Geotechnical Data Report
I-55 Ramp/SR-14 Bridge Replacement and Interchange Improvement
Shelby County, Tennessee

TDOT Project No. 79005-1175-14
TDOT PIN No. 128674.00

Prepared for:

Kimley-Horn
Memphis, Tennessee

Prepared by:

Geotechnology, LLC
Memphis, Tennessee

Date:

July 21, 2023

Geotechnology Project No.:

J042144.01

SAFETY
QUALITY
INTEGRITY
PARTNERSHIP
OPPORTUNITY
RESPONSIVENESS

July 21, 2023

Ms. Heather Lewis, P.E.
Kimley-Horn
6750 Poplar Avenue, Suite 600
Memphis, Tennessee 38138

Re: Geotechnical Data Report
I-55 Ramp/SR-14 Bridge Replacement and Interchange Improvement
Shelby County, Tennessee
Geotechnology Project No. J042144.01
TDOT Project No. 79005-1175-14
TDOT Pin No. 128674.00

Dear Ms. Lewis:

Presented in this report are the results of the geotechnical exploration performed by Geotechnology, LLC for the referenced project. The report includes our understanding of the project, observed site conditions and data as listed in the Table of Contents.

We appreciate the opportunity to provide geotechnical services for this project. If you have any questions regarding this report, or if we can be of any additional service to you, please do not hesitate to contact us.

Respectfully submitted,

GEOTECHNOLOGY, LLC

Amber Meadows
Project Engineer

Ashraf S. Elsayed, Ph.D., P.E.
Chief Engineer – South Region

ABM/DBA/ASE:sas

Copies submitted: Client (email)

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I-55 Ramp/SR-14 Bridge Replacement and Interchange Improvement
Shelby County, Tennessee

July 21, 2023 | Geotechnology Project No. J042144.01

1.0 SCOPE OF SERVICES

Presented in this report is the data from geotechnical exploration for the design and construction of the proposed bridge replacements and interchange improvements at the Interstate 55 (I-55) and State Route 14 (SR-14) interchange. The referenced project includes demolition of the existing SR-14 bridge over I-55 and the I-55 exit ramp bridge over I-55 and SR-14, and the construction of new bridges, roadway, median, ditches and side slopes. The project location is shown on Figure 1 included in Appendix B.

Results of the sounding, borings, in-situ sampling, testing, sampling, in-situ laboratory testing, and site-specific seismic study are included in this report. A total of six borings and one CPT sounding were performed in the vicinity of the site as shown on Figure 2 included in Appendix B. The CPT sounding and boring logs, along with field and laboratory test results, are enclosed. The collected data has been analyzed and the physical properties of the in-situ soils summarized. Unless noted otherwise, all dimensions, measurements, depths, and locations in this report should be considered approximate. Important information prepared by the Geotechnical Business Council (GBC) of the Geoprofessional Business Association for studies of this type is presented in Appendix A for your review.

2.0 GENERAL INFORMATION

Existing Structure and Planned Modifications

The project at I-55 and SR-14 (South 3rd Street) in Shelby County includes the SR-14 bridge over I-55 and the northbound I-55 exit ramp bridge to southbound SR-14. The planned modifications are discussed in the following paragraphs.

<u>Bridge One: State Route 14 Bridge over Interstate 55</u>

The existing bridge is a 192-foot-long, 95-foot-wide, four-span, pre-stressed, concrete bridge with 14 feet of vertical clearance. The bridge consists of four, 12-foot-wide travel lanes and outside shoulders, a 4-foot-wide median, and sidewalks with curb and gutter.

The existing bridge will be replaced with a 200-foot long, 95-foot-wide, two-span, pre-stressed, concrete bridge. Up to 4 feet of cut and 9 feet of fill will be required to achieve design grades. Sides slopes are anticipated to range from two horizontal units for every vertical unit (2H:1V) and (6H:1V).

Bridge Two: Interstate 55 Exit Ramp Bridge to State Route 14

The existing bridge is a 512-foot-long, 27-foot-wide, seven span, pre-stressed, concrete and steel girder bridge with 15 feet of vertical clearance. The bridge consists of one 16-foot-wide travel lane with 2-foot shoulders. The existing roadway approach has shoulder widths of 5 to 6 feet.

The existing bridge will be replaced with a 660-foot-long, 43-foot-wide, five-span, steel girder bridge. Up to 8 feet of cut and 15 feet of fill will be required to achieve design grades. Sides slopes are anticipated to range from 2H:1V to 6H:1V.

Ancillary Modifications

Additional improvements will be made to the interchange including modifying the existing side slopes of the westbound I-55 exit ramp for southbound SR-14, eastbound I-55 exit ramp for southbound SR-14, northbound SR-14 exit ramps for eastbound and westbound I-55, and the southbound SR-14 exit ramps for eastbound I-55. Up to 12 feet of cut and 15 feet of fill will be required to achieve design grades. Sides slopes are anticipated to range from 2H:1V to 6H:1V.

Topography and Drainage

Based on the provided plans, the existing topography at the site ranges from approximately El 205[1] to El 250 along the existing I-55 and SR-14 (South 3rd Street). The site is within the floodplain of Nonconnah Creek, which drains west into the Mississippi River.

Geology

The site is located in Shelby County in the southwestern corner of Tennessee on the Gulf Coastal Plain. The Coastal Plain in the project area is characterized by flat to hilly topography and is dissected in many places by rivers and creeks. Approximately 200 feet of relief occur across the county.

Geologically, the site is near the north-central part of the Mississippi Embayment, a trough-like depression plunging southward along an axis approximating the present course of the Mississippi River. Sediment depth in the project area is approximately 2,900 feet. The unconsolidated sediments consist of clay, silt (aeolian, alluvial and marine), sand, gravel and lignite, ranging in age from Cretaceous to recent. Except for some local beds of ferruginous and calcareous sandstone and limestone, there is no well-consolidated rock above the Paleozoic Formation, which forms the rock below the sediments.

The uppermost formation over most of Shelby County is Pleistocene Epoch Loess, which consists of clayey silts and silty clays. Loess is predominantly silt, but with varying amounts of clay, and is generally buff-colored and uniform in texture. The thickness of the loess is usually about 20 to 30 feet, but typically is greater than 60 feet along the Mississippi River. The loess cap thins to the east, commonly terminating at the Mississippi Embayment boundary.

[1] Elevations are in units of feet referenced to the North American Vertical Datum 1988

The next formation in succession is a discontinuous series of alluvial deposits referred to as the Terrace Deposits. The terrace deposits are tertiary period in age, thin gradually eastward, and are absent in many places as a result of erosion or non-deposition. The alluvial deposits are composed mostly of coarse-grained quartz sand, fine-grained iron-stained quartz and chert gravel. Lenses of yellowish-brown clay are frequently present locally in the lower part of the deposits. These materials are typically red or brown, dense and well graded, and the thickness ranges from 0 to 200 feet. They generally occur 35 to 50 feet below the ground surface.

Underlying the terrace deposits is the Jackson Formation, which is a series of marine deposits of Eocene age consisting of hard blue, gray or brown clays interbedded with very dense white fine sands and seams of lignite. The thickness of the Jackson Formation in the area ranges up to 350 feet. The Jackson Formation overlies the Tertiary Period Claiborne/Wilcox Formation, which is characterized as irregularly bedded sand, locally interbedded with lenses and beds of gray to white clay, silty clay, lignitic clay and lignite; the thickness of this formation is typically more than 400 feet.

Sediments deposited by streams along the channels and on the flood plains during flood periods are referred to as Alluvium. These materials are from the Holocene Period, and are composed of clay, silt, sand, and gravel. The alluvium in the Memphis area is generally confined to strips along the Mississippi River and its tributaries, and it is frequently subjected to flooding and reworking. Lignite, peat, and carbonaceous materials are distributed irregularly throughout the upper level. Alluvial sands are usually white or gray, loose to medium dense, and poorly graded. The loose and poorly graded alluvial sands can be susceptible to liquefaction during seismic events.

3.0 GEOTECHNICAL EXPLORATION

The geotechnical exploration consisted of six borings, designated as Borings B-1 through -4 and R-1 and -2 and one cone penetration test sounding designated as CPT-1. Borings B-1 through -4 were located near the proposed bridge supports, and Borings R-1 through -2 were located near the proposed retaining walls. Approximate locations of the borings and sounding are shown on Figure 2 (Aerial Photograph of Site and Boring Locations) in Appendix B.

The borings were drilled between March 28 and April 4, 2023 with a track-mounted rotary drill rig (Diedrich D-50) advancing hollow stem augers and rotary wash drilling methods to a maximum depth of 100 feet. Sampling of the soils was accomplished ahead of the augers at the depths indicated on the boring logs, using 2-inch-outside-diameter (O.D.) split-spoons and 3-inch-O.D., thin-walled Shelby tube samplers in general accordance with the procedures outlined by ASTM D1586 and ASTM D1587, respectively. Standard Penetration Tests (SPTs) were conducted at

2.5- and 5-foot depth intervals using an automatic hammer to obtain the standard penetration resistance, or N-value[2], of the sampled material.

A Geotechnology engineer recorded the subsurface profile noting the soil types and stratifications, groundwater, SPT results, and other pertinent data. Observations for groundwater were made in the borings during drilling.

Representative portions of the split-spoon samples were placed in glass jars to preserve sample moisture. The Shelby tubes were capped and taped at their ends to preserve sample moisture and unit weight, and the tubes were transported and stored in an upright position. The glass jars and Shelby tubes were marked and labeled in the field for identification, then returned to our laboratory in Memphis.

The samples were examined in the laboratory by a geotechnical professional who prepared descriptive logs of the materials encountered. Logs of the borings are presented in Appendix C. An explanation of the terms and symbols used on the boring logs is also provided in Appendix C. Listed in Table 1 are in situ tests and measurements made as part of the fieldwork and recorded on the boring logs.

Table 1. Field Tests and Measurements.

Item	Test Method
Soil Classification	ASTM D 2488/ D 3282
Standard Penetration Test (SPT)	ASTM D 1586/ AASHTO T206
Thin-Walled (Shelby) Tube Sampling	ASTM D 1587/ AASHTO T207
Electronic Friction Cone and Piezocone Penetration Test	ASTM D 5778

Geotechnology also conducted one cone penetration test (CPT) sounding, designated as CPT-1, on March 28, 2023. The approximate CPT location is shown on Figure 2 in Appendix B. The CPT location was staked in the field by Geotechnology personnel using a handheld GPS device.

The CPT sounding was advanced to 100 feet using a 20-ton, track-mounted Vertek direct-push rig. The data was collected using a Vertek 15 square-centimeter end area, seismic piezometric cone with a u_2 pore pressure location (i.e., behind the cone). Plots of the CPT measurements are presented in Appendix C along with interpreted soil behavior types.

[2] The standard penetration resistance, or N-value, is defined as the number of blows required to drive the split-spoon sampler 12 inches with a 140-pound hammer falling 30 inches. Since the split-spoon sampler is driven 18 inches or until refusal, the blows for the first 6 inches are for seating the sampler, and the number of blows for the final 12 inches is the N-value. Additionally, “refusal” of the split-spoon sampler occurs when the sampler is driven less than 6 inches with 50 blows of the hammer.

A seismic cone penetration test (SCPT) was performed at approximately 3-foot depth intervals in Sounding CPT-1 to collect shear wave velocity data. A plot of shear wave velocity measurements versus depth is presented in the site-specific seismic study in Appendix E.

The boring logs and related information depict subsurface conditions only at the specific locations and times where sampling was conducted. The passage of time could result in changes in conditions, interpreted to exist, at or between the locations where sampling was conducted.

4.0 LABORATORY REVIEW AND TESTING

Laboratory testing was performed on soil samples to assess engineering and index properties. Moisture contents, Atterberg limits, percent fines, pH, resistivity, and UU test results are presented on the boring logs in Appendix C. Laboratory test results for Atterberg, grain size (sieve) analysis, resistivity, and unconsolidated-undrained triaxial compression (UU), are presented in Appendix D. Laboratory tests and corresponding test method standards are listed in Table 2.

Table 2. Summary of Laboratory Tests and Methods.

Number of Tests	Laboratory Test	ASTM	AASHTO
59	Moisture Content	D 2216	T 265
16	Atterberg Limits	D 4318	T 98
19	Grain Size Analysis by Sieving	D 6913	T 88
6	Percent Fines	D 1140	T 11
12	Unconsolidated-Undrained Triaxial Compression (UU)	D 2850	T 296
8	Soil Electrical Resistivity	G 57	T 288
8	Soil pH	D 4972	T 289

The boring logs were prepared from field logs, visual classification of the soil samples, and laboratory test results. Terms and symbols used on the boring logs are presented on the Boring Log: Terms and Symbols in Appendix C. Stratification lines on the boring logs indicate approximate changes in strata. The transition between materials could be gradual or could occur between recovered samples. The stratification given on the boring logs, or described herein, is for use by Geotechnology in its analyses and should not be used as the basis of design or construction cost estimates without realizing that there can be variation from that shown or described.

5.0 SUBSURFACE CONDITIONS

General Stratigraphy

The ground surface at the locations of the borings was covered with 3 to 6 inches of topsoil. The stratigraphy encountered in the borings generally consisted of intermixed layers of predominantly fine-grained soils classified as silt, lean clay, fat clay, with varying amounts of sand, and predominantly coarse-grained soils classified as sand, gravel, clayey sand and silty sand. Detailed descriptions of the stratigraphy encountered are presented below and on the boring logs and CPT sounding in Appendix C.

Upper, Fine-Grained Soils. The upper, fine-grained soils were generally classified as silt (ML), lean clay (CL), and fat clay (CH) with varying amounts of sand by the Unified Soil Classification System (USCS) and A-4, A-6, and A-7-6 by the AASHTO method and extended to maximum depths of 18 to 33 feet. Moisture contents of tested samples ranged from 15 to 28 percent. Liquid limits and plasticity indices of the tested samples ranged from 30 to 47 percent and 9 to 30 percent, respectively. SPT N-values ranged from 3 to 22 blows per foot (bpf). The UU tests performed on relatively undisturbed Shelby Tube samples yielded an undrained shear strength range of 475 to 3,315 pounds per square foot (psf). The results of field and laboratory testing were indicative of soft to very stiff consistencies in this upper fine-grained stratum.

Coarse-Grained Soils. The coarse-grained soils were generally classified as clayey sand (SC), intermixed sand (SP, SP-SC, SP-SM), gravel (GP), and silty sand (SM) by USCS and A-1-a, A-1-b, and A-3 by the AASHTO method and generally extended to depths of 98 feet in Borings B1 through -4. SPT N-values measured in the coarse-grained soils ranged from 10 to greater than 50 bpf. Based on the SPT N-values and CPT results, the conditions in the coarse-grained soils ranged from very loose to very dense.

Lower Fine-Grained Soils. The lower, fine-grained soils encountered underlying the coarse-grained soils in Borings B1 through -4 were classified as fat clay (CH) with varying amounts of sand by the Unified Soil Classification System (USCS) and A-7-6 by the AASHTO method and extended in the 100-foot borings to termination depth. Moisture contents of tested samples ranged from 21 to 38 percent. SPT N-values ranged from 10 to 40 blows per foot (bpf). The results of field testing were indicative of medium stiff to hard consistencies in this lower fine-grained stratum.

Groundwater

Groundwater was encountered during drilling at depths of 16,18,11,28 and 24 feet in Borings B-1, -3, -4, and R-1 and -2, respectively. Groundwater was not encountered in Boring B-2; however, groundwater levels might have been masked by the use of rotary wash drilling, which introduced water into the borehole. Based on pore pressure data from the CPT sounding, ground water was interpreted at a depth of 16 feet at the location of CPT-1. Groundwater levels could vary significantly over time due to the effects of seasonal variation in precipitation, recharge from Nonconnah Creek, or other factors not evident at the time of exploration.

6.0 CONCLUSIONS AND RECOMMENDATIONS

Seismic Considerations

Earthquake Risk. The project area is located within the New Madrid Seismic Zone (NMSZ). The NMSZ is located in the northern part of the Mississippi Embayment and trends in a northeast to southwest direction from southern Illinois to northeast Arkansas. In December 1811, a series of large magnitude earthquakes occurred, which were centered near New Madrid, Missouri. Three strong earthquakes occurred over a 3-month period and smaller aftershocks continued until at least 1817. According to researchers, the magnitudes of these three events ranged from 7.5 to 8.0.

Earthquake Forces. It is our understanding that the proposed bridge will be designed in accordance with the AASHTO publication "LRFD Bridge Design Specifications", ninth edition (2020).

Site-Specific Seismic Study

A site-specific seismic study was performed for the project site to develop a seismic design response spectrum. The process included downhole, seismic cone testing to measure the shear wave velocity of the soil profile, performing probabilistic seismic hazard analyses to determine probabilistic consistent magnitudes and epicentral distances, generation of time histories, and evaluation of the near-surface soil effects. Data measured using the seismic cone resulted in an average shear wave velocity in the upper 100 feet ($V_{S,100}$) of CPT-1 of 738 feet per second as shown on Table 1 of the appended Site-Specific Seismic Study report in Appendix E. The shear wave velocity profile is shown in Figure 3 in Appendix B.

The results of the seismic study indicate that the site is a Site Class D, "stiff soil", profile based on a $V_{S,100}$ of 738 feet per second, per the criteria stated in Chapter 3 of the AASHTO LRFD Bridge Specifications. The site class is based on the average shear wave velocity of the soil profile in the top 100 feet as measured in CPT-1. According to the results of the site-specific seismic study, the recommended site-specific design accelerations are presented in Table 3.

Table 3. Site-Specific Design Spectral Acceleration Coefficients.

Seismic Event	Site-Adjusted Peak Ground Acceleration (A_s)	0.2-Second Acceleration (S_{DS})	1.0-Second Acceleration (S_{D1})
7% PE* in 75 Years	0.389g	0.829g	0.564g

* Probability of Exceedance

7.0 LIMITATIONS

This report has been prepared on behalf of, and for the exclusive use of, the client for specific application to the named project as described herein. If this report is provided to other parties, it should be provided in its entirety with all supplementary information. In addition, the client should make it clear the information is provided for factual data only, and not as a warranty of subsurface conditions presented in this report.

Geotechnology has attempted to conduct the services reported herein in a manner consistent with the level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality and under similar conditions. The recommendations and conclusions contained in this report are professional opinions. The report is not a bidding document and should not be used for that purpose.

Our scope for this phase of the project did not include any environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below or around this site. Any statements in this report or on the boring logs regarding odors noted or unusual or suspicious items or conditions observed are strictly for the information of our client. Our scope did not include an assessment of the effects of flooding and erosion of creeks or rivers adjacent to or on the project site.

Our scope did not include: any services to investigate or detect the presence of mold or any other biological contaminants (such as spores, fungus, bacteria, viruses, and the by-products of such organisms) on and around the site; or any services, designed or intended, to prevent or lower the risk of the occurrence of an infestation of mold or other biological contaminants.

The analyses, conclusions, and recommendations contained in this report are based on the data obtained from the geotechnical exploration. The field exploration methods used indicate subsurface conditions only at the specific locations where samples were obtained, only at the time they were obtained, and only to the depths penetrated. Consequently, subsurface conditions could vary gradually, abruptly, and/or nonlinearly between sample locations and/or intervals.

The conclusions or recommendations presented in this report should not be used without Geotechnology's review and assessment if the nature, design, or location of the facilities is changed, if there is a lapse in time between the submittal of this report and the start of work at the site, or if there is a substantial interruption or delay during work at the site. If changes are contemplated or delays occur, Geotechnology must be allowed to review them to assess their impact on the findings, conclusions, and/or design recommendations given in this report. Geotechnology will not be responsible for any claims, damages, or liability associated with any other party's interpretations of the subsurface data or with reuse of the subsurface data or engineering analyses in this report.

The recommendations included in this report have been based in part on assumptions about variations in site stratigraphy that can be evaluated further during earthwork and foundation construction. Geotechnology should be retained to perform construction observation and continue its geotechnical engineering service using observational methods. Geotechnology cannot assume liability for the adequacy of its recommendations when they are used in the field without Geotechnology being retained to observe construction.

Appendix A

IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL-ENGINEERING REPORT

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are not final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

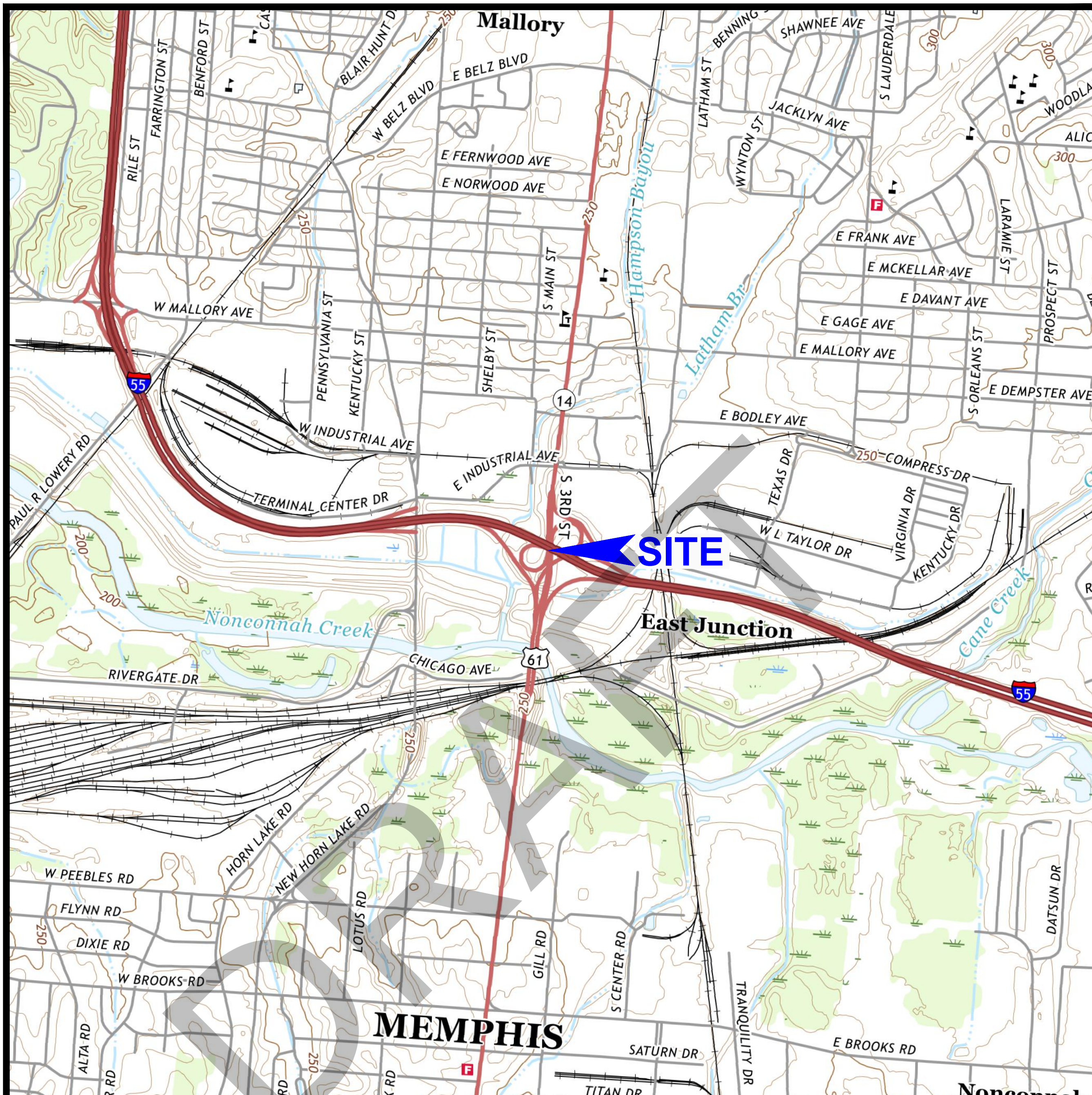
Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

Appendix B
FIGURES



NOTES

1. Plan adapted from a 7.5 minute U.S.G.S. map for Southwest Memphis, Tennessee-Arkansas quadrangle, last revised in 2019.

