October 28, 2024

Gresham Smith
2095 Lakeside Centre Way, Suite 120
Knoxville, Tennessee 37922

ATTENTION: Mr. Jason Brady
Jason.Brady@greshamsmith.com

Subject: **REPORT OF PRELIMINARY GEOTECHNICAL EXPLORATION**
State Route 353 Emergency Bridge Replacement
Washington County, Tennessee
Bridge No. 90S238600011
UES Project A24109.02271

Dear Mr. Brady:

We are submitting the results of the geotechnical exploration performed for the subject project. The geotechnical exploration was performed in accordance with our Proposal No. 24109.02271, dated October 1, 2024. The following report presents our findings and recommendations for the proposed project. Should you have any questions regarding this report, or if we can be of any further assistance, please contact us at your convenience.

Sincerely,
UES, LLC

10-28-24

Ibrahim M. Aklouk, P.E.
Geotechnical Project Manager
TN PE 127,662

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Senior Geotechnical Engineer

Stephen R. Martin, P.E.
Geotechnical Department Manager

REPORT OF
GEOTECHNICAL EXPLORATION

State Route 353 Emergency Bridge Replacement

Bridge No. 90S238600011
Washington County, Tennessee

UES Project No. A24109.02271.000

Submitted to:

Gresham Smith
2095 Lakeside Centre Way, Suite 120
Knoxville, Tennessee 37922

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BRIDGE FOUNDATION REPORT
SR-353 Emergency Bridge Repair

PREPARED FOR: Gresham Smith
PREPARED BY: UES, LLC

Project Reference SR-353 Bridge No. 90S238600011 Region 1
Project Number To Be Determined County Washington
Location Station 17+70.00 - Station 22+35.00
Geotechnical Engineer Ibrahim M. Aklouk, P.E. Drill Crew Tri-State Drilling, LLC
Total Number of Borings 2
Total Footage 66.5 linear feet of soil drilling and 60 linear feet of rock core
Date Drilled October 2 to October 4, 2024

1.0 INTRODUCTION AND PURPOSE

1.1 PURPOSE

The purpose of our services was to evaluate the subsurface conditions for bridge foundation construction. This information will be utilized to provide geotechnical recommendations, including design parameters for abutment and intermediate column bent foundation support. Driven H-piles are anticipated for the abutments and rock-bearing drilled shaft are anticipated for the interior bents.

The proposed site is located along Bailey Bridge Road in Chuckey, Tennessee (Washington County). This report provides a discussion of the project, site geology, subsurface conditions, our recommendations for H-pile and drilled shaft foundation design, as well as seismic design parameters. This exploration and report have been performed in general accordance with the TDOT Geotechnical Guidelines, dated October 2023.

1.2 PROJECT AND SITE DESCRIPTION

Our understanding of the project information has been developed during email correspondence and a site visit with Mr. Jason Brady, P.E. and Mr. Patrick Fiveash, P.E. of Gresham Smith and Mr. Stephen Martin, P.E. of UES on September 30, 2024. The previous State Route 353 (SR-353) bridge (Bridge ID No. 90S238600011) over the Nolichucky River in Washington County, Tennessee was demolished during the recent flooding after Hurricane Helene. We have been provided with a site plan of the proposed bridge titled *Present Plan*, as prepared by the Tennessee Department of Transportation (TDOT).

We understand the previous design was for a 4-span concrete box beam bridge; however, due to the significant erosion/destruction caused, the bridge will be completed under a design build and design may change.

The current plans indicate the proposed new bridge will begin at approximately Station 17+70.00 and continue until Station 22+35.00, a distance of about 465 feet. No structural loading information is available at this time. Based on experience with similar projects, we anticipate the proposed bridge will be supported at the abutments using driven H-piles and at the interior column bents using drilled shafts.

This report includes recommendations for drilled shaft foundation design in accordance with AASHTO LRFD Bridge Design Specifications and TDOT requirements, as applicable. This report has been prepared based on the soil test boring and rock core information obtained during this exploration. Based on the requirements outlined in the State of Tennessee Department of Transportation Geotechnical Engineering Guidelines (TDOT GES) dated October 2023, U.S. DOT Federal Highway Administration Publication No. FHWA NHI-01-031 dated May 2002 and AASHTO LRFD Bridge Design Specifications, Nineth Edition dated 2020.

1.3 SCOPE OF STUDY

The geotechnical exploration involved a site reconnaissance, field exploration, laboratory testing, and engineering analysis. The following sections of this report present discussions of the field exploration, laboratory testing programs, site conditions, and conclusions and recommendations. Following the text of this report, Appendix A presents figures, general notes and test boring records, Appendix B presents photographic logs of the rock cores, Appendix C presents Laboratory Test Results, and Appendix D shows the engineering calculations.

The geotechnical scope of services did not include an environmental assessment for determining the presence or absence of wetlands, or hazardous or toxic materials in the soil, bedrock, surface water, groundwater, or air, on, or below, or around this site. Any statements in this report or on the boring logs regarding odors, colors, and unusual or suspicious items or conditions are strictly for informational purposes.

2.0 EXPLORATION AND TESTING PROGRAMS

2.1 FIELD EXPLORATION

The site subsurface conditions were explored by drilling two (2) soil test borings at accessible locations in the immediate vicinity of the existing bridge: one (1) on the southwestern side (B-1) of the existing bridge and (1) on the northeastern side (B-2) of the existing bridge.

The soil test borings were drilled between October 2 and 4, 2024, and advanced using hollow stem augers and a track-mounted drill rig. The approximate locations of the test borings are shown in Figure 2 in Appendix A of this report. The depths referenced in this report are those that existed at the time of the field exploration and the ground surface elevations referenced in this report were provided by Gresham Smith. Detailed logs for the borings can be found in Appendix A of this report.

Standard Penetration Tests (SPT) and split-spoon sampling were performed at approximately 2½-foot intervals in the upper 10 feet and 5-foot intervals thereafter. The drill crew worked in general accordance with ASTM D6151 for Hollow Stem Auger (HSA) drilling. SPT and split-spoon sampling were performed in accordance with ASTM D1586. In addition, two relatively undisturbed (Shelby tube) samples were attempted to be collected from each boring in general accordance with ASTM D 1587. However, they were found to have been disturbed during the sampling, extrusion and testing process.

In the split–spoon sampling, a standard 2-inch O.D. split-spoon sampler is driven into the bottom of the boring with a 140-pound hammer falling a distance of 30 inches. The number of blows required to advance the sampler the last 12 inches of the standard 18 inches of total penetration is recorded as the Standard Penetration Resistance (N-value). These N-values are indicated on the boring logs at the test depth and provide an indication of the consistency of fine-grained soils and relative density of coarse-grained soils.

Upon encountering auger refusal, rock coring was performed in each boring using a rotary drill rig utilizing an NQ2 core barrel equipment in general accordance with ASTM D2113. The core barrel is rotated at high speeds and is capable of cutting hard rock. Samples of cored material from the swivel-mounted inner barrel are removed, then classified and the recovery ratio (REC) and Rock Quality Designation (RQD) are determined. The sample REC is defined as the length of core retained divided by the total length core expressed as a percent. The RQD is defined as the cumulative sum of recovered hard core pieces 4 inches and longer divided by the total length cored. The sample recovery and RQD are a measure of the character and continuity of the material penetrated and are indications of the quality of the rock.

A log of the rock core material encountered at the boring locations was prepared in the field. After recovery, each sample was removed from the sampler and the full sections of extracted rock were placed in sturdy core boxes for transport to the laboratory facility for visual classification.

2.2 LABORATORY TEST PROGRAM

After completion of the field drilling and sampling phase of this project, the soil samples were returned to our laboratory where they were visually-manually classified in general accordance with FHWA NHI-01-031 by a UES geotechnical professional. Select samples were then tested for moisture content (ASTM D2216), Atterberg limits (ASTM D4318), grain size nest of sieves (ASTM D6913), and unconfined compressive strength of rock (ASTM D7012). The laboratory testing are discussed herein and presented in Appendix B of this report.

3.0 GEOLOGIC CONDITIONS

3.1 GEOLOGIC CONDITIONS

The project site lies in the Appalachian Valley and Ridge Physiographic Province of East Tennessee. This province is characterized by elongated, northeasterly-trending ridges formed on highly resistant sandstone and shale. Between ridges, broad valleys and rolling hills are formed primarily on less resistant limestone, dolomite, and shale.

Published geologic information indicates that the site is underlain by bedrock of the Knox Group. The Knox Group is composed of the Mascot Dolomite, Kingsport Dolomite, Longview Dolomite, Chepultepec Dolomite, and Copper Ridge Dolomite Formations. However, the Knox Group is not differentiated into its individual formations in this area. The Knox Group, where undivided, consists of siliceous dolomite with inter-bedded limestone. These rock units weather to produce a thick residual clay overburden. Silica in the form of chert is resistant to weathering and is scattered in various quantities throughout the clay residuum.

The site geology has also been influenced by water-deposited (alluvial) materials within the flood plain of the nearby Nolichucky River. These alluvial materials are usually soft and compressible, having never been consolidated by pressures in excess of their present overburden. Alluvial material composed of black, tan, brown, orangish brown, light gray, and white silts and sands were encountered underlying existing fill material at this site.

Since the bedrock underlying this site contains carbonate rock (i.e., limestone/dolomite), it is susceptible to the hazards of irregular weathering, cave and cavern conditions, and overburden sinkholes. Carbonate rock, while appearing very hard and resistant, is soluble in slightly acidic water. This characteristic, plus differential weathering of the bedrock mass is responsible for these hazards. Of these hazards, the occurrence of sinkholes is potentially the most damaging to overlying soil-supported structures. Sinkholes occur primarily due to differential weathering of the bedrock mass and flushing of overburden soil into the cavities within the bedrock. This loss of solids creates a cavity, or dome, within the overburden. Growth of the cavity over time, or excavation over the dome, can create a condition in which rapid subsidence, or collapse, of the roof of the dome occurs.

3.2 SUBSURFACE CONDITIONS

The following subsurface description is of a generalized nature to highlight the subsurface stratification features and material characteristics at the boring locations. The boring logs attached to this report should be reviewed for specific information at each boring location. Information on actual subsurface conditions exists only at the specific boring locations and is relevant only to the time that this exploration was performed. Variations may occur and should be expected at the site.

Surficial Materials

Initially, boring B-1 encountered a surficial layer consisting of approximately 10 inches of asphalt, followed by 10 inches of gravel. We note that boring B-2 did not encounter surficial layer, as that portion of the site was eroded from the hurricane. As such, we anticipate the actual depth of surficial materials may vary across the site and between our widely spaced borings.

Fill

Underlying the surficial materials and from the ground surface B-2, both boring encountered apparent fill materials. Fill is a material which has been transported and placed by man and machine. The fill materials generally consisted of brown, reddish brown, orangish brown, tan, and black fat (high plasticity) and lean (low plasticity) clayey soils with varying amounts of gravel, sand, silt, and mica. In addition, boring B-1 encountered a layer of tan, brown, and gray clayey and sandy gravels with varying amount of silt and rock fragments. We note that boring B-1 encountered asphalt fragments within its fill matrix. The fill materials extended to depths ranging from approximately 8 to 17 feet below existing grade.

The United Soil Classification System (USCS) Group Symbols for the fill soils are CH, CL, and GC-GC. The American Association of State Highway and Transportation Officials (AASHTO) Sub-Groups of these soils are A-7, A-6, A-1-a (hereafter denoted as CH, A-7; CL, A-6; GC-GC, A-1-a).

The SPT N-values in the fill ranged from 7 blows per foot (bpf) to 50/1" (50 blows per 1 inch of penetration), indicating firm to hard consistencies within the fine-grained materials and medium dense to very dense relative densities in the coarse-grained materials. We note that SPT N-values greater than 15 bpf may have been influenced by the presence of dense materials, such as gravel, rock fragments, and dense sand within the fill matrix.

Alluvium

Underlying the fill materials, boring B-2 encountered alluvial materials. Alluvium is a material which has been deposited by water. The alluvial materials generally consisted of black, tan, and brown lean clays (CL, A-6) with mica, and silt. In addition, orangish brown, light gray, tan, white, and brown coarse-grained sand (SP, A-1-b) with rounded rock fragments was encountered. The alluvial extended to 32.3 feet below existing grade, including auger refusal depth.

The N-values in the loess ranged from 9 bpf to 50/1", indicating a soil consistency of stiff to very stiff in the fine-grained materials and relative densities of medium dense to very dense.

Residuum

Beneath the fill materials, boring B-1 encountered apparent residual soils. Residual soils were formed by the weathering of the parent bedrock. These materials generally consisted of tan, orangish brown, reddish brown, light gray fat (CH, A-7) and lean (CL, A-6) clays with varying amounts of black manganese nodules, silt, mica, and rock fragments.

The SPT N-values within the residual materials ranged from 14 bpf to 50/1"indicating firm to hard consistencies in the coarse-grained materials. The exception was the isolated samples between 25 and 30 feet, which had SPT N-values of 0 to 2 bpf, indicating very soft consistencies in the fine-grained materials.

Refusal

Auger refusal was encountered in both locations (B-1 and B-2) at approximately 34.2 to 32.3 feet below the existing grade (~1392.54 to 1392.77), respectively. Auger refusal is a designation applied to materials that cannot be penetrated by the power auger. Auger refusal may indicate hard materials, such as rock boulders, ledges or pinnacles, or the top of continuous bedrock.

Upon encountering refusal materials, rock coring was attempted in both borings. The cores in B-1 and B-2 were extended to depths of approximately 69.2 to 57.3 feet below existing grade using rock coring techniques, respectively. The rock was classified as light to dark gray, white, and black slightly to moderately weathered dolomite and shaly dolomite with varying amounts of calcium seams and vugs.

The percent recovery (amount of rock recovered from the core barrel versus the total depth cored) obtained from these borings ranged from 8 to 100 percent, while the Rock Quality Designation (RQD) values between 0 to 92 percent, indicating very poor to excellent quality from an engineering standpoint. The lower recovery values in the case of boring B-1 are thought to be the result of shaley materials washing away during the coring process and indications of voids or discontinuities were not noted during drilling.

We note that the rock data presented in this report is preliminary and was only conducted to determine the top of rock elevation. An additional exploration will be required at each of the proposed foundation locations, following TDOT guidelines.

Groundwater

Groundwater was encountered in both boring (B-1 and B-2) at depths of approximately 22 to 19 feet below the existing ground surface (~1404.74 to 1406.07) at the time of drilling, respectively. We note that stabilized water levels can sometimes be difficult to obtain as some of the encountered soils are known to be relatively impermeable. In addition, each boring was backfilled upon completion in consideration of safety so delayed water levels were not recorded.

It is possible for groundwater to exist within the depths explored during other times of the year depending upon climatic and rainfall conditions. Additionally, discontinuous zones of perched water may exist within the overburden materials. The depth of groundwater will approximate the level of surface water in the Nolichucky River in areas near the river and this depth will vary with changes in river water level. The groundwater information presented in this report is the information that was collected at the time of our field activities.

4.0 LABORATORY TEST RESULTS

Natural moisture contents, liquid limit, plastic limit, and plasticity index tests (collectively referred to herein as Atterberg limits); and grain size tests were performed on selected split-spoon samples. These tests were used to confirm our visual-manual classifications and classify the soil samples using the USCS system.

Laboratory testing of select samples indicated in-situ moisture content values ranging from 5.4 to 44.4 percent, which varied with depth. In addition, Atterberg limit testing was performed on select samples from two borings (B-1 and B-2) between approximately 3.5 to 10 feet below existing grade. The samples yielded liquid limits between 26 and 37 and plasticity indices between 8 and 23, which indicated a soil classification of lean clay (CL) – based on the plasticity testing alone, where no grain size testing was performed. Along with that, soil gradation analysis was performed on a select sample from borings B-1 between approximately 8.5 to 10 feet below existing grade. Table 1 summarizes the Atterberg limits of the selected samples, while Table 2 tabulates the grain size data.

Table 1 – Atterberg Limits Summary

Test Location	Depth (ft)	Atterberg Limits			Soil Classification
		Liquid Limit (LL)	Plastic Limited (PL)	Plasticity Index (PI)	
B-1	3.5-5	37	14	23	Lean CLAY (CL)
B-2	8.5-10	26	18	8	Lean CLAY (CL)

Table 2 – Grain Size Summary

Test Location	Depth (ft)	Grain Size Data				
		Gravel (%)	Sand			Passing #200 (%)
			Coarse Sand (%)	Medium Sand (%)	Fine Sand (%)	
B-1	8.5-10	59.3	9.1	10.3	13.3	8.0

Unconfined compressive strength testing of select rock cores from both borings was performed at depths between approximately 44.2 to 50.42 feet below existing. The cores from borings B-1 and B-2 indicated unconfined compressive strengths of 24,476 and 19,054 psi, respectively. We note that we were unable to perform any tests on the deeper rock cores from boring B-2 due to their shaly structure.

