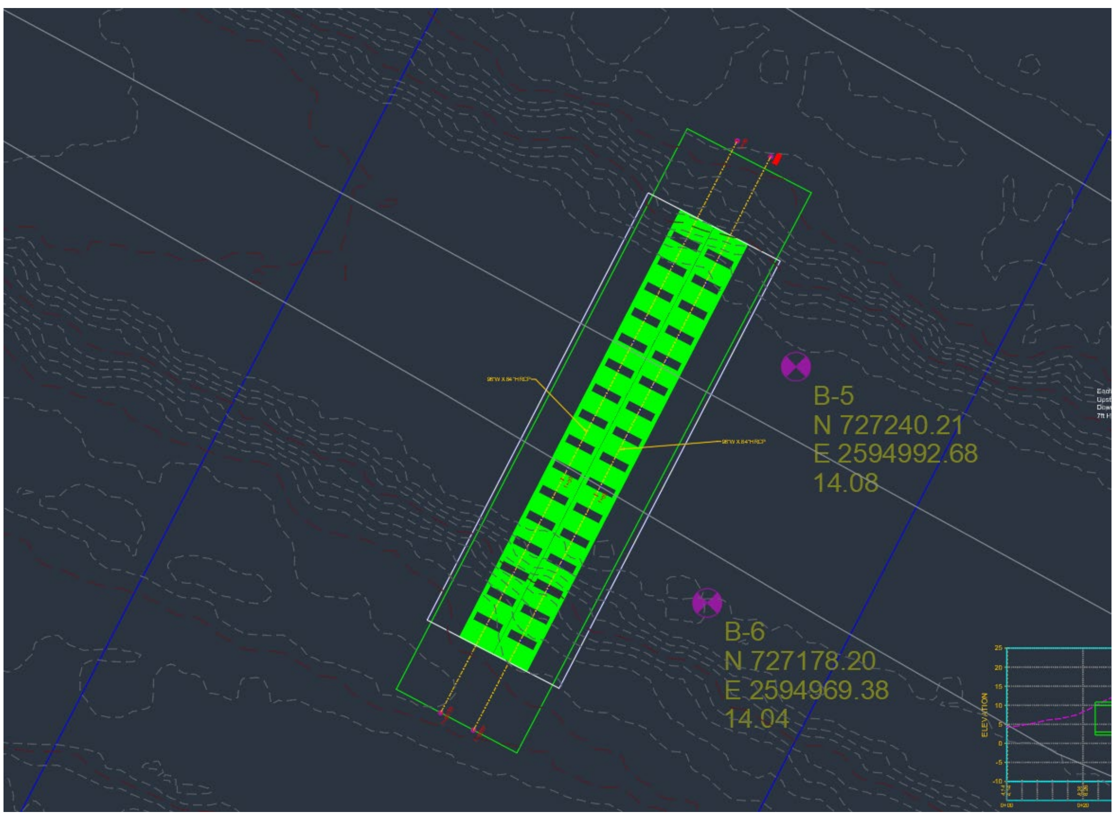


Figure C-18: Site B, 501 Business Bridge boring locations.