0 2,000 4,000

SCALE IN FEET

N

Drawn By: WAH	Ck'd By: SAS	App'vd By: ABM
Date: 7-13-23	Date: 7-18-23	Date: 7-18-23

GEOTECHNOLOGY
A UES Company

I-55 Ramp/SR-14 Bridge Replacement and Interchange Improvement
Memphis, Tennessee

SITE LOCATION AND TOPOGRAPHY

Project Number J042144.01

FIGURE 1

NOTES

1. Plan adapted from a July 14, 2022 aerial photograph courtesy of Google Earth.
2. Borings and Sounding were located in the field with reference to site features and are shown approximate only.

LEGEND

● Borings and Sounding Location

Drawn By: WAH	Ck'd By: SAS-	App'vd By: ABM
Date: 7-13-23	Date: 7-18-23	Date: 7-18-23

GEOTECHNOLOGY
A UES Company

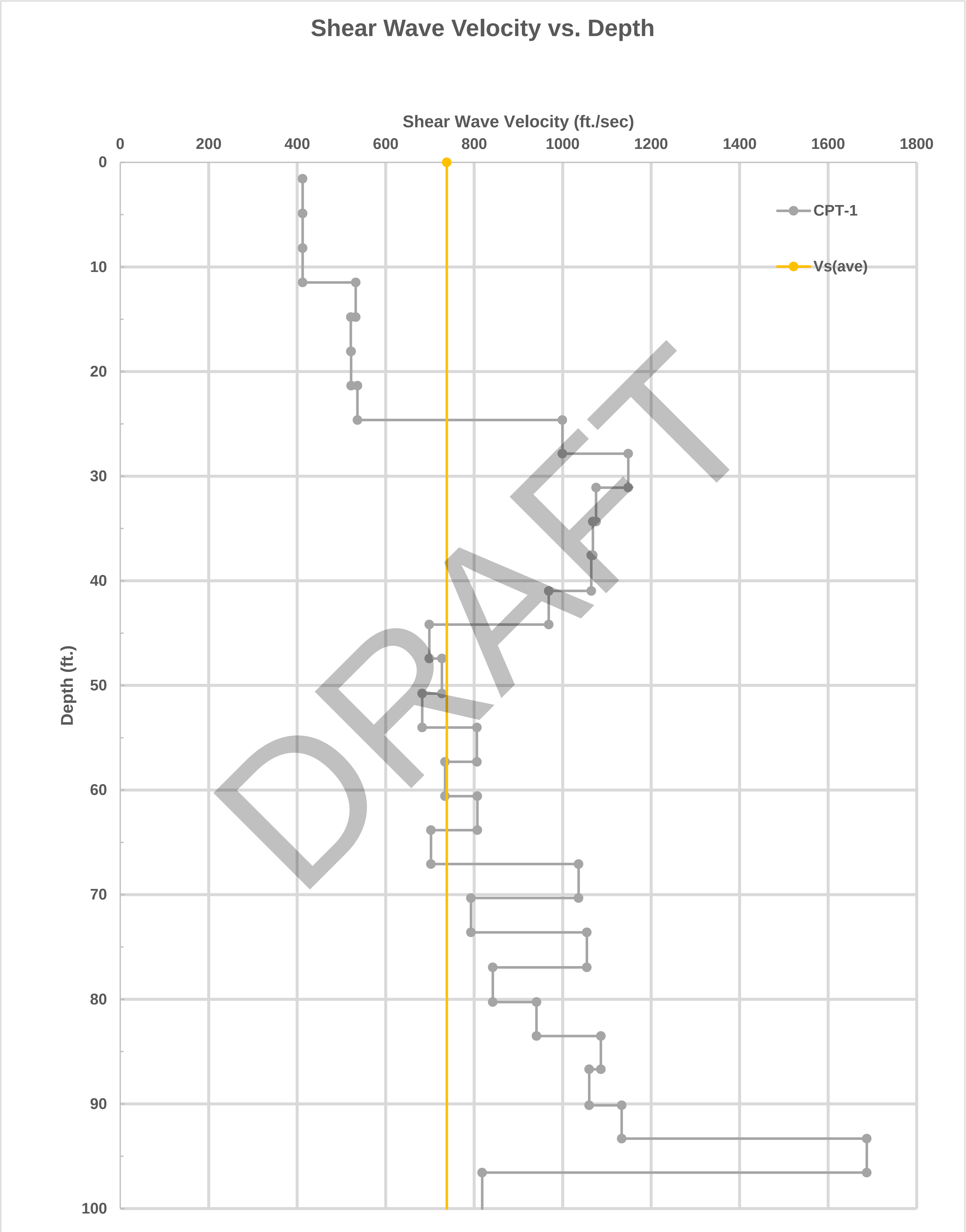
I-55 Ramp/SR-14 Bridge Replacement and Interchange Improvement
Memphis, Tennessee

AERIAL PHOTOGRAPH OF SITE AND BORING LOCATIONS

Project Number J042144.01

FIGURE 2

Figure 3
Shear Wave Velocity Profile



Appendix C
BORING AND SOUNDING INFORMATION

Surface Elevation: N/A

Datum NAVD 88

Completion Date: 3/28/23

DEPTH IN FEET	DESCRIPTION OF MATERIAL
	Topsoil: 3 inches
	Medium stiff, brown, LEAN CLAY - CL
5	Medium stiff, gray, FAT CLAY - CH
	Stiff to medium stiff, gray, LEAN CLAY - (CL)
10	94% passing No. 200 sieve
15	Very loose, gray, CLAYEY SAND - SC
	Very stiff, brown and gray, LEAN CLAY - (CL)
	87% passing No. 200 sieve
20	Medium dense to dense, brown SAND, some gravel - (SP)
	3% passing No. 200 sieve
25	
30	pH = 7.48
35	resistivity = 7,233.30 ohms-cm
40	
45	trace clay
50	Medium dense to dense, gray to tan and yellow SAND, little clay - (SP-SC)
	12% passing No. 200 sieve
55	pH = 6.98
	resistivity = 7,615.20 ohms-cm
60	
65	
70	Very stiff, gray, sandy, LEAN CLAY - CL
75	
80	
85	
90	Dense, gray, CLAYEY SAND - SC
95	
100	Hard, gray, FAT CLAY, trace sand - CH
	Boring terminated at 100 feet.

Groundwater symbol ∇ at 16 feet in the graphic log column.

DRY UNIT WEIGHT (pcf) / SPT BLOW COUNTS / CORE RECOVERY/RQD	SAMPLES
4-4-4	SS1
2-2-4	SS2
104	ST3
2-2-4	SS4
3-2-2	SS5
107	ST6
6-9-11	SS7
10-12-14	SS8
10-12-12	SS9
7-16-20	SS10
8-16-19	SS11
9-11-12	SS12
6-7-10	SS13
10-16-20	SS14
9-9-15	SS15
6-8-13	SS16
6-8-13	SS17
12-20-21	SS18
10-14-18	SS19

SHEAR STRENGTH, tsf

Δ - UU/2 ○ - QU/2 ▯ - SV

0.5 1.0 1.5 2.0 2.5

STANDARD PENETRATION RESISTANCE

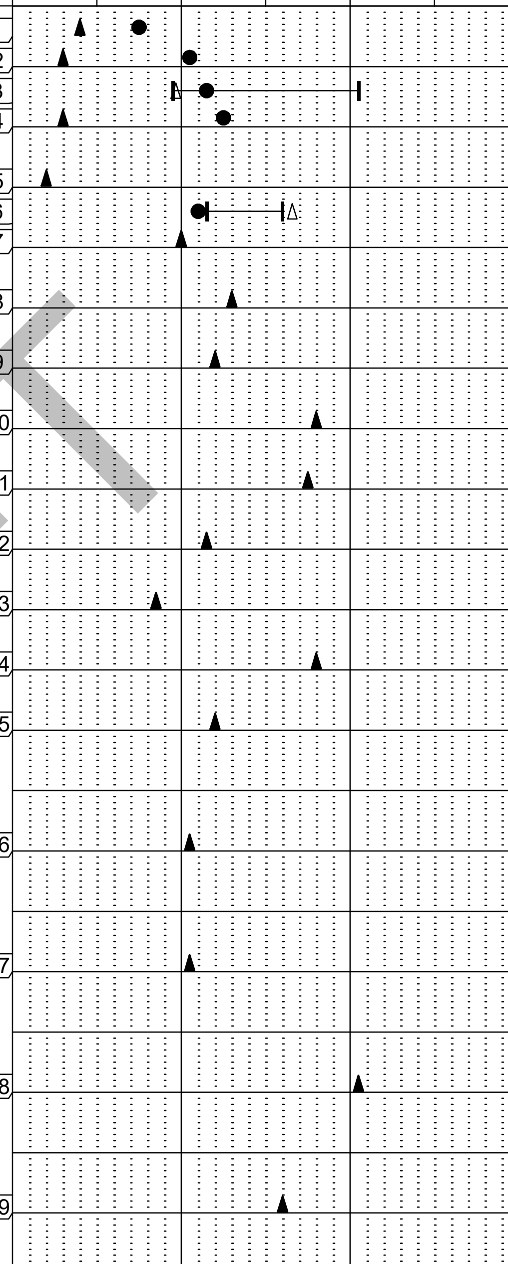
(ASTM D 1586)

▲ N-VALUE (BLOWS PER FOOT)

WATER CONTENT, %

PL ⊢——●——⊣ LL

10 20 30 40 50



GROUNDWATER DATA

ENCOUNTERED AT 16 FEET ∇

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM

WASHBORING FROM 20 FEET

JCG DRILLER EER LOGGER

Diedrich D-50 DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 93 %

REMARKS:

Drawn by: LLP	Checked by: SAS	App'vd. by: ABM
Date: 3/31/23	Date: 7/16/23	Date: 7/20/23

GEOTECHNOLOGY

A UES Company

I-55/SR-14 Bridge Replacement and Interchange Improvement Memphis, Tennessee

LOG OF BORING: B-1

Project No. J042144.01

NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

LOG OF BORING 2020 JDM J042144.01.GPJ GTINC 0638301.GPJ 7/21/23

Surface Elevation: N/A

Datum NAVD 88

Completion Date: 4/3/23

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Topsoil: 4 inches	1-2-3	SS1
5	Soft to very stiff, brown to gray, silty, LEAN CLAY - (CL)	3-2-1	SS2
	97% passing No. 200 sieve	106	ST3
10	91% passing No. 200	2-3-4	SS4
		103	ST5
15	96% passing No. 200 sieve	3-4-5	SS6
		99	ST7
20	pH = 7.53	2-3-4	SS8
25		2-3-4	SS9
30	resistivity = 1,468.32 ohms-cm	2-2-3	SS10
35	Dense, orange GRAVEL, some sand - GP pH = 7.13	9-15-16	SS11
40		4-15-20	SS12
45	resistivity = 6,840 ohms-cm	8-29-21	SS13
50	Medium dense, orange SAND, some gravel - SP	24-17-13	SS14
55	Medium dense to dense, gray to tan and orange SAND, trace clay - (SP-SC)	6-7-10	SS15
60	8% passing No. 200 sieve	10-6-8	SS16
65		7-8-8	SS17
70		9-8-8	SS18
75		11-11-13	SS19
80	8% passing No. 200 sieve	17-14-19	SS20
85		12-6-7	SS21
90	Stiff, gray, FAT CLAY - CH	4-4-6	SS22
95	Hard, gray, sand, FAT CLAY - CH	7-19-21	SS23
100	Very stiff, gray, FAT CLAY - CH Boring terminated at 100 feet.	6-7-9	SS24

GRAPHIC LOG

SHEAR STRENGTH, tsf

Δ - UU/2 ○ - QU/2 ▯ - SV

0.5 1.0 1.5 2.0 2.5

STANDARD PENETRATION RESISTANCE

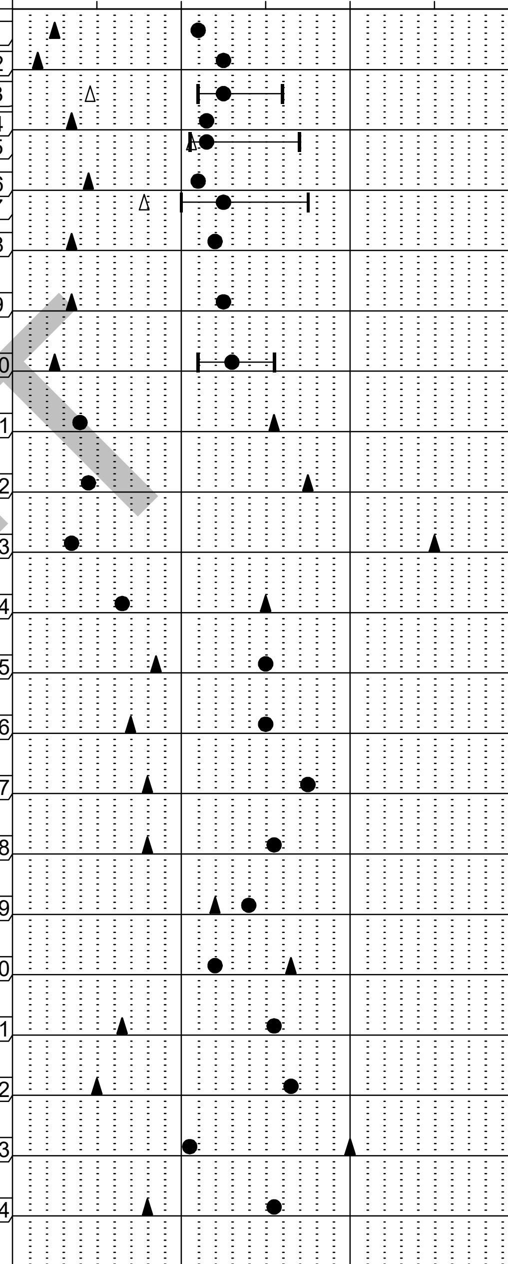
(ASTM D 1586)

▲ N-VALUE (BLOWS PER FOOT)

WATER CONTENT, %

PL LL

10 20 30 40 50



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM

WASHBORING FROM 30 FEET

JCG DRILLER RSP LOGGER

Diedrich D-50 DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 93 %

REMARKS:

Drawn by: RSP	Checked by: SAS	App'vd. by: ABM
Date: 4/6/23	Date: 7/16/23	Date: 7/20/23

GEOTECHNOLOGY
A UES Company

I-55/SR-14 Bridge Replacement and Interchange Improvement Memphis, Tennessee

LOG OF BORING: B-2

Project No. J042144.01

Surface Elevation: N/A

Datum NAVD 88

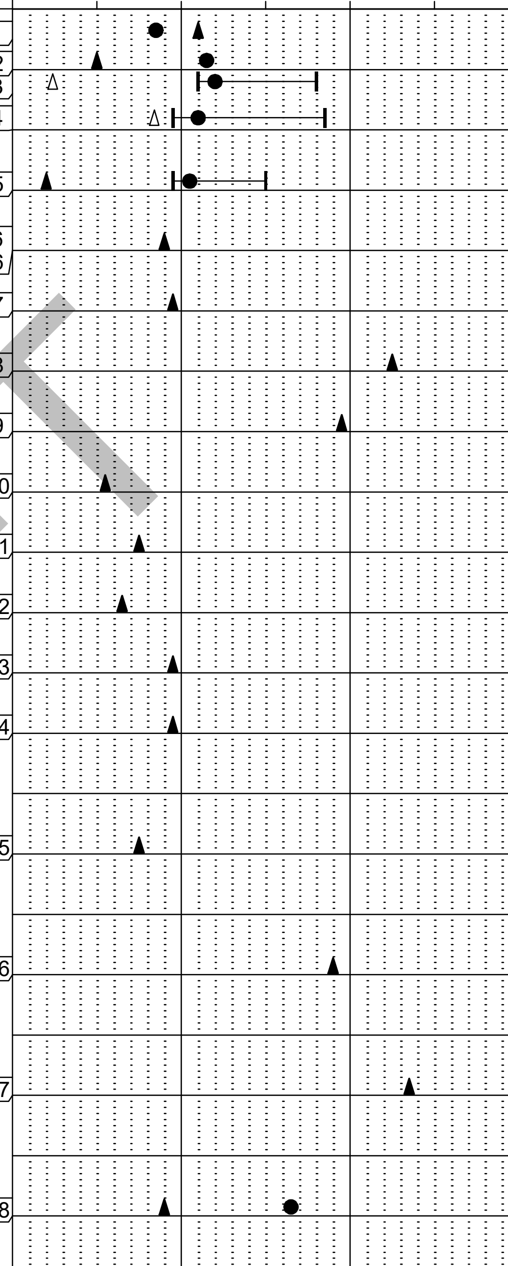
Completion Date: 4/4/23

SHEAR STRENGTH, tsf: Δ - UU/2, ○ - QU/2, ▯ - SV (0.5, 1.0, 1.5, 2.0, 2.5)

STANDARD PENETRATION RESISTANCE (ASTM D 1586): ▲ N-VALUE (BLOWS PER FOOT)

WATER CONTENT, %: PL |—●—| LL (10, 20, 30, 40, 50)

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) / SPT BLOW COUNTS / CORE RECOVERY/RQD	SAMPLES
0	Topsoil: 4 inches		
	Very stiff to stiff, brown to gray, silty, LEAN CLAY - (CL)	9-11-11	SS1
		4-4-6	SS2
5	93% passing No. 200 sieve	102	ST3
	87% passing No. 200 sieve	105	ST4
10			
15	Soft, brown and gray, sandy, LEAN CLAY - (CL) 52% passing No. 200 sieve	2-2-2	SS5
20	Medium dense, tan SAND, trace gravel - (SP) 2% passing No. 200 sieve	4-8-10	ST6 SS6
25	Medium dense to dense, orange SAND, some gravel - SP	12-11-8	SS7
30	resistivity = 11,400 ohms-cm	3-16-29	SS8
35		7-18-21	SS9
40	Medium dense, orange GRAVEL - (GP) 5% passing No. 200 sieve	5-5-6	SS10
45	Medium dense, gray SAND - SP pH = 6.42	6-10-5	SS11
50	Medium dense, orange GRAVEL - (GP) 1% passing No. 200 sieve	8-8-5	SS12
55		8-10-9	SS13
60	Medium dense to dense, gray to tan SAND, trace clay - (SP-SC) pH = 7.05	9-10-9	SS14
65			
70	8% passing No. 200 sieve	7-7-8	SS15
75			
80		14-17-21	SS16
85			
90		12-20-27	SS17
95			
100	Very stiff, gray, FAT CLAY - CH	7-8-10	SS18
	Boring terminated at 100 feet.		



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 18 FEET ∇

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM

WASHBORING FROM 20 FEET

JCG DRILLER RSP LOGGER

Diedrich D-50 DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 93 %

REMARKS:

Drawn by: RSP	Checked by: SAS	App'vd. by: ABM
Date: 4/6/23	Date: 7/16/23	Date: 7/20/23

GEOTECHNOLOGY
A UES Company

I-55/SR-14 Bridge Replacement and Interchange Improvement Memphis, Tennessee

LOG OF BORING: B-3

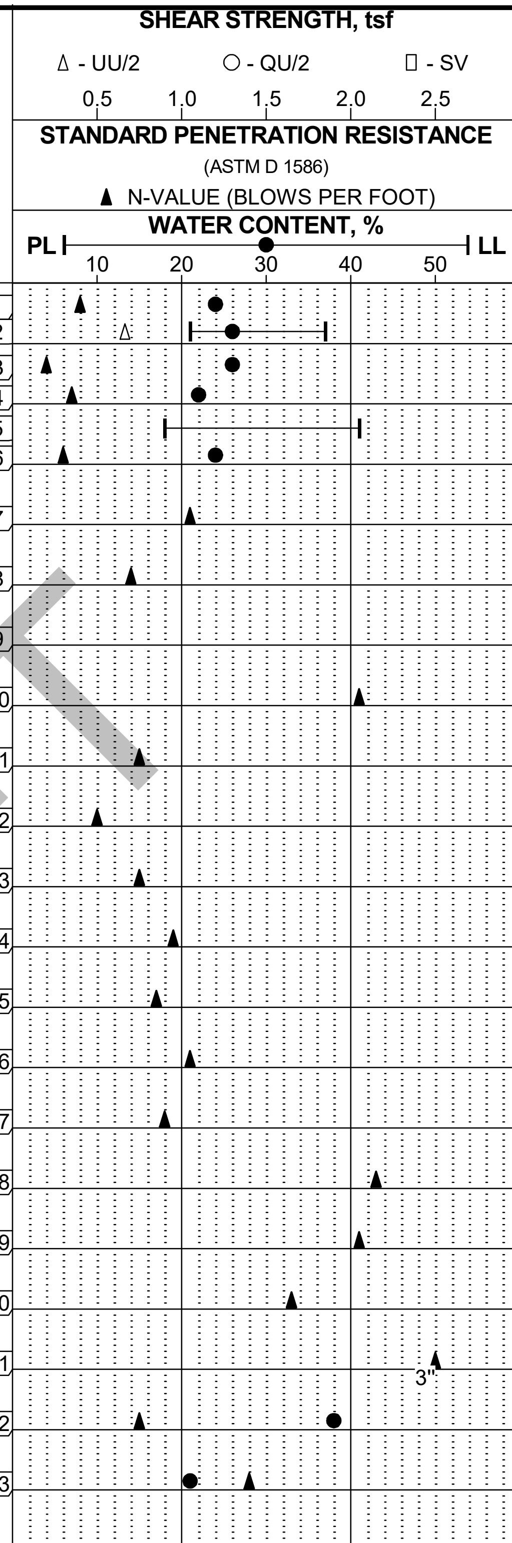
Project No. J042144.01

Surface Elevation: N/A

Datum NAVD 88

Completion Date: 3/31/23

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) / SPT BLOW COUNTS / CORE RECOVERY/RQD	SAMPLES
	Topsoil: 6 inches		3-4-4	SS1
5	Stiff to soft, brown to gray, silty, LEAN CLAY - (CL) 95% passing No. 200 sieve		104	ST2
			1-2-2	SS3
10		∇	3-3-4	SS4
	90% passing No. 200 sieve			ST5
15	Medium stiff, brown, sandy, LEAN CLAY - CL		1-3-3	SS6
20	Medium dense to very dense, brown SAND, little gravel, trace silt - (SP-SM) pH = 6.20		3-9-12	SS7
25	9% passing No. 200 sieve		5-7-7	SS8
30	resistivity = 22,230 ohms-cm		14-24-36	SS9
35			15-23-18	SS10
40	Medium dense, gray to orange, SILTY SAND - SM pH = 5.28		5-8-7	SS11
45	Loose, gray to orange SAND, trace silt (SP-SM) 8% passing No. 200 sieve		5-5-5	SS12
50	Medium dense to very dense, gray to orange, SILTY SAND - (SM) 14% passing No. 200 sieve		5-6-9	SS13
55	resistivity = 912 ohms-cm		7-9-10	SS14
60			6-7-10	SS15
65			6-8-13	SS16
70			6-8-10	SS17
75			13-20-23	SS18
80			13-18-23	SS19
85			6-12-21	SS20
90			16-50/3"	SS21
95	Stiff to very stiff, gray, FAT CLAY - CH		5-7-8	SS22
100			8-12-16	SS23
	Boring terminated at 100 feet.			



