

ARKANSAS DEPARTMENT OF TRANSPORTATION



SUBSURFACE INVESTIGATION

STATE JOB NO. 101017

FEDERAL AID PROJECT NO. ACNHPP-0056(57)

TYRONZA RIVER & DITCH NO. 6 STRS. & APPRS. (S)

STATE HIGHWAY 118 SECTION 2

IN POINSETT COUNTY

The information contained herein was obtained by the Department for design and estimating purposes only. It is being furnished with the express understanding that said information does not constitute a part of the Proposal or Contract and represents only the best knowledge of the Department as to the location, character and depth of the materials encountered. The information is only included and made available so that bidders may have access to subsurface information obtained by the Department and is not intended to be a substitute for personal investigation, interpretation and judgment of the bidder. The bidder should be cognizant of the possibility that conditions affecting the cost and/or quantities of work to be performed may differ from those indicated herein.



ARKANSAS DEPARTMENT OF TRANSPORTATION

ArDOT.gov | IDriveArkansas.com | Scott E. Bennett, P.E., Director

MATERIALS DIVISION

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December 23, 2019

TO: Mr. Trinity Smith, Engineer of Roadway Design

SUBJECT: Job No. 101017
Tyronza River Str. & Apprs. (S)
Route 118 Section 2
Poinsett County

Based on soil information from projects in the surrounding area, an estimated R-Value of less than five is appropriate for pavement design.

Listed below is the additional information requested for use in developing the plans:

Asphalt Concrete Hot Mix

| Type | Asphalt Cement % | Mineral Aggregate % |
|----------------|------------------|---------------------|
| Surface Course | 5.0 | 95.0 |
| Binder Course | 4.1 | 95.9 |
| Base Course | 3.9 | 96.1 |



Michael C. Benson
Materials Engineer

MCB:pt:bjj
Attachment

cc: State Constr. Eng. – Master File Copy
District 10 Engineer
System Information and Research Div.
G. C. File

GEOTECHNOLOGY

A UES Company



**GEOTECHNICAL REPORT
TYRONZA RIVER & DITCH No. 6
STRS. & APPRS. (S)
BRIDGE 1 – HWY. 118 OVER
TYRONZA RIVER
POINSETT COUNTY, ARKANSAS**

**ARKANSAS DEPARTMENT OF TRANSPORTATION
STATE PROJECT No. 101017**

Prepared for:
**ARKANSAS DEPARTMENT OF TRANSPORTATION (ARDOT)
LITTLE ROCK, ARKANSAS**

Prepared by:
**GEOTECHNOLOGY, LLC
MEMPHIS, TENNESSEE**

Date:
FEBRUARY 9, 2024

Geotechnology Project No.:
J042992.01

**SAFETY
QUALITY
INTEGRITY
PARTNERSHIP
OPPORTUNITY
RESPONSIVENESS**



February 9, 2024

Mr. Paul Tierney
Geotechnical Engineer
Arkansas Department of Transportation (ARDOT)
PO Box 2261
Little Rock, Arkansas 72203

Re: Geotechnical Report
Tyronza River & Ditch No. 6 Strs. & Apprs. (S)
Bridge 1 - Hwy. 118 Over Tyronza River
Poinsett County, Arkansas
ARDOT Project No. 101017
Geotechnology Project No. J042992.01

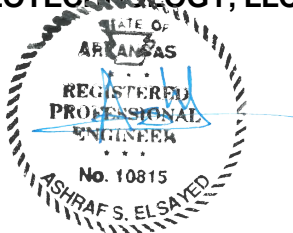
Dear Mr. Tierney:

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, conclusions and/or recommendations, and support 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



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

Jacob Monroe, P.E.
Project Engineer

JDM/ASE/DBA:jdm

Copies submitted: Client (email)



TABLE OF CONTENTS

| | |
|---|----|
| 1.0 Scope of Services..... | 1 |
| 2.0 General Information | 1 |
| Planned Modifications..... | 1 |
| Topography | 2 |
| Drainage | 2 |
| Geology | 2 |
| 3.0 Geotechnical Exploration | 2 |
| Cone Penetration Testing | 2 |
| Drilling and Sampling..... | 2 |
| 4.0 Laboratory Review and Testing | 3 |
| 5.0 Subsurface Conditions..... | 4 |
| Subgrade Materials..... | 4 |
| Creek Bed Material Mean Grain Size | 4 |
| Groundwater..... | 5 |
| 6.0 Engineering Evaluation, Analysis, and Recommendations | 5 |
| Site Preparation and Earthwork | 5 |
| Seismic Considerations | 7 |
| AASHTO LRFD 2020 Seismic Site Classification and Seismic Design | |
| Parameters | 7 |
| Site-Specific Ground Motion Assessment..... | 7 |
| Liquefaction and Dynamic Settlement..... | 8 |
| Approach Embankment Settlement | 9 |
| Global Stability..... | 10 |
| Deep Foundations | 11 |
| Downdrag | 15 |
| Corrosion Potential | 16 |
| 7.0 Recommended Additional Services | 17 |
| 8.0 Limitations..... | 17 |