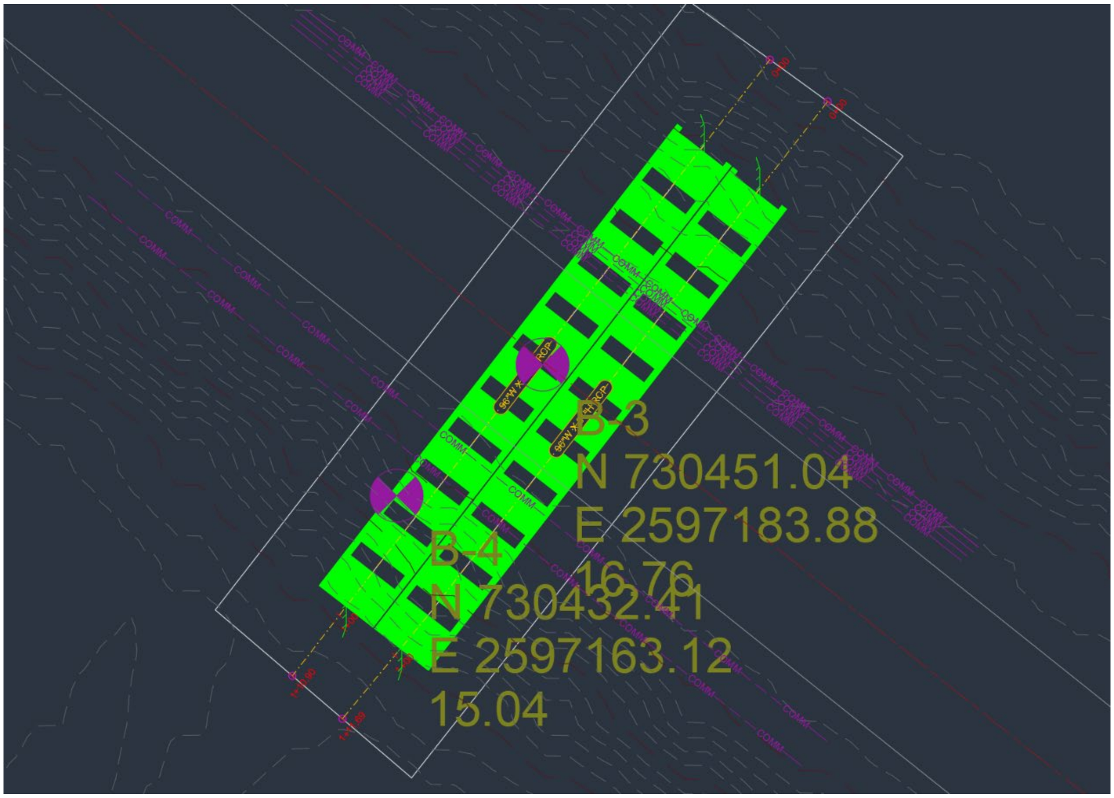


Figure C-19: Site C, 501 Bridge boring locations.

<u>Standard Penetration Tests (SPT)</u>

Borings B-1 through B-6 were drilled by mechanically twisting a continuous flight hollow stem steel auger into the soil. Standard penetration testing was performed in general accordance with ASTM D1586. At assigned intervals, soil samples were obtained with standard 1.4-inch I.D., 2-inch O.D., split-tube sampler. The sampler was first seated 6 inches to penetrate any loose cuttings and then driven an additional 12 inches with blows of a 140-pound hammer falling 30 inches. The number of hammer blows required to drive the sampler the final 12 inches was recorded and is designated the "penetration resistance". The penetration resistance, when properly evaluated, is an index to the strength of the soil and foundation supporting capability. For gravelly and fine-grained soils, SPT can be unreliable for obtaining strength and stiffness properties.

In some soils it is not always practical to drive a split-spoon sampler the full three consecutive 6-inch increments. Whenever more than 50 blows are required to drive the sampler over a 6-inch increment, or the sampler is observed not to penetrate after 50 blows, the condition is called split-spoon refusal. Split-spoon refusal conditions may occur because of obstructions or because the earth materials being tested are very dense or very hard. When split-spoon refusal occurs, often little or no sample is recovered. The SPT N-value for split-spoon refusal conditions is typically estimated as greater than 100 blows per foot (bpf). Where the sampler is observed not to penetrate after 50 blows, the N-value is reported as 50/0". Otherwise, the depth of penetration after 50 blows is reported in inches, i.e. 50/5", 50/2", etc.

An automatic hammer was used to advance the split-barrel samplers in the SPT borings. A significantly greater efficiency is achieved with the automatic hammers as compared to conventional safety hammers operated with a cathead and rope. Hammer efficiency was verified in accordance with ASTM D4633 and the energy ratio, C_E is denoted on the boring logs.

Disturbed SPT samples were obtained during drilling at designated test intervals. At boring B-1 (34 to 36-ft bgs) and B-6 (27.5 to 29.5-ft bgs),a relatively undisturbed sample was also obtained using a 3-in. O.D. Shelby tube in general accordance with ASTM D1587.