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

DRAFT

GROUNDWATER DATA

ENCOUNTERED AT 11 FEET ∇

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM

WASHBORING FROM 20 FEET

JCG DRILLER EER LOGGER

Diedrich D-50 DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 93 %

REMARKS:

Drawn by: RSP	Checked by: SAS	App'vd. by: ABM
Date: 4/6/23	Date: 7/16/23	Date: 7/20/23

GEOTECHNOLOGY
A UES Company

I-55/SR-14 Bridge Replacement and Interchange Improvement Memphis, Tennessee

LOG OF BORING: B-4

Project No. J042144.01

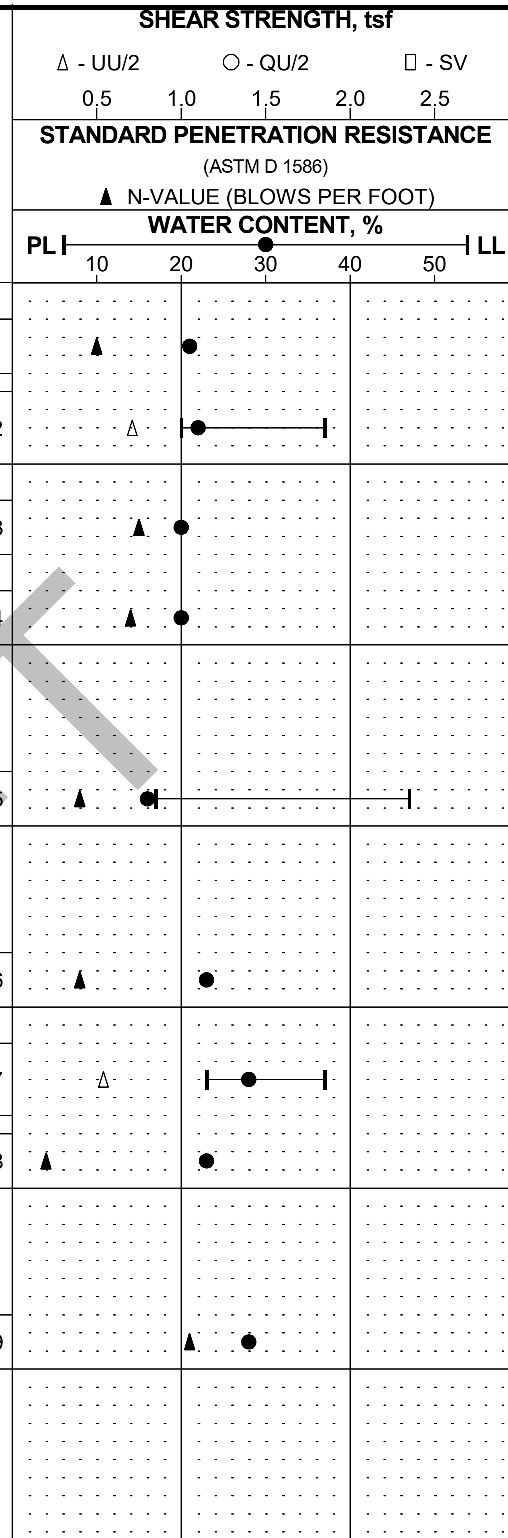
LOG OF BORING 2020 JDM J042144.01.GPJ GTINC 0638301.GPJ 7/21/23

Surface Elevation: N/A

Datum: NAVD 88

Completion Date: 4/4/23

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Topsoil: 4 inches			
	Stiff to soft, brown to gray, silty, LEAN CLAY - (CL)		5-6-4	SS1
	98% passing No. 200 sieve		111	ST2
5				
			4-7-8	SS3
			4-7-7	SS4
10				
	99% passing No. 200 sieve		2-4-4	SS5
15				
			3-3-5	SS6
20				
	97% passing No. 200 sieve		100	ST7
			1-2-2	SS8
25				
	Medium dense, brown, SILTY SAND, some gravel - SM	▽	1-3-18	SS9
30	Boring terminated at 30 feet.			



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 28.5 FEET ▽

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM

WASHBORING FROM __ FEET

JCG DRILLER RSP LOGGER

Diedrich D-50 DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 93 %

REMARKS:

Drawn by: RSP	Checked by: SAS	App'vd. by: ABM
Date: 4/6/23	Date: 7/16/23	Date: 7/20/23

GEOTECHNOLOGY
A UES Company

I-55/SR-14 Bridge Replacement and Interchange Improvement Memphis, Tennessee

LOG OF BORING: R-1

Project No. J042144.01

Surface Elevation: N/A

Datum: NAVD 88

Completion Date: 3/28/23

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) / SPT BLOW COUNTS / CORE RECOVERY/RQD	SAMPLES
	Topsoil: 4 inches			
	Stiff, brown, clayey SILT - ML		6-7-6	SS1
	Medium stiff to very stiff, brown and gray to gray, LEAN CLAY - (CL)		6-8-6	SS2
5				
			4-8-12	SS3
	trace gravel		6-9-10	SS4
10				
	93% passing No. 200 sieve		103	ST5
15				
			2-3-3	SS6
20				
	87% passing No. 200 sieve		98	ST7
	Medium stiff, brown, sandy, LEAN CLAY - CL	⊻	2-1-4	SS8
25				
	Medium dense, brown SAND, trace gravel - (SP) 1% passing No. 200 sieve		4-7-6	SS9
30	Boring terminated at 30 feet.			

SHEAR STRENGTH, tsf

∆ - UU/2 ○ - QU/2 ▯ - SV

0.5 1.0 1.5 2.0 2.5

STANDARD PENETRATION RESISTANCE

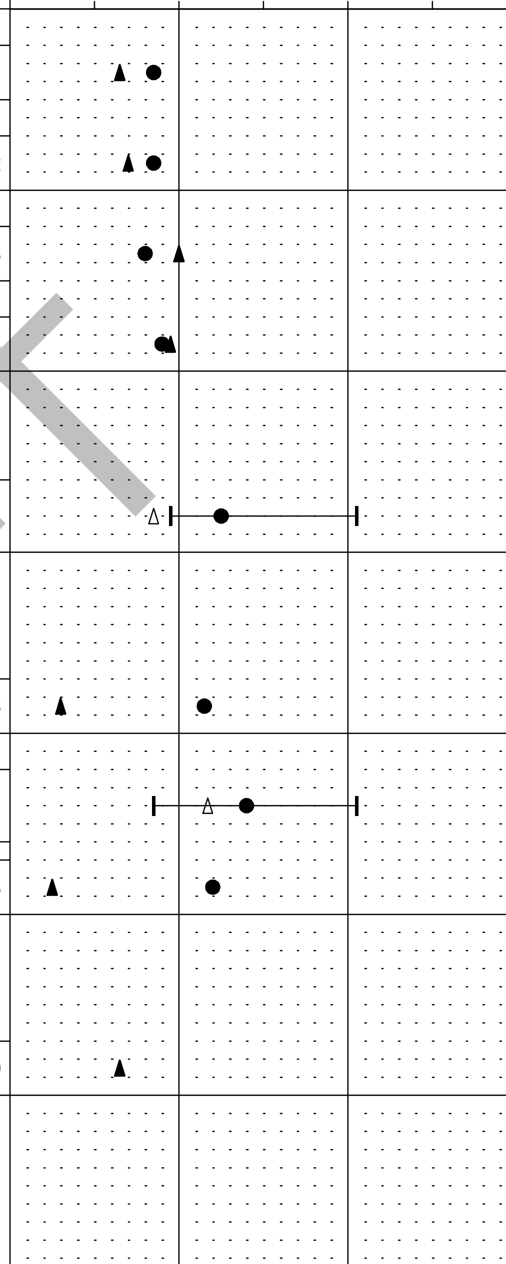
(ASTM D 1586)

▲ N-VALUE (BLOWS PER FOOT)

WATER CONTENT, %

PL ⊢ ● ⊣ LL

10 20 30 40 50



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 24 FEET ⊻

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM

WASHBORING FROM __ FEET

JCG DRILLER EER LOGGER

Diedrich D-50 DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 93 %

REMARKS:

Drawn by: LLP	Checked by: SAS	App'vd. by: ABM
Date: 3/31/23	Date: 7/16/23	Date: 7/20/23

GEOTECHNOLOGY

A UES Company

I-55/SR-14 Bridge Replacement and Interchange Improvement Memphis, Tennessee

LOG OF BORING: R-2

Project No. J042144.01

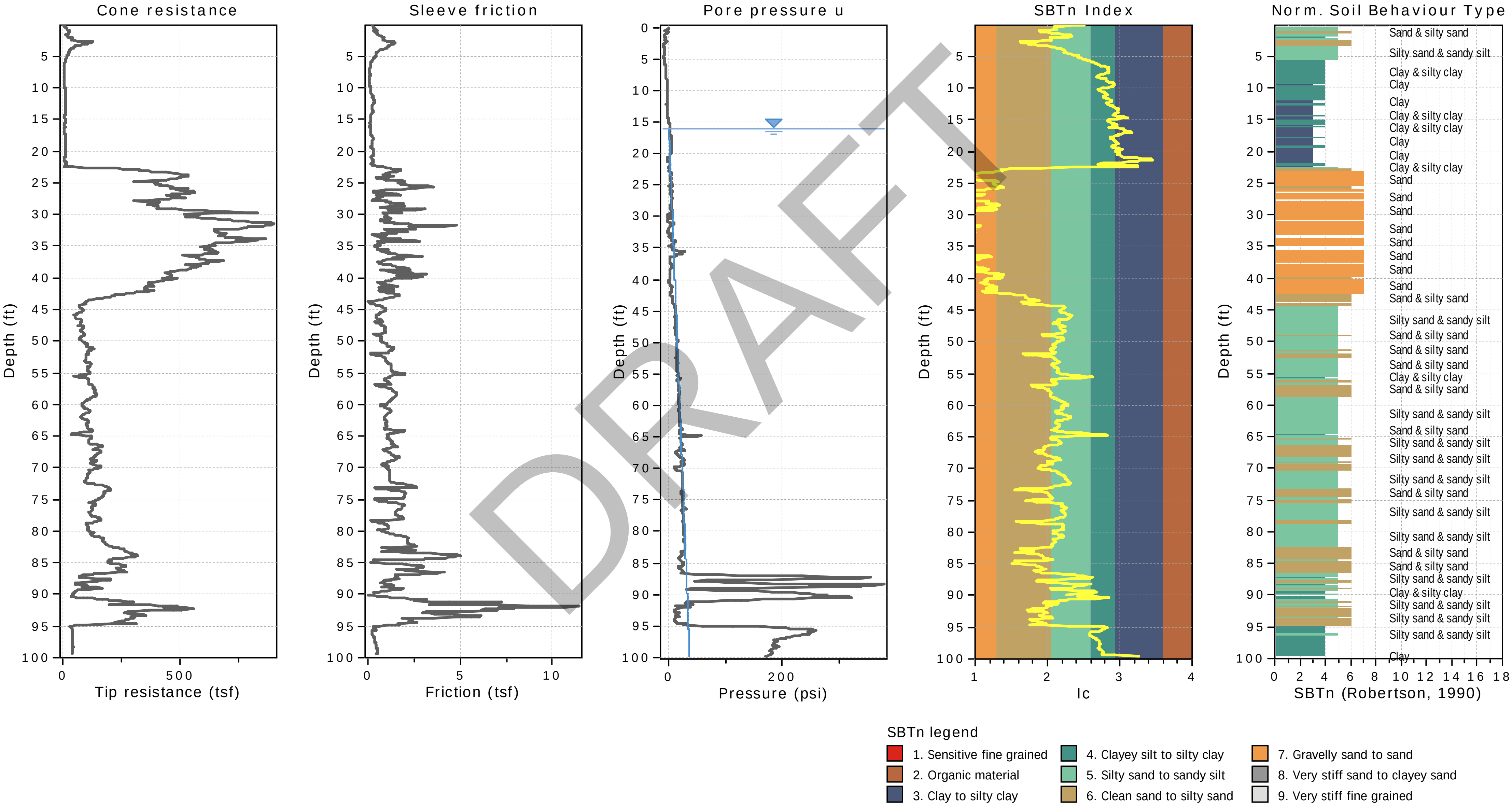
Geotechnology, LLC
3312 Winbrook Drive
Memphis, Tennessee
http://www.geotechnology.com

Project: I-55 & South 3rd Street (SR-14) Interchange

Location: Memphis, Tennessee

CPT-1

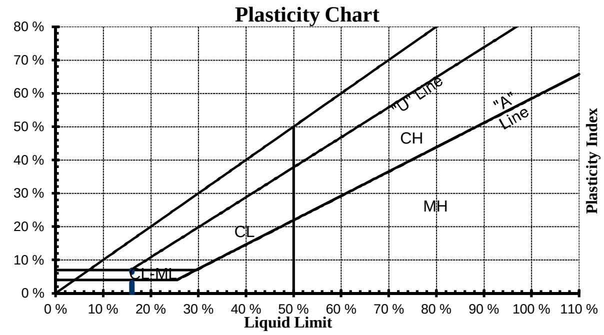
Total depth: 99.81 ft, Date: 6/1/2023



BORING LOG: TERMS AND SYMBOLS

LEGEND

CS	Continuous Sampler
GB	Grab Sample
NQ	NQ Rock Core
PST	Three-Inch Diameter Piston Tube Sample
SS	Split-Spoon Sample (Standard Penetration Test)
ST	Three-Inch Diameter Shelby Tube Sample
*	Sample Not Recovered
PL	Plastic Limit (ASTM D4318)
LL	Liquid Limit (ASTM D4318)
SV	Shear Strength from Field Vane (ASTM D2573)
UU	Shear Strength from Unconsolidated-Undrained Triaxial Compression Test (ASTM D2850)
QU	Shear Strength from Unconfined Compression Test (ASTM D2166)



SOIL GRAIN SIZE

US STANDARD SIEVE

	12"		3"		3/4"		4		10		40		200		

BOULDERS	COBBLES	GRAVEL		SAND			SILT	CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE		

	300		76.2		19.1		4.76		2.00		0.42		0.074		0.005	

SOIL GRAIN SIZE IN MILLIMETERS

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions			***Symbol***	***Description***
Coarse-Grained Soils (More than 50% Larger than No. 200 Sieve Size)	Gravel and Gravelly Soil	Clean Gravels Little or no Fines	GW	Well-Graded Gravel, Gravel- Sand Mixture
			GP	Poorly-Graded Gravel, Gravel-Sand Mixture
		Gravels with Appreciable Fines	GM	Silty Gravel, Gravel-Sand-Silt Mixture
			GC	Clayey-Gravel, Gravel-Sand-Clay Mixture
	Sand and Sandy Soils	Clean Sands Little or no Fines	SW	Well-Graded Sand, Gravelly Sand
			SP	Poorly-Graded Sand, Gravelly Sand
		Sands with Appreciable Fines	SM	Silty Sand, Sand-Silt Mixture
			SC	Clayey-Sand, Sand-Clay Mixture
Fine-Grained Soils (More than 50% Smaller than No. 200 Sieve Size)	Silts and Clays	Liquid Limit Less Than 50	ML	Silt, Sandy Silt, Clayey Silt, Slight Plasticity
			CL	Lean Clay, Sandy Clay, Silty Clay, Low to Medium Plasticity
			OL	Organic Silts or Lean Clays, Low Plasticity
	Silts and Clays	Liquid Limit Greater Than 50	MH	Silt, High Plasticity
			CH	Fat Clay, High Plasticity
			OH	Organic Clay, Medium to High Plasticity
	Highly Organic Soils		PT	Peat, Humus, Swamp Soil

STRENGTH OF COHESIVE SOILS

Consistency	***Undrained Shear Strength (tsf)***	***Unconfined Comp. Strength (tsf)***
Very Soft	less than 0.125	less then 0.25
Soft	0.125 to 0.25	0.25 to 0.5
Medium Stiff	0.25 to 0.5	0.5 to 1.0
Stiff	0.5 to 1.0	1.0 to 2.0
Very Stiff	1.0 to 2.0	2.0 to 3.0
Hard	greater than 2.0	greater than 4.0

DENSITY OF GRANULAR SOILS

Descriptive Term	***Approximate N_{60}-Value Range***
Very Loose	0 to 4
Loose	5 to 10
Medium Dense	11 to 30
Dense	31 to 50
Very Dense	>50

N-Value (Blow Count) is the last two, 6-inch drive increments (i.e. 4/7/9, N = 7 + 9 = 16). Values are shown as a summation on the grid plot and shown in the Unit Dry Weight/SPT column.

RELATIVE COMPOSITION

Trace	0 to 10%
Little	10 to 20%
Some	20 to 35%
And	35 to 50%

OTHER TERMS

- Layer - Inclusion greater than 3 inches thick.
- Seam - Inclusion 1/8-inch to 3 inches thick
- Parting - Inclusion less than 1/8-inch thick
- Pocket - Inclusion of material that is smaller than sample diameter

GEOTECHNOLOGY
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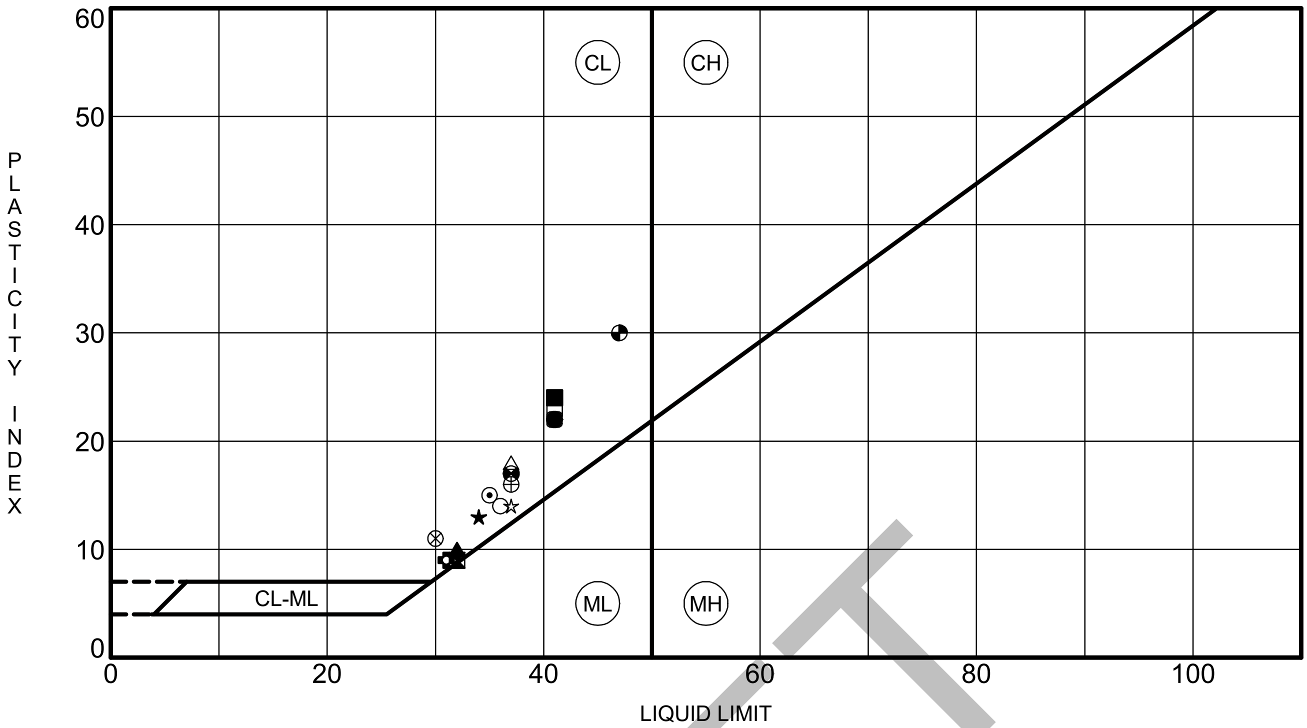
Relative composition and Unified Soil Classification System (USCS) designations are based on visual descriptions and are approximate only. If laboratory tests were performed to classify the soil, the USCS designation is shown in parenthesis.