5.0 ENGINEERING AND DESIGN RECOMMENDATIONS

5.1 H-PILES

We understand TDOT typically uses point bearing HP10x42 or HP12x53 sections for driven pile support of bridge abutments. It is anticipated that the H-piles driven to practical refusal on bedrock will be used for this project. The H-pile yield stress has been taken as 60 kips per square inch (ksi). The lateral soil resistance for the H-pile was calculated based on the alpha method, while the tip resistance was based on the rock resistance. Both methods were obtained from the FHW-NHI-16-009 (FHWA GEC 012). The total lateral soil resistance and tip resistance were summed to obtained total resistance. The total resistance was compared to the nominal capacity of the H-pile. The capacity of the H-pile was the controlling resistance for both H-piles. Table 3 provides the anticipated pile tip elevations and capacity.

Table 3 – Minimum Tip Elevations For H-Piles

Boring	Pile Type	Refusal Elevation (ft MSL)	Estimated Pile Tip Elevation (ft MSL)	Factored Resistance (Kips)
B-1	HP10x42	1392.54	1392.54	538.7
B-2	HP10x42	1392.77	1392.77	538.7
B-1	HP12x53	1392.54	1392.54	717.2
B-2	HP12x53	1392.77	1392.77	717.2

Notes: Elevations were provided by Gresham Smith and should be considered approximate.

The fills at the ends of the bridge shall be in place and thoroughly compacted before any abutment piles are driven. The minimum pile length (measured from the bottom of the footing, abutment beam, or wing beam) to the pile tip shall be 7 feet for driven point bearing piles. Where the recommended pile penetration cannot be achieved without exceeding the refusal criteria, other penetration aids such as predrilling will be required to reach the required depth. Where oversized predrilled pile holes are required such that all four corners of the H-pile are not in immediate contact with the surrounding soil, then lateral pile stability should be restored by filling the spaces between the pile and the sides of the hole using approved clean sand. Piles installed in preformed holes should be driven to the recommended refusal criteria at the recommended minimum tip elevation (and after the clean sand has been placed, if required) once predrilling has been completed. Where predrilling is required, the H-piles shall have a minimum embedment of at least 10 feet.

We recommend an uplift capacity of the pile weight plus the weight of the pile cap be utilized for design. We expect that any settlement associated with the piles will be elastic settlement within the pile itself. Maximum total and differential settlements area expected to be less than ½ inch.

No reduction in pile load capacity is necessary for end bearing piles installed in groups. A minimum spacing of four times the pile diameter is recommended between adjacent piles. All foundation piles should be installed by specialty contractors who have experience in the installation of piles in conditions such as those at this site. Installation shall be by the continuous driving of the pile section to virtual refusal. Virtual refusal, for the purposes of this project, shall be defined as a driving resistance of 20 blows per inch, or equivalent resistances for increments less than an inch (e.g. 5 blows per quarter inch). The number of blows required for refusal may be adjusted based on the evaluation of the driving conditions. Pile driving should be performed in general accordance with Section 606 of the TDOT Standard Specifications for Road and Bridge Construction.

The installation of each pile should be observed by the geotechnical engineer, or a staff professional working under the direction of the geotechnical engineer. The installation observations should include the following:

- Keeping a record of pile installation and driving procedure.
- Verify that the piles are installed to the proper driving resistance and to a depth indicative of the intended bearing stratum.
- Generally confirm that the pile driving equipment is operating as anticipated.
- Confirm from visual appearance that the piles are not damaged during installation and inspecting the piles prior to installation for detective workmanship. The geotechnical engineer should review all driving records prior to pile cap construction.

5.2 DRILLED SHAFTS

5.2.1 Drilled Shaft Design

Rock-bearing drilled shafts are recommended for support of the proposed bridge column interior bents. The drilled shafts should extend through the overburden and zones of weathered rock to bear on continuous dolomite bedrock. Based on the preliminary borings of this exploration, we recommend the shaft tip elevations presented in Table 4 below.

Table 4 – Drilled Shaft Tip Elevations

Boring	Station	Ground Surface Elevation	Refusal Elevation	Competent Rock Depth	Competent Rock Elevation	Estimated Shaft Tip Elevation
B-1	17+70.00	1426.74	1392.5	68.5	1358.2	1354.2
B-2	22+35.00	1425.07	1392.8	45	1380.1	1376.1

Notes: Elevations were provided by Gresham Smith in feet MSL. Depths in feet.

The estimated tip elevations are based on an assumed minimum 4-foot rock socket below competent rock. The required socket depth may change based on the design of the shafts and should be confirmed prior to plan development.

The TDOT Geotechnical Guidelines defines competent rock as having no more than three instances of rock discontinuities, voids, or very weathered seams greater than 2-inches or a single discontinuity of greater than 6 inches in a 10-foot core run. We note that incomplete core recovery was present in boring B-1; however, this is anticipated to be the result of shaley materials washing away during the coring process and indications of voids or discontinuities were not noted during drilling. Pre-coring prior to construction should be performed at each shaft location to verify and establish the shaft tip elevations.

Side and tip resistance values and resistance factors are presented in Table 5. Both side and tip resistance values were calculated as outlined in FHWA-NHI 18-024 (FHWA GEC 10) and conservative assumptions were made due to the variability of the rock competency. In addition, the frication angle of the rock was assumed to be 30 degrees. A side resistance value for the socket (for uplift resistance) was calculated to be at 15ksf ultimate, while the tip resistance of 250 ksf ultimate was calculated. These conditions can be achieved if a clean shaft is constructed and founded on competent rock. The side and tip resistance factors were obtained from AASHTO LRFD 2020 Table 10.5.5.2.4-1 and are based on a side resistance in clay and a tip resistance in rock.

Table 5 – Recommended Drilled Shaft Axial Design Parameters

Property	Ultimate	Resistance Factor (AASHTO Table 10.5.5.2.4-1)	Allowable
Side Resistance Rock	15ksf	0.45	7.5ksf
Tip Resistance Rock	250ksf	0.5	125ksf

From the TDOT Structural Design Guidelines (SDG 10), the shaft capacity for axial load shall be based on either end bearing capacity or side fiction capacity, but not a combination of both. The minimum rock socket length shall be as directed by the geotechnical engineer, but in no case shall be less than 1.5 times the rock socket diameter.

Lateral analysis shall be performed once loading demands are determined for the project. The standard of practice for drilled shafts is to use the Beam on Winkler Springs method included in the software LPILE and/or Group by Ensoft, Inc. Group effects shall be considered for shaft groups based on guidance from FHWA GEC 10. Ultimate soil p-y parameters for the site are summarized in Table 4. Resistance factors shall be applied as necessary.

Table 6 – Recommended Drilled Shaft Lateral Design Parameters

Layer Depth	Description	p-y model	Unconfined Compressive Strength for Rock (psi)	Undrained Shear Strength for Clays (psf)
0-17ft	Fat Clay/Fill	Stiff Clay Without Free Water	-	800
17ft-35ft	Fat Clay/Residuum & Alluvium	Stiff Clay Without Free Water	-	1,000
35ft-100ft	Rock	Strong Rock/Vuggy Dolomite	10,000	-

At this time, we have not been provided with foundation bearing elevations. However, based on the depths to competent rock, the shaft lengths will be on the order of 49 to 73 feet based only on depth to competent rock. Top of competent rock elevations ranged from about 1358.2to 1380.1 feet MSL. Shaft lengths will be determined once axial and lateral loading conditions are analyzed.

The drilled shafts should be installed only by a specialty contractor with proven experience in the installation of drilled shafts in similar geologic conditions. The shafts should be cased at all times, as required by the Occupational Safety and Health Administration (OSHA), for the protection of workers entering the excavation.

5.2.2 Drilled Shaft Installation

Typical drilled shaft construction in this area utilizes a temporary steel casing and the dry hole method. The steel casing is installed deep enough to seal the excavation to allow workers to safely excavate, clean, observe and test the drilled shaft, but leaving the sides of any rock socket exposed for inspection. The protective steel casing may be extracted as the concrete is placed should conditions allow. A sufficient head of at least 5 feet of concrete should be maintained above the bottom of the casing during withdrawal and the contractor should prevent concrete from "hanging-up" inside the shell which can cause soil and water intrusion below the casing.

If the dry hole method is utilized, water encountered during shaft construction should be removed from the excavation to a depth of no greater than 6 inches prior to the placement of concrete. Water removed from the shafts should be directed to an off-site outfall, away from the construction area, and in accordance with any environmental engineering recommendations (outside the scope of this document). If it is not possible to adequately remove this water, concrete should be placed using the tremie method. The concrete should be placed in a manner to prevent segregation of the aggregate or the creation of honeycomb structures or other voids in the completed shaft. The geotechnical engineer should observe each portion of the drilled shaft construction.

Prior to entry of the drilled shafts, testing should be performed of the atmosphere within the confined space to ensure adequate oxygen levels and to monitor for the presence of flammable, explosive, or toxic vapors or substances, in accordance with OSHA standards. Air monitoring should be performed at representative intervals through the full depth of the drilled shaft to confirm the safety of personnel. Other OSHA requirements will also apply such as safety harnesses, lifelines, continuous monitoring of down-hole personnel by an attendant at the surface, ventilation and other requirements, as applicable. Please refer to the most current OSHA guidelines regarding drilled shaft construction.

In this geologic setting, discontinuous rock with mud seams tend to overly competent bedrock. It is likely that 10 to 25 feet (in some cases more) of weathered rock/soil seam penetration will be required to reach moderately hard, continuous bedrock upon which the drilled shafts will bear. The socket depth could increase, based upon the slope, orientation, and surface of the bedrock.

As discussed above, it is likely that drilled shafts would need to be extended several feet beyond the refusal depths of the soil borings using rock auger or coring procedures to penetrate the upper zone of discontinuous/weathered limestone bedrock. While common practice for drilled shaft construction, this rock removal (coring or drilling) could result in project delays and greater than anticipated construction costs given the high degree of bedrock variability in this geologic setting. It is our experience that rock having compressive strengths similar to those of this exploration require contractor consideration for means and methods of removal. Competent rock, and zones of partially weathered rock containing uneven zones of weathered competent rock, may present challenges to excavation that need to be considered by the specialty contractor and the general contractor with regard to pricing and schedule.

Pre-coring of the drilled shafts is recommended to verify and confirm competent rock extends to the depths specified by TDOT below the shaft tip elevation. The TDOT Geotechnical Guidelines defines competent rock as having no more than three instances of rock discontinuities, voids, or very weathered seams greater than 2-inches or a single discontinuity of greater than 6 inches in a 10-foot core run. The pre-coring and installation of each shaft should be observed by the geotechnical engineer.

5.3 SEISMIC DESIGN CRITERIA

In accordance with the AASHTO LRFD Bridge Design Specifications 2020, we are providing the following seismic design information. After evaluating the SPT N-value data from previous soil test borings performed at the site, it was determined that the site subsurface conditions most closely matched the description for “Seismic Site Class D” or “Stiff Soil”. Table 7 provides the spectral response accelerations for both short and 1-second periods, which may be used for design.

Table 7 – Seismic Design Parameters

Structure	PGA (AASHTO Figure 3.10.2-.1-1)	Ss (AASHTO Figure 3.10.2.1-2)	S1 (AASHTO Figure 3.10.2.1-3)	Site Class
SR 353 Emergency Bridge Repair – Washington County, Tennessee	0.21	0.384	0.151	D

5.4 SPECIFIC SEISMIC HAZARDS – GROUND FAULT RUPTURE AND LIQUEFACTION

No active faults at or near the site are currently known but minor ground shaking is possible at the site. At all sites subject to ground shaking, ground fault rupture is possible but the risks at this site are deemed negligible.

Liquefaction occurs when a saturated soil (below the water table) with little to no cohesion experiences a temporary reduction or loss of strength as a result of transient pore pressure increases generated by strong ground motion. The design peak ground acceleration (PGA) at this site is relatively low and the residual soils are generally cohesive in nature so the potential of liquefaction triggering for the subject site is deemed negligible. Lateral spread requires liquefaction on a shear zone with a clear path to exit the surface and allow lateral movement of the liquefied soils and non-liquefied crust above the liquefied zone. As the liquefaction potential is negligible, potential for lateral spread at this site is also deemed negligible.

6.0 LIMITATIONS

This report has been prepared in accordance with generally accepted geotechnical engineering practice for specific application to this project. This report is for our geotechnical work only, and no environmental assessment efforts have been performed. The conclusions and recommendations contained in this report are based upon applicable standards of our practice in this geographic area at the time this report was prepared. No other warranty, express or implied, is made.

The analyses and recommendations submitted herein are based, in part, upon the data obtained from the exploration. The nature and extent of variations between the borings will not become evident until construction. If variations appear evident, then we will re-evaluate the recommendations of this report. In the event that any changes in the nature, design, or location of the structure is planned, the conclusions and recommendations contained in this report will not be considered valid unless the changes are reviewed, and conclusions modified or verified in writing. Also, if the scope of the project should change significantly from that described herein, these recommendations may need to be re-evaluated.

7.0 REFERENCES

AASHTO LRFD bridge design specifications, 2020. American Association of State Highway and Transportation Officials, Washington, D.C.

Federal Highway Administration (FHWA), 2016. Design and Construction of Driven Pile Foundations – Volumes I. Report No. FHWA-NHI-16-009, U.S. Department of Transportation, Washington, D.C.

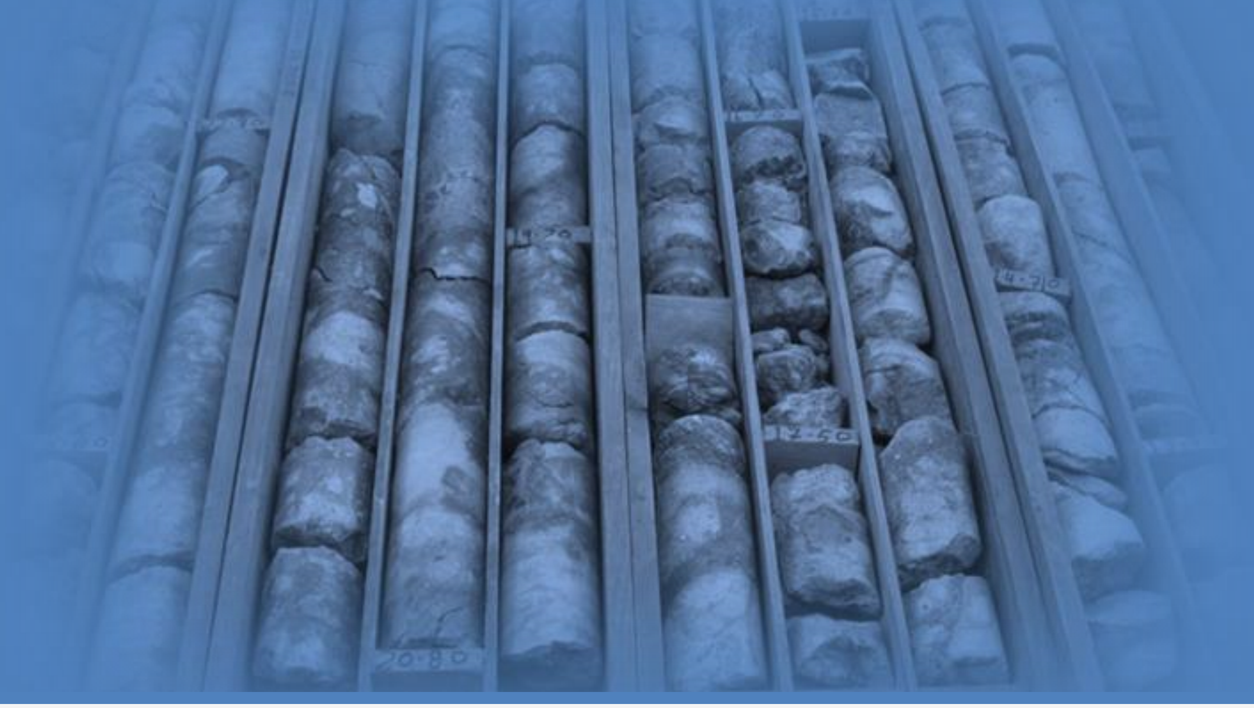
Federal Highway Administration (FHWA), 2018. Drilled Shafts: Construction Procedures and Design Methods. Report No. FHWA-NHI-18-024, U.S. Department of Transportation, Washington, D.C.