Classification of the soil samples was performed in general accordance with ASTM D2488 (Visual-Manual Procedure for Description of Soils). The soil classifications include the use of the Unified Soil Classification System described in ASTM D2487 (Classification of Soils for Engineering Purposes). Since the soil descriptions and classifications are based on visual examination and laboratory testing, they should be considered approximate. Where encountered, depth to groundwater is denoted on the boring logs provided in Appendix A-4 Attachments.

Laboratory Soil Testing

Laboratory tests were performed on selected samples in general accordance with ASTM standards. The laboratory testing performed consisted of classification and index property testing (D6913, D4318, D2216, D2487). In addition, permeability testing (ASTM D5084, Method F) was performed on each undisturbed sample. Laboratory test results are provided in Appendix A-4 Attachments.

Surface Wave Testing

Surface wave tests were performed at the locations depicted in Figures C-17 through C-19. Due to site constraints the test arrays consisted of linear array placed along the grass shoulder parallel to the roadways. The test data were collected using 4.5-Hz surface geophones. The linear arrays utilized 24 geophones spaced at 10-foot intervals resulting in array lengths of 230 feet. The data were recorded using a DAQlink, 24-bit seismic acquisition system using a 1 millisecond sampling rate. Individual record lengths were 15 seconds in duration, and approximately 22 individual recordings were performed at each site. The energy waves captured in the recordings consisted of both passive ambient background noise as well as generated active noise created by personnel running along the geophone arrays and hits on the ground with sledgehammer blows. Shear wave velocity results are provided in Appendix A-4 Attachments.

Compression wave velocity profiles were also requested to be provided. The Vp values were estimated using the Vs values determined. The following equation was used to determine the Vp values (n = Poisson's ratio). The Vp profiles and values are also provided in Appendix A-4 Attachments.

$$Vp = \frac{Vs}{\left[1 - \frac{0.5}{1 - v}\right]^{0.5}}$$

Poisson's ratio was assumed to be 0.35 for all soil layers.

C.3.3 Site and Subsurface Conditions

C3.2.3 Site Description

The relief culverts will be located east of Conway, SC and traverse existing highway embankments that vary up to about 7½ -ft in height.

Site A: the culvert location will traverse SC Hwy 905 approximately 0.6 miles east of N. Main street in Conway, SC. Contours of adjacent slopes and review of project as-built data indicate the road embankment has a height of approximately 6-ft at this location.

Site B: the culvert location will traverse SC Business Hwy 501 approximately 1.1 miles southeast of Hwy 905 in Conway, SC. Contours of adjacent slopes and review of project as-built data indicate the road embankment has a height that varies up to approximately 7.5-ft at this location.

Site C: the culvert location will traverse SC Hwy 501 approximately 1.5 miles southeast of SC Hwy 701 in Conway, SC. Contours of adjacent slopes and review of project as-built data indicate the road embankment has a height that varies up to about 7.5-ft at this location.

C3.2.4 Area Geology and Regional Seismicity

The regional geology of the Waccamaw River basin in Conway, SC is characterized by relatively young, unconsolidated sediments of the Coastal Plain province, which range in age from the Cretaceous to the Quaternary. The topography is low-relief, and the river itself is a blackwater system that meanders through swamps and tidal wetlands. The surface and near-surface geology are dominated by these extensive, loose sandy and clayey deposits, which overlie older, more consolidated formations at depth.

A significant underlying geological unit is the Late Cretaceous Pee Dee Formation, which does not typically outcrop at the surface in the immediate Conway area but is a key part of the subsurface stratigraphy of the lower Coastal Plain. This formation consists of marine shales, clayey sands, and sands, and is often found in the subsurface unconformably below younger Cenozoic sediments. The Pee Dee Formation is a significant part of the regional aquifer system and is also known for its fossil content.