Appendix D
LABORATORY TEST DATA

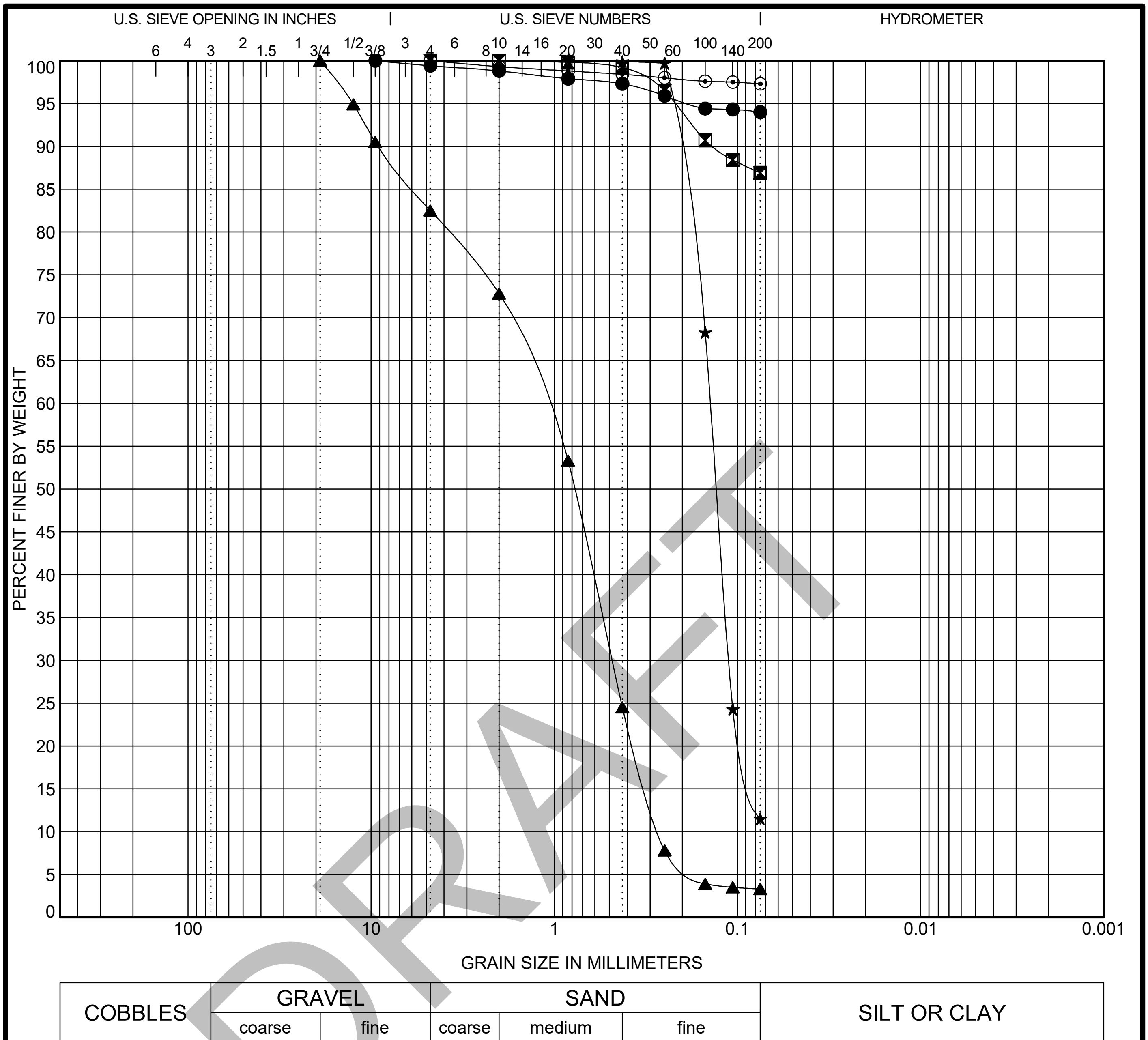


	Specimen Identification		LL	PL	PI	Fines	Classification
●	B-1	6.0	41	19	22	94	LEAN CLAY(CL), A-7-6 (22)
⧗	B-1	16.0	32	23	9	87	LEAN CLAY(CL), A-4 (8)
▲	B-2	6.0	32	22	10	97	LEAN CLAY(CL), A-4 (10)
★	B-2	10.0	34	21	13	91	LEAN CLAY(CL), A-6 (12)
⊙	B-2	15.0	35	20	15	96	LEAN CLAY(CL), A-6 (15)
✿	B-2	28.5	31	22	9		LEAN CLAY(CL), A-4 (8)
○	B-3	5.0	36	22	14	93	LEAN CLAY(CL), A-6 (14)
△	B-3	8.0	37	19	18	87	LEAN CLAY(CL), A-6 (15)
⊗	B-3	13.5	30	19	11	52	SANDY LEAN CLAY(CL), A-6 (3)
⊕	B-4	3.0	37	21	16	95	LEAN CLAY(CL), A-6 (16)
□	B-4	12.0	41	18	23	90	LEAN CLAY(CL), A-7-6 (21)
◉	R-1	3.0	37	20	17	98	LEAN CLAY(CL), A-6 (18)
◕	R-1	13.5	47	17	30	99	LEAN CLAY(CL), A-7-6 (32)
☆	R-1	21.0	37	23	14	97	LEAN CLAY(CL), A-6 (15)
✣	R-2	13.0	41	19	22	93	LEAN CLAY(CL), A-7-6 (21)
■	R-2	21.0	41	17	24	87	LEAN CLAY(CL), A-7-6 (21)

GEOTECHNOLOGY
A UES Company

ATTERBERG LIMITS RESULTS

I-55/SR-14 Bridge Replacement
and Interchange Improvement
Memphis, Tennessee
J042144.01



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	B-1	6.0	**LEAN CLAY(CL), A-7-6 (22)**	41	19	22		
⊠	B-1	16.0	**LEAN CLAY(CL), A-4 (8)**	32	23	9		
▲	B-1	18.5	**POORLY GRADED SAND(SP) with GRAVEL, A-1-b (0)**				0.77	4.22
★	B-1	48.5	**POORLY GRADED SAND with CLAY(SP-SC), A-2-4 (0)**				1.22	1.95
⊙	B-2	6.0	**LEAN CLAY(CL), A-4 (10)**	32	22	10		

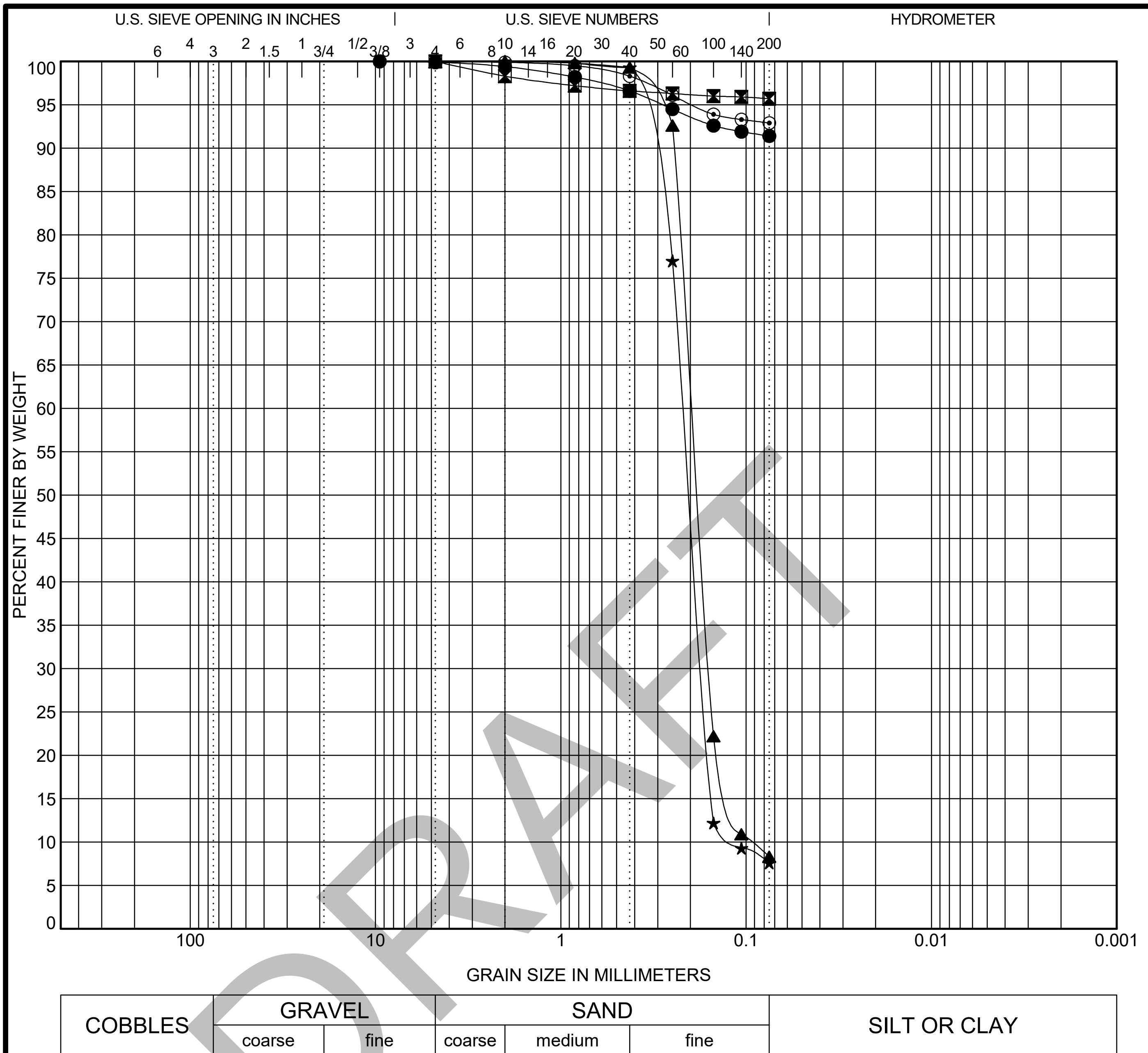
Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-1	6.0	9.5				0.6	5.4	94.0	
⊠	B-1	16.0	4.75				0.0	13.1	86.9	
▲	B-1	18.5	19	1.132	0.484	0.268	17.5	79.2	3.3	
★	B-1	48.5	0.84	0.14	0.111		0.0	88.5	11.5	
⊙	B-2	6.0	4.75				0.0	2.7	97.3	

GEOTECHNOLOGY
A UES Company

GRAIN SIZE DISTRIBUTION

I-55/SR-14 Bridge Replacement and Interchange Improvement
Memphis, Tennessee
J042144.01

US GRAIN SIZE J042144.01.GPJ US LAB.GDT 7/21/23



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	B-2	10.0	**LEAN CLAY(CL), A-6 (12)**	34	21	13		
⧗	B-2	15.0	**LEAN CLAY(CL), A-6 (15)**	35	20	15		
▲	B-2	58.5	**POORLY GRADED SAND with CLAY(SP-SC), A-3 (0)**				1.36	2.10
★	B-2	78.5	**POORLY GRADED SAND with SILT(SP-SM), A-3 (0)**				1.18	1.90
⊙	B-3	5.0	**LEAN CLAY(CL), A-6 (14)**	36	22	14		

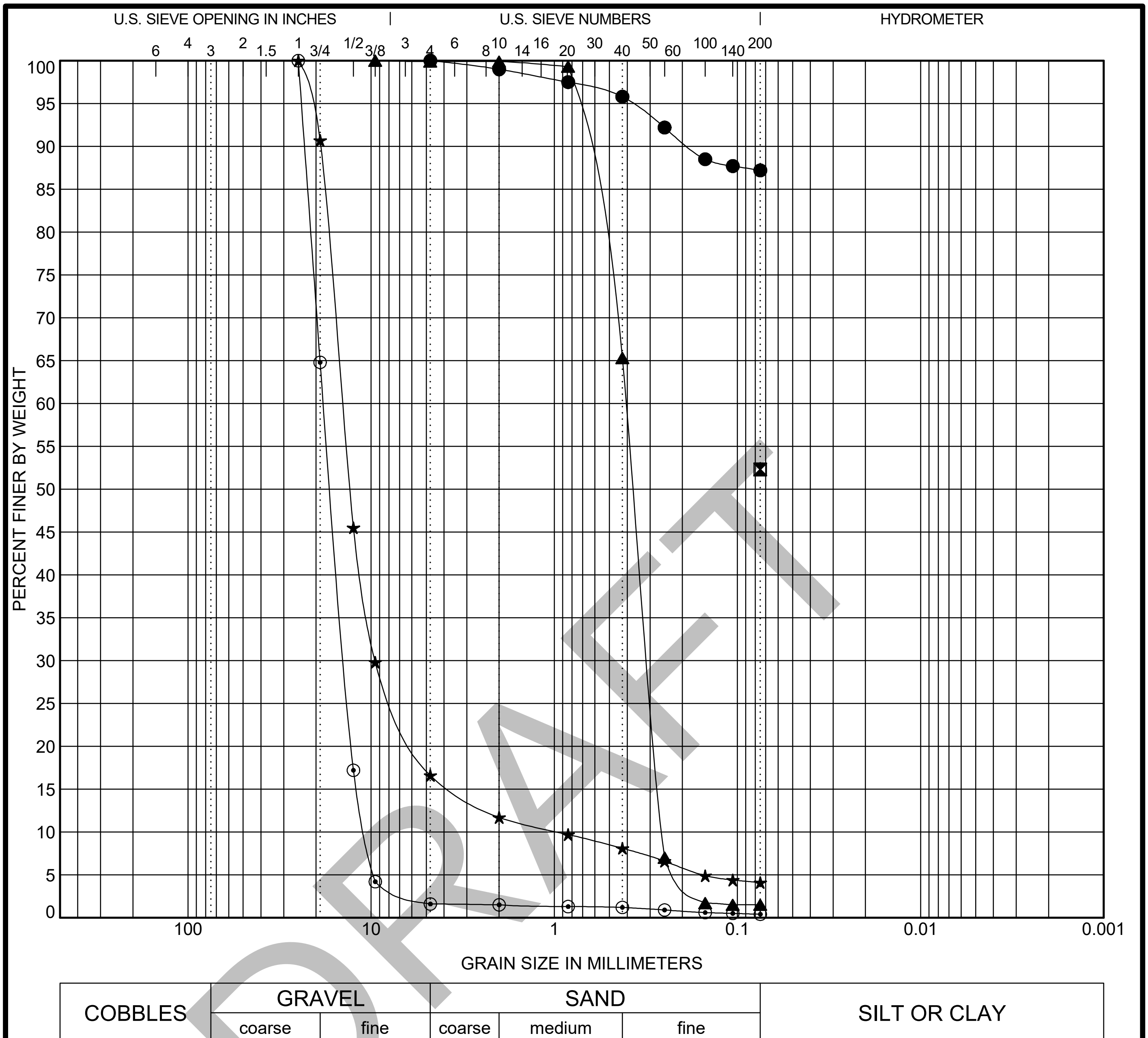
Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-2	10.0	9.5				0.1	8.5	91.4	
⧗	B-2	15.0	4.75				0.0	4.3	95.7	
▲	B-2	58.5	2	0.197	0.159	0.094	0.0	91.7	8.3	
★	B-2	78.5	2	0.219	0.173	0.115	0.0	92.4	7.6	
⊙	B-3	5.0	4.75				0.0	7.1	92.9	

GEOTECHNOLOGY
A UES Company

GRAIN SIZE DISTRIBUTION

I-55/SR-14 Bridge Replacement and Interchange Improvement
Memphis, Tennessee
J042144.01

US_GRAIN_SIZE_J042144.01.GPJ_US_LAB.GDT 7/21/23



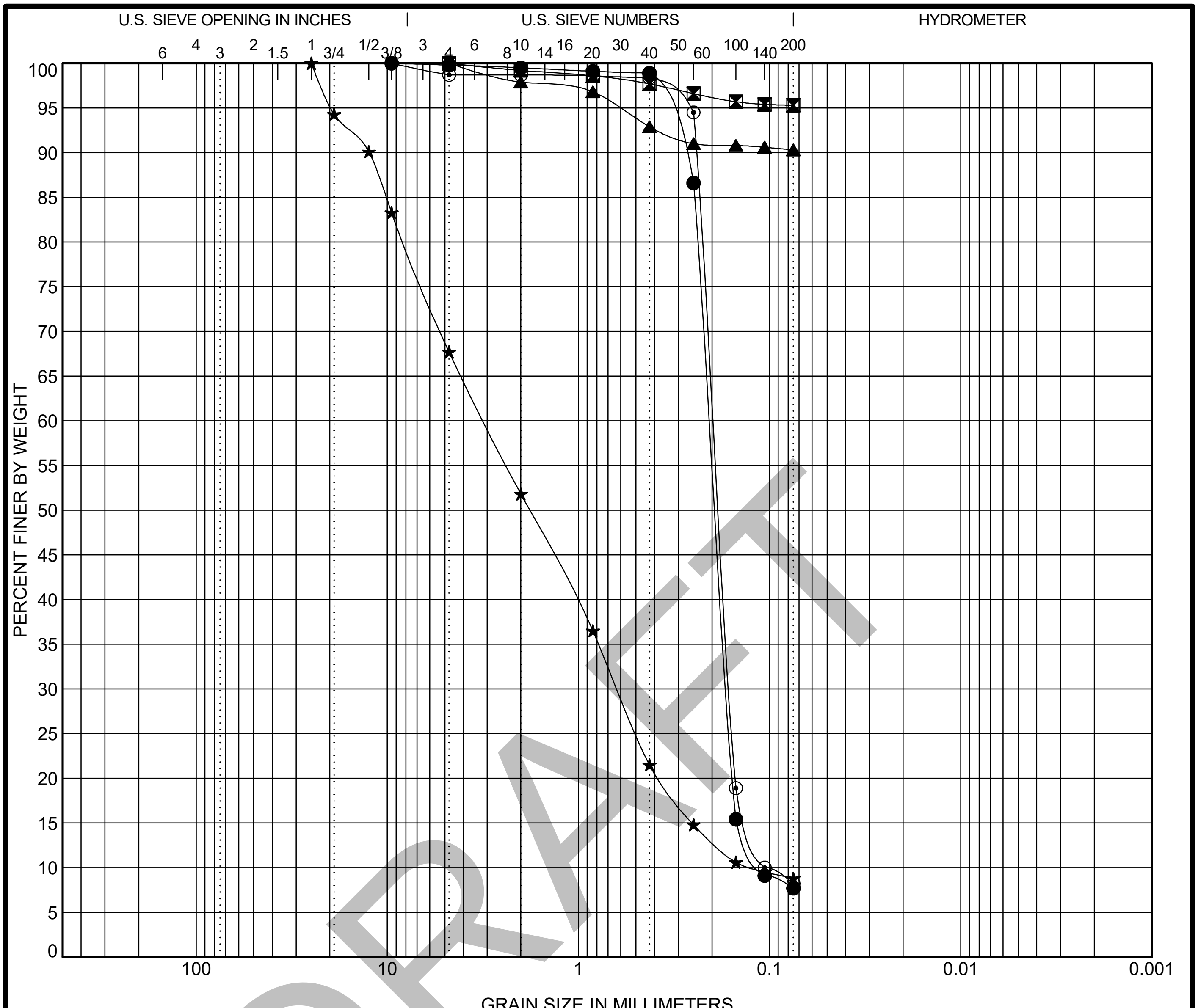
Specimen Identification		Classification	LL	PL	PI	Cc	Cu
● B-3	8.0	**LEAN CLAY(CL), A-6 (15)**	37	19	18		
⊠ B-3	13.5	**SANDY LEAN CLAY(CL), A-6 (3)**	30	19	11		
▲ B-3	18.0	**POORLY GRADED SAND(SP), A-3 (0)**				0.91	1.58
★ B-3	38.5	**POORLY GRADED GRAVEL(GP), A-1-a (0)**				6.64	14.94
⊙ B-3	48.5	**POORLY GRADED GRAVEL(GP), A-1-a (0)**				1.00	1.70

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● B-3	8.0	4.75				0.0	12.8	87.2	
⊠ B-3	13.5	0.075				0.0	0.0	52.3	
▲ B-3	18.0	9.5	0.405	0.308	0.257	0.1	98.4	1.5	
★ B-3	38.5	25	14.297	9.533	0.957	83.4	12.5	4.1	
⊙ B-3	48.5	25	18.214	13.99	10.737	98.4	1.2	0.4	

GEOTECHNOLOGY
A UES Company

GRAIN SIZE DISTRIBUTION

I-55/SR-14 Bridge Replacement and Interchange Improvement
Memphis, Tennessee
J042144.01



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification		Classification	LL	PL	PI	Cc	Cu
● B-3	68.5	POORLY GRADED SAND with CLAY(SP-SC), A-3 (0)				1.21	1.85
⧗ B-4	3.0	LEAN CLAY(CL), A-6 (16)	37	21	16		
▲ B-4	12.0	LEAN CLAY(CL), A-7-6 (21)	41	18	23		
★ B-4	23.5	P GRADED SAND with SILT and GRAVEL(SP-SM), A-1-b (0)				1.01	25.17
⊙ B-4	43.5	POORLY GRADED SAND with SILT(SP-SM), A-3 (0)				1.25	1.87

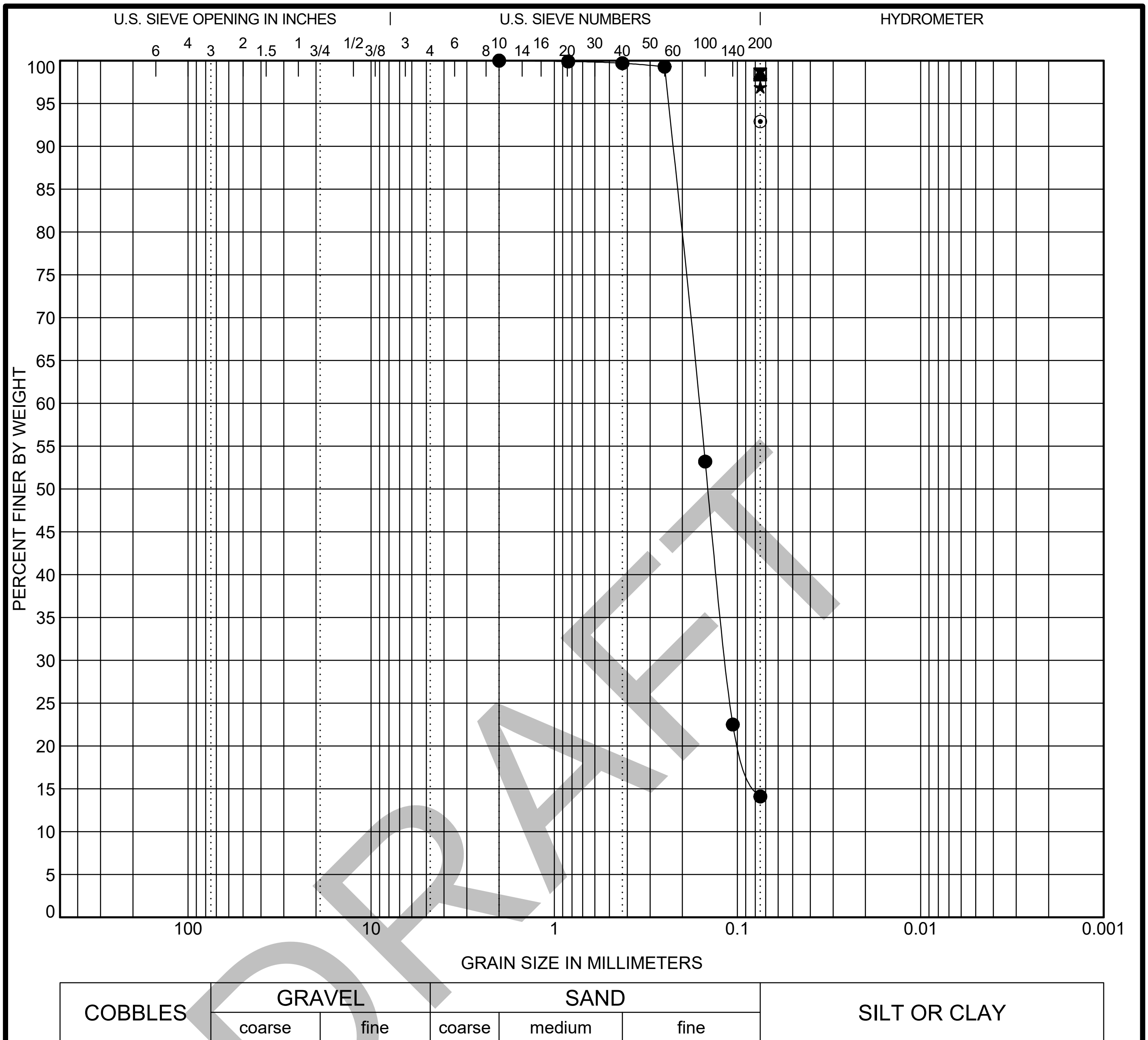
Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● B-3	68.5	9.5	0.207	0.167	0.111	0.2	92.1	7.7	
⧗ B-4	3.0	4.75				0.0	4.7	95.3	
▲ B-4	12.0	4.75				0.0	9.7	90.3	
★ B-4	23.5	25	3.124	0.625	0.124	32.3	58.9	8.8	
⊙ B-4	43.5	9.5	0.198	0.162	0.106	1.3	90.5	8.2	