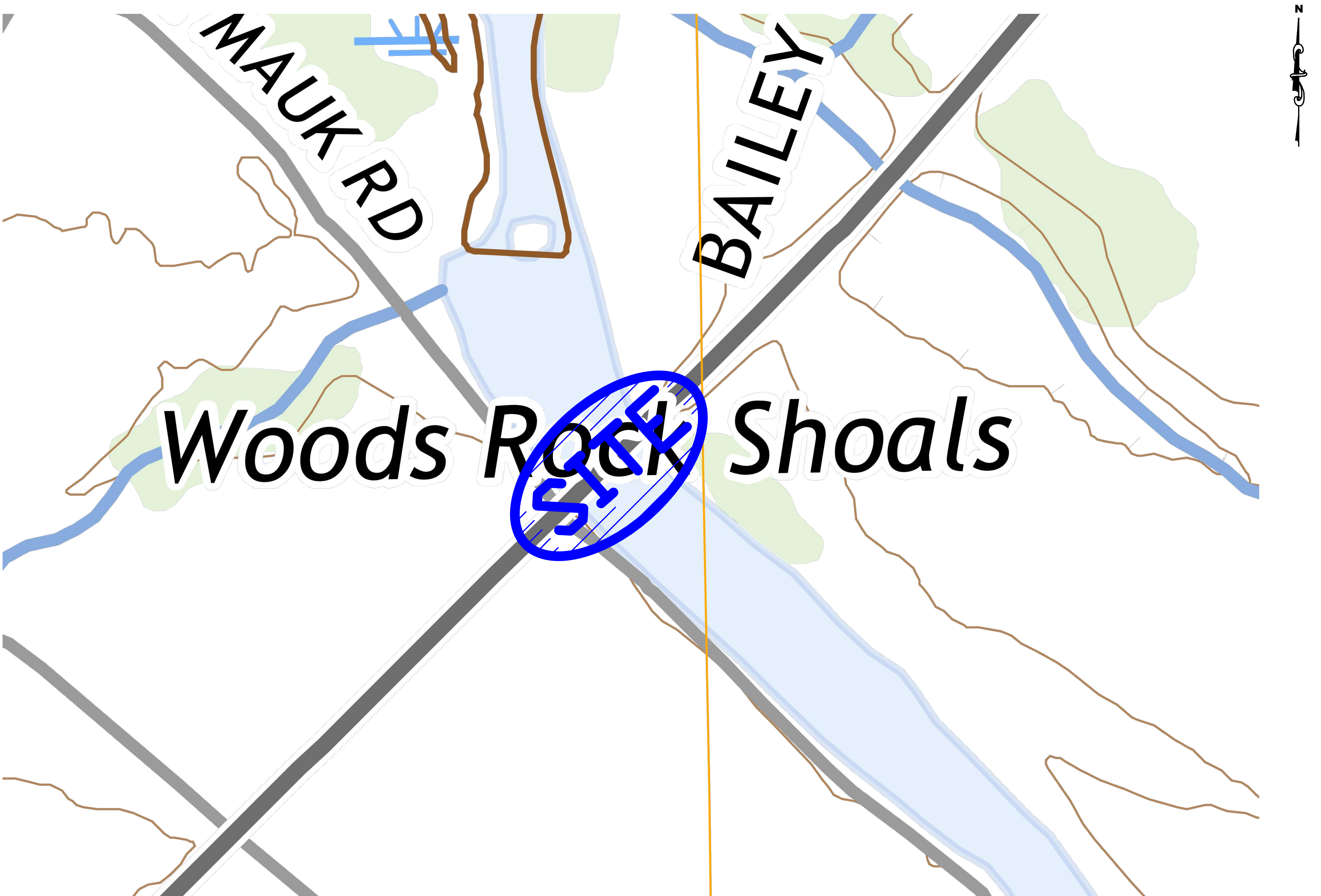
Hoek, E., Carter, T.G., Diederichs, M.S., 2013. Quantification of the Geological Strength Index Chart. *American Rock Mechanic Association. 13-672.*

APPENDICES

APPENDIX A

Figures, General Notes, Boring Logs, & Boring Profiles





NOTES:

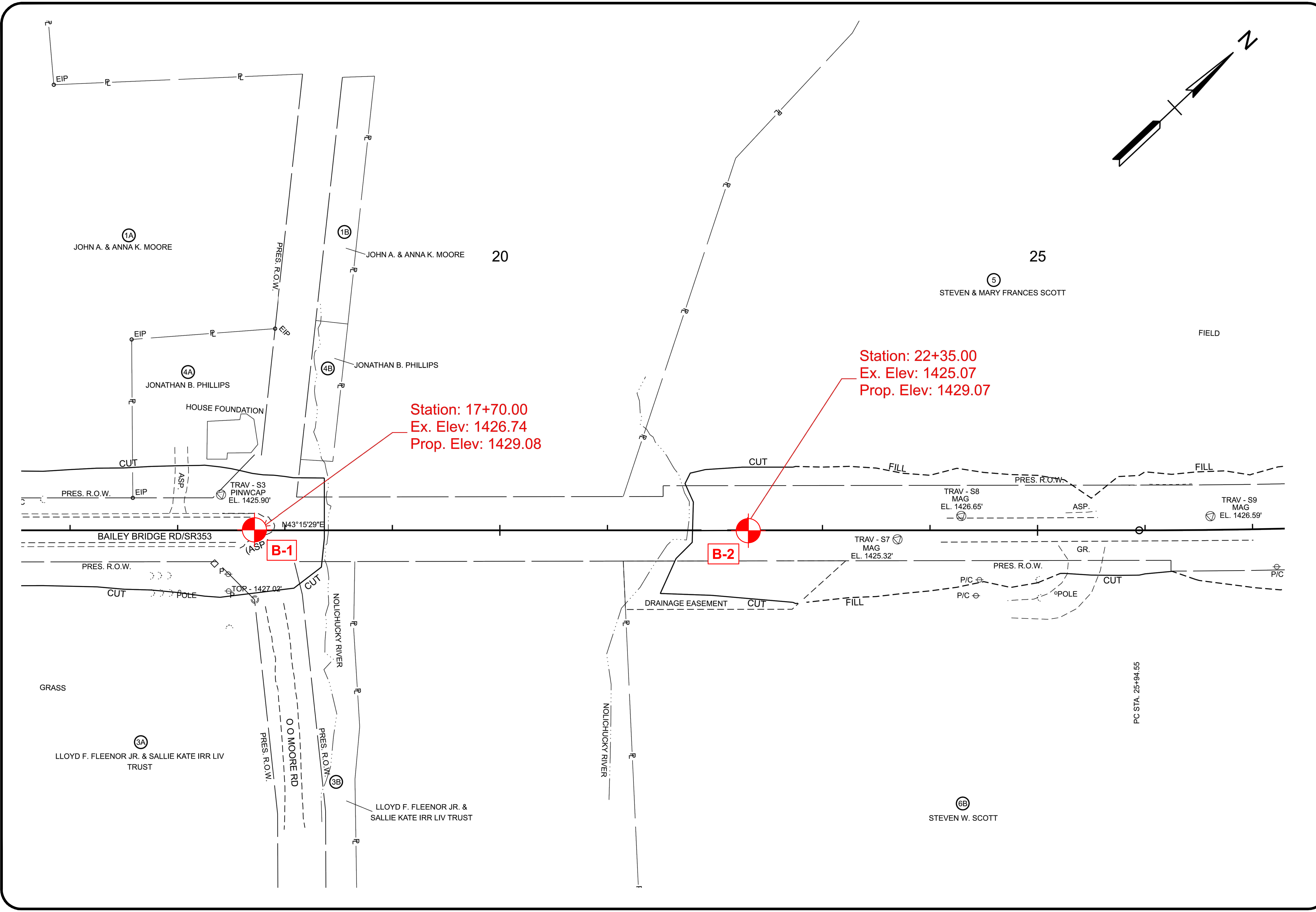
1.) BASE MAP: USGS QUADRANGLE (TELFORD, TENNESSEE)

UES™

2561 Willow Point Way, Knoxville, TN 37931

Office: 865-539-8242 Fax: 865-539-8252

SITE VICINITY MAP	DRAWN BY: KSR	FIGURE
STATE ROUTE 353	APPROVED BY: SRM	1
EMERGENCY BRIDGE REPLACEMENT	SCALE: N.T.S.	
WASHINGTON COUNTY, TENNESSEE	JOB NO.: A24109.02271.000	
	DATE: 10/13/24	



NOTES:
1.) BORING LOCATIONS ARE SHOWN IN GENERAL ARRANGEMENT ONLY.
2.) DO NOT USE BORING LOCATIONS FOR DETERMINATIONS OF DISTANCES OR QUANTITIES.
3.) BASE MAP PROVIDED BY: State of Tennessee Department of Transportation

LOCATION OF SOIL TEST BORINGS

SOIL TEST BORING LOCATION PLAN WITH LAYOUT

STATE ROUTE 353
EMERGENCY BRIDGE REPLACEMENT
WASHINGTON COUNTY, TENNESSEE

DRAWN BY:	KSR
APPROVED BY:	SRM
SCALE:	N.T.S.
JOB NO:	A24109.02271.000
DATE:	10/13/24

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Figure 2

SOIL CLASSIFICATION CHART PER USCS (ASTM D 2488)

PRIMARY DIVISIONS			SECONDARY DIVISIONS: GROUP SYMBOL	SECONDARY DIVISIONS: GROUP NAME
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	GRAVEL more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines	GW	well-graded GRAVEL
			GP	poorly-graded GRAVEL
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines	GW-GM	well-graded GRAVEL with silt
			GP-GM	poorly-graded GRAVEL with silt
			GW-GC	well-graded GRAVEL with clay
			GP-GC	poorly-graded GRAVEL with clay
		GRAVEL with FINES more than 12% fines	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND 50% or more of coarse fraction retained on No. 4 sieve	CLEAN SAND less than 5% fines	SW	well-graded SAND
			SP	poorly-graded SAND
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines	SW-SM	well-graded SAND with silt
			SP-SM	poorly-graded SAND with silt
			SW-SC	well-graded SAND with clay
			SP-SC	poorly-graded SAND with clay
		SAND with FINES more than 12% fines	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	SILT and CLAY liquid limit less than 50%	INORGANIC	CL	lean CLAY
			ML	SILT
			CL-ML	silty CLAY
		ORGANIC	OL (PI > 4)	organic CLAY
			OL (PI < 4)	organic CLAY
	SILT and CLAY liquid limit 50% or more	INORGANIC	CH	fat CLAY
			MH	elastic SILT
		ORGANIC	OH (plots on or above 'A'-line)	organic CLAY
			OH (plots below 'A'-line)	organic SILT
	Highly Organic Soils		PT	Peat

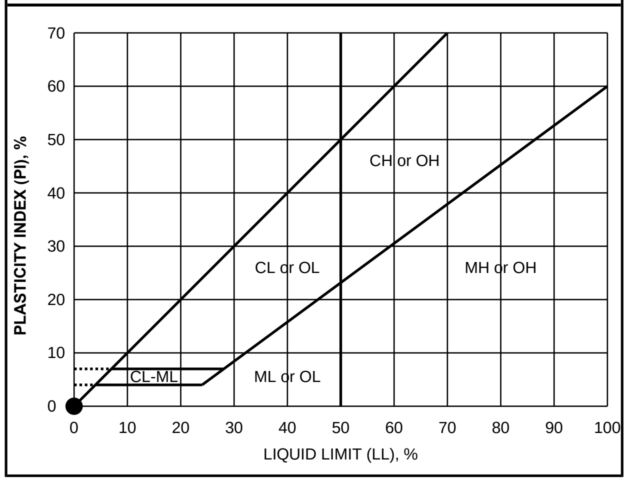
GRAPHIC COLORS

COLOR	MODE OF DEPOSITION
	FILL
	ALLUVIUM
	COLLUVIUM
	MARINE
	RESIDUUM
	GLACIAL (Till / Outwash)
	CULTIVATED
	LOESS
	VOID
	WEATHERED ROCK

SAMPLE GRAPHICS

GRAPHIC	SAMPLE TYPE & ABBREVIATION
	Large Split Spoon - LSS
	Standard Penetration Test - SPT
	Small Split Spoon - SS
	Shelby Tube - ST
	Liner Sample - Liner
	Grab Sample - Grab
	Bulk Sample - Bulk
	Penetrometer Sample - Pen
	Coring (Bedrock / Asphalt / Concrete) - Core
	Hand Auger Sample - Hand

PLASTICITY CHART



COARSE-GRAINED SOIL

APPARENT DENSITY	N-Value (blows/foot)
Very Loose	≤ 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

FINE-GRAINED SOIL

CONSISTENCY	N-Value (blows/foot)
Very Soft	≤ 2
Soft	3 - 4
Firm	5 - 8
Stiff	9 - 15
Very Stiff	16 - 30
Hard	> 30

ROCK CORING PROPERTIES

ROCK CORING QUALITY DESIGNATION (RQD): Percent RQD	Quality of Rock
90 TO 100	EXCELLENT
75 TO 90	GOOD
50 TO 75	FAIR
25 TO 50	POOR
O TO 25	VERY POOR

ROCK HARDNESS	
VERY SOFT:	ROCK DISINTEGRATES OR EASILY COMPRESSES TO TOUCH: CAN BE HARD TO VERY HARD SOIL.
SOFT:	ROCK IS COHERANT BUT BREAKS EASILY TO THUMB PRESSURE AT SHARP EDGES AND IT CRUMBLES WITH FIRM HAND PRESSURE.
MODERATELY HARD:	SMALL PIECES CAN BE BROKEN OFF ALONG SHARP EDGES BY CONSIDERABLE HARD THUMB PRESSURE: CAN BE BROKEN BY LIGHT HAMMER BLOWS.
HARD:	ROCK CAN NOT BE BROKEN BY THUMB PRESSURE, BUT CAN BE BROKEN BY MODERATE HAMMER BLOWS.
VERY HARD:	ROCK CAN BE BROKEN BY HEAVY HAMMER BLOWS.

PARTICLE SIZE

BOULDERS:	Greater Than 300 mm	MEDIUM SAND:	0.425 mm to 2 mm
COBBLES:	75 mm to 300 mm	FINE SAND:	0.075 mm to 0.425 mm
GRAVEL:	4.74 mm to 75 mm	SILTS & CLAYS:	Less Than 0.075 mm
COARSE SAND:	2 mm to 4.74 mm		

UES™

SOIL BORING NUMBER: B-1 (17+70.00)

PROJECT NAME State Route 353 Emergency Bridge
PROJECT NUMBER A24109.02271.000
ELEVATION 1426.74 feet **PROPOSED ELEVATION** 1429.08
PROJECT LOCATION Bridge No. 90S238600011
DATE DRILLED 10/02/2024
LATITUDE / LONGITUDE 36.155761, -82.590824
DRILLING CONTRACTOR Tri-State Drilling, LLC
DRILL RIG CME-55 & NQ Rock Coring
GROUNDWATER: AT TIME OF DRILLING 22.0 feet
24 HOURS TNP **TOTAL DEPTH** 69.2 feet

Elevation (ft)	Depth (ft)	Sample Type	Sample Number	MATERIAL DESCRIPTION	Blow Counts (N/Refusal)	Recovery (%)	RQD (%)	Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
1426.7				**ASPHALT** - 10 Inches							
		SS	NR	**BASESTONE** - 10 Inches	50/2" (50/2")						
1425				**LEAN CLAY** (CL) - with gravel, trace asphalt fragments, and strong organic odor - black, dark brown, and reddish brown - moist (**FILL**)							
		SS	2		3-3-4 (7)			17.80	37	14	23
	5			**GRAVEL** (GW-GC) - with sand and clay - tan, brown, and dark gray - dry (**FILL**)							
1420		SS	3		5-8-9 (17)			5.80			
		SS	4		5-8-6 (14)			5.40			
1415	10	ST	1			0					
		SS	5	**FAT CLAY** (CH) - with rock fragments, sand, silt, trace roots, and organic odor - tan, black, brown, and orangish brown - moist (**FILL**)	4-9-50/1" (50/1")			17.20			
1410	15			**LEAN CLAY** (CL) - micaceous with silt, black manganese nodules, and rock fragments - tan, orangish brown, and brown - stiff - moist to very moist (**RESIDUUM**)							
		SS	6		10-9-5 (14)			24.40			

REMARKS
WOH = Weight of Hammer
No water return was encountered while rock coring beginning at 47.0 feet and continuing to coring termination at 69.2 feet.