Overlying the older units like the Pee Dee Formation and the Pliocene Bear Bluff Formation are the materials that define the modern landscape, notably the Pliocene and Pleistocene Waccamaw Formation and the Late Pleistocene Conway Formation. The Waccamaw Formation, which can be found in outcrops north of the area in North Carolina, is composed of soft limestones and loose, gray to buff, fine quartz sands, often rich in marine fossils such as large shells and echinoids. In the Conway area, the Waccamaw Formation is typically found in the shallow subsurface, often overlain by even younger Pleistocene terrace deposits and Holocene alluvium along the river.

The most recent geological activity has shaped the present-day Waccamaw River basin and its distinctive features like the nearby Carolina Bays. The river basin's surface is mainly composed of recent alluvium (swamp and marsh deposits of Holocene age) and Pleistocene terrace deposits. The Waccamaw River flows through a low-gradient landscape, and its current position is the result of a regional, southwestward migration of the ancestral Pee Dee river system over millions of years, driven by factors including barrier-island deposition and subsurface paleotopography. The water in the river is tea-colored due to organic matter from decaying vegetation in the surrounding swamps.

The Waccamaw River basin near Conway, South Carolina, is situated in an area of the state with a moderate to high seismic risk, despite not being a region typically associated with major earthquakes globally. South Carolina experiences an average of 10 to 20 minor earthquakes annually, with most being too small to be felt and generally registering below magnitude 3.0. The most seismically active area in the state is the Coastal Plain, particularly around Summerville and Bowman, which experienced the catastrophic 1886 Charleston earthquake (estimated magnitude 7.3). While the Waccamaw basin is not in the most active specific cluster, the potential for seismic activity is a recognized hazard for the broader region.

C3.2.5 Subsurface Soil Conditions

Borings B-1 & B-2

The soil borings were advanced through approximately 12 inches of asphalt pavement. Underlying the asphalt pavement, embankment fill sampled as (USCS SP-SM, SM) was encountered to depths of 6-ft bgs and 3-ft bgs in borings B-1 and B-2, respectively. SPT N_{60}-values in the fill varied from 11 to 28 bpf. Underlying the road embankment, interbedded sands and sandy clays (USCS: SC-SM, SP-SM, SM, CL, SM, SP, SC type soils) were encountered to depths as great as 31½ ft-bgs. SPT N_{60}-values varied from 4 to 29 in the sands and 2 to 16 bpf in the sandy clays.

In the deeper boring (B-1), fat clay (USCS: CH) was encountered to boring termination at 44 ft-bgs. SPT N_{60}-values in the clay varied from 18 to 50/5”.

Seismic Array No. 1: Site A: SC 905

Based on the shear wave velocity profile in the upper 100 feet, the $V_{s,100}$ values at Site A varied between 380 ft/s and 820 ft/s. The average shear wave velocity within the upper 100-ft, $V_{s,100}$ = 707 ft/s which classifies as a Site Class D in accordance with *2020 LRFD Bridge Design Specifications*, refer to Table C-7.

Table C-7: Site A Shear Wave and Compression Wave Velocity Profile

Depth Interval (ft-bgs)		Shear Wave Velocity, V_s (ft/s)	Compression Wave Velocity, V_p (ft/s)
0	6.5	545	1,135
6.5	10.7	689	1,434
10.7	15.74	589	1,226
15.74	21.79	380	791
21.79	35.27	488	1,016
35.27	46.47	669	1,393
46.47	56.92	807	1,680
56.92	78.68	820	1,707
78.68	100	887	1,846

Average $V_{s,100}$ = 707 ft/s

Borings B-3 & B-4

The soil borings were advanced through approximately 12 inches of asphalt pavement. Underlying the asphalt pavement, embankment fill sampled as (USCS: SP-SM, SM, SP) was encountered to depths of 7½ and 5½ ft bgs in borings B-3 and B-4, respectively. SPT N_{60}-values in the fill varied from 2 to 31 bpf. Underlying the road embankment, interbedded sands and sandy clays (USCS: SP-SM, SM, CL, SM, SP, SC type soils) were encountered to depths as great as 49 ft-bgs. SPT N_{60}-values varied from 5 to 35 in the sands and 2 to 16 bpf in the sandy clays. In the deeper boring (B-4), marl-stone was encountered at a depth of 49 feet in boring B-4. SPT N_{60} values exceeded 100 bpf.