GEOTECHNOLOGY
A UES Company

GRAIN SIZE DISTRIBUTION

I-55/SR-14 Bridge Replacement and Interchange Improvement
Memphis, Tennessee
J042144.01

US_GRAIN_SIZE_J042144.01.GPJ_US_LAB.GDT 7/21/23



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	B-4	48.5	**SILTY SAND(SM), A-3 (0)**					
⌧	R-1	3.0	**LEAN CLAY(CL), A-6 (18)**	37	20	17		
▲	R-1	13.5	**LEAN CLAY(CL), A-7-6 (32)**	47	17	30		
★	R-1	21.0	**LEAN CLAY(CL), A-6 (15)**	37	23	14		
⊙	R-2	13.0	**LEAN CLAY(CL), A-7-6 (21)**	41	19	22		

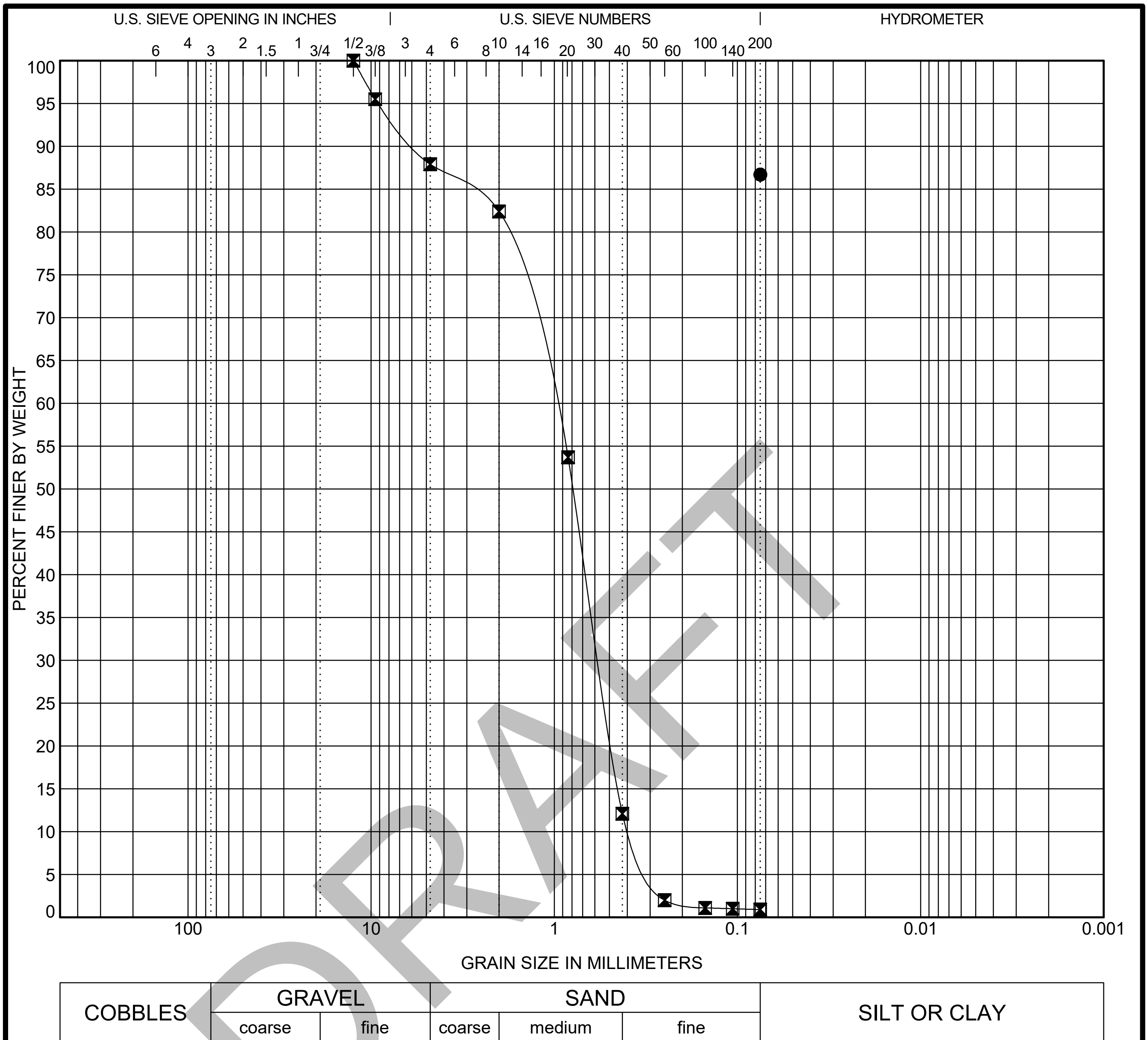
Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-4	48.5	2	0.162	0.115		0.0	85.9	14.1	
⌧	R-1	3.0	0.075				0.0	0.0	98.4	
▲	R-1	13.5	0.075				0.0	0.0	98.5	
★	R-1	21.0	0.075				0.0	0.0	96.9	
⊙	R-2	13.0	0.075				0.0	0.0	92.9	

US GRAIN SIZE J042144.01.GPJ US LAB.GDT 7/21/23

GEOTECHNOLOGY
A UES Company

GRAIN SIZE DISTRIBUTION

I-55/SR-14 Bridge Replacement and Interchange Improvement
Memphis, Tennessee
J042144.01



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	R-2	21.0	**LEAN CLAY(CL), A-7-6 (21)**	41	17	24		
⊠	R-2	28.5	**POORLY GRADED SAND(SP), A-1-b (0)**				0.84	2.67

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	R-2	21.0	0.075				0.0	0.0	86.7	
⊠	R-2	28.5	12.5	1.016	0.57	0.381	12.1	87.0	0.9	

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GRAIN SIZE DISTRIBUTION

I-55/SR-14 Bridge Replacement and Interchange Improvement
Memphis, Tennessee
J042144.01

GEOTECHNOLOGY
A Universal Engineering Sciences Company

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Kimley-Horn and Associates, Inc. DATE: 5/16/2023
PROJECT NO.: J042144.01
PROJECT: I-55 & South 3rd Street (SR-14) Interchange Modifications Preliminary
LOCATION: Memphis, Tennessee

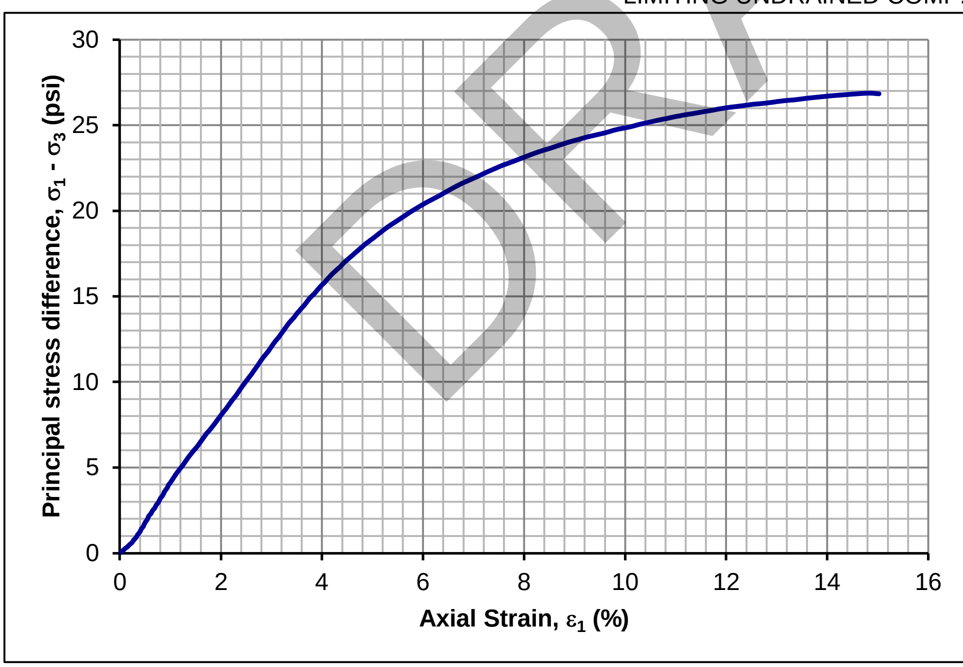
BORING NO.: B-1 SAMPLE NO.: ST-3 DEPTH (ft.): 6.0-8.0
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Stiff, gray, LEAN CLAY - (CL)

LIQUID LIMIT (%): 41 PLASTIC LIMIT (%): 19 PLASTICITY INDEX (%): 22 USCS: CL

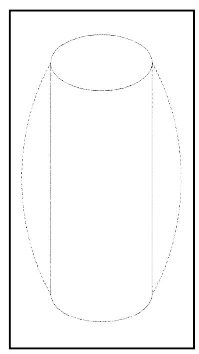
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.84
HEIGHT (in.):	5.91
HEIGHT TO DIAMETER RATIO:	2.08
WET UNIT WEIGHT (pcf):	127.6
DRY UNIT WEIGHT (pcf):	104.0
VOID RATIO:	0.65
MOISTURE CONTENT (%)*:	22.8
DEGREE OF SATURATION (%):	96.2

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**22.2**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**14.8**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**26.9**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**4.1**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**31.0**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**3,870**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**1,935**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**3,575**



FAILURE SHAPES



REMARKS :

* Initial moisture content determined from sample cuttings.
** Final moisture content determined from entire sample.
*** Failure stress values have been corrected for membrane effects.

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Kimley-Horn and Associates, Inc. DATE: 1/0/1900
PROJECT NO.: J042144.01
PROJECT: I-55 & South 3rd Street (SR-14) Interchange Modifications Preliminary
LOCATION: Memphis, Tennessee

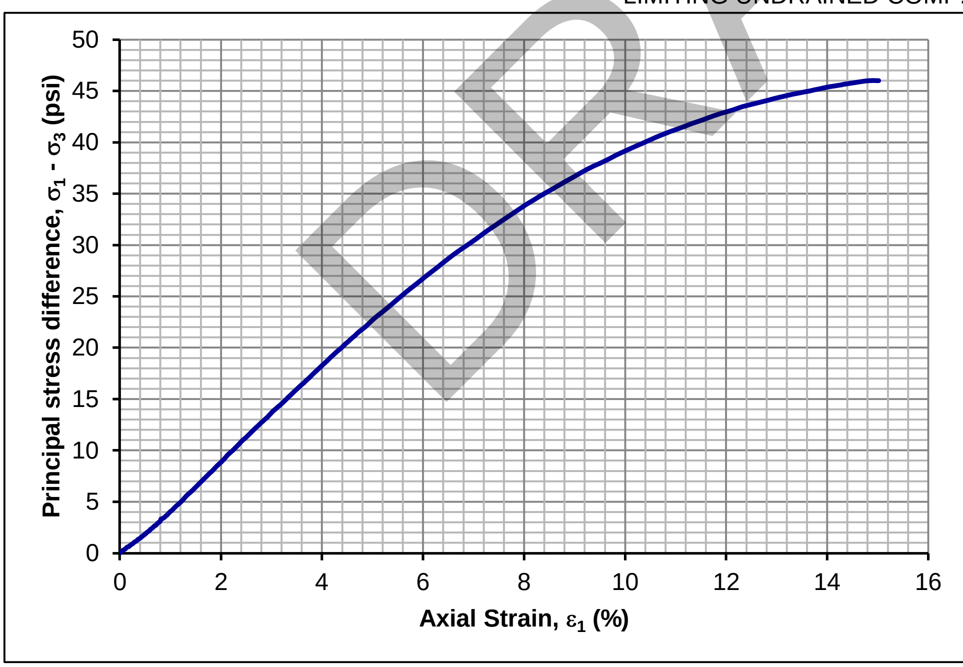
BORING NO.: B-1 SAMPLE NO.: ST-6 DEPTH (ft.): 16.0-18.0
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Very stiff, gray, LEAN CLAY - (CL)

LIQUID LIMIT (%): 32 PLASTIC LIMIT (%): 23 PLASTICITY INDEX (%): 9 USCS: CL

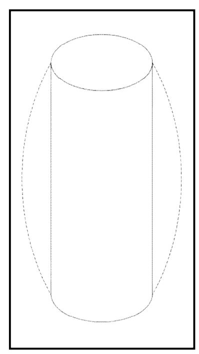
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.79
HEIGHT (in.):	5.90
HEIGHT TO DIAMETER RATIO:	2.11
WET UNIT WEIGHT (pcf):	131.5
DRY UNIT WEIGHT (pcf):	107.4
VOID RATIO:	0.60
MOISTURE CONTENT (%)*:	22.4
DEGREE OF SATURATION (%):	100.0

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**23.9**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**14.8**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**46.0**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**10.0**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**56.0**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**6,630**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**3,315**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**5,640**



FAILURE SHAPES



REMARKS :

* Initial moisture content determined from sample cuttings.
** Final moisture content determined from entire sample.
*** Failure stress values have been corrected for membrane effects.

GEOTECHNOLOGY
A Universal Engineering Sciences Company

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Kimley-Horn and Associates, Inc. DATE: 4/20/2023
PROJECT NO.: J042144.01
PROJECT: I-55 & South 3rd Street (SR-14) Interchange Modifications Preliminary
LOCATION: Memphis, Tennessee

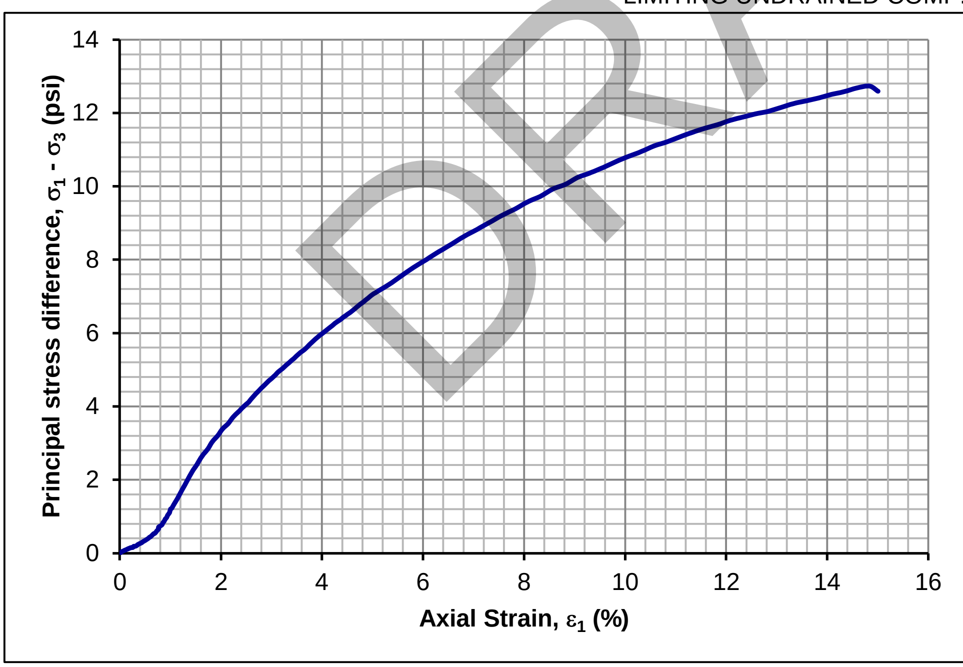
BORING NO.: B-2 SAMPLE NO.: ST-3 DEPTH (ft.): 6.0-8.0
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Medium stiff, brown and gray, LEAN CLAY - (CL)

LIQUID LIMIT (%): 32 PLASTIC LIMIT (%): 22 PLASTICITY INDEX (%): 10 USCS: CL

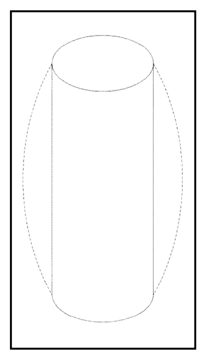
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.77
HEIGHT (in.):	6.02
HEIGHT TO DIAMETER RATIO:	2.18
WET UNIT WEIGHT (pcf):	132.7
DRY UNIT WEIGHT (pcf):	106.3
VOID RATIO:	0.61
MOISTURE CONTENT (%)*:	24.8
DEGREE OF SATURATION (%):	100.0

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**25.3**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**14.8**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**12.7**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**4.1**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**16.8**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**1,830**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**915**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**1,550**



FAILURE SHAPES



REMARKS :

* Initial moisture content determined from sample cuttings.
** Final moisture content determined from entire sample.
*** Failure stress values have been corrected for membrane effects.

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Kimley-Horn and Associates, Inc. DATE: 4/13/2023
PROJECT NO.: J042144.01
PROJECT: I-55 & South 3rd Street (SR-14) Interchange Modifications Preliminary
LOCATION: Memphis, Tennessee

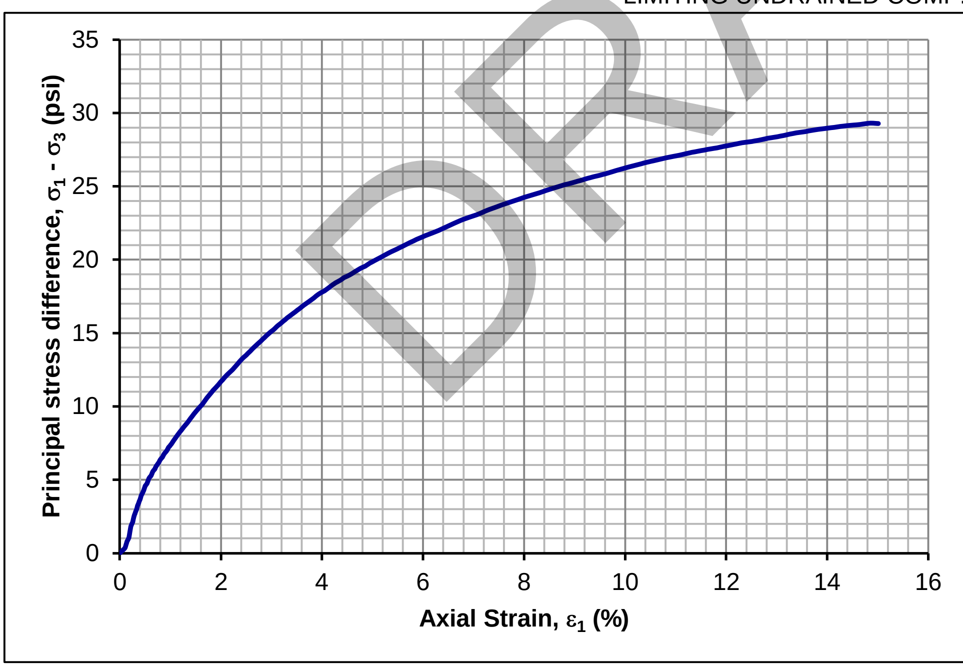
BORING NO.: B-2 SAMPLE NO.: ST-5 DEPTH (ft.): 10.0-12.0
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Very stiff, brown and gray, LEAN CLAY - (CL)

LIQUID LIMIT (%): 34 PLASTIC LIMIT (%): 21 PLASTICITY INDEX (%): 13 USCS: CL

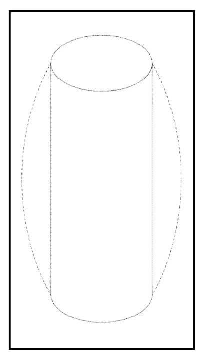
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.82
HEIGHT (in.):	5.92
HEIGHT TO DIAMETER RATIO:	2.10
WET UNIT WEIGHT (pcf):	130.4
DRY UNIT WEIGHT (pcf):	103.2
VOID RATIO:	0.66
MOISTURE CONTENT (%)*:	26.4
DEGREE OF SATURATION (%):	100.0

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**22.8**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**14.8**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**29.3**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**6.4**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**35.7**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**4,220**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**2,110**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**3,780**



FAILURE SHAPES



REMARKS :

* Initial moisture content determined from sample cuttings.
** Final moisture content determined from entire sample.
*** Failure stress values have been corrected for membrane effects.

GEOTECHNOLOGY

A Universal Engineering Sciences Company

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Kimley-Horn and Associates, Inc. DATE: 4/13/2023
PROJECT NO.: J042144.01
PROJECT: I-55 & South 3rd Street (SR-14) Interchange Modifications Preliminary
LOCATION: Memphis, Tennessee

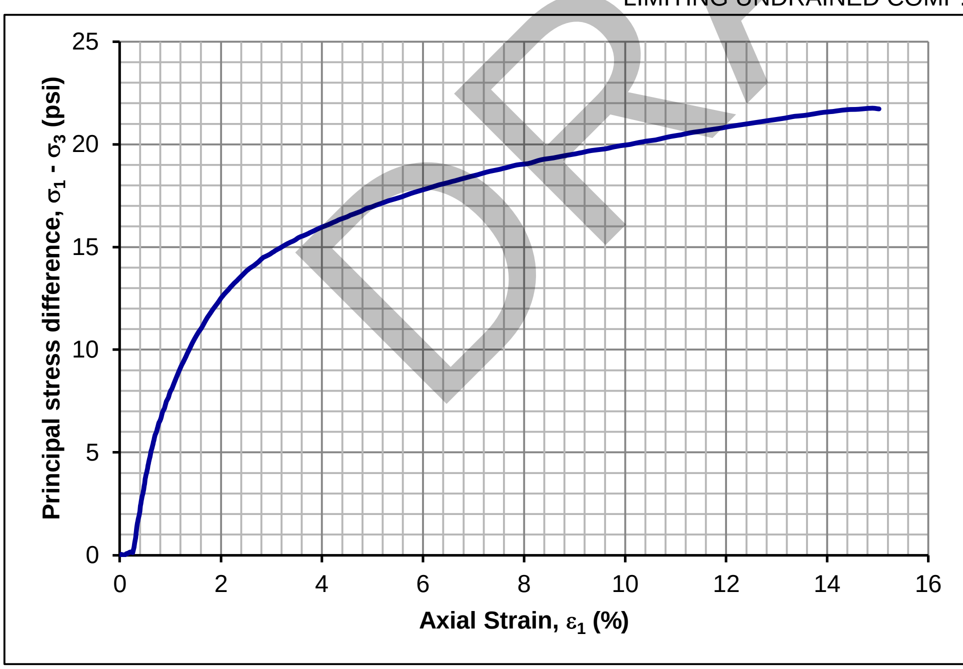
BORING NO.: B-2 SAMPLE NO.: ST-7 DEPTH (ft.): 15.0-17.0
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Very stiff, gray, LEAN CLAY - (CL)

LIQUID LIMIT (%): 35 PLASTIC LIMIT (%): 20 PLASTICITY INDEX (%): 15 USCS: CL

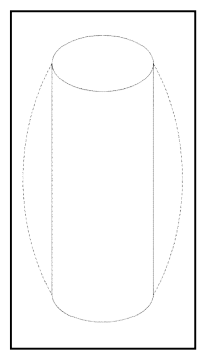
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.83
HEIGHT (in.):	5.96
HEIGHT TO DIAMETER RATIO:	2.10
WET UNIT WEIGHT (pcf):	123.6
DRY UNIT WEIGHT (pcf):	99.4
VOID RATIO:	0.73
MOISTURE CONTENT (%)*:	24.3
DEGREE OF SATURATION (%):	92.0

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**24.8**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**14.9**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, σ_1 - σ_3 (psi):	**21.8**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**9.3**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**31.1**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**3,130**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**1,565**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**2,875**



FAILURE SHAPES



REMARKS :

* Initial moisture content determined from sample cuttings.
** Final moisture content determined from entire sample.
*** Failure stress values have been corrected for membrane effects.