PROJECT NAME State Route 353 Emergency Bridge
PROJECT NUMBER A24109.02271.000
ELEVATION 1426.74 feet **PROPOSED ELEVATION** 1429.08
PROJECT LOCATION Bridge No. 90S238600011
DATE DRILLED 10/02/2024
LATITUDE / LONGITUDE 36.155761, -82.590824
DRILLING CONTRACTOR Tri-State Drilling, LLC
DRILL RIG CME-55 & NQ Rock Coring
GROUNDWATER: AT TIME OF DRILLING ▽ 22.0 feet
24 HOURS ▼ TNP
TOTAL DEPTH 69.2 feet

Elevation (ft)	Depth (ft)	Sample Type	Sample Number	MATERIAL DESCRIPTION	Blow Counts (N/Refusal)	Recovery (%)	RQD (%)	Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
1405				**LEAN CLAY** (CL) - micaceous with silt, black manganese nodules, and rock fragments - tan, orangish brown, and brown - stiff - moist to very moist (**RESIDUUM**)							
				FAT CLAY (CH) - with silt and rock fragments - tan, orangish brown, reddish brown, and brown - very soft - moist (**RESIDUUM**)							
		SS	7		2-1-1 (2)			26.90			
1400	25										
		SS	8		WOH-WOH-WOH			44.40			
1395	30										
				LEAN CLAY (CL) - with rock fragments - light gray, tan, reddish brown, orangish brown, and brown - hard - wet (**RESIDUUM**)							
		SS	9		15-15-50/1" (50/1")			39.50			
				Auger Refusal at 34.2 Feet (Began Coring)							
	35	Core	1	**DOLOMITE** - with partially healed calcium seams - 0.0 to 45.0 degree dip angle - slight HCl reaction - light gray, dark gray, black, and white - slightly to moderately weathered - moderately to highly fractured		66	42				
1390											
		Core	2			8	0				

REMARKS
WOH = Weight of Hammer
No water return was encountered while rock coring beginning at 47.0 feet and continuing to coring termination at 69.2 feet.

SOIL BORING NUMBER: B-1 (17+70.00)

PROJECT NAME State Route 353 Emergency Bridge
PROJECT NUMBER A24109.02271.000
ELEVATION 1426.74 feet **PROPOSED ELEVATION** 1429.08
PROJECT LOCATION Bridge No. 90S238600011
DATE DRILLED 10/02/2024
LATITUDE / LONGITUDE 36.155761, -82.590824
DRILLING CONTRACTOR Tri-State Drilling, LLC
DRILL RIG CME-55 & NQ Rock Coring
GROUNDWATER: AT TIME OF DRILLING 22.0 feet **24 HOURS** TNP **TOTAL DEPTH** 69.2 feet

Elevation (ft)	Depth (ft)	Sample Type	Sample Number	MATERIAL DESCRIPTION	Blow Counts (N/Refusal)	Recovery (%)	RQD (%)	Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
		Core	2	**DOLOMITE** - with partially healed calcium seams - 0.0 to 45.0 degree dip angle - slight HCl reaction - light gray, dark gray, black, and white - slightly to moderately weathered - moderately to highly fractured		8	0				
1385											
	45	Core	3			66	58				
1380											
	50	Core	4			42	28				
1375											
	55	Core	5			28	8				
1370											
		Core	6			68	50				

REMARKS
WOH = Weight of Hammer
No water return was encountered while rock coring beginning at 47.0 feet and continuing to coring termination at 69.2 feet.

SOIL BORING NUMBER: B-1 (17+70.00)

PROJECT NAME State Route 353 Emergency Bridge **PROJECT NUMBER** A24109.02271.000
ELEVATION 1426.74 feet **PROPOSED ELEVATION** 1429.08 **PROJECT LOCATION** Bridge No. 90S238600011
DATE DRILLED 10/02/2024 **LATITUDE / LONGITUDE** 36.155761, -82.590824
DRILLING CONTRACTOR Tri-State Drilling, LLC **DRILL RIG** CME-55 & NQ Rock Coring
GROUNDWATER: AT TIME OF DRILLING 22.0 feet **24 HOURS** TNP **TOTAL DEPTH** 69.2 feet

Elevation (ft)	Depth (ft)	Sample Graphic	Sample Type	Sample Number	Graphic Log	MATERIAL DESCRIPTION	Samples: Blow Counts (N/Refusal)	Samples: Recovery (%)	Samples: RQD (%)	Lab: Moisture Content (%)	Lab: Liquid Limit	Lab: Plastic Limit	Lab: Plasticity Index
1365			Core	6		**DOLOMITE** - with partially healed calcium seams - 0.0 to 45.0 degree dip angle - slight HCl reaction - light gray, dark gray, black, and white - slightly to moderately weathered - moderately to highly fractured		68	50				
1360	65		Core	7				50	8				

Coring terminated at 69.2 feet

REMARKS

WOH = Weight of Hammer
No water return was encountered while rock coring beginning at 47.0 feet and continuing to coring termination at 69.2 feet.

SOIL BORING NUMBER: B-2 (22+35.00)

PROJECT NAME State Route 353 Emergency Bridge
PROJECT NUMBER A24109.02271.000
ELEVATION 1425.07 feet **PROPOSED ELEVATION** 1429.07
PROJECT LOCATION Bridge No. 90S238600011
DATE DRILLED 10/04/2024
LATITUDE / LONGITUDE 36.156390, -82.590052
DRILLING CONTRACTOR Tri-State Drilling, LLC
DRILL RIG CME-55 & NQ Rock Coring
GROUNDWATER: AT TIME OF DRILLING 19.0 feet
24 HOURS TNP
TOTAL DEPTH 57.3 feet

Elevation (ft)	Depth (ft)	Sample Type	Sample Number	MATERIAL DESCRIPTION	Blow Counts (N/Refusal)	Recovery (%)	RQD (%)	Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
1425.1	0			**LEAN CLAY** (CL) - with rock fragments and mica - reddish brown - moist (**FILL**)							
		SS	1		50/1" (50/1")			24.40			
				FAT CLAY (CH) - with mica, silt, rock fragments, and organic odor - reddish brown, tan, and brown - moist (**FILL**)							
		SS	2		3-3-8 (11)			29.40			
1420	5										
		SS	3		4-4-10 (14)			29.40			
				LEAN CLAY (CL) - micaceous with silt - tan, black, dark brown, and brown - very stiff to stiff - slightly moist to moist (**ALLUVIUM**)							
		SS	4		6-10-8 (18)			14.30	26	18	8
1415	10										
		SS	5		4-5-5 (10)			17.40			
1410	15	ST	1			0					
		SS	6		5-4-5 (9)			22.20			

REMARKS
No water return was encountered while rock coring beginning at 52.3 feet and continuing to coring termination at 57.3 feet.

SOIL BORING NUMBER: B-2 (22+35.00)

PROJECT NAME State Route 353 Emergency Bridge
PROJECT NUMBER A24109.02271.000
ELEVATION 1425.07 feet
PROPOSED ELEVATION 1429.07
PROJECT LOCATION Bridge No. 90S238600011
DATE DRILLED 10/04/2024
LATITUDE / LONGITUDE 36.156390, -82.590052
DRILLING CONTRACTOR Tri-State Drilling, LLC
DRILL RIG CME-55 & NQ Rock Coring
GROUNDWATER: AT TIME OF DRILLING 19.0 feet
24 HOURS TNP
TOTAL DEPTH 57.3 feet

Elevation (ft)	Depth (ft)	Sample Type	Sample Number	MATERIAL DESCRIPTION	Blow Counts (N/Refusal)	Recovery (%)	RQD (%)	Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
				LEAN CLAY (CL) - micaceous with silt - tan, black, dark brown, and brown - very stiff to stiff - slightly moist to moist (**ALLUVIUM**)							
				Coarse grained SAND (SP) - with rounded rock fragments - orangish brown, light gray, tan, white, and brown - medium dense to very dense - very moist (**ALLUVIUM**)							
		SS	7		2-8-10 (18)			12.30			
1400	25										
		SS	NR		50/1" (50/1")						
1395	30										
				Auger Refusal at 32.3 Feet (Began Coring)							
		Core	1	**DOLOMITE** - with partially healed to healed calcium seams - 0.0 to 15.0 degree dip angle - slight HCl reaction - light gray, white, black, and tan - moderately to highly weathered - slightly fractured		20	0				
1390	35										
		Core	2			28	8				

REMARKS
No water return was encountered while rock coring beginning at 52.3 feet and continuing to coring termination at 57.3 feet.

UES™

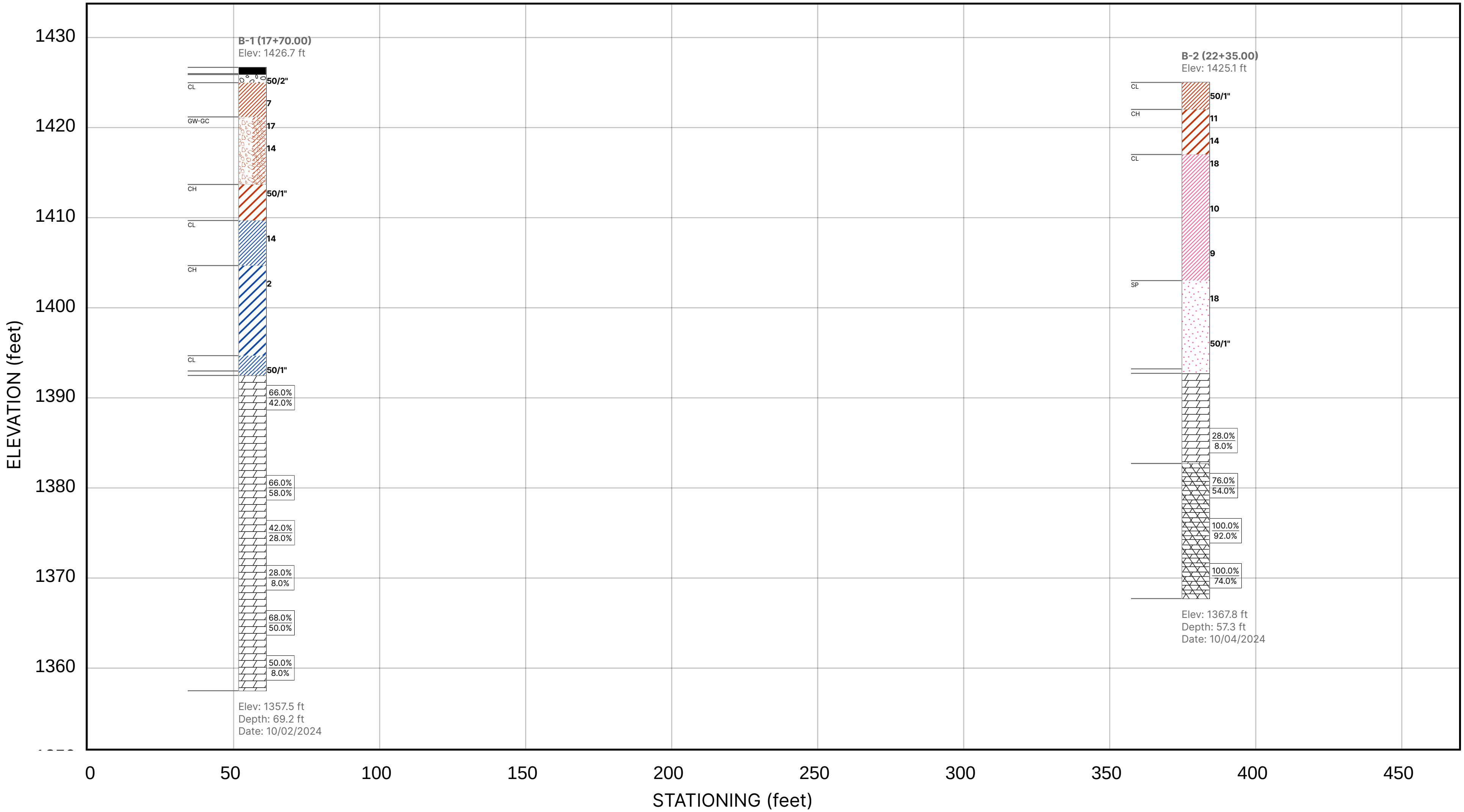
SOIL BORING NUMBER: B-2 (22+35.00)

PROJECT NAME State Route 353 Emergency Bridge
PROJECT NUMBER A24109.02271.000
ELEVATION 1425.07 feet
PROPOSED ELEVATION 1429.07
PROJECT LOCATION Bridge No. 90S238600011
DATE DRILLED 10/04/2024
LATITUDE / LONGITUDE 36.156390, -82.590052
DRILLING CONTRACTOR Tri-State Drilling, LLC
DRILL RIG CME-55 & NQ Rock Coring
GROUNDWATER: AT TIME OF DRILLING ▽ 19.0 feet
24 HOURS ▼ TNP
TOTAL DEPTH 57.3 feet

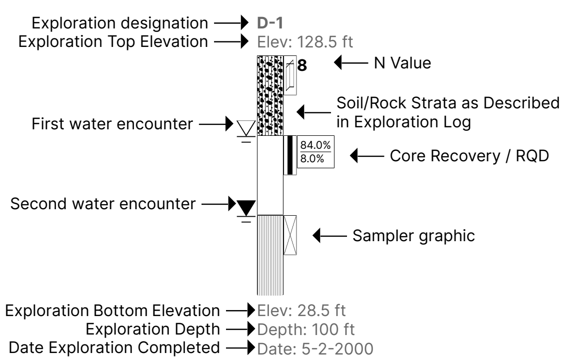
Elevation (ft)	Depth (ft)	Sample Graphic	Sample Type	Sample Number	Graphic Log	MATERIAL DESCRIPTION	Samples: Blow Counts (N/Refusal)	Samples: Recovery (%)	Samples: RQD (%)	Lab: Moisture Content (%)	Lab: Liquid Limit	Lab: Plastic Limit	Lab: Plasticity Index
			Core	2		**DOLOMITE** - with partially healed to healed calcium seams - 0.0 to 15.0 degree dip angle - slight HCl reaction - light gray, white, black, and tan - moderately to highly weathered - slightly fractured		28	8				
			Core	3		**SHALY DOLOMITE** - with partially healed to healed calcium seams and trace vugs - 15.0 to 45.0 degree dip angle - slight HCl reaction - light gray, white, black, and tan - slightly weathered - moderately to highly fractured		76	54				
1380	45												
			Core	4				100	92				
1375	50												
			Core	5				100	74				
1370	55												

Coring terminated at 57.3 feet

REMARKS No water return was encountered while rock coring beginning at 52.3 feet and continuing to coring termination at 57.3 feet.



EXPLORATION LOG LEGEND



LEGEND KEY

- Asphalt
- Basestone
- CL
- GW-GC
- CH
- CL
- CH
- Dolomite
- null
- CL
- SP
- Shaly Dolomite

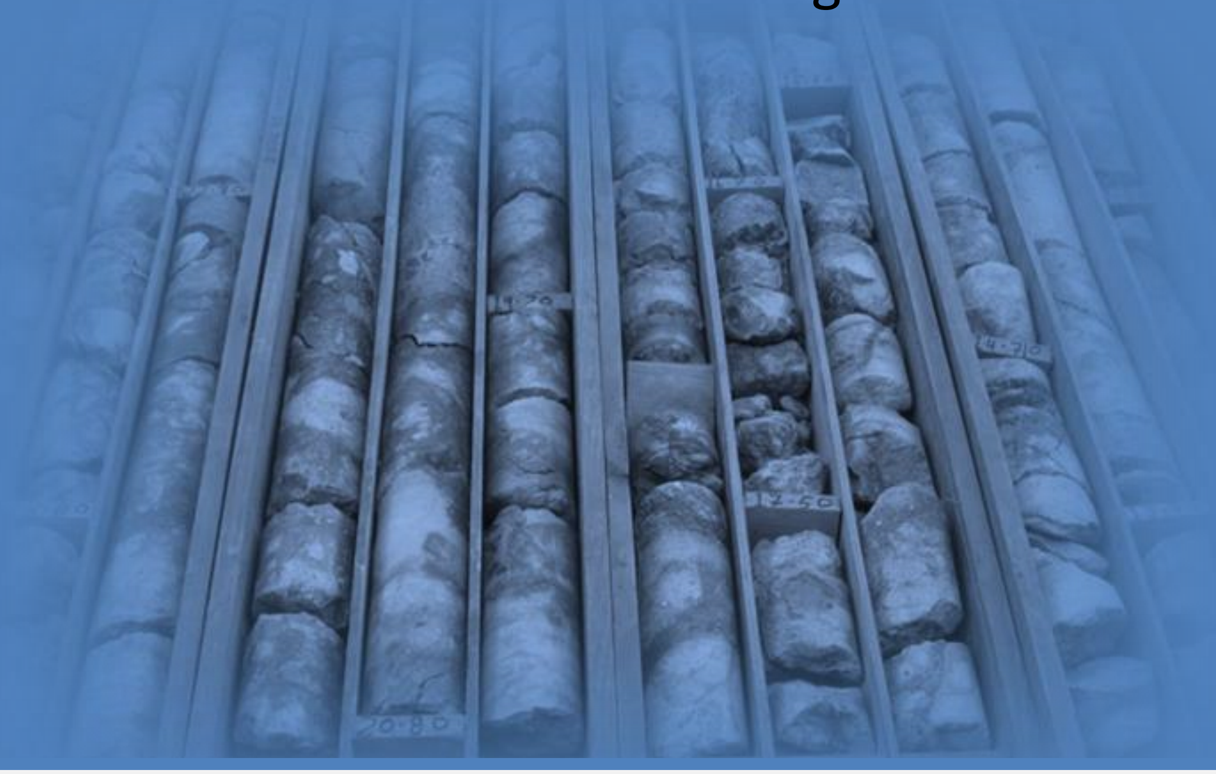
State Route 353 Emergency Bridge Replacement
Bridge No. 90S238600011