Seismic Array No. 2: Site B: US Hwy 501B

Based on the shear wave velocity profile in the upper 100 feet, the $V_{s,100}$ values at Site B varied between 384 ft/s to 1,271 ft/s. The average shear wave velocity within the upper 100-ft, $V_{s,100}$ = 753 ft/s which classifies as a Site Class D in accordance with *2020 LRFD Bridge Design Specifications*, refer to Table C-8.

Table C-8: Site A Shear Wave and Compression Wave Velocity Profile

Depth Interval (ft-bgs)		Shear Wave Velocity, V_s (ft/s)	Compression Wave Velocity, V_p (ft/s)
0	3.12	638	1,328
3.12	6.85	570	1,187
6.85	11.34	710	1,478
11.34	21.33	384	799
21.33	31.49	558	1,162
31.49	39.24	809	1,684
39.24	51.19	1,009	2,100
51.19	65.09	921	1,917
65.09	85.9	895	1,863
85.9	100	1,271	2,646

Average $V_{s,100}$ = 753 ft/s

Borings B-5 & B-6

The soil borings were advanced through approximately 12 inches of asphalt pavement. Underlying the pavement, embankment fill sampled as (USCS: SP, SP-SM) was encountered to depths of 3½ and 4-ft bgs in borings B-5 and B-6, respectively. SPT N_{60} values in the fill varied from 7 to 35 bpf. Underlying the road embankment, interbedded sands and sandy clays (USCS: SP-SM, SM, CL, SM, SP, SC type soils) were encountered to depths as great as 18½ ft-bgs. SPT N_{60}-values varied 5 to 58 in the sands and weight of hammer to 8 bpf in the clays. In the deeper soil test boring (B-6), fat clay (USCS: CH) was encountered from approximately 18½ to 30 ft bgs. SPT N60 values in the fat clay varied from weight hammer to 2 bpf.

Note: A ‘weight of hammer’ event occurs during standard penetration testing when the hammer and drill string (drilling rod and split spoon sampler) are allowed to rest on the bottom of the borehole, and they sink under their own weight.
Underlying the fat clay, marl-stone was encountered to the boring termination depth of 32½ ft bgs in boring B-6. SPT N_{60} values exceeded 100 bpf.

Seismic Array No. 3: Site C: SC 905

Based on the shear wave velocity profile in the upper 100 feet, the $V_{s,100}$ values at Site C varied between 480 ft/s and 1,208 ft/s. The average shear wave velocity within the upper 100-ft, $V_{s,100}$ = 811 ft/s which classifies as a Site Class D in accordance with *2020 LRFD Bridge Design Specifications*, refer to Table C-9.

Table C-9: Site A Shear Wave and Compression Wave Velocity Profile

Depth Interval (ft-bgs)		Shear Wave Velocity, V_s (ft/s)	Compression Wave Velocity, V_p (ft/s)
0	3.13	480	999
3.13	12.06	1,208	2,515
12.06	14.58	665	1,384
14.58	17.60	399	831
17.60	22.26	413	860
22.26	26.62	512	1,066
26.62	41.07	909	1,892
41.07	64.40	868	1,807
64.40	90.07	999	2,080
90.07	100	1,016	2,115

Average $V_{s,100}$ = 811 ft/s

The subsurface descriptions are of a generalized nature to highlight the major subsurface stratification features and material characteristics. The logs of the SPT borings should be reviewed for specific information at individual boring locations. The stratifications shown on the logs represent the soil conditions only at the specific exploration location. Variations may occur and should be expected between locations. The stratification lines shown represent the approximate boundary between the subsurface materials; the actual transition may differ.

C3.2.6 Groundwater Conditions

Groundwater was encountered at the boring locations at depths between 7½ and 10 ft bgs during drilling activities. At Site B, groundwater was encountered at depths between 7.5 and 9 feet bgs. It is expected that groundwater depths at Sites A and C will be influenced by water levels in the Waccamaw River. Table C-10 presents typical and key historical water level data for the Waccamaw River at the Conway Marina gauge (USGS 021107004) near Conway, SC.