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Kimley-Horn and Associates, Inc. DATE: 5/16/2023
PROJECT NO.: J042144.01
PROJECT: I-55 & South 3rd Street (SR-14) Interchange Modifications Preliminary
LOCATION: Memphis, Tennessee

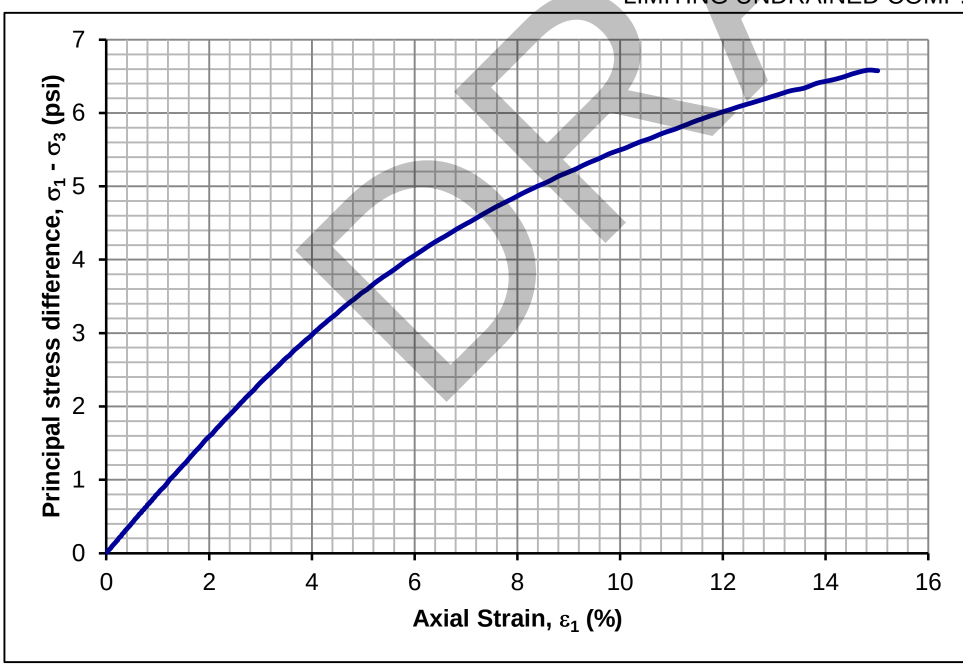
BORING NO.: B-3 SAMPLE NO.: ST-3 DEPTH (ft.): 5.0-7.0
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Soft, gray, LEAN CLAY - (CL)

LIQUID LIMIT (%): 36 PLASTIC LIMIT (%): 22 PLASTICITY INDEX (%): 14 USCS: CL

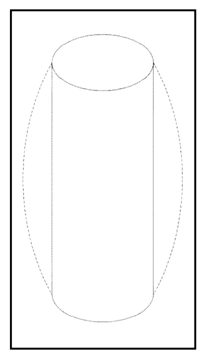
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.77
HEIGHT (in.):	5.78
HEIGHT TO DIAMETER RATIO:	2.08
WET UNIT WEIGHT (pcf):	126.7
DRY UNIT WEIGHT (pcf):	102.5
VOID RATIO:	0.67
MOISTURE CONTENT (%)*:	23.6
DEGREE OF SATURATION (%):	96.4

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**27.5**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**14.8**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**6.6**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**3.5**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**10.1**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**950**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**475**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**790**



FAILURE SHAPES



REMARKS :

* Initial moisture content determined from sample cuttings.
** Final moisture content determined from entire sample.
*** Failure stress values have been corrected for membrane effects.

GEOTECHNOLOGY
A Universal Engineering Sciences Company

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Kimley-Horn and Associates, Inc. DATE: 5/16/2023
PROJECT NO.: J042144.01
PROJECT: I-55 & South 3rd Street (SR-14) Interchange Modifications Preliminary
LOCATION: Memphis, Tennessee

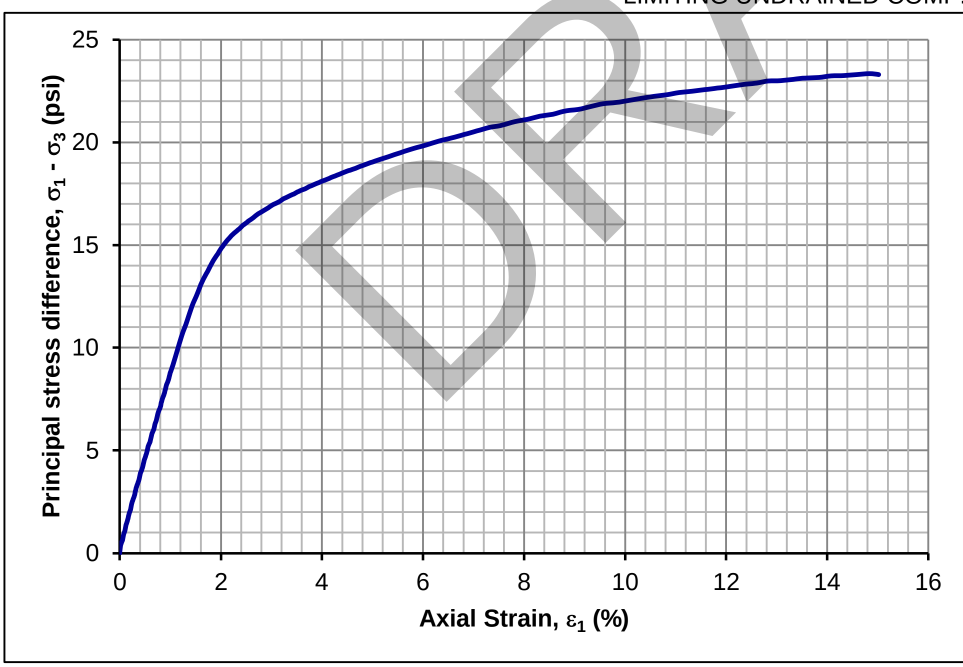
BORING NO.: B-3 SAMPLE NO.: ST-4 DEPTH (ft.): 8.0-10.0
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Stiff, gray, LEAN CLAY - (CL)

LIQUID LIMIT (%): 37 PLASTIC LIMIT (%): 19 PLASTICITY INDEX (%): 18 USCS: CL

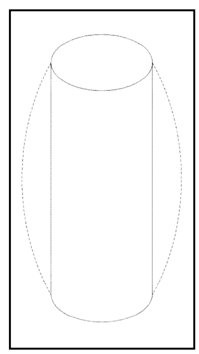
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.82
HEIGHT (in.):	5.91
HEIGHT TO DIAMETER RATIO:	2.10
WET UNIT WEIGHT (pcf):	128.0
DRY UNIT WEIGHT (pcf):	105.2
VOID RATIO:	0.63
MOISTURE CONTENT (%)*:	21.7
DEGREE OF SATURATION (%):	94.4

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**22.3**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**14.8**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, σ_1 - σ_3 (psi):	**23.3**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**5.3**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**28.6**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**3,360**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**1,680**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**3,170**



FAILURE SHAPES



REMARKS :

* Initial moisture content determined from sample cuttings.
** Final moisture content determined from entire sample.
*** Failure stress values have been corrected for membrane effects.

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Kimley-Horn and Associates, Inc. DATE: 5/16/2023
PROJECT NO.: J042144.01
PROJECT: I-55 & South 3rd Street (SR-14) Interchange Modifications Preliminary
LOCATION: Memphis, Tennessee

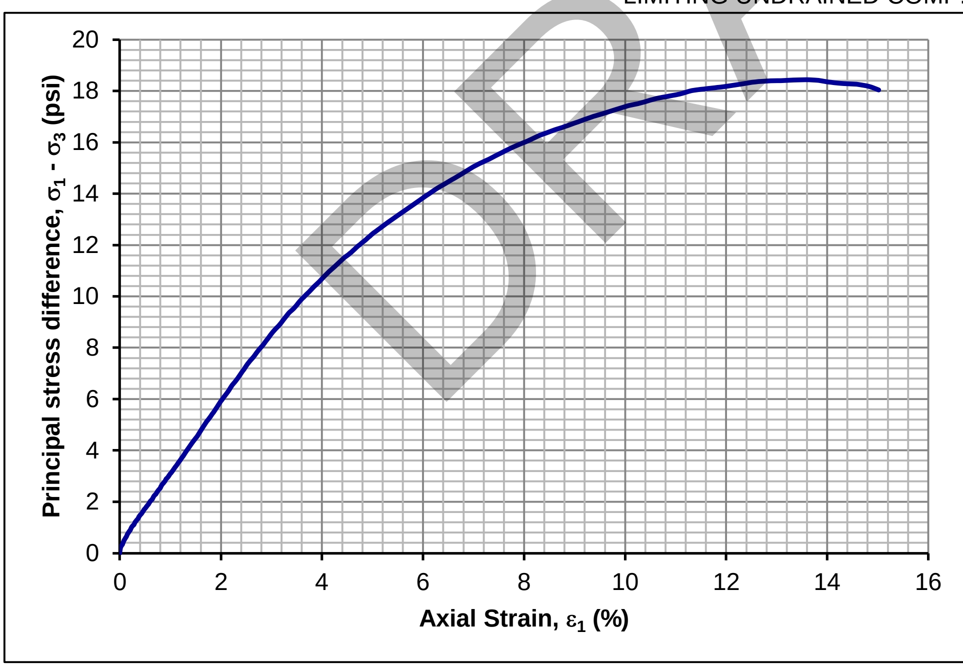
BORING NO.: B-4 SAMPLE NO.: ST-2 DEPTH (ft.): 3.0-5.0
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Stiff, gray, LEAN CLAY - (CL)

LIQUID LIMIT (%): 37 PLASTIC LIMIT (%): 21 PLASTICITY INDEX (%): 16 USCS:

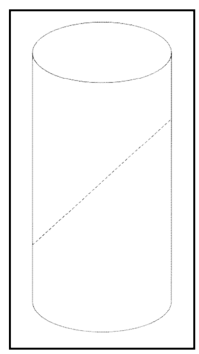
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.79
HEIGHT (in.):	5.97
HEIGHT TO DIAMETER RATIO:	2.14
WET UNIT WEIGHT (pcf):	130.6
DRY UNIT WEIGHT (pcf):	103.8
VOID RATIO:	0.65
MOISTURE CONTENT (%)*:	25.8
DEGREE OF SATURATION (%):	100.0

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**24.4**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**13.6**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**18.4**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**2.3**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**20.7**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**2,660**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**1,330**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**2,505**



FAILURE SHAPES



REMARKS :

* Initial moisture content determined from sample cuttings.
** Final moisture content determined from entire sample.
*** Failure stress values have been corrected for membrane effects.

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UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Kimley-Horn and Associates, Inc. DATE: 4/12/2023
PROJECT NO.: J042144.01
PROJECT: I-55 & South 3rd Street (SR-14) Interchange Modifications Preliminary
LOCATION: Memphis, Tennessee

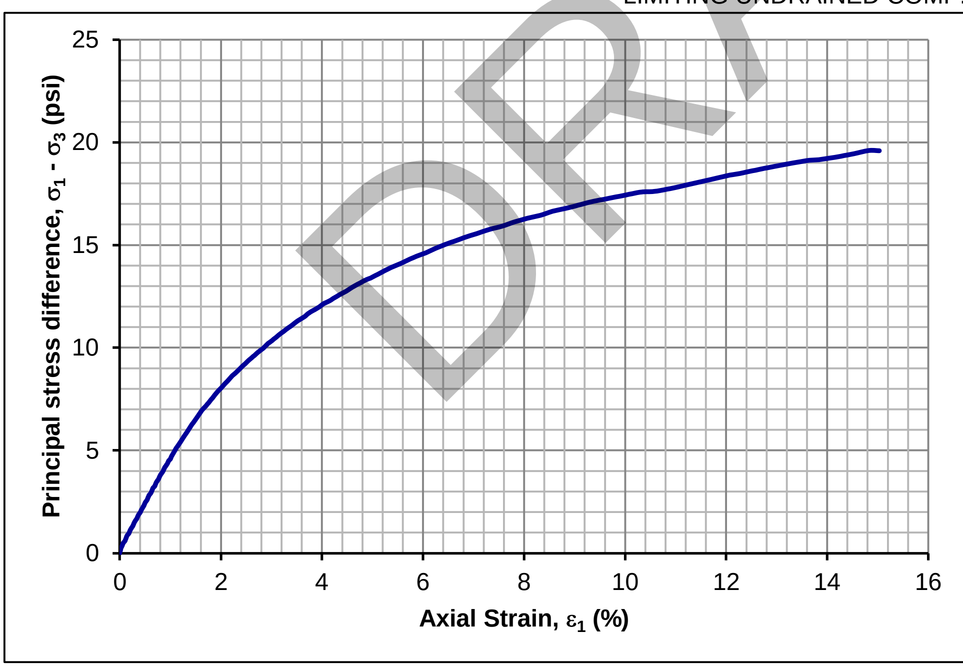
BORING NO.: R-1 SAMPLE NO.: ST-2 DEPTH (ft.): 3.0-5.0
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Stiff, gray, LEAN CLAY - (CL)

LIQUID LIMIT (%): 37 PLASTIC LIMIT (%): 20 PLASTICITY INDEX (%): 17 USCS: CL

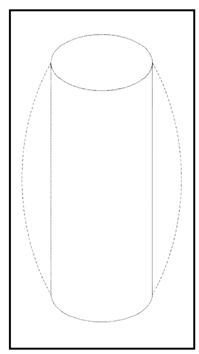
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.80
HEIGHT (in.):	5.90
HEIGHT TO DIAMETER RATIO:	2.11
WET UNIT WEIGHT (pcf):	132.7
DRY UNIT WEIGHT (pcf):	110.9
VOID RATIO:	0.55
MOISTURE CONTENT (%)*:	19.7
DEGREE OF SATURATION (%):	98.9

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**22.2**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**14.8**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**19.6**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**2.3**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**21.9**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**2,820**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**1,410**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**2,510**



FAILURE SHAPES



REMARKS :

* Initial moisture content determined from sample cuttings.
** Final moisture content determined from entire sample.
*** Failure stress values have been corrected for membrane effects.

GEOTECHNOLOGY
A Universal Engineering Sciences Company

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Kimley-Horn and Associates, Inc. DATE: 4/12/2023
PROJECT NO.: J042144.01
PROJECT: I-55 & South 3rd Street (SR-14) Interchange Modifications Preliminary
LOCATION: Memphis, Tennessee

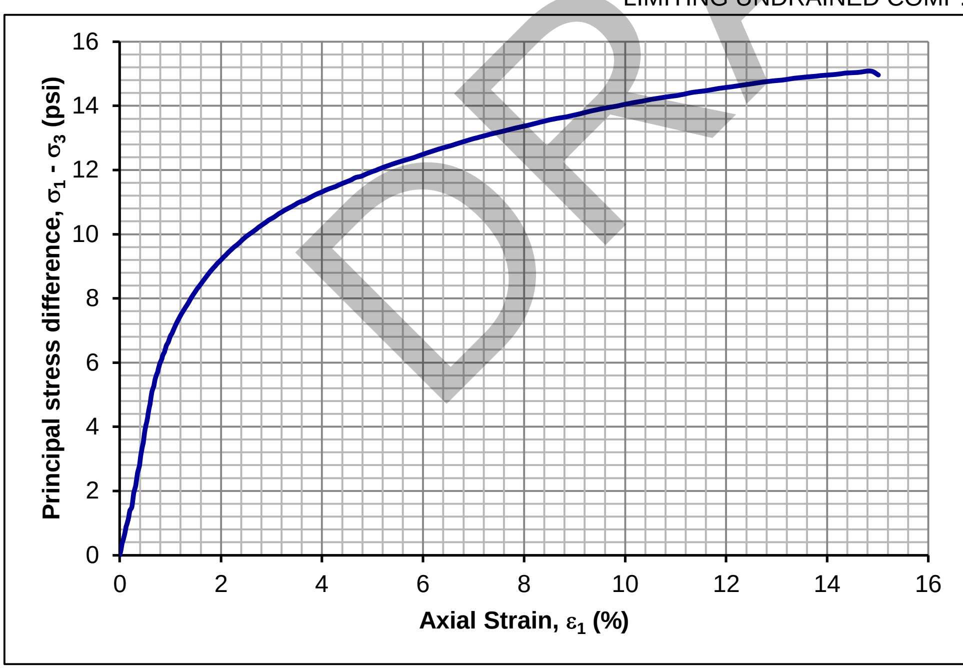
BORING NO.: R-1 SAMPLE NO.: ST-7 DEPTH (ft.): 21.0-23.0
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Stiff, gray, LEAN CLAY - (CL)

LIQUID LIMIT (%): 37 PLASTIC LIMIT (%): 23 PLASTICITY INDEX (%): 14 USCS: CL

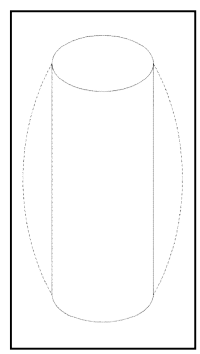
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.81
HEIGHT (in.):	5.94
HEIGHT TO DIAMETER RATIO:	2.11
WET UNIT WEIGHT (pcf):	125.9
DRY UNIT WEIGHT (pcf):	99.5
VOID RATIO:	0.72
MOISTURE CONTENT (%)*:	26.5
DEGREE OF SATURATION (%):	100.0

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**28.1**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**14.9**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**15.1**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**12.8**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**27.9**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**2,170**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**1,085**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**2,025**



FAILURE SHAPES



REMARKS :

* Initial moisture content determined from sample cuttings.
** Final moisture content determined from entire sample.
*** Failure stress values have been corrected for membrane effects.

GEOTECHNOLOGY
A Universal Engineering Sciences Company

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Kimley-Horn and Associates, Inc. DATE: 4/13/2023
PROJECT NO.: J042144.01
PROJECT: I-55 & South 3rd Street (SR-14) Interchange Modifications Preliminary
LOCATION: Memphis, Tennessee

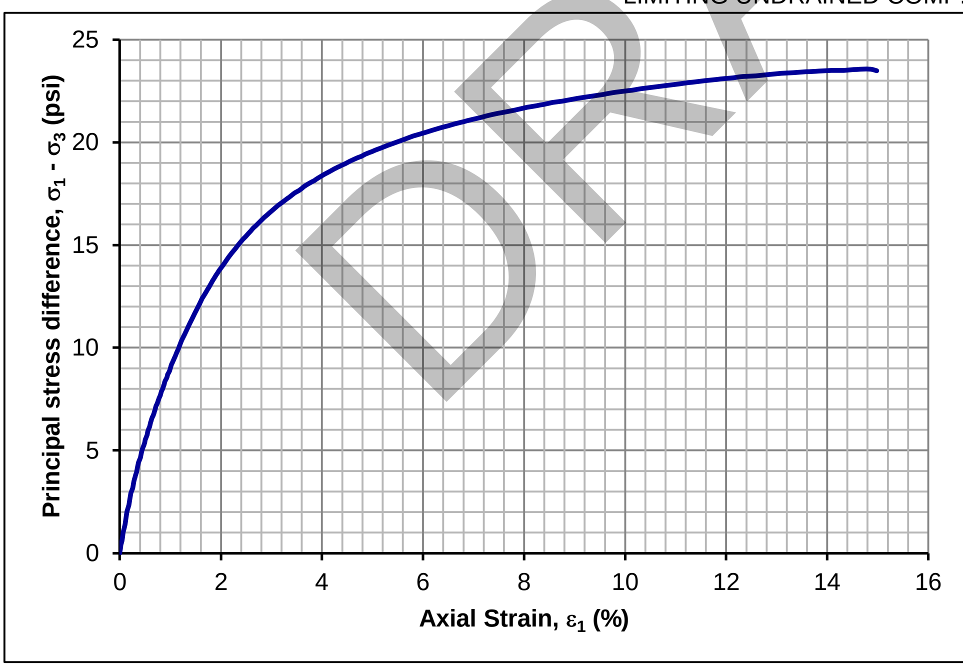
BORING NO.: R-2 SAMPLE NO.: ST-5 DEPTH (ft.): 13.0-15.0
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Stiff,gray, LEAN CLAY - (CL)

LIQUID LIMIT (%): 41 PLASTIC LIMIT (%): 19 PLASTICITY INDEX (%): 22 USCS: CL

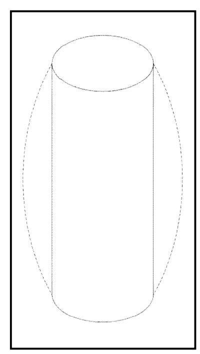
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.81
HEIGHT (in.):	5.82
HEIGHT TO DIAMETER RATIO:	2.07
WET UNIT WEIGHT (pcf):	128.0
DRY UNIT WEIGHT (pcf):	103.4
VOID RATIO:	0.66
MOISTURE CONTENT (%)*:	23.9
DEGREE OF SATURATION (%):	99.5

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**24.5**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**14.8**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**23.6**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**8.2**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**31.8**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**3,390**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**1,695**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**3,240**



FAILURE SHAPES



REMARKS :

* Initial moisture content determined from sample cuttings.
** Final moisture content determined from entire sample.
*** Failure stress values have been corrected for membrane effects.