Appendices

- Appendix A – Important Information about This Geotechnical-Engineering Report
- Appendix B – Figures
- Appendix C – Boring Information
- Appendix D – CPT-1 Sounding Plot
- Appendix E – Laboratory Test Data
- Appendix F – Site-Specific Seismic Study
- Appendix G – AASHTO and USCS Classifications
- Appendix H – Global Stability Analyses
- Appendix I – Soil Parameters for Synthetic Profiles
- Appendix J – Nominal Resistance Curves



LIST OF TABLES

| | |
|--|----|
| Table 1. Field Tests and Measurements..... | 3 |
| Table 2. Summary of Laboratory Tests and Methods. | 4 |
| Table 3. Approximate Mean Grain Size (D_{50}) of Creek Bed Material. | 5 |
| Table 4. Seismic Design Parameters (7% Probability of Exceedance in 75 years)..... | 7 |
| Table 5. Design Acceleration Parameters (7% Probability of Exceedance in 75 Years). | 8 |
| Table 6. Results of Liquefaction Analyses. | 9 |
| Table 7. Summary of Estimated Settlement..... | 10 |
| Table 8. Results of Slope Stability Analyses. | 11 |
| Table 9. Nominal Axial Resistance of Driven Closed-Ended Pipe Piles..... | 12 |
| Table 10. Resistance Factors Based on Static Analysis Methods. | 13 |
| Table 11. Resistance Factors for Driven Piles. | 13 |
| Table 12. Results of pH and Soil Resistivity Testing. | 16 |

GEOTECHNICAL REPORT
TYRONZA RIVER & DITCH NO. 6 STRS. & APPRS. (S)
BRIDGE 1 - HWY. 118 OVER TYRONZA RIVER
POINSETT COUNTY, ARKANSAS
February 9, 2024 | Geotechnology Project No. J042992.01

1.0 SCOPE OF SERVICES

Presented in this report are the results of the geotechnical exploration and recommendations for design and construction of the proposed replacement of the existing Bridge No. A3872 over Tyronza River along Highway 118 (Hwy. 118) in Poinsett County, Arkansas. The referenced project includes the construction of a new bridge to replace the existing Bridge No. A3872. It is our understanding the anticipated foundation type for support of the new bridge will be driven 18-inch-diameter, closed-ended pipe piles at the external (abutment) bents and interior bents as provided by ARDOT. The project location is shown on Figure 1 included in Appendix B.

The recommendations presented in this report are based on the geology, provided plans and project information, and the results of the geotechnical exploration. Results of the borings, Cone Penetration Test (CPT) sounding, in-situ testing, sampling, and laboratory testing are included in the report. A total of six borings and one seismic CPT sounding were performed in the vicinity of the site as shown on Figure 2 included in Appendix B. The boring logs and CPT sounding plot, along with field and laboratory test results, are enclosed. The collected data have been analyzed and the physical properties of the in-situ soils summarized. General site conditions are discussed, along with recommendations for subgrade preparation. 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

Planned Modifications

The existing two-lane, approximately 277.5-foot-long, 30-foot-wide, five-span Hwy. 118 Bridge No. A3872 over Tyronza River will be replaced by a two-lane, 332-foot-long, 32.5-foot-wide, three span bridge. It is our understanding the existing bridge will be demolished to facilitate construction of the replacement bridge and the centerline of the bridge will remain unchanged.

The abutment slopes are anticipated to be two horizontal units for every vertical unit (2H:1V) and side slopes are anticipated to be 2H:1V. Up to 8 feet of cut and 2 feet of fill will be required at the bridge abutments to achieve design grades.



Topography

The proposed Hwy. 118 bridge replacement is located in Poinsett County, Arkansas. According to the provided plans¹, existing elevations at the south and north abutments of the proposed bridge location are approximately El 222.5 which slope down to the bottom of Tyronza River at approximately El 190.

Drainage

Drainage at the site consists of Tyronza River which is aligned east-west. The drainage system in the project area consists of the Lower St. Francis Watershed. The Lower-St. Francis Watershed, in turn, is part of the overall drainage system of the St. Francis River Basin.

Geology

Poinsett County is located in northeast Arkansas in the Mississippi Embayment. The Mississippi Embayment is a trough-like depression dipping southward along an axis approximately following the Mississippi River. Site geology generally consists of Holocene-aged alluvial deposits of clay and silt underlain by fine-grained sand.

3.0 GEOTECHNICAL EXPLORATION

Cone Penetration Testing

One seismic cone penetration testing (CPT) sounding was performed in the proposed bridge northern approach for continuous soil data collection and to measure the average shear wave velocity. The CPT sounding was performed to a depth of approximately 100 feet.