Table C-10: Gauge Data for Waccamaw River at Conway Marina [1]

Normal River Stage	4.0 to 10.0-ft
Minor Flood Stage	11.0-ft
Moderate Flood Stage	12.0-ft
Major Flood Stage	14.0-ft
Highest Recorded Crest	21.16-ft

Notes:

1. Specific normal levels vary due to the river being tidally influenced, causing daily fluctuations, as well as seasonal precipitation changes. USGS Gauge 02117004, Datum El = 127.5 NAVD88.

Fluctuations in the elevation of the groundwater should be anticipated with changing climatic and rainfall conditions and will be particularly affected locally by water levels in the creek. Therefore, water levels during construction at other times in the life of the project may be higher or lower than the levels indicated on the boring logs. The possibility of water level fluctuations should be considered when developing the design and construction plans for the project.

In addition, perched water could develop in sand seams and above lower permeability clayey soils following periods of heavy or prolonged precipitation. A perched-water condition occurs when water seeping downward is slowed by a low permeability soil layer, such as silt or clay, and saturates the more permeable overlying soil. The perched-water level can be any number of feet above the true groundwater level. Due to the prevalence of interbedded sand and clays at the project site, the successful design-build contractor should expect to encounter perched water during construction.

C.3.4 Geotechnical Considerations

C3.4.1 Construction Dewatering and Water Diversion

Construction of culverts through the road embankments (having varying height up to about 7.5-feet) will necessitate temporary measures for managing both surface water flow and potential groundwater infiltration, despite the normal groundwater table being 5 feet deep at each location. Given the low height of the embankment and the standard depth of the culvert installation, traditional dewatering wells or well-points may not be necessary if the work is timed correctly during dry periods. However, a contingency plan is required for dewatering the immediate excavation area during active construction to ensure a stable working platform and dry conditions for culvert placement and backfill.

Effective water management for this project will likely require diversion techniques to manage periodic surface water issues. Upstream and downstream cofferdams or sandbag dikes can be utilized to block the existing flow path through the embankment area, channeling the water around the work zone via temporary flumes, pipes, or diversion channels. We anticipate standard sump pumping will likely be sufficient to remove incidental rainwater or minor seepage, keeping the groundwater level below the working depth.

Construction during drier seasons of the year could alleviate potential problems associated with the surface and groundwater conditions. Saturated soils comprising the sides of the proposed excavation are likely to slough into the excavation. It is recommended that construction traffic be excluded from the culvert slab and wingwall foundation areas as this traffic could loosen the bearing soils. A crushed aggregate working platform or concrete mud mat placed in the base of the wet excavation may be required to limit construction activity damage.

C3.4.2 Open-Cut Excavations

For open-cut excavations, either excavated side slopes or vertical cut excavations are feasible. For vertical cut excavations greater than 5 feet in depth, excavations will require the use of a trench box or shoring and bracing to prevent sloughing and caving of the soil into the excavation. The contractor should use a trench box or shoring and bracing as necessary to maintain a safe and clean excavation which meets Occupational Safety and Health Administration (OSHA) requirements, reference also EM 385-1-1.

In lieu of shoring, bracing, or trench boxes for excavations greater than 5 feet, OSHA standards provide recommendations for the design of temporary sloped excavations with a depth less than 20 feet. The OSHA standards provide maximum allowable slopes contingent on three designated soil types: Type A, Type B, or Type C. According to OSHA standards, temporary sloped excavations should be no steeper than

0.75-horizontal on 1-vertical (0.75H:1V) for Type A soils, 1H:1V for Type B soils, and 1.5H:1V for Type C soils. The surface slopes should be protected from deterioration and weathering if they are left open for significant periods of time. For situations where a higher strength soil is underlain by a lower strength soil, and the excavation extends into the lower strength soil, the slope of the entire excavation is governed by that required for the lower strength soil.