CROSS SECTIONS REPORT

UES

APPENDIX B

Rock Core Photo Logs



1	**Remarks**	B-1 – From 34.2 Feet to 54.2 Feet

2	**Remarks**	B-1 – From 54.2 Feet to 69.2 Feet

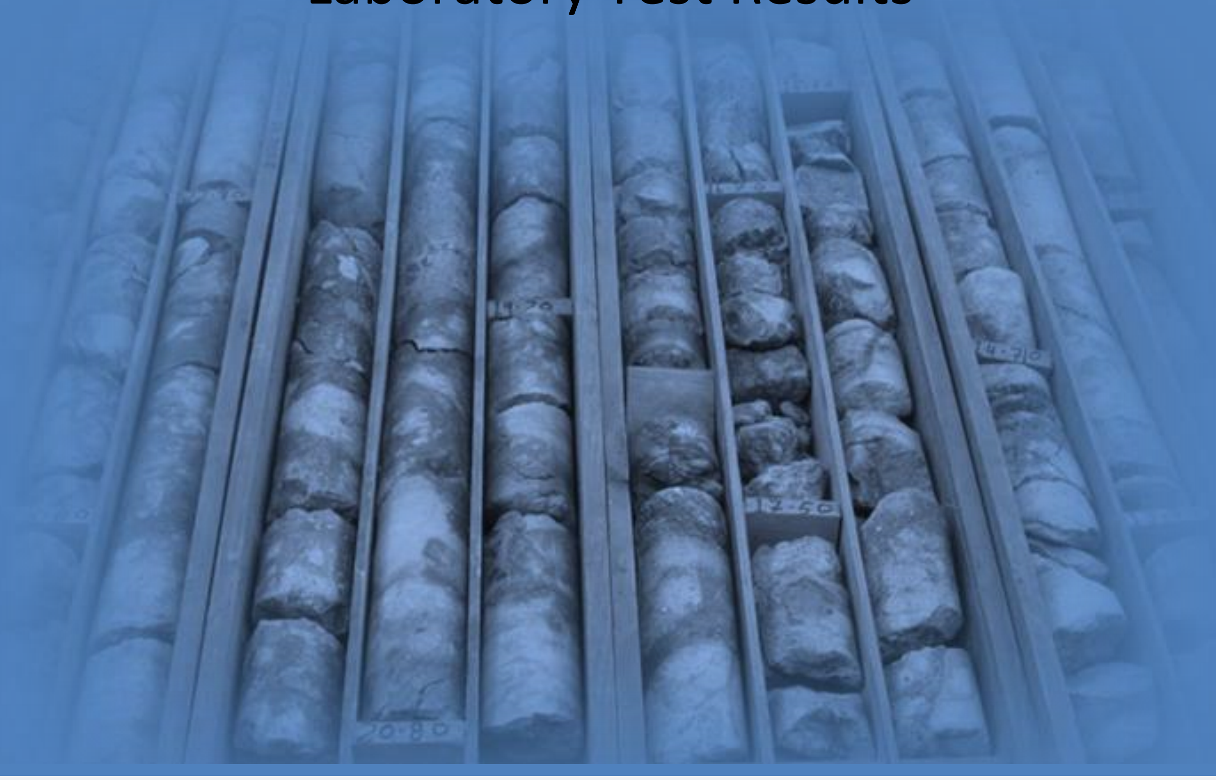
SR-353 Emergency Bridge Repair
Coring Photographs

3	**Remarks**	B-2 – From 32.3 Feet to 52.3 Feet

4	**Remarks**	B-3 – From 52.3 Feet to 57.3 Feet

APPENDIX C

Laboratory Test Results



SR-353 Emergency Bridge Repair

Project No. A24109.02271.000

October 21, 2024

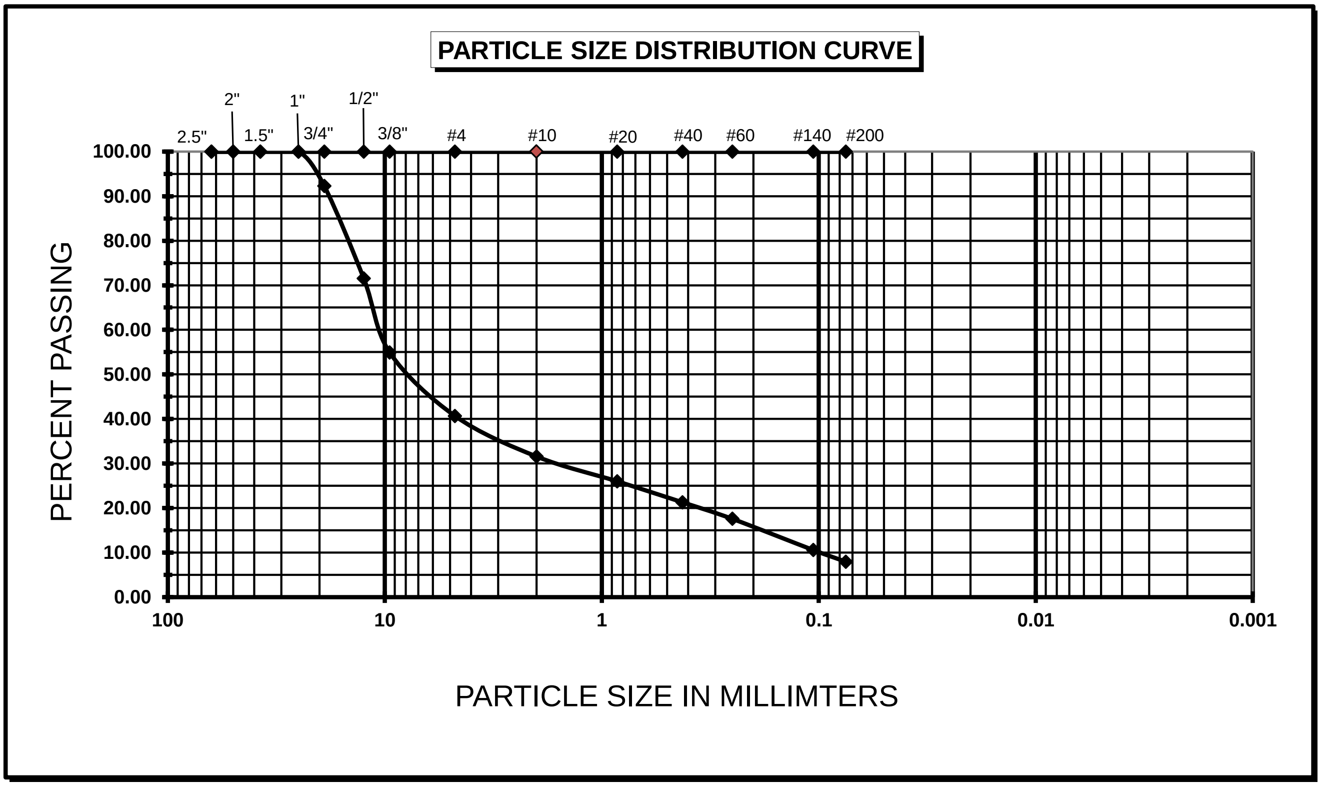
SOIL DATA SUMMARY

Boring Number	Sample Number	Depth (feet)	Natural Moisture Content	Atterberg Limits			Soil Type	Percent Organic Content
				LL	PL	PI		
B-1	2	3.5' - 5'	17.8%	37	14	23	CL	
	3	6' - 7.5'	5.8%					
	4	8.5' - 10'	5.4%					
	5	13.5' - 15'	17.2%					
	6	18.5' - 20'	24.4%					
	7	23.5' - 25'	26.9%					
	8	28.5' - 30'	44.4%					
	9	33.5' - 35'	39.5%					
B-2	1	1' - 2.5'	24.4%					
	2	3.5' - 5'	29.4%					
	3	6' - 7.5'	29.4%					
	4	8.5' - 10'	14.3%	26	18	8	CL	
	5	13.5' - 15'	17.4%					
	6	18.5' - 20'	22.2%					
	7	23.5' - 25'	12.3%					

PARTICLE SIZE ANALYSIS
ASTM D422

GEOS Project Name:	SR-353 Emergency Bridge Repair	Report Date:	October 21, 2024
GEOS Project Number:	A24109.02271.000	Date Received:	October 15, 2024

Sample Location:	B-1, S-3	GEOS Log #:	
Sample Depth:	8.5' - 10'		
Sample Description:	Gravel with Sand and Clay		



GRAIN SIZE DATA	
% GRAVEL	**59.3**
% TOTAL SAND	**32.7**
% Coarse Sand	**9.1**
% Medium Sand	**10.3**
% Fine Sand	**13.3**
% PASSING #200	**8.0**

Particle Size Ranges	
Gravel	< 75 mm and > 4.75 mm
Sands	< 4.75 mm & > 0.075 mm
Coarse Sand	< 4.75 mm and >2.00 mm
Medium Sand	< 2.00 mm and > 0.425 mm
Fine Sand	< 0.425 mm and > 0.075 mm
Silts & Clays	< 0.075 mm

ATTERBERG LIMIT DATA			
Liquid Limit	**N/A**	**Plasticity Index**	**N/A**
Plastic Limit	**N/A**	**Fine Soil Type**	**N/A**

Notes:

PROCEDURE FOR VERIFYING SHAPE CONFORMANCE - INTACT ROCK CORE
ASTM D4543

Project Name:	SR-353 Emergency Bridge Repair	Diameter (in):	1.86	Tested By:	JBB
Project Number:	A24109.02271	Length (in):	4.28	Test Date:	10/16/2024
Client:	Gresham Smith - Knoxville	Length to Diameter:	2.30	Reviewed By:	BKP
Log Number:	B-1, (44.2')	Unit Weight (pcf):	168.2		

Flatness (Procedure FP2)						
Travel	End 1, Ø1	End 1, Ø2	End 1, Ø3	End 2, Ø1	End 2, Ø2	End 2, Ø3
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1/8	0.0001	-0.0010	-0.0002	0.0013	-0.0010	-0.0002
2/8	0.0019	-0.0030	-0.0009	0.0030	-0.0022	-0.0006
3/8	0.0037	-0.0046	-0.0013	0.0048	-0.0040	-0.0010
4/8	0.0048	-0.0064	-0.0016	0.0060	-0.0059	-0.0015
5/8	0.0068	-0.0081	-0.0022	0.0072	-0.0082	-0.0022
6/8	0.0080	-0.0106	-0.0024	0.0086	-0.0102	-0.0027
7/8	0.0090	-0.0123	-0.0031	0.0104	-0.0123	-0.0034
1	0.0106	-0.0141	-0.0033	0.0121	-0.0138	-0.0040
1 1/8	0.0121	-0.0163	-0.0037	0.0133	-0.0158	-0.0044
1 2/8	0.0140	-0.0184	-0.0043	0.0146	-0.0178	-0.0049
1 3/8	0.0153	-0.0200	-0.0048	0.0160	-0.0197	-0.0054
1 4/8	0.0167	-0.0223	-0.0052	0.0179	-0.0214	-0.0058
1 5/8	0.0180	-0.0240	-0.0058	0.0187	-0.0235	-0.0069
1 6/8	0.0185	-0.0253	-0.0060	0.0194	-0.0248	-0.0078
1 7/8						

Length to Diameter Ratio (2.0 to 2.5) ☑ Yes ☐ No

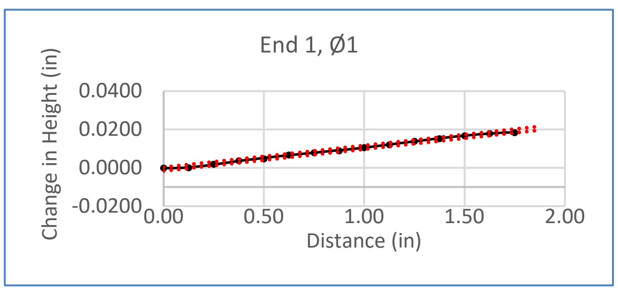
Minimum Diameter (1-7/8 inch) ☐ Yes ☑ No

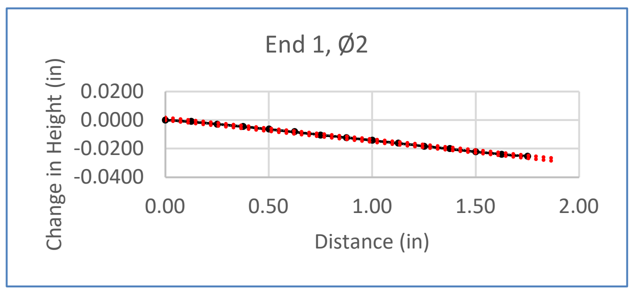
Side Straightness (Procedure S1) - Is the maximum gap between the specimen and the flat surface less than or equal to 0.020-in.?

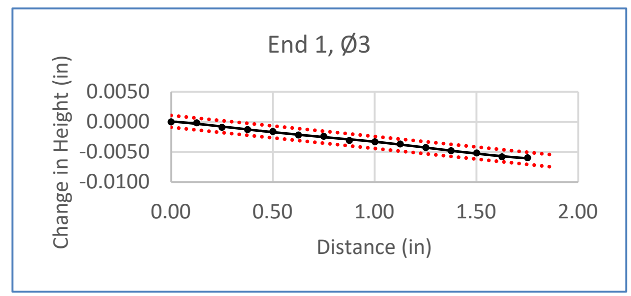
Maximum gap ≤0.020-in.? ☐ Yes ☑ No

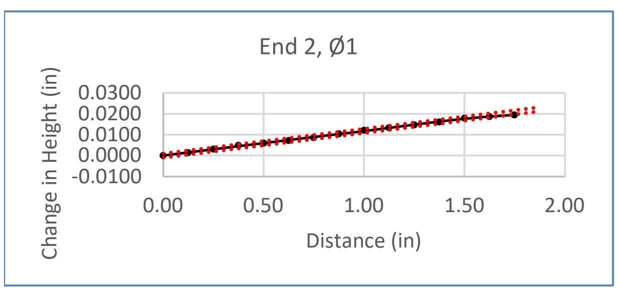
Does specimen meet straightness criteria? ☐ Yes ☑ No

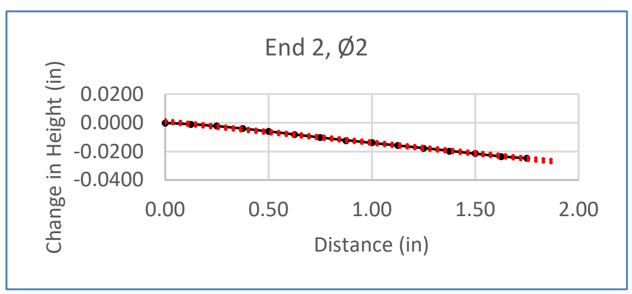
Flatness Requirement - The flatness tolerance is met when the smooth curve so determined does not depart from a visual best-fit straight line by more than 0.001 in.
Does the Specimen meet Flatness Requirements? ☑ Yes ☐ No

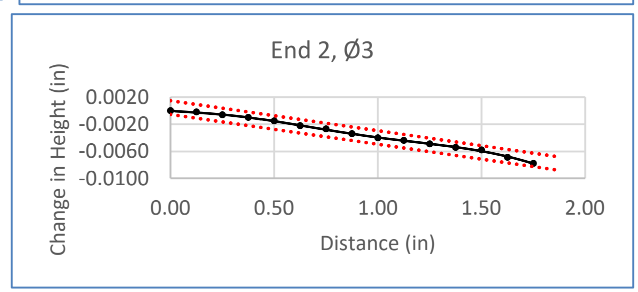










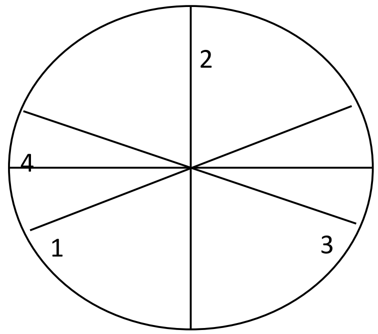


References: ASTM D4543

PROCEDURE FOR VERIFYING SHAPE CONFORMANCE - INTACT ROCK CORE
ASTM D4543

GEOS Project Name:	SR-353 Emergency Bridge Repair	Diameter (in):	1.86	Tested By:	JBB
GEOS Project Number:	A24109.02271	Length (in):	4.28	Test Date:	10/16/2024
GEOS Client:	Gresham Smith - Knoxville	Length to Diameter:	2.30	Reviewed By:	BKP
GEOS Log Number:	B-1, (44.2')	Unit Weight (pcf):	168.2		