GEOTECHNOLOGY

A Universal Engineering Sciences Company

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Kimley-Horn and Associates, Inc. DATE: 4/20/2023
PROJECT NO.: J042144.01
PROJECT: I-55 & South 3rd Street (SR-14) Interchange Modifications Preliminary
LOCATION: Memphis, Tennessee

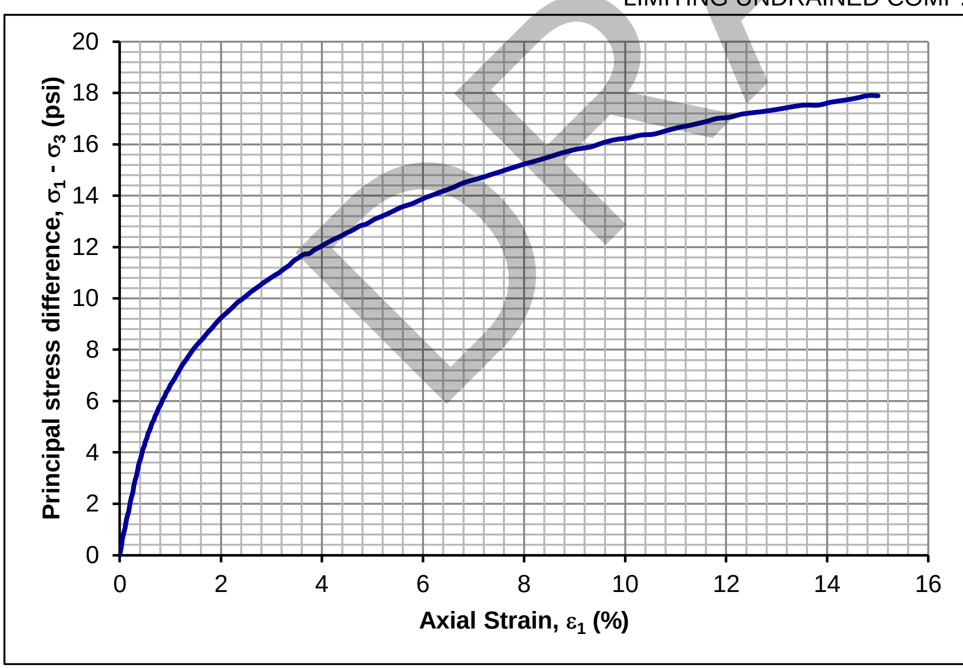
BORING NO.: R-2 SAMPLE NO.: ST-7 DEPTH (ft.): 21.0-23.0
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Stiff, brown, LEAN CLAY - (CL)

LIQUID LIMIT (%): 41 PLASTIC LIMIT (%): 17 PLASTICITY INDEX (%): 24 USCS: CL

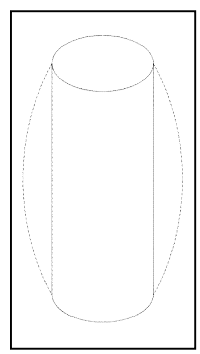
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.81
HEIGHT (in.):	5.96
HEIGHT TO DIAMETER RATIO:	2.12
WET UNIT WEIGHT (pcf):	126.2
DRY UNIT WEIGHT (pcf):	98.4
VOID RATIO:	0.74
MOISTURE CONTENT (%)*:	28.2
DEGREE OF SATURATION (%):	100.0

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**27.4**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**14.8**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**17.9**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**12.8**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**30.7**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**2,580**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**1,290**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**2,335**



FAILURE SHAPES



REMARKS :

* Initial moisture content determined from sample cuttings.
** Final moisture content determined from entire sample.
*** Failure stress values have been corrected for membrane effects.

SOIL RESISTIVITY TEST REPORT

TDOT PIN:	**128674.00**	**July 17th, 2023**
Project No.:	**J042144.01**	
Project Name:	**I-55 & South 3rd Street (SR-14) Interchange Modifications**	
Boring Number:	**B-1**	
Sample ID:	**SS-10**	
Depth (ft):	**33.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	35,800	0.57	20,406.00	9.4
#2	20,450	0.57	11,656.50	16.5
#3	16,720	0.57	9,530.40	22.9
#4	14,800	0.57	8,436.00	28.7
#5	12,690	0.57	7,233.30	33.1
#6	14,730	0.57	8,396.10	42.0

Minimum Soil Resistivity **7,233.30**

SOIL RESISTIVITY TEST REPORT

TDOT PIN:	**128674.00**	**July 17th, 2023**
Project No.:	**J042144.01**	
Project Name:	**I-55 & South 3rd Street (SR-14) Interchange Modifications**	
Boring Number:	**B-1**	
Sample ID:	**SS-14**	
Depth (ft):	**53.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	32,100	0.57	18,297.00	9.9
#2	19,710	0.57	11,234.70	16.6
#3	15,300	0.57	8,721.00	24.4
#4	13,360	0.57	7,615.20	26.7
#5	14,080	0.57	8,025.60	39.0

Minimum Soil Resistivity **7,615.20**

SOIL RESISTIVITY TEST REPORT

TDOT PIN:	**128674.00**	**July 17th, 2023**
Project No.:	**J042144.01**	
Project Name:	**I-55 & South 3rd Street (SR-14) Interchange Modifications**	
Boring Number:	**B-2**	
Sample ID:	**SS-10**	
Depth (ft):	**28.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	7,160	0.57	4,081.20	10.9
#2	3,828	0.57	2,181.96	17.7
#3	3,725	0.57	2,123.25	26.2
#4	2,576	0.57	1,468.32	36.7
#5	2,776	0.57	1,582.32	45.9

Minimum Soil Resistivity **1,468.32**

SOIL RESISTIVITY TEST REPORT

TDOT PIN: 128674.00 **June 1, 2023**
Project No.: J042144.01
Project Name: I-55 & South 3rd Street (SR-14) Interchange Modifications
Boring Number: B-2
Sample ID: SS-13
Depth (ft): 43.5

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	23,000	0.57	13,110.00	10.5
#2	16,000	0.57	9,120.00	16.8
#3	13,000	0.57	7,410.00	23.9
#4	12,000	0.57	6,840.00	33.0
#5	15,000	0.57	8,550.00	26.4
	Minimum Soil Resistivity		**6,840.00**	

SOIL RESISTIVITY TEST REPORT

TDOT PIN: 128674.00 **June 1, 2023**
Project No.: J042144.01
Project Name: I-55 & South 3rd Street (SR-14) Interchange Modifications
Boring Number: B-3
Sample ID: SS-8
Depth (ft): 28.5

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	34,000	0.57	19,380.00	9.7
#2	23,500	0.57	13,395.00	16.7
#3	20,000	0.57	11,400.00	22.8
#4	21,000	0.57	11,970.00	21.6
	Minimum Soil Resistivity		**11,400.00**	

SOIL RESISTIVITY TEST REPORT

TDOT PIN:	**128674.00**	**July 17th, 2023**
Project No.:	**J042144.01**	
Project Name:	**I-55 & South 3rd Street (SR-14) Interchange Modifications**	
Boring Number:	**B-3**	
Sample ID:	**SS-14**	
Depth (ft):	**58.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	29,170	0.57	16,626.90	9.7
#2	17,180	0.57	9,792.60	16.9
#3	14,050	0.57	8,008.50	24.3
#4	12,760	0.57	7,273.20	33.6
#5	13,270	0.57	7,563.90	43.4

Minimum Soil Resistivity **7,273.20**

SOIL RESISTIVITY TEST REPORT

TDOT PIN: 128674.00 **June 1, 2023**
Project No.: J042144.01
Project Name: I-55 & South 3rd Street (SR-14) Interchange Modifications
Boring Number: B-4
Sample ID: SS-9
Depth (ft): 28.5

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	63,000	0.57	35,910.00	9.7
#2	46,000	0.57	26,220.00	16.4
#3	39,000	0.57	22,230.00	22.6
#4	43,000	0.57	24,510.00	25.5
	Minimum Soil Resistivity		**22,230.00**	

SOIL RESISTIVITY TEST REPORT

TDOT PIN:	**128674.00**	**June 1, 2023**
Project No.:	**J042144.01**	
Project Name:	**I-55 & South 3rd Street (SR-14) Interchange Modifications**	
Boring Number:	**B-4**	
Sample ID:	**SS-14**	
Depth (ft):	**53.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	3,100	0.57	1,767.00	9.1
#2	2,200	0.57	1,254.00	16.5
#3	1,800	0.57	1,026.00	24.4
#4	1,650	0.57	940.50	31.9
#5	1,600	0.57	912.00	40.7
#6	1,700	0.57	969.00	52.7
	Minimum Soil Resistivity		**912.00**	

Appendix E
SITE-SPECIFIC SEISMIC STUDY

Site-Specific Seismic Study
I55/SR-14 Bridge Replacement
Memphis, Tennessee

By

Shahram Pezeshk, Ph.D., P.E.
Email: s.pezeshk@aol.com
901-606-6934

June 18, 2023

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Site-Specific Seismic Study
I55/SR-14 Bridge Replacement
Memphis, Tennessee

1.0. EXECUTIVE SUMMARY

The executive summary provides an overview of my understanding of the project and recommendations. Information and recommendations presented in the executive summary should not be used without reviewing the entire Report.

- The location of the study site is 35.07961°N and 90.05711°W (See Appendix A).
- Based on the recommendations of the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions, A_S (zero-period), S_{DS} (short period), and S_{D1} (long period) are provided in Table 3.
- Site-specific recommendations following the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions are provided in Table 5 and Table 6.

2.0. SCOPE OF WORK

The purpose of our study is to estimate the design spectra following the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions. The structural design of new buildings allows two procedures for determining design ground motions:

1. *General Procedure*. In this method, the response spectrum is determined using the following steps: (1) develop the rock spectrum using seismic design maps for values of Peak Ground Acceleration (PGA) and spectral acceleration at periods of 0.2 and 1.0 seconds; (2) determine the Site Class using the shear-wave velocity (V_s) measurements from the upper 100 feet of the soil profile, and (3) adjust the rock spectrum for site class to develop the general response spectrum.

2. *Site-Specific Procedure*. In this method, the response spectrum is determined using a combination of probabilistic seismic hazard and site response analyses. The site-specific response spectrum may not be less than 2/3 of the general response spectrum.

Briefly, the scope of our services for the site-specific investigation included the following steps:

1. Perform probabilistic seismic hazard analysis (PSHA) to estimate ground motions in the rock underlying the site;
2. Determine Uniform Hazard Response Spectrum (UHRS) at the rock level;
3. Determine probabilistic consistent magnitude and distances from deaggregation;
4. Select ground motions consistent with magnitude and distances obtained in step 3;
5. Perform spectral matching to match the selected ground motions to the UHRS of Step 2;
6. Perform one-dimensional equivalent linear site-specific ground response analysis using the site-specific earthquake time histories by using the computer program SHAKE91 (Idriss and Sun, 1992) and considering the uncertainties associated with the shear-wave velocity and layer thicknesses for the soil profile; and
7. Develop site-specific response spectra for the existing subsurface conditions using the procedure outlined in the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, with 2022 Interim Revisions, based on 7 percent probability of exceedance in 75 years and 5 percent damping for a single degree of freedom (SDOF) structure.

3.0. SUBSURFACE CONDITIONS

This study is based on the available information on the soil stratigraphy provided by Geotechnology and the shear-wave velocity profile obtained using Seismic Cone Penetration Testing (SCPT).

4.0. SHEAR-WAVE VELOCITY PROFILE

Seismic Cone Penetration Testing (SCPT) was performed by Geotechnology (a UES Company). Table 1 provides the shear-wave velocity obtained from SCPT.

Table 1. Shear-Wave Velocities Measured.

Depth1 (ft)	**Depth2 (ft)**	V_s **(ft/sec)**
1.57	4.89	412.26
4.89	8.20	412.26
8.20	11.48	412.26
11.48	14.79	532.38
14.79	18.07	521.32
18.07	21.35	522.01
21.35	24.63	536.31
24.63	27.85	999.38
27.85	31.09	1148.33
31.09	34.34	1075.58
34.34	37.56	1068.26
37.56	40.97	1064.72
40.97	44.18	968.55
44.18	47.43	698.77
47.43	50.77	727.21
50.77	54.02	682.63
54.02	57.30	806.29
57.30	60.58	734.20
60.58	63.83	807.31
63.83	67.08	702.38
67.08	70.32	1036.05
70.32	73.60	792.84
73.60	76.95	1054.72
76.95	80.26	842.17
80.26	83.51	940.93
83.51	86.69	1086.37
86.69	90.13	1060.03
90.13	93.32	1133.40
93.32	96.56	1687.23
96.56	100.01	818.16

5.0 GENERAL INFORMATION

For this project, we have been requested to perform a site-specific seismic study to produce the ground surface response spectrum and a set of time series based on the seismic parameters used in the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions, which include: seismic hazards related to 7 percent probability of exceedance in 75 years and 5 percent damping for SDOF structure.

6.0. REGIONAL SEISMICITY

Petersen et al. (2019) used fault models from the 2014 NSHM to model large earthquakes and apply gridded, smoothed seismicity models from an earthquake catalog to account for smaller earthquakes on and off the faults. They developed new seismicity catalogs for the CEUS and WUS, including earthquakes from 2013 through 2017 that occurred since the last model was constructed. Between 2013, when the catalog was last updated, and 2018, strongly felt earthquakes (magnitude 4+) occurred in almost half of the states in the United States. Figure 1 shows the USGS 2018 declustered catalog for CEUS.

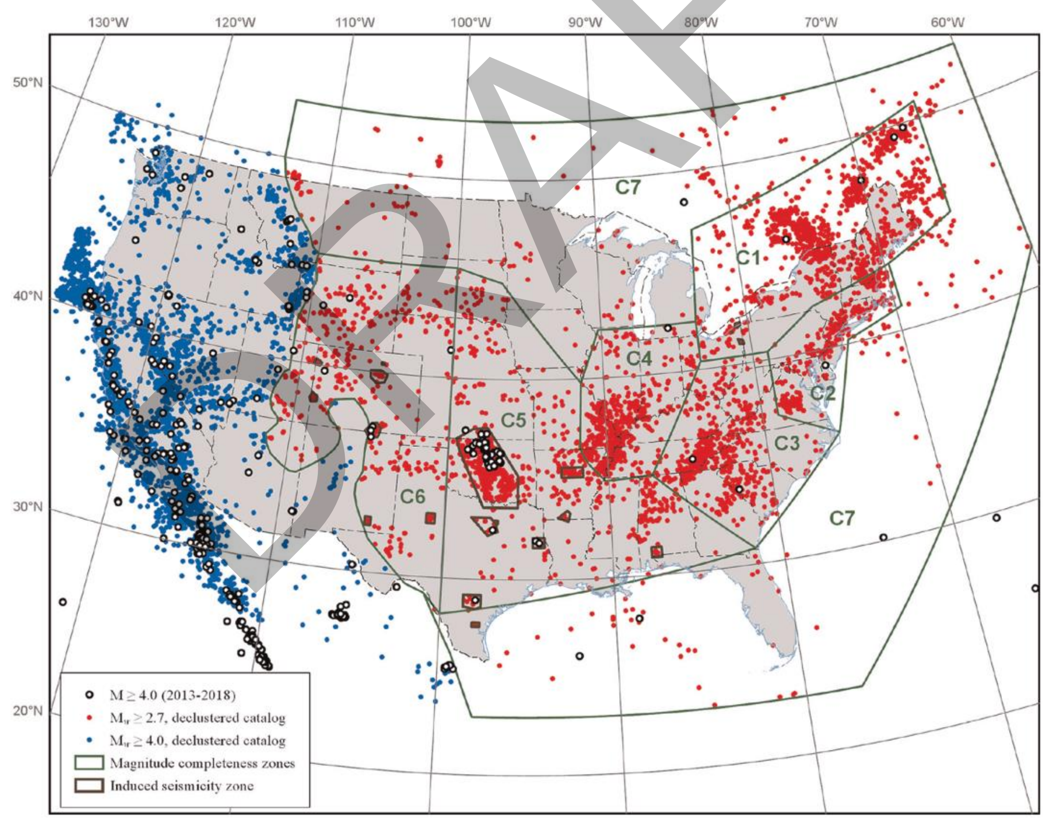

Figure 1. The 2018 NSHM Declustered Catalog for Central and Eastern United States (red) and Western United States (blue).

7.0. SEISMIC HAZARD ANALYSIS

A PSHA was performed to estimate the seismic ground motions for a rock site condition. The analytical model used for the PSHA is based on models developed initially by Cornell (1968). These models' underlying assumption is that earthquakes occur in space and time within a particular seismic zone is entirely random (i.e., a Poisson process). This type of probabilistic model is commonly used for seismic hazard analyses of essential facilities throughout the world.

The two primary components of the probabilistic model are:

1. The seismic source models specify the spatial, temporal, and magnitude distribution of earthquake occurrences expected in each of the seismic sources, and

2. The ground-motion attenuation models which determine the distribution of ground motions expected at the site for a potential earthquake occurrence (characterized by magnitude and location, and usually by other factors) on a seismic source.

The above two components comprise the inputs to the PSHA. In the PSHA, probability-of-exceedance rates (hazard curves) are computed for a range of horizontal ground motions. These ground motions are expressed in terms of peak ground acceleration (PGA) and 5 percent-damped pseudo absolute spectral accelerations (S_a) at various single-degree-of-freedom oscillator periods. From the probability-of-exceedance rates, the Uniform Hazard Response Spectrum (UHRS) corresponding to average return periods of 7% probability of exceedance in 75 years is computed.

7.1. SEISMIC SOURCE MODELS

The USGS seismic source models have been used for this project. The USGS addressed the causes of earthquakes in the Central and Eastern United States in two ways: (1) earthquake fault; and (2) background or smoothed seismicity models, which forecast the occurrence rates and magnitudes of potential seismic events.

7.2. GROUND MOTION MODELS

In general, the characteristics of the fault source, such as distance, type, magnitude, and site conditions, are used to estimate the magnitude of an earthquake parameter (spectral acceleration, peak ground acceleration, etc.) via ground-motion models (GMMs) or ground-motion prediction equations (GMPEs), also known as attenuation relationships. Various attenuation relationships have developed for specific regions using a database of appropriate ground motion records.

Petersen et al. (2020a) presented only a summary of the CEUS GMM updates, which included comparisons of the 2018 weighted median GMMs to the 2014 National Seismic Hazard Model (NSHM) and an overview of the aleatory variability (GMM standard deviation) and site-effect models. Rezaeian et al. (2021) discuss the CEUS GMM updates and implementation in the 2018 NSHM in detail. These updates consist of (1) 31 new GMMs, including the state-of-the-art Next Generation Attenuation relationships for central and eastern North America (NGA-East) (Goulet et al., 2018, 2017, 2021; Pacific Earthquake Engineering Research Center (PEER), 2015a), (2) an associated model of aleatory variability (based on Al Atik, 2015; Goulet et al., 2017; Stewart et al., 2019), and (3) a new site-effect model (for amplification or deamplification) specific to the CEUS (Hashash et al., 2020; Stewart et al., 2020). In the following, we discuss the individual GMMs in terms of their medians, assigned weights, weighted averages, attenuations with distance, and epistemic uncertainty.

According to Rezaeian et al. (2021), NSHM 2018 was updated to generate national seismic hazard maps for the Central and Eastern United States. The logic tree weights are based on the distance and the geometric spreading term used by each model. The models with a faster geometric spreading term are given more weight. The New Madrid seismic zone is the most likely seismic source that could affect the considered site. NSHM removed the attenuation relationships not applicable beyond 500 km, and weights were renormalized.

Table 2 lists the selected GMMs from the NSHM 2018 models with their associated weights. Three of the models were developed by Pezeshk and his colleagues [Pezeshk et al. 2015; 2018 (PZCT15-M1SS, PZCT15-M2ES), Shajouei and Pezeshk (2016) (SP16)].

Table 2. Ground Motion Models (GMMs).

CEUS GMMs (Acronyms)	Authorship	Weight
14 Updated Seed GMMs (used by USGS in 2018 NSHM)		**0.333**
B-bca10d	Boore	0.02209
B-ab95	Boore	0.00736
B-bs11	Boore	0.00736
2CCSP	Darragh-Abrahamson-Silva-Gregor	0.01841
2CVSP	Darragh-Abrahamson-Silva-Gregor	0.01841
Graizer16	Graizer	0.01813
Graizer17	Graizer	0.01813
PZCT15-M1SS	Pezeshk-Zandieh-Campbell-Tavakoli	0.01813
PZCT15-M2ES	Pezeshk-Zandieh-Campbell-Tavakoli	0.01813
SP16	Shahjouei-Pezeshk	0.03626
YA15	Yenier-Atkinson	0.03736
HA15	Hassani-Atkinson	0.03736
Frankel15	Frankel	0.03737
PEER-GP	Hollenback-Kuehn-Goulet-Abrahamson	0.03850
Other NGA-East Adjusted Seed GMMs (not used by USGS in 2018 NSHM)		**0**
B-a04	Boore	0
B-ab14	Boore	0
B-sgd02	Boore	0
1CCSP	Darragh-Abrahamson-Silva-Gregor	0
1CVSP	Darragh-Abrahamson-Silva-Gregor	0
SP15 (replaced with SP16 by USGS)	Shahjouei-Pezeshk	0
Graizer (replaced with Graizer16 & Graizer17 by USGS)	Graizer	0
PEER-EX	Hollenback-Kuehn-Goulet-Abrahamson	0
~~ANC15~~ (see Note 1)	~~Al Noman-Cramer~~	0
17 NGA-East GMMs (used by USGS in 2018 NSHM)		**0.667**
Models 1 to 17	NGA-East Project	Period-dependen[a]

CEUS: central and eastern United States; USGS: U.S. Geological Survey; NSHM: National Seismic Hazard Model.
[a]See Figure 6 for example weights at periods PGA, 0.2, 1, 2, and 5 s.

7.3. TREATMENT OF UNCERTAINTIES

Seismic-hazard studies distinguish between two types of uncertainty, namely epistemic and aleatory. Aleatory uncertainty is probabilistic variability that results from a natural physical process. For example, the size, location, and time of the next earthquake on a fault and the details of the ground motion are considered aleatory uncertainties. In advanced seismic hazard studies, integration is performed over aleatory uncertainties to get a single hazard curve—the epistemic uncertainty results from a lack of knowledge about earthquakes and their effects. In principle, epistemic uncertainties are addressed by multiple models and parameters. The most well-known epistemic uncertainties associated with the input parameters in seismic hazard analysis include the uncertainties in seismic source models (i.e., tectonic stresses, geological features, geometries, etc.), seismicity (i.e., activity rate, slip rate, etc.), and attenuation relationships (source, path, and site effects). The USGS 2014 procedure (Petersen *et al.*, 2014) is followed in this project to address the uncertainty in seismic-source characterization, which is quantified by considering alternative geometries, multiple magnitude-recurrence parameters, and multiple maximum magnitudes.

8.0. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, 2022 Interim Revisions

Time-averaged shear-wave velocity in the top 100 ft (30 m) is defined as V_{S30}. The V_{S30} for the study site is determined to be 738 ft/sec, which according to the Guide Specifications, the study site is determined to be a Site Class "D" (Table 3.4.2.1-1, Site Class Definitions). Site coefficients F_{pga}, F_a, and F_v for the study site following Tables 3.4.2.3-1 and 3.4.2.302 mapped spectral acceleration are summarized in Table 3.

8.1. Dynamic Soil Properties

Low-strain soil shear modulus and damping are the required dynamic soil properties for seismic ground response analysis. A brief discussion of these properties is given below.

8.1.1. Low Strain Soil Shear Modulus

A key parameter necessary to evaluate the dynamic response of soils is the dynamic shear modulus, $G_{s,}$ or shear wave velocity, which is also related to the dynamic shear modulus. Values of shear wave velocity or shear modulus can be determined either by measuring in the laboratory on undisturbed soil samples or by performing seismic field tests. Shear modulus is not a constant property of soil but decreases nonlinearly with increasing strain. For initial design purposes, shear modulus measured at small shear strain amplitudes (less than 10^{-4} percent), referred to as G_{max}, is the desired design parameter.

Laboratory measurement of shear wave velocity or low-strain soil shear modulus was beyond the scope of our services. Various correlations and typical values are available in the literature to estimate the approximate value of shear-wave velocity and G_{max}.