C3.4.3 Trenchless Methods

For Site C, Hwy 501, trenchless method may be required; however, existing utilities, settlement risk, and movement of the overlying road are likely to be the biggest challenges. In general, soil cover between the top of the culvert and below the compacted subgrade should be at least one pipe diameter to prevent road damages and limit settlement of the roadway to tolerable levels. For groundwater conditions below the culvert invert elevation, multiple methods appear feasible and are listed below along with their advantages/disadvantages:

Open-Shield Pipe Jacking:

- Steerable
- Limited ability to stabilize face and therefore potential over-excavation in running ground conditions (e.g. loose clean sand)
- Requires something to push against (back wall of a shaft/pit or reaction structure)
- Multiple rigid pipe material alternatives (steel, RCP, etc.)

Pipe Ramming:

- Non-steerable
- Risk of potential settlement due to vibration if the sand is very loose/loose
- Potential issues with instability at the face in running ground conditions like loose clean sand (additional risk of settlement)
- Risk of heave given the low clearance from the road
- Noise/vibration from hammer
- Steel pipe only

Auger Boring:

- Minimal steering capability unless installed in conjunction with pilot tubes (e.g. guided auger boring)
- Risk of potential settlement, particularly if face is unstable (e.g. loose sand)
- 96-inch pipe would be on the upper end of typical diameter
- Steel pipe only

Box Jacking:

- Difficult to maintain stability at the excavation face under shallow cover
- Increased jacking forces in sandy soil due to high frictional resistance
- High potential for ground settlement, soil collapse into overcut or annular space
- Precast box culverts of standard sizes up to 12-ft x 12-ft, possibly larger

C3.4.4 Strength and Service Limit States

We anticipate the bridge culverts will be designed using LRFD specifications. For service limit design of the culvert bearing slabs and wingwall foundations bearing on approved ground, a preliminary factored bearing resistance (q_r) of 2.0 ksf may be used in and should limit settlements to tolerable levels. At the service limit state, the LRFD resistance factor (φ_b) is 1.0. The factored bearing resistance assumes that the bearing surfaces are relatively dry 'dewatered', undisturbed, and clean of loose soil. The service limit state will likely control design.

Since the roadway embankments have been in service for many years, stresses induced by the culverts are estimated to be less than those exerted by the road embankment. Therefore, settlement will be controlled by the quality and degree of soil compaction achieved in the open-cut excavations.

For trenchless methods, settlement can be estimated once the configuration of the culvert and type of installation have been determined. Regardless of which trenchless method is used, a soil cover (between the bottom of the road subbase and top of pipe) of at least one pipe diameter is recommended to reduce damages to the roadway. Settlement monitoring of the roadway should be anticipated during and after construction where trenchless methods are utilized. Settlement is anticipated to occur rather quickly in the sandy soils. Therefore, grouting of the annular space outside of the culvert should be accomplished immediately after construction.

C3.4.5 Lateral Earth Pressures

The determination of lateral earth pressures will be required for culvert design. The "at rest" ($\mathrm{K_o}$) earth pressure should be used in the case of unyielding walls (e.g., box culverts). While passive earth pressure can provide some resistance, it is not reliably mobilized due to the amount of displacement required and can be conservatively ignored.

Beyond the static at-rest pressures, the design of the box culvert must also consider the surcharge loads imposed by road traffic and the potential for increased lateral pressures during dynamic events. The vertical stress from the embankment fill and surface traffic is distributed laterally, applying additional horizontal load onto the culvert walls. These pressures can be estimated using elastic theory or established design methods like the AASHTO LRFD specifications. Accounting for these additional live loads will be necessary for structural design and overall project viability.

C3.4.6 Seismic Design Parameters

The 2020 AASHTO LRFD Bridge Design Specifications provide guidelines for determining the seismic hazard at a bridge site. The seismic hazard for a bridge site is characterized by the acceleration response spectrum and the site factors for the relevant site classification. Table C-11 presents a summary of the seismic site classification, hazard coefficients, and the site factors.