Parallelism (Procedure FP2)				
Travel	End 1, Ø2	End 1, Ø4	End 2, Ø2	End 2, Ø4
0	0.0000	0.0000	0.0000	0.0000
1/8	-0.0010	0.0000	-0.0010	0.0004
2/8	-0.0030	0.0006	-0.0022	0.0010
3/8	-0.0046	0.0014	-0.0040	0.0014
4/8	-0.0064	0.0018	-0.0059	0.0023
5/8	-0.0081	0.0026	-0.0082	0.0024
6/8	-0.0106	0.0035	-0.0102	0.0026
7/8	-0.0123	0.0038	-0.0123	0.0030
1	-0.0141	0.0045	-0.0138	0.0036
1 1/8	-0.0163	0.0050	-0.0158	0.0038
1 2/8	-0.0184	0.0052	-0.0178	0.0046
1 3/8	-0.0200	0.0058	-0.0197	0.0049
1 4/8	-0.0223	0.0062	-0.0214	0.0051
1 5/8	-0.0240	0.0072	-0.0235	0.0060
1 6/8	-0.0253	0.0081	-0.0248	0.0062
1 7/8				
Slope	-0.0151	0.0046	-0.0149	0.0035



Rock Core Diametral Lines

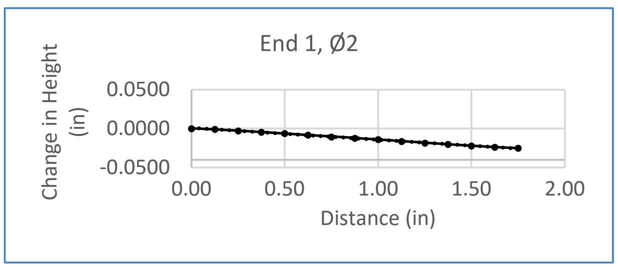
End 1, Ø2 Best-Fit Straight Line Angular Slope: -0.87 °
End 2, Ø2 Best-Fit Straight Line Angular Slope: -0.85 °

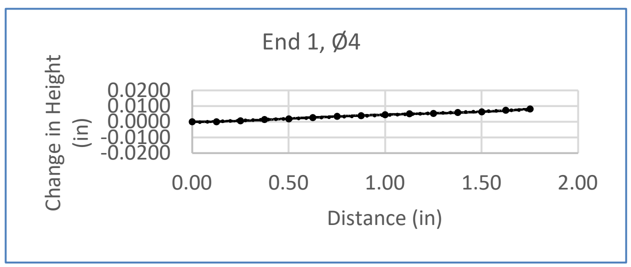
End 1, Ø4 Best-Fit Straight Line Angular Slope: 0.26 °
End 2, Ø4 Best-Fit Straight Line Angular Slope: 0.20 °

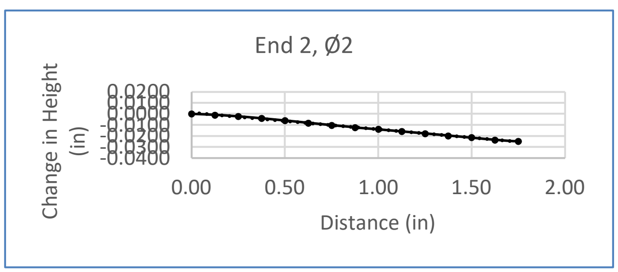
The parrallelism tolerance is met when the max. angular difference between the opposing best-fit straight line on each specimen end is not more than 0.25° for spherically seated test machines.

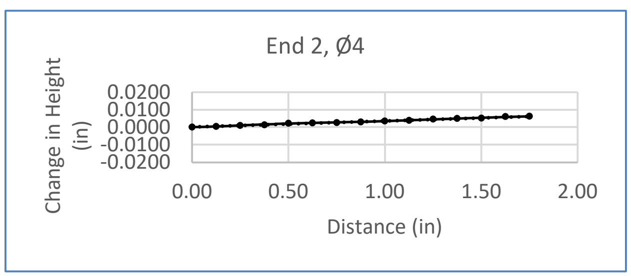
Max. Angular Difference (opposing best-fit straight lines)
Max. Angular Difference (Diameter 2): 0.01 °
Max. Angular Difference (Diameter 4): 0.06 °

Does the specimen meet parrallelism requirements?
☑ Yes ☐ No









<u>Perpendicularity (Procedure P2)</u>

The ends of the specimen meet perpendicularity when the gap, Δ, divided by the specimen length, L, is less than 1 part in 230, that is 0.0043.

Max .gap between the square and the top of specimen =

Δ = 0.063 inches Length = 4.28 inches
Gap / Length = 0.0147

Does the specimen meet perpendicularity requirements?
☐ Yes ☑ No

Property	**Pass / Fail**
Straightness	Fail
Flatness	Pass
Parallelism	Pass
Perpendicularity	Fail
Best Effort Applied	

References: ASTM D4543

PROCEDURE FOR VERIFYING SHAPE CONFORMANCE - INTACT ROCK CORE
ASTM D4543

Project Name:	SR-353 Emergency Bridge Repair	Diameter (in):	1.87	Tested By:	JBB
Project Number:	A24109.02271	Length (in):	4.12	Test Date:	10/16/2024
Client:	Gresham Smith - Knoxville	Length to Diameter:	2.20	Reviewed By:	BKP
Log Number:	B-2, (46.3')	Unit Weight (pcf):	168.6		

Flatness (Procedure FP2)						
Travel	End 1, Ø1	End 1, Ø2	End 1, Ø3	End 2, Ø1	End 2, Ø2	End 2, Ø3
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1/8	0.0000	0.0003	0.0002	-0.0001	0.0000	0.0000
2/8	-0.0002	0.0006	0.0001	-0.0004	0.0000	-0.0003
3/8	-0.0004	0.0008	0.0000	-0.0005	-0.0001	-0.0006
4/8	-0.0006	0.0007	-0.0002	-0.0010	-0.0004	-0.0008
5/8	-0.0010	0.0004	-0.0005	-0.0014	-0.0002	-0.0012
6/8	-0.0012	0.0004	-0.0008	-0.0015	-0.0005	-0.0012
7/8	-0.0017	0.0001	-0.0010	-0.0019	-0.0005	-0.0017
1	-0.0018	0.0002	-0.0012	-0.0022	-0.0006	-0.0018
1 1/8	-0.0022	0.0001	-0.0013	-0.0025	-0.0007	-0.0018
1 2/8	-0.0024	0.0002	-0.0015	-0.0028	-0.0009	-0.0020
1 3/8	-0.0027	-0.0001	-0.0018	-0.0032	-0.0010	-0.0021
1 4/8	-0.0030	-0.0001	-0.0019	-0.0033	-0.0011	-0.0026
1 5/8	-0.0032	-0.0003	-0.0022	-0.0037	-0.0010	-0.0026
1 6/8	-0.0038	-0.0005	-0.0028	-0.0039	-0.0012	-0.0030
1 7/8						

Length to Diameter Ratio (2.0 to 2.5) ☑ Yes ☐ No

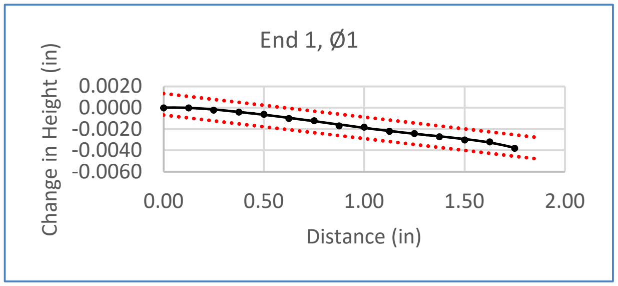
Minimum Diameter (1-7/8 inch) ☐ Yes ☑ No

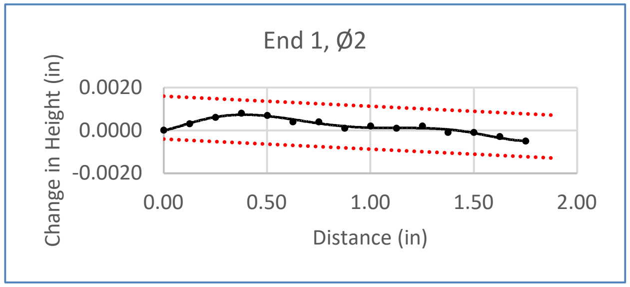
Side Straightness (Procedure S1) - Is the maximum gap between the specimen and the flat surface less than or equal to 0.020-in.?

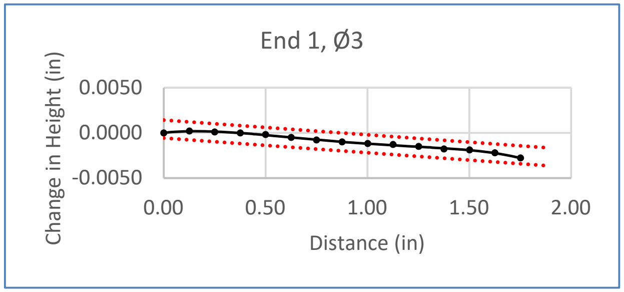
Maximum gap ≤0.020-in.? ☑ Yes ☐ No

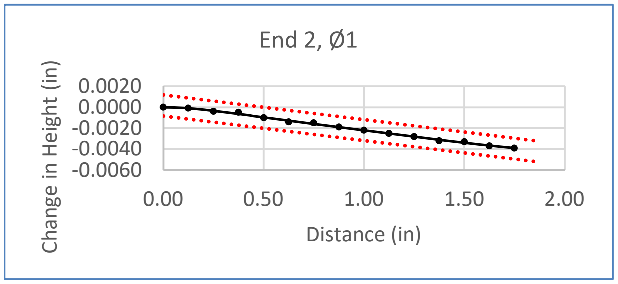
Does specimen meet straightness criteria? ☑ Yes ☐ No

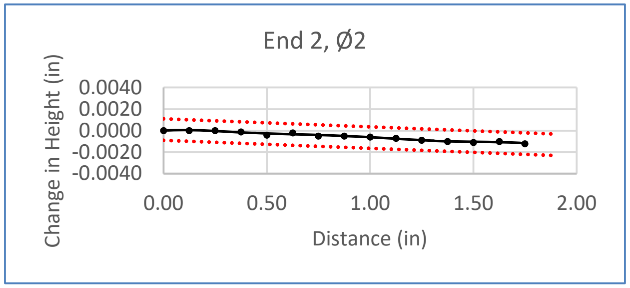
Flatness Requirement - The flatness tolerance is met when the smooth curve so determined does not depart from a visual best-fit straight line by more than 0.001 in.
Does the Specimen meet Flatness Requirements? ☑ Yes ☐ No

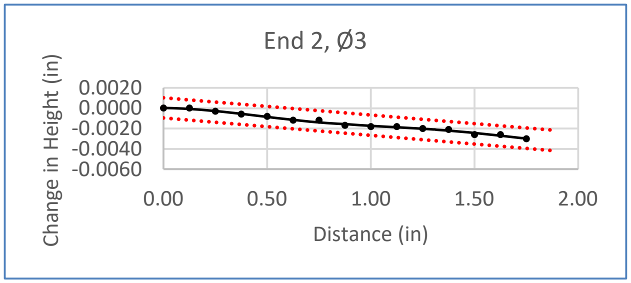












References: ASTM D4543

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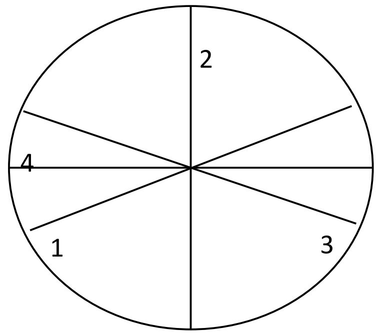
PROCEDURE FOR VERIFYING SHAPE CONFORMANCE - INTACT ROCK CORE
ASTM D4543

GEOS Project Name:	SR-353 Emergency Bridge Repair	Diameter (in):	1.87	Tested By:	JBB
GEOS Project Number:	A24109.02271	Length (in):	4.12	Test Date:	10/16/2024
GEOS Client:	Gresham Smith - Knoxville	Length to Diameter:	2.20	Reviewed By:	BKP
GEOS Log Number:	B-2, (46.3')	Unit Weight (pcf):	168.6		

Parallelism (Procedure FP2)				
Travel	End 1, Ø2	End 1, Ø4	End 2, Ø2	End 2, Ø4
0	0.0000	0.0000	0.0000	0.0000
1/8	0.0003	0.0000	0.0000	0.0000
2/8	0.0006	-0.0003	0.0000	-0.0004
3/8	0.0008	-0.0004	-0.0001	-0.0005
4/8	0.0007	-0.0006	-0.0004	-0.0007
5/8	0.0004	-0.0010	-0.0002	-0.0010
6/8	0.0004	-0.0012	-0.0005	-0.0013
7/8	0.0001	-0.0016	-0.0005	-0.0018
1	0.0002	-0.0019	-0.0006	-0.0019
1 1/8	0.0001	-0.0022	-0.0007	-0.0023
1 2/8	0.0002	-0.0024	-0.0009	-0.0025
1 3/8	-0.0001	-0.0028	-0.0010	-0.0028
1 4/8	-0.0001	-0.0029	-0.0011	-0.0031
1 5/8	-0.0003	-0.0034	-0.0010	-0.0034
1 6/8	-0.0005	-0.0042	-0.0012	-0.0042
1 7/8				
Slope	-0.0005	-0.0023	-0.0007	-0.0023



Rock Core Diametral Lines

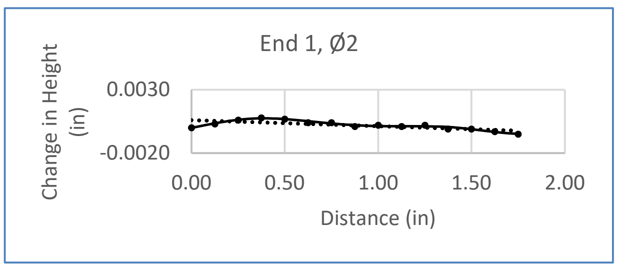
End 1, Ø2 Best-Fit Straight Line Angular Slope: -0.03 °
End 2, Ø2 Best-Fit Straight Line Angular Slope: -0.04 °

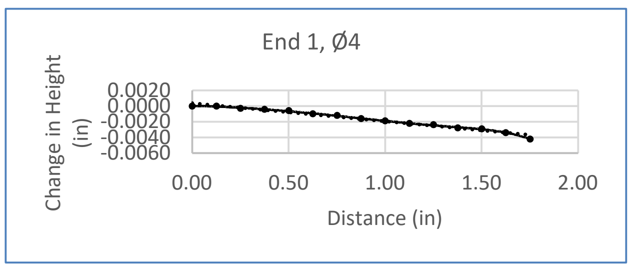
End 1, Ø4 Best-Fit Straight Line Angular Slope: -0.13 °
End 2, Ø4 Best-Fit Straight Line Angular Slope: -0.13 °

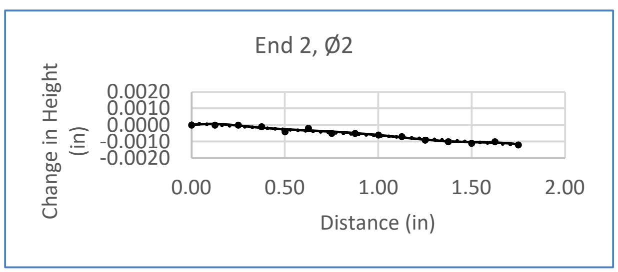
The parrallelism tolerance is met when the max. angular difference between the opposing best-fit straight line on each specimen end is not more than 0.25° for spherically seated test machines.

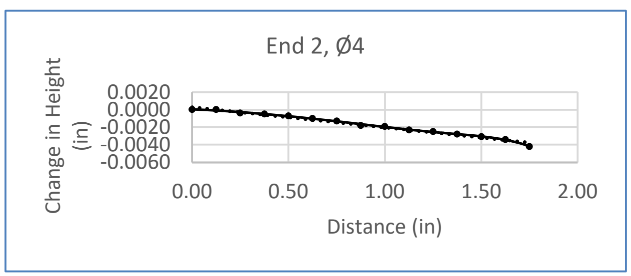
Max. Angular Difference (opposing best-fit straight lines)
Max. Angular Difference (Diameter 2): 0.02 °
Max. Angular Difference (Diameter 4): 0.00 °

Does the specimen meet parrallelism requirements?
☑ Yes ☐ No









<u>Perpendicularity (Procedure P2)</u>

The ends of the specimen meet perpendicularity when the gap, Δ, divided by the specimen length, L, is less than 1 part in 230, that is 0.0043.