8.1.2. Damping

The inelastic behavior of soil (discussed later) also gives rise to the energy absorption characteristics of soil, known as material damping. Damping is generally expressed as a percentage of critical damping. Low strain damping of approximately 5 to 10 percent of the critical damping is commonly used for soils. Damping of 5 percent of critical was used for the analysis. However, this damping was modified in the study based on the strain levels in the soil, as explained in subsequent sections of this Report.

8.1.3. Effect of Strain on Dynamic Soil Properties

It is well understood that the stress-strain relationship of soils is nonlinear. This means that the soil shear modulus is not a constant value but degrades nonlinearly with increasing strain in the soil. Dynamic analyses considering the true nonlinear behavior of soil are complicated and are an active and current research area. Accordingly, an equivalent linear analysis is typically used in practice. Equivalent linear analyses consist of performing a series of linear analyses in an iterative process, using, for each analysis, soil properties consistent with the strains resulting from the previous one. An equivalent linear site response analysis is used in the present study. Many studies have been performed in the past to establish a relationship between modulus degradation with strain.

9.0. CODE-BASED DESIGN APPROACH

9.1. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, 2022 Interim Revisions

Using the United States Geological Survey (USGS) Hazard Maps and the project location, the mapped 0.2-second spectral response acceleration (S_s) and the mapped 1.0-second spectral response acceleration (S_1) are provided in Table 3. Based on the average shear-wave velocities of the top 100 ft of soil, the site class has been determined to be site class "D." Based on the mapped spectral acceleration and site class D, the site coefficients F_{PGA}, F_a, and F_v are provided in Table 3. provides a summary of these parameters.

Table 3. Mapped Provisional Design Response Spectrum Parameters at 5% Damping.

Parameter	Value
F_a	1.252
F_v	2.097
F_{PGA}	1.140
S_S	0.685
S_1	0.176
S_{DS}	0.858
S_{D1}	0.369
PGA	0.360
A_s	0.410

10.0. SITE-SPECIFIC PROCEDURE

The probabilistic seismic hazard analysis (PSHA) considers all potential earthquake sources that will contribute to hazards at a specific site. The PSHA factors in contributions from all magnitudes, distances, and probability of occurrence for all sources. This study used PSHA to estimate PGA and spectral acceleration at various periods for a B/C NEHRP site condition for a 7% probability of exceedance in 75 years.

The PSHA was performed to obtain a uniform hazard response spectrum (UHRS). The PSHA and de-aggregation results were used to select earthquakes for the site response analyses. Eleven horizontal components (total of 11) of previously recorded earthquakes within the range of de-aggregation magnitudes and distances were selected.

Table *4* provides the mean and the modal deaggregation magnitude and distances for various periods. The UHRS was selected as the target spectrum, and the chosen time histories were matched with the target spectrum. As an example, acceleration, velocity, and displacement time histories for a typically selected earthquake are illustrated in Figure 2. The same process was repeated for all eleven earthquakes for both components.

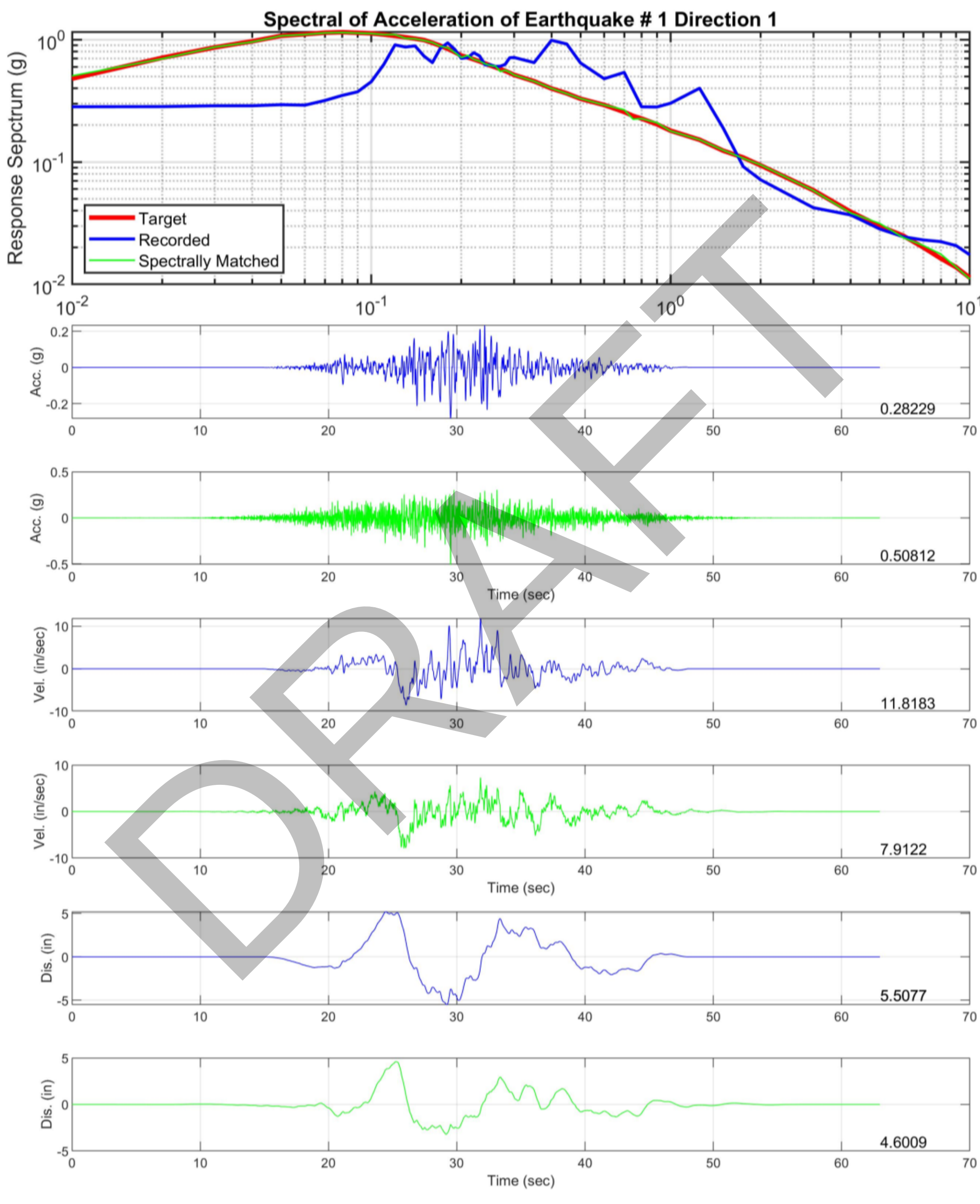

Figure 2. Time Histories Before and After the Spectral Matching Process for Earthquake #1. The numbers Shown in the Bottom right of Each Figure Represent the Absolute Maximum Value of the Graph.

Table 4. Deaggregation.

Mean and Mode Deaggregation Parameter at 1,033 Years					
Mean			**Mode**		
Period	**M**	**R (km)**	**Period**	**M**	**R (km)**
PGA	7.03	57.11	PGA	7.52	62.39
0.01	7.01	56.17	0.01	7.55	62.17
0.02	6.97	55.27	0.02	7.55	62.29
0.03	6.97	55.21	0.03	7.76	62.14
0.05	7.03	57.98	0.05	7.54	62.65
0.075	7. 03	60.84	0.075	7.53	63.07
0.10	7.12	63.98	0.10	7.54	62.87
0.15	7.21	68.31	0.15	7.55	62.65
0.20	7.25	71.56	0.20	7.54	62.83
0.25	7.31	74.75	0.25	7.54	62.99
0.30	7.33	76.52	0.30	7.54	63.11
0.40	7.37	78.91	0.40	7.76	132.66
0.50	7.40	80.86	0.50	7.77	133.03
0.75	7.45	84.78	0.75	7.54	62.78
1.00	7.48	87.08	1.00	7.55	62.63
1.50	7.52	89.31	1.50	7.77	132.59
2.00	7.55	91.36	2.00	7.77	132.68
3.00	7.58	92.91	3.00	7.55	62.40
4.00	7.60	94.05	4.00	7.55	62.20
5.00	7.61	94.76	5.00	7.56	62.32
7.50	7.63	95.09	7.50	7.53	63.09
10.00	7.64	96.00	10.00	7.76	132.83

10.1. Seismic Hazard Analysis

The uniform hazard response spectrum (UHRS) and the magnitude and distance deaggregation for a 7 percent probability of exceedance in 75 years (equivalent to a return period of about 1033 years) are calculated from the PSHA. The seismic hazard is calculated for the uniform firm site condition with 760 m/s shear-wave velocity in the upper 30 m (V_{s30}), representing the boundary between NEHRP site classes B and C.

10.2. Variability in Soil's Shear-Wave and Thickness Profile

A probabilistic characterization of the soil shear-wave velocity profile was used to simulate shear-wave profiles. Two separate components; one for the thickness of each layer called the layering model that captures the variability in the thickness of soil layers, and one for the shear-wave velocity associated with each layer called the velocity model to account for the variability in the shear-wave velocity of each layer are used. A non-homogeneous Poisson model is used with a depth-dependent rate to account for the fact that the soil thickness of layers increases with depth.

In this project, the variability in the shear-wave velocity are considered. The model used statistically captures the soil layer shear-wave velocity and thickness uncertainties and their correlation with depth. A total of 60 cases were generated. These 60 soil profiles are used to capture the soil layer shear-wave velocity and thickness uncertainties and their correlation with depth.

10.3. Site-Specific Results

Following the procedure outlined above, the site-specific response spectra were obtained, analyzing sixty profiles for each matched ground motion with the UHRS.

The site-specific results were obtained by performing PSHA using all seismic sources and faults and appropriate and recent ground motion prediction equations for Central and Eastern United States following the provisions of the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions. All uncertainties associated with each aspect of the site-specific analysis were carefully considered. Figure 3 shows the design response spectra, Guide Specifications, and 2/3 of Guide Specifications design spectra. In this figure, the site-specific spectrum is not limited to 2/3 of the Guide Specifications response spectrum for illustration.

Site-specific seismic design recommendations following the Guide Specifications provisions are provided in Table 5 and Table 6. The recommendation is to use the design Sa values provided in Table 5. Figure 4 shows the design response spectra, Guide Specifications, 2/3 of Guide Specifications design spectra, and the site-specific design spectrum constructed based on three periods of PGA, 0.2 sec and 1 sec. In Figure 4, the site-specific response spectrum is adjusted not to be less than 2/3 of the Guide Specifications design response spectrum.

11.0. DESIGN RESPONSE SPECTRAL PARAMETERS

The design spectral response acceleration parameters listed in Table 5 were developed following Guide Specifications.

Table 5. Site-Specific Spectral Acceleration Considering 5% Damping following the Guide Specifications.

Period	Site-Specific Response Spectra
(s)	(g)
0.010	0.389
0.030	0.394
0.040	0.412
0.050	0.447
0.070	0.516
0.100	0.572
0.150	0.673
0.200	0.829
0.250	0.798
0.300	0.788
0.400	0.843
0.500	0.875
0.750	0.705
1.000	0.564
1.500	0.357
2.000	0.245
3.000	0.177
4.000	0.172
5.000	0.136
7.500	0.080
10.000	0.037

Table 6. Site-Specific Response Accelerations Considering 5% Damping.

PARAMETER	DESIGN ACCELERATION PARAMETERS (g)
S_{DS}	0.829
S_{D1}	0.564
S_{MS}	0.829
S_{M1}	0.564
MCE_G	0.389

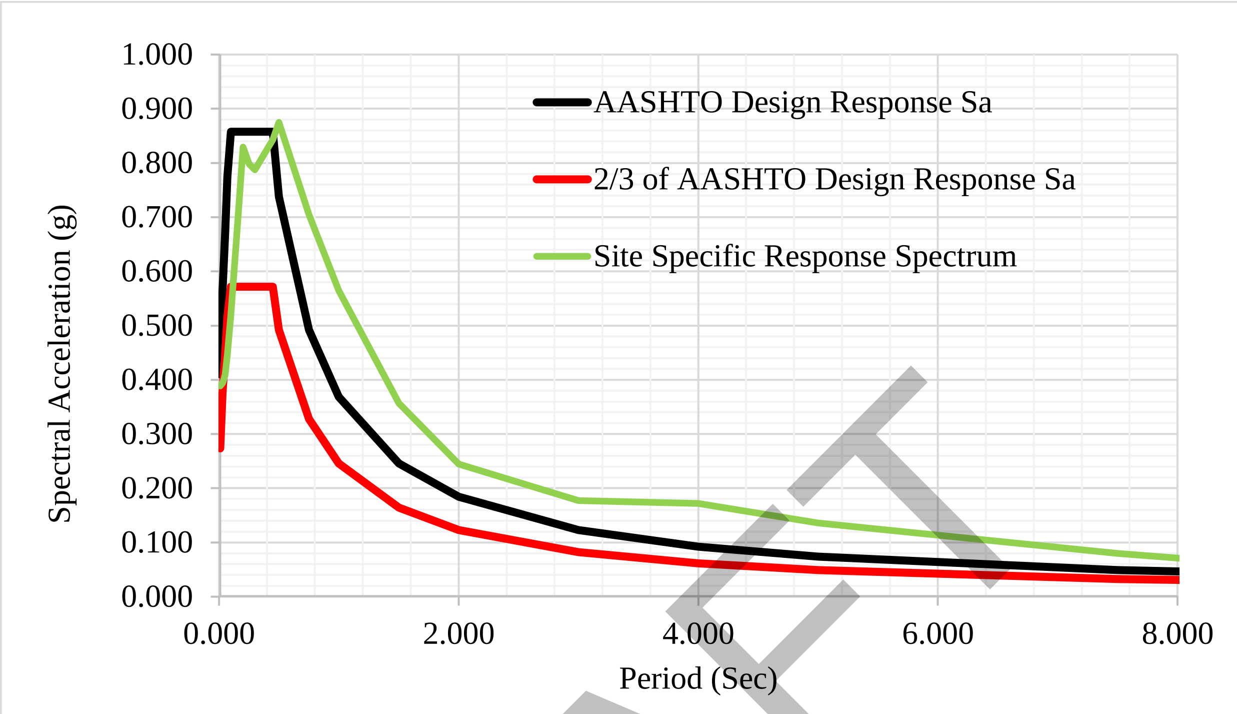

Figure 3. Site-Specific Design Response Spectrum, AASHTO Guide Specifications Design Response Spectrum, and 2/3 of the AASHTO Guide Specifications Design Response Spectrum.

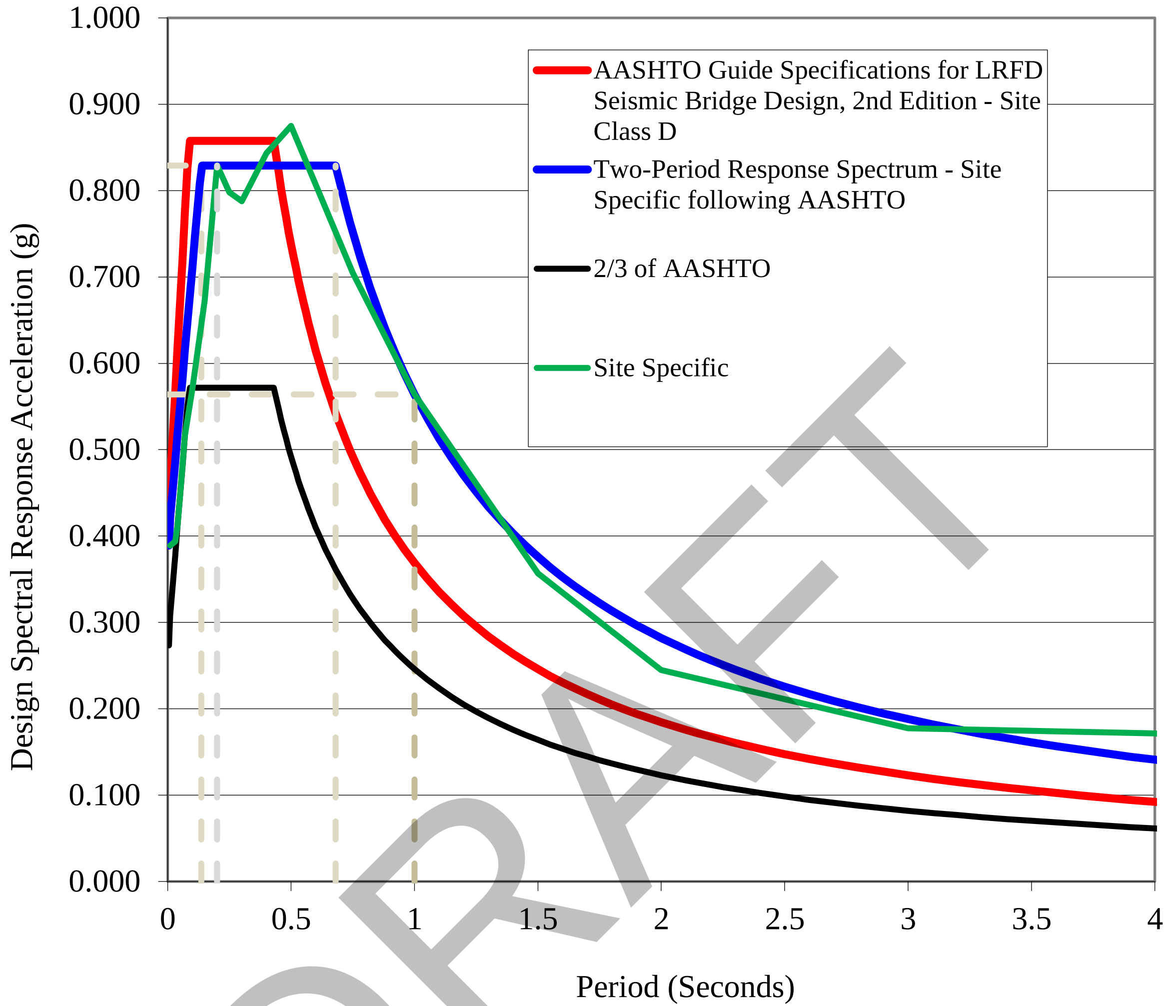

Figure 4. Design Response Spectrum based on AASHTO Guide Specifications, 2/3 of the AASHTO Guide Specifications Site-Specific, and Design Response Spectrum Based on PGA, 0.2, and 1 Second.

12.0 LIMITATIONS OF THE REPORT

The analyses, conclusions, and recommendations presented in this Report are professional opinions based on the site conditions and project layout described herein and further assume that the conditions provided in the geotechnical Report are representative of the subsurface conditions throughout the site, i.e., that the subsurface conditions elsewhere on the site are the same as those disclosed by the borings. If, during construction, subsurface conditions different from those encountered in the exploratory boring are observed or appear to be present, the Client must contact us immediately so that we can make changes to this Report if needed. The scope of our services did not include an assessment of the effects of flooding and natural erosion on the project site. No liquefaction studies were performed. This study is based on the condition that soil will not liquefy.

This Report is copy-righted and was prepared for the exclusive use of the owner, architect, and engineer to evaluate the project's design related to the ground response discussed in this Report.

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APPENDIX A. Site Location

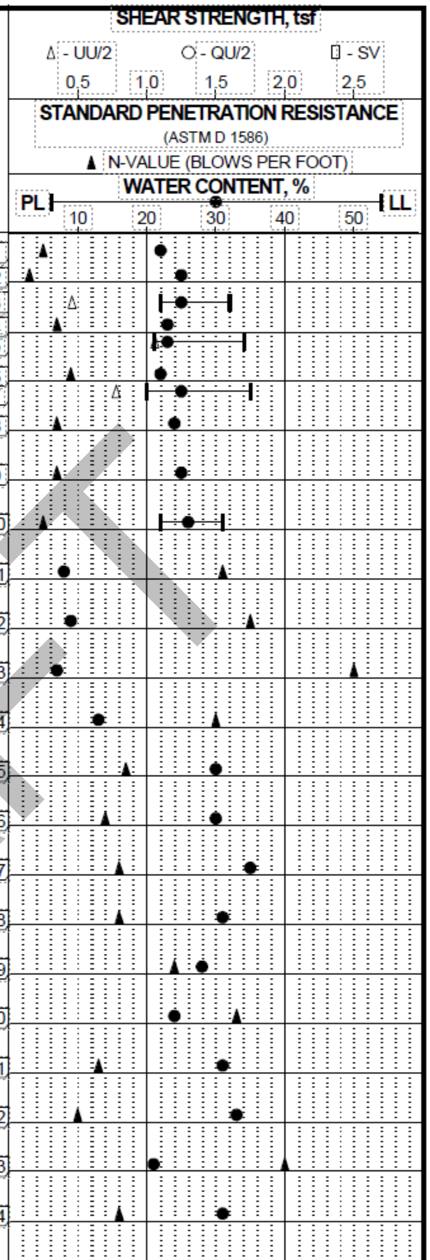
Figure A.1. The Location of the Study Site.

APPENDIX B. Boring Log

Surface Elevation: ______ Completion Date: **4/3/23**

Datum **N/A**

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) / SPT BLOW COUNTS / CORE RECOVERY/RQD	SAMPLES
	Topsoil: 4 inches Soft to very stiff, brown to gray, silty, LEAN CLAY - (CL)	1-2-3	SS1
		3-2-1	SS2
5	97% passing No. 200 sieve	106	ST3
		2-3-4	SS4
10	91% passing No. 200	103	ST5
		3-4-5	SS6
15	96% passing No. 200 sieve	99	ST7
20	pH = 7.53	2-3-4	SS8
25		2-3-4	SS9
30		2-2-3	SS10
35	Dense, orange GRAVEL, some sand - GP pH = 7.13	9-15-16	SS11
40		4-15-20	SS12
45	resistivity = 6,840 ohms-cm	8-29-21	SS13
50	Medium dense, orange SAND, some gravel - SP	24-17-13	SS14
55	Medium dense to dense, gray to tan and orange SAND, trace silt - (SP-SM)	6-7-10	SS15
60	9% passing No. 200 sieve	10-6-8	SS16
65		7-8-8	SS17
70		9-8-8	SS18
75		11-11-13	SS19
80		17-14-19	SS20
85		12-6-7	SS21
90	Stiff, gray, FAT CLAY - CH	4-4-6	SS22
95	Hard, gray, sand, FAT CLAY - CH	7-19-21	SS23
100	Very stiff, gray, FAT CLAY - CH Boring terminated at 100 feet.	6-7-9	SS24



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

AUGER 3 3/4" HOLLOW STEM

WASHBORING FROM 30 FEET

JCG DRILLER RSP LOGGER

Diedrich D-50 DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 93 %

REMARKS:

Drawn by: RSP	Checked by:	App'vd. by:
Date: 4/6/23	Date:	Date:

GEOTECHNOLOGY
A UES Company

I-55/SR-14 Bridge Replacement and Interchange Improvement Memphis, Tennessee

LOG OF BORING: B-2

Project No. J042144.01

LOG OF BORING 2020 JDM J042144.01.GPJ GTINC 0638301.GPJ 6/6/23