Table C-11: Seismic Site Classification and Ground Motion Parameters

Site A, 905 Bridge (Relief Culvert)	
Design Parameter	Value
Seismic site class	D [1]
Geometric-Mean Maximum Considered Earthquake (MCE_G) peak acceleration for the user-specified site class	PGA_M = 0.21g
Design spectral response acceleration (0.2s period)	S_{DS} = 0.32g
Design spectral response acceleration (1.0s period)	S_{D1} = 0.17g
Liquefaction Potential in Event of an Earthquake	Not susceptible
Site B, 501 Business Bridge (Relief Culvert)	
Design Parameter	Value
Seismic site class	D [1]
Geometric-Mean Maximum Considered Earthquake (MCE_G) peak acceleration for the user-specified site class	PGA_M = 0.21g
Design spectral response acceleration (0.2s period)	S_{DS} = 0.32g
Design spectral response acceleration (1.0s period)	S_{D1} = 0.17g
Liquefaction Potential in Event of an Earthquake	Not susceptible
Site C, 501 Bridge (Relief Culvert)	
Design Parameter	Value
Seismic site class	D [1]
Geometric-Mean Maximum Considered Earthquake (MCE_G) peak acceleration for the user-specified site class	PGA_M = 0.22g
Design spectral response acceleration (0.2s period)	S_{DS} = 0.32g
Design spectral response acceleration (1.0s period)	S_{D1} = 0.17g
Liquefaction Potential in Event of an Earthquake	Not susceptible

Surface shear wave velocity measurements were utilized to determine the seismic site classification per the *2020 LRFD Bridge Design Specifications*.

Based on the coefficients and factors in Table C-11 above, the relief culvert sites are assigned to Seismic Performance Zone 2 in accordance with the 2020 AASHTO LRFD Bridge Design Specifications. It is recommended that the structure be designed based on 2020 AASHTO LRFD Bridge Design Specifications, Sections 3.10.9 and 4.7.4 (for seismic) specifications. Further seismic analyses were not performed.

C3.4.7 Additional Subsurface Exploration

Further exploration and evaluation may be required prior to final design and should comply with the requirements of the SCDOT. The scope of additional work will depend on the culvert location/installation method, final grade elevations, and the loading conditions, among other things. It may include additional borings to obtain additional soil information and possible laboratory testing to determine engineering characteristics of soil strata. Additional data regarding the existing pavement section and ground-water table may also be necessary.

C.4 Weir Removal

Removal of the existing weirs on Socastee Creek in the Socastee focus area was proposed to improve conveyance. The demolition of these weirs will require sediment control BMPs to mitigate sediment transport downstream. The side slopes where the existing weirs are located will need to be permanently stabilized to mitigate erosion post-demolition. No site-specific geotechnical data for these structures were obtained and further geotechnical considerations for these measures will be developed.

D NONSTRUCTURAL MEASURES

D.1 Elevation

Elevation of residential structures has been proposed in all focus areas. Structure elevation will likely include a deep pile foundation. Based on limited information of the regional geology, difficult excavation due to rock is unlikely. However, debris and other unsuitable material may be encountered during the excavation operations.

Excavation trenches near the existing structures should be graded such that rainwater does not saturate the soils beneath the existing foundation. Temporary unwatering measures, by sump pumps, drainage ditches, or other methods as determined by the contractor, may be needed to control surface water during excavation operations.

E REPORT LIMITATIONS

The geological information provided in this report is based on general data obtained for the SC coastal plain area, SCDNR Geologic Quadrangle Maps, and limited geotechnical reports from adjacent construction in the area. This report does not account for human placed materials, existing organic materials, and/or surficial deposits that may overlay the geological formation. Site specific groundwater information is not available at the time of this report. Groundwater can vary based on site topography, seasons, rainfall, and other factors. Impermeable to semi-impermeable surfaces, such as concrete, rock, clay, debris, etc., can cause perched groundwater conditions. Site specific investigations can help the engineers and contractors have a better understanding of the subsurface conditions at the proposed work sites.

F REFERENCES

U.S. Army Corps of Engineers, Engineering Manual (EM) 1110-2-2502 (Floodwalls and Other Hydraulic Retaining Walls). Issued 2022

UFC 3-220-10. Soil Mechanics. Unified Facilities Criteria (UFC), 2022.