Max .gap between the square and the top of specimen =

Δ = 0.004 inches Length = 4.12 inches
Gap / Length = 0.0010

Does the specimen meet perpendicularity requirements?
☑ Yes ☐ No

Property	Pass / Fail
Straigtness	Pass
Flatness	Pass
Parallelism	Pass
Perpendicularity	Pass
Best Effort Applied	

References: ASTM D4543

PROJECT SUMMARY REPORT

GEOS Project Name:	SR-353 Emergency Bridge Repair	GEOS Log Number:	As Identified Below
GEOS Project Number:	A24109.02271.000	Material ID:	Rock Core (See Below)
GEOS Client:	Gresham Smith - Knoxville	Tested By:	BKP
Project Location:	Knoxville, Tennessee	Approved By:	JBB

UNCONFINED COMPRESSIVE STRENGTH OF ROCK CORE
ASTM D7012 METHOD C

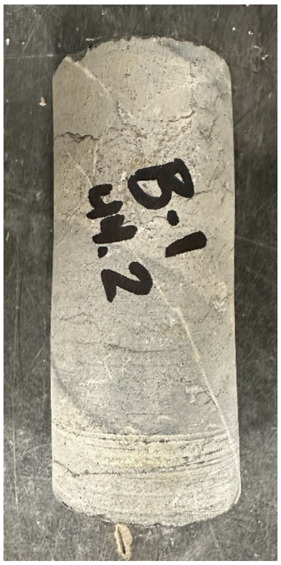
Core ID	Depth (ft)	Diameter (in)	Length (in)	Length/Diameter Ratio	Area (in^2)	Unit Weight (pcf)	Shape Key	Ultimate Load (lbs)	Compressive Strength (psi)
B-1	44.2	1.86	4.28	2.30	2.72	168.2	E	66,574	24,476
B-2	46.3	1.87	4.12	2.20	2.75	168.6	A	52,398	19,054

Shape Key	Description
A	Specimen met the requirements as stated in ASTM D4543-19 for straigtness, flatness, parallelism and perpendicularity.
B	Specimen did not meet the straightness requirements in ASTM D4543-19. Specimen met the requirements as stated in ASTM D4543-19 for flatness, parallelism and perpendicularity. Best effort applied.
C	Specimen did not meet the straightness and parallelism requirements in ASTM D4543-19. Specimen met the requirements as stated in ASTM D4543-19 for flatness and perpendicularity. Best effort applied.
D	Specimen did not meet the straightness, parallelism and perpendicularity requirements in ASTM D4543-19. Specimen met the requirements as stated in ASTM D4543-19 for flatness. Best effort applied.
E	Specimen did not meet the straightness and perpendicularity requirements in ASTM D4543-19. Specimen met the requirements as stated in ASTM D4543-19 for flatness and parallelism. Best effort applied.

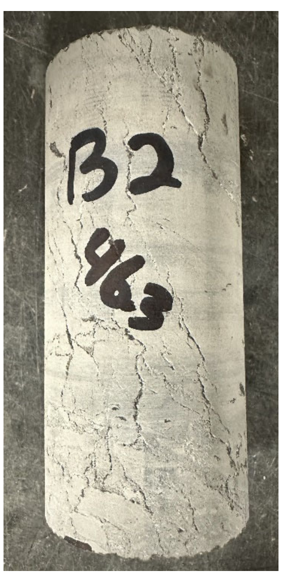
Comments:

PROJECT PHOTOGRAPHS

GEOS Project Name:	SR-353 Emergency Bridge Repair	Date:	10/21/2024
GEOS Project Number:	A24109.02271.000		



1	Sample Identification / Description:	B-1, 44.2'
	Test Identification:	ASTM D7012 Method C



2	Sample Identification / Description:	B-2, 46.3'
	Test Identification:	ASTM D7012 Method C

APPENDIX D

Engineering Calculations

Calculation Review and Approval Status Sheet

UES, LLC

UES Project Number: **A24109.02271**

Bridge Number: **90S238600011**

Project: **State Route 353 Emergency Bridge Replacement**

Title of Calculations:
H-Pile Nominal Capacity
H - Pile Nominal Capacity in Soil
Drilled Shaft Resistance
Seismic

IBRAHIM M. AKLOUK
REGISTERED ENGINEER
AGRICULTURE
COMMERCE
No. 127662
STATE OF TENNESSEE

10-28-24

Prepared by: Ibrahim M. Aklouk, P.E.
Geotechnical Project Engineer

Reviewed by: Aashish Sharma, PhD, EIT
Geotechnical Designer

H-Pile Nominal Capacity

HP10x42

In[∘]:= $P_n = F_{cr} * A_g$

A_g = Cross - sectional Area = 12.4 in^2 = 0.086 ft^2

$$\frac{KL}{r} \le 4.71\sqrt{\frac{E}{F_y}}$$

K = Effective length factor = 1

L = Unbraced length (in) = 120 in

In[∘]:= E = Young ' s modulus of steel = 29 000 Ksi

F_y = Steel strength of steel = 60 ksi

r = radius of gyration of controling axis = 2.41 in

In[∘]:= $$\frac{1 * 120\ \text{in}}{2.41\ \text{in}} \le 4.71\sqrt{\frac{29\,000\ \text{Ksi}}{60\ \text{ksi}}}$$

49.79 ≤ 103.54

In[∘]:= $$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) * F_y$$

In[∘]:= $$F_e = \frac{\pi^2 * E}{\left(\frac{K*L}{r}\right)^2} = 115.44\ \text{ksi}$$

In[∘]:= $$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) * F_y = 48.27\ \text{ksi}$$

P_n = 48.27 ksi * 12.4 in^2 = 598.5 k

- use $\phi = 0.9$

ϕP_n = 538.69 k

HP12x53

$P_n = F_{cr} * A_g$

A_g = Cross - sectional Area = $15.5\ in^2 = 0.108\ ft^2$

$$\frac{KL}{r} \le 4.71\sqrt{\frac{E}{F_y}}$$

K = Effective length factor = 1

L = Unbraced lenth (in) = 120 in

E = Young ' s modulus of steel = 29 000 Ksi

F_y = Steel strength of steel = 60 ksi

r = radius of gyration of controling axis = 2.86 in

$$\frac{1 * 120\ in}{2.86\ in} \le 4.71\sqrt{\frac{29\,000\ Ksi}{60\ ksi}}$$

$41.96 \le 103.54$

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) * F_y$$

$$F_e = \frac{\pi^2 * E}{\left(\frac{K*L}{r}\right)^2} = 162.58\ ksi$$

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) * F_y = 51.41\ ksi$$

$P_n = 51.41\ ksi * 15.5\ in^2 = 796.89\ k$

- use $\phi = 0.9$

$\varphi P_n = 717.2\ k$

H - Pile Nominal Capacity in Soil

HP12x53

- Two soils layer were used in this method to represent the fill and residual/alluvial soils
- Layer one is from 0-17 ft and layer two is from 17 to 35 ft

Side Resistance - Alpha Method From FHWA- NHI-16-009

$f_s = C_a = \alpha * S_u$ Eq. 7 - 10

In[ ]:= C_a = adhesion (ksf)
S_u = Undrained Shear Strength (ksf)
α = ashesion factor

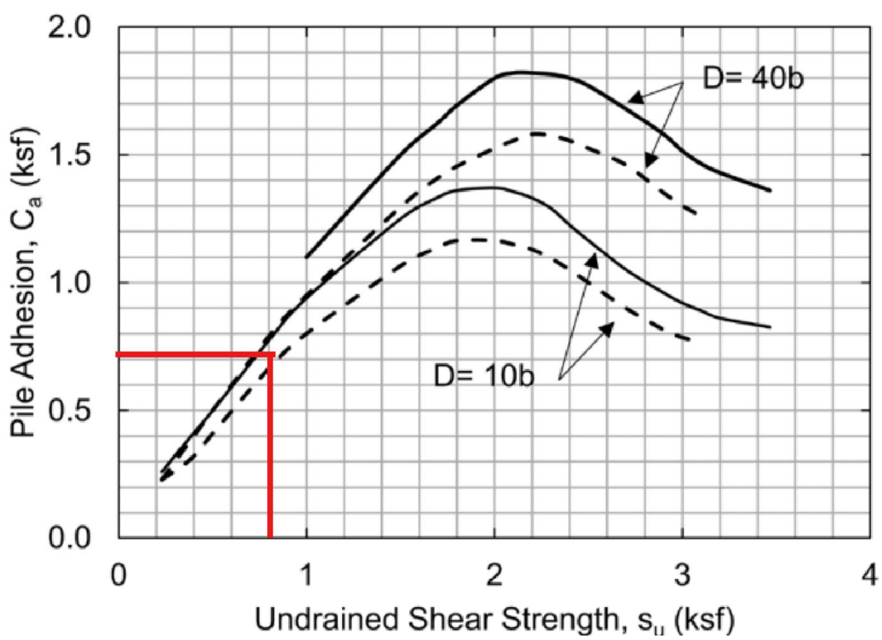
- Use Figure 7-17 to determine C_a
- 0-17 ft
- Use S_u = 800 ksf for the fill soils

D = Distance from ground surface to bottom of clay layer = 17 ft

In[ ]:= b = Pile Diamter = b_f = 12 inch = 1 ft

d = pile depth = 11.8 in

D / b = 17



- f_s = C_a = 0.70 based on graph

R_s = C_a * A_s

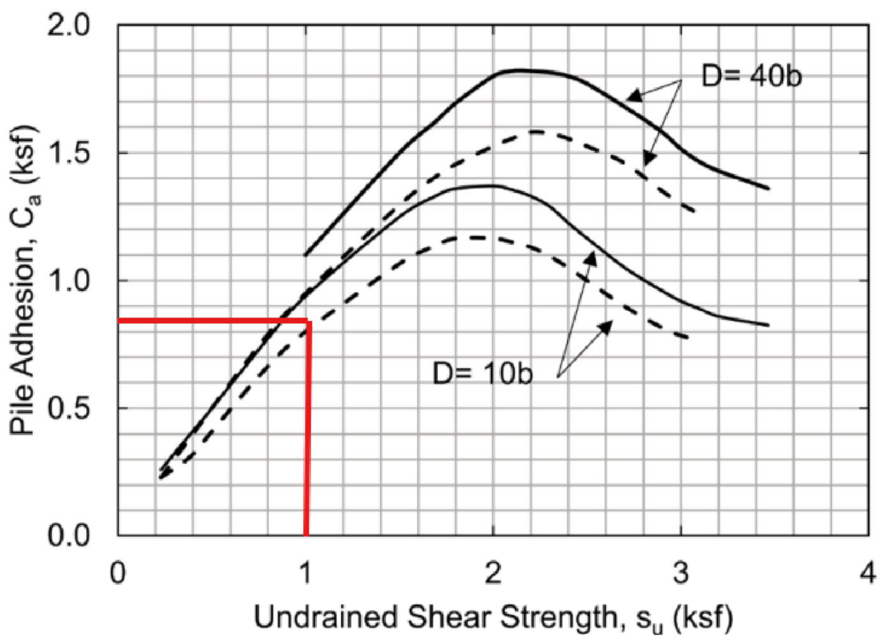
$$A_s = \text{Surface area} = \frac{(2*12\text{ in}) + (2*11.8\text{ in})}{12} * 17\text{ ft} = 67.49\text{ ft}^2$$

R_s = 0.7 ksf * 67.49 ft^2 = 47.24 k

- 17 - 35 ft
- Use S_u = 1, 000 ksf for the residual / alluvial soils

D = Distance from ground surface to bottom of clay layer = 18 ft

In[ ]:= D / b = 18



$f_s = C_a = 0.85$ based on graph

$R_s = C_a * A_s$

A_s = Surface area $= \frac{(2*12\text{ in}) + (2*11.8\text{ in})}{12} * 18\text{ ft} = 71.46\text{ ft}^2$

$R_s = 0.85\text{ ksf} * 71.46\text{ ft}^2 = 60.74\text{ k}$

$\sum R_s = 108.0\text{ k}$

Tip Resistance From FHWA - NHI - 16 - 009

$q_p = P_s * s_u * N_c + \gamma * D * N_q + P_t * \gamma * \left(\frac{b * N_\gamma}{2}\right)$ Eq. 7 – 34

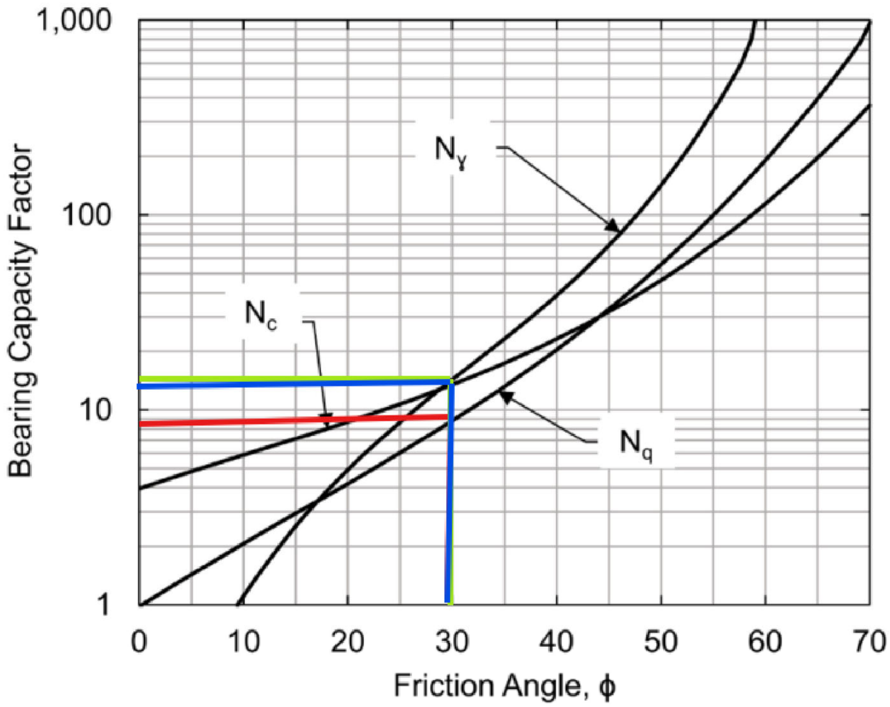
s_u = undrained shear resistance of the rock (ksf)
γ = effective density of the rock mass (kcf)
D = pile penetration below the rock surface (ft)

b = pile width or diameter (ft)

In[ ]:= P_s = pile toe shape factor of 1.25 for square pile or 1.2 for a circular Pile

P_t = pile base factor of 0.8 for a square pile or 0.7 for a circular pile. N_c, N_q, and N_γ are bearing capacity factors from Figure 7 – 22.

- Assume rock friction angle of 30 degrees.



- From Figure: N_c = 15, N_q = 8, and N_γ = 8
- Assume γ = 0.140 kcf
- During our rock core testing were unable to test a portion of the cored samples due to their shaly structure. As a result, 10,000 psi (1,440 ksf) will be used as the undrained shear resistance of the rock.

$$q_p = 1.25 * 1440\ \text{ksf} * 15 + 0.140 * 0 * 8 + 0.8 * 0.140 * \left(\frac{1*8}{2}\right) = 27\,000.4\ \text{ksf}$$

$$R_p = q_p * A_g$$

$$R_p = 27\,000.4\ \text{ksf} * 0.108\ \text{ft}^2 = 2916.05\ \text{k}$$

Total Resistance = 2916.05 k + 108.0 k = 3024.05 K

- Use 717.2 k, as it controls

HP10x42

- Two soils layer were used in this method to represent the fill and residual/alluvial soils
- Layer one is from 0-17 ft and layer two is from 17 to 35 ft

b = Pile Diamter = b_f = 10.1 inch = 0.84 ft

Side Resistance - Alpha Method From FHWA - NHI - 16 - 009

$f_s = C_a = \alpha * S_u$ Eq. 7 – 10

C_a = adhesion (ksf)
S_u = Undrained Shear Strength (ksf)
α = ashesion factor

- C_a values for this H-pile are the same as the one above since D/b is the same.

$R_s = C_a * A_s$

- 0 to 17 ft

D / b = 20

$f_s = C_a$ = 0.70 based on graph above

$$A_s = \text{Surface area} = \frac{(2 * 10.1\,\text{in}) + (2 * 9.7\,\text{in})}{12} * 17\,\text{ft} = 56.1\,\text{ft}^2$$

$R_s = 0.7\,\text{ksf} * 56.1\,\text{ft}^2 = 39.27\,\text{k}$

- 17 to 35 ft

D / b = 21

$f_s = C_a$ = 0.85 based on graph above

$$A_s = \text{Surface area} = \frac{(2 * 10.1\,\text{in}) + (2 * 9.7\,\text{in})}{12} * 18\,\text{ft} = 59.4\,\text{ft}^2$$

$R_s = 0.85\,\text{ksf} * 59.4\,\text{ft}^2 = 50.49\,\text{k}$

$\sum R_s = 89.76\,\text{k}$

Tip Resistance From FHWA - NHI - 16 - 009

- q_p - same as H-pile above

$R_p = q_p * A_g$

$R_p = 27\,000.4\,\text{ksf} * 0.086\,\text{ft}^2 = 2322.04\,\text{k}$

Total Resistance = 2322.04 k + 89.76 k = 2411.8 k

- Use 538.69 k, as it controls

Drilled Shafts

Side Resistance FHWA-NHI 18-024

$$\frac{f_{SN}}{P_a} = 0.65 * \alpha_E * \sqrt{\frac{q_u}{P_a}} \quad \text{Eq. } 10 - 22$$

q_a = mean value of uniaxial compressive strength for the rock layer

P_a = atmospheric pressure in the same units as qu

C = a regression coefficient used to analyze load test results

- Although lab tested compressive strength of cored rock was between 19, 000 and 20, 0000 psi; 10, 000 psi will be used as the compressive strength. This to account for the shaly material, which we were unable to test.

TABLE 10-3 SIDE RESISTANCE REDUCTION FACTOR FOR CAVING ROCK

RQD (%)	Joint Modification Factor, α_E	
	Closed joints	Open or gouge-filled joints
100	1.00	0.85
70	0.85	0.55
50	0.60	0.55
30	0.50	0.50
20	0.45	0.45

- Table 10-3 was used determine α_E and 20% RQD was selected to match the site conditions. Therefore, α_E = 0.45

$$\frac{f_{SN}}{14.7} = 0.65 * 0.45 * \sqrt{\frac{10,000 \text{ psi}}{14.7}}$$

f_{SN} = 112.15 psi = 16.1 ksf

- Use 15 ksf

Tip Resistance FHWA - NHI 18 - 024

$q_{BN} = N^*_{cr} * q_u$

q_u = 10, 000 psi

- Use Figure 10-8 to determine N^*_{cr}. Based Figure N^*_{cr} = 0.2

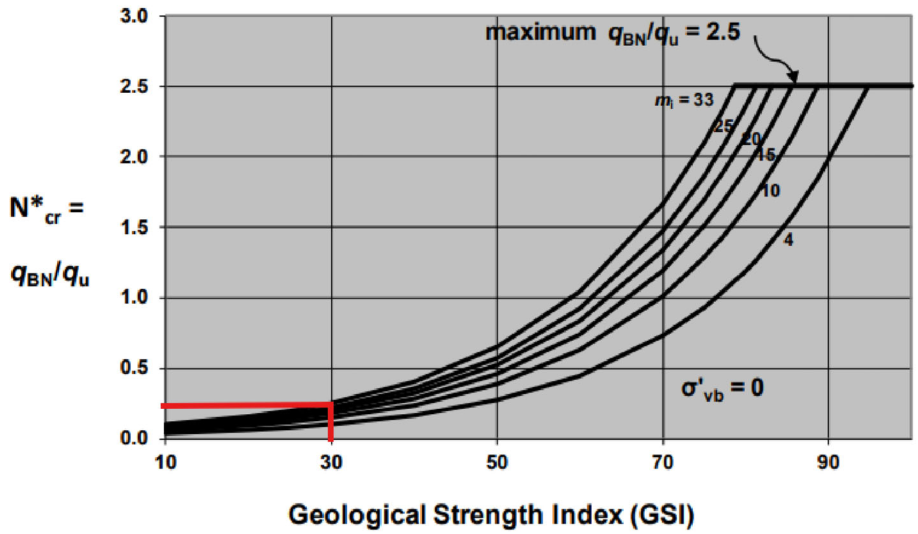


Figure 10-8 Bearing Capacity Factor N^*_{cr} versus GSI for Base Resistance of Fractured Rock Mass

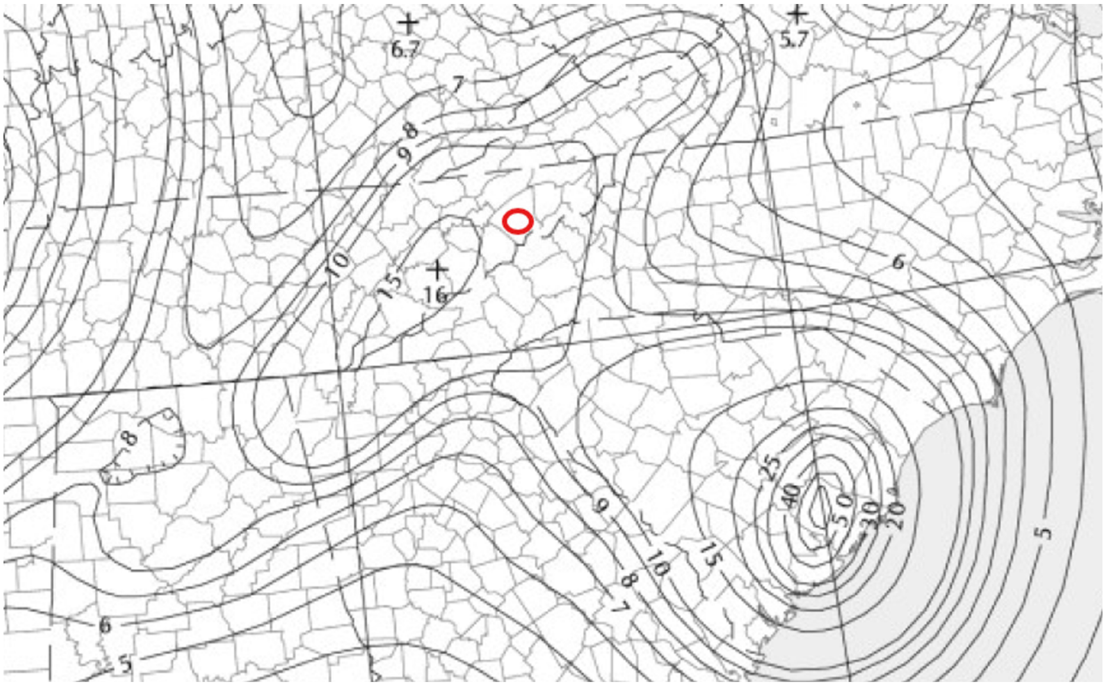
$$0.2 = \frac{q_{BN}}{10,000\ \text{psi}}$$

$$q_{BN} = 2,000\ \text{psi} = 288\ \text{ksf}$$

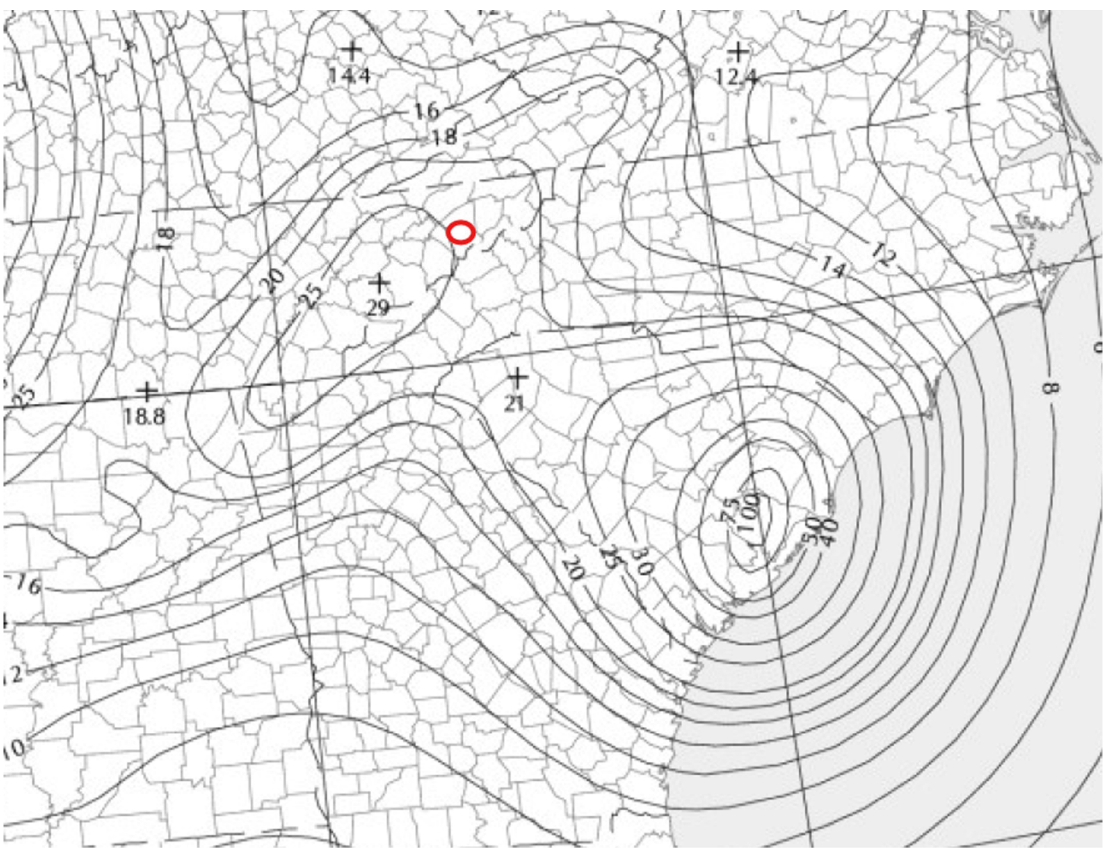
- Use 250 ksf

Seismic

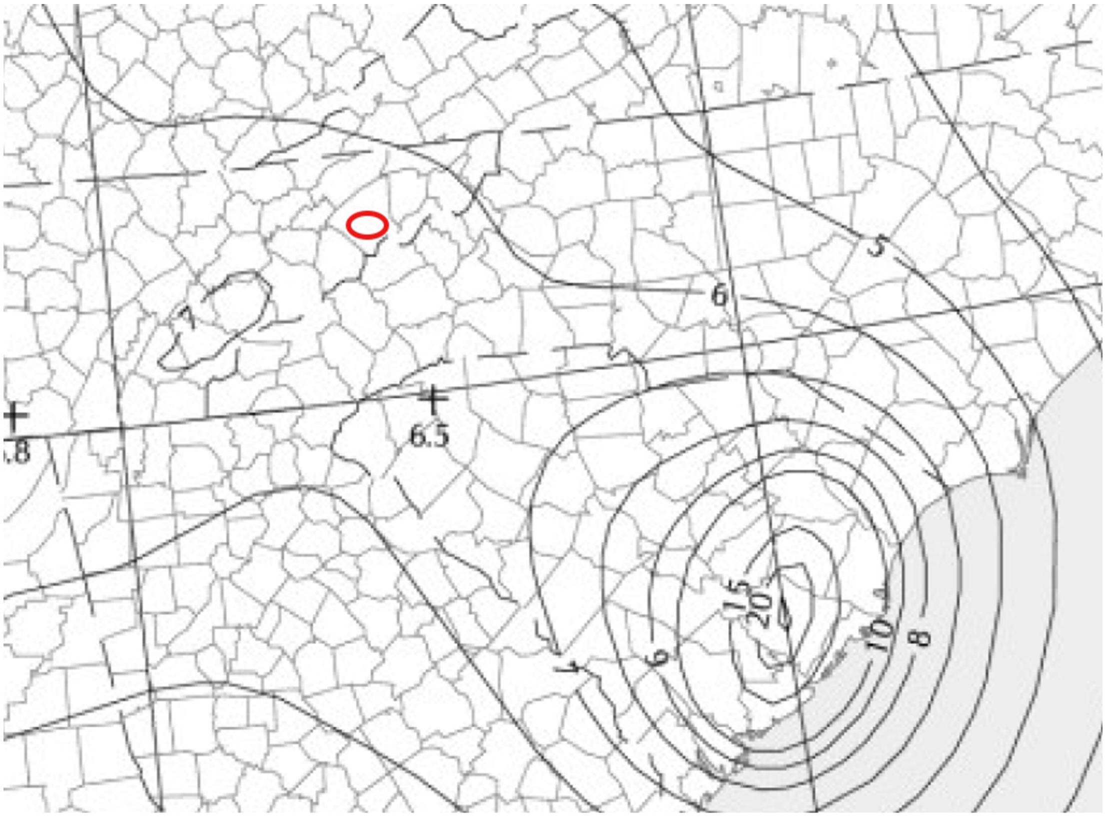
- Use Figure 3.10.2-.1-1 From AASHTO Figure to determine PGA. The site contour is 0.14.



- Use Figure 3.10.2-.1-2 From AASHTO Figure to determine S_S. The site contour is 0.24.



- Use Figure 3.10.2-.1-3 From AASHTO Figure to determine S_S. The site contour is 0.63.



Site Class - $\overline{N}$ Method

$$\overline{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$$

d_i = thickness of a layer between 0 and 100 ft

N_i = Standard Penetration Test blow count of a layer (not to exceed 100 blows / ft in the above expression)

B - 1

$$\overline{N} = \frac{2.5\text{ ft}}{7} + \frac{2.5\text{ ft}}{7} + \frac{2.5\text{ ft}}{17} + \frac{2.5\text{ ft}}{14} + \frac{5\text{ ft}}{50} + \frac{5\text{ ft}}{14} + \frac{5\text{ ft}}{2} + \frac{5\text{ ft}}{1} + \frac{5\text{ ft}}{50} + \frac{65\text{ ft}}{100} = 11.64$$

B - 2

$$\bar{N} = \frac{2.5\,ft}{11} + \frac{2.5\,ft}{11} + \frac{2.5\,ft}{14} + \frac{2.5\,ft}{18} + \frac{5\,ft}{10} + \frac{5\,ft}{9} + \frac{5\,ft}{18} + \frac{5\,ft}{50} + \frac{70\,ft}{100} = 34.42$$

- Average of $\bar{N}$ from both borings = 23.03. Based Table below, the site class is D.

Table 3.10.3.1-1—Site Class Definitions

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\bar{v}_s > 5{,}000$ ft/s
B	Rock with 2,500 ft/sec < $\bar{v}_s$ < 5,000 ft/s
C	Very dense soil and soil rock with 1,200 ft/sec < $\bar{v}_s$ < 2,500 ft/s, or with either $\bar{N}$ > 50 blows/ft, or $\bar{s}_u$ > 2.0 ksf
D	Stiff soil with 600 ft/s < $\bar{v}_s$ < 1,200 ft/s, or with either 15 < $\bar{N}$ < 50 blows/ft, or 1.0 < $\bar{s}_u$ < 2.0 ksf
E	Soil profile with $\bar{v}_s$ < 600 ft/s or with either $\bar{N}$ < 15 blows/ft or $\bar{s}_u$ < 1.0 ksf, or any profile with more than 10.0 ft of soft clay defined as soil with PI > 20, w > 40 percent and $\bar{s}_u$ < 0.5 ksf
F	Soils requiring site-specific evaluations, such as: • Peats or highly organic clays (H > 10.0 ft of peat or highly organic clay where H = thickness of soil) • Very high plasticity clays (H > 25.0 ft with PI > 75) • Very thick soft/medium stiff clays (H >120 ft)

- Use Table 3.10.3.2-1 to factor PGA. Based on table use a factor 1.5.

Table 3.10.3.2-1—Values of Site Factor, F_{pga}, at Zero-Period on Acceleration Spectrum

Site Class	Peak Ground Acceleration Coefficient (PGA)[1]				
	PGA < 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA > 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F[2]	*	*	*	*	*

PGA = 1.5 * 0.14 = 0.21

- Use Table 3.10.3.2-2 to factor S_S. Based on table use a factor 1.6.

Table 3.10.3.2-2—Values of Site Factor, F_a, for Short-Period Range of Acceleration Spectrum

Site Class	Spectral Acceleration Coefficient at Period 0.2 sec (S_S)[1]				
	$S_S <$ 0.25	$S_S =$ 0.50	$S_S =$ 0.75	$S_S =$ 1.00	$S_S >$ 1.25
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F[2]	*	*	*	*	*

S_s = 1.6 * 0.24 = 0.384

- Use Table 3.10.3.2-3 to factor S_1. Based on table use a factor 2.4.

Table 3.10.3.2-3—Values of Site Factor, F_v, for Long-Period Range of Acceleration Spectrum

Site Class	Spectral Acceleration Coefficient at Period 1.0 sec (S_1)[1]				
	$S_1 <$ 0.1	$S_1 =$ 0.2	$S_1 =$ 0.3	$S_1 =$ 0.4	$S_1 >$ 0.5
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F[2]	*	*	*	*	*

S_1 = 2.4 * 0.063 = 0.1512