CPT-1 was advanced using a 20-ton, track-mounted Vertek direct-push rig on May 3, 2023. The data were collected using a Vertek 15 square-centimeter end area, seismic piezometric cone with a u_2 pore pressure location (behind the cone) following the procedures outlined in ASTM D3441 and D5778. A plot of the CPT measurements is presented in Appendix D along with interpreted soil behavior types. Seismic CPT tests were performed in CPT-1 at approximately 1-meter intervals to collect shear wave velocity data. A plot of the shear wave velocity profile is presented in Figure 3 in Appendix B.

Drilling and Sampling

A total of six borings were drilled to approximate depths of 50 to 100 feet at select locations in the proposed bridge approaches and through the existing bridge deck. The borings were drilled June 11 through 22 and July 11, 2023 using rotary drill rigs (CME 550X and Diedrich D-50), hollow-stem augers, and wet rotary methods. Sampling procedures included Standard Penetration Test (SPT) and thin-wall (Shelby) tube methods. SPT's were conducted at 2.5, 5, and 10-foot depth intervals

¹ *Arkansas Department of Transportation, Construction Plans for State Highway, Tyronza River & Ditch No. 6 Strs. & Apprs. (S), Poinsett County, Route 118, Section 2, Job 101017, Fed. Aid Proj. NHPP-0056(57). Provided by Garver USA, dated September 14, 2023.*



using an automatic hammer. Thin-walled Shelby tube samples were collected in cohesive soils at selected depths. Groundwater observations were made during drilling operations.

The collected samples were visually examined by field staff and transported to our laboratory for further evaluation and testing. The samples were examined in the laboratory by a geotechnical professional who prepared descriptive logs of the materials encountered. The boring logs are presented in Appendix C along with an explanation of the terms and symbols used on the boring logs. Included on each boring log are elevation data estimated from the provided plans. Included 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 |

The boring logs and CPT sounding plot represent conditions observed at the time of exploration and have been edited to incorporate results of the laboratory tests. Unless noted on the boring logs, the lines designating the changes between various strata represent approximate boundaries. The transition between materials could be gradual or 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.

The boring logs and CPT sounding plot 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. Most of the laboratory test results are presented on the boring logs in Appendix C. The Atterberg limits, grain size analyses, percent finer than No. 200 sieve (hydrometer), unconsolidated-undrained triaxial compression (UU), one-dimension consolidation, consolidated-undrained triaxial compression (CU), pH, and soil resistivity test results are also provided in Appendix E. The laboratory tests and corresponding test method standards are presented in Table 2.



Table 2. Summary of Laboratory Tests and Methods.

| Laboratory Test | ASTM | AASHTO |
|---|--------|--------|
| Moisture Content | D 2216 | T 265 |
| Atterberg Limits | D 4318 | T 98 |
| Grain Size Analysis | D 422 | T 88 |
| Percent Finer Than No. 200 Sieve | D 1140 | T 11 |
| Unconsolidated-Undrained Triaxial Compression | D 2850 | T 296 |
| One-Dimensional Consolidation | D 2435 | T 216 |
| Consolidated-Undrained Triaxial Compression | D 4767 | T 297 |
| pH of Soil | D 4972 | T 289 |
| Soil Electrical Resistivity | G 57 | T 288 |

The boring logs and CPT1-1 plot were prepared by a project geotechnical engineer from the field logs, visual classification of the soil samples in the laboratory, 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 strata could be abrupt or gradual.

5.0 SUBSURFACE CONDITIONS

Subgrade Materials

Borings B1-1 through -3, -5, and -6 were performed in the alignment of the existing and proposed bridge replacement. Boring B1-4 and CPT1-1 were performed adjacent to an interior bent of the existing bridge. Boring B1-3 was drilled through the existing bridge deck and mudline of Tyronza River. Surficial material at Borings B1-1, -2, -5, and -6 consisted of approximately 7 inches of asphalt pavement; at Boring B1-4, surficial material consisted of topsoil. Below the surficial material in Borings B-1, -2, and -4 through -6, at the ground surface at CPT1-1, and below the mudline at Boring B1-3, the soils generally consisted of predominately fine-grained soils overlying coarse-grained soils that extended to the boring and CPT termination depths. The boring logs and CPT sounding plot, with more detailed descriptions, are included in Appendix C and D, respectively. Laboratory testing was used to determine the AASHTO classifications as presented in Appendix G.

The upper, fine-grained soils were classified as high plasticity “fat” clay (CH), A-7-6; high plasticity silt (MH), A-7-5; and low plasticity “lean” clay (CL), A-6. The fine-grained soils ranged from soft to stiff in consistency.

The lower, coarse-grained soils were classified as poorly graded sand (SP), A-3, A-1-b; poorly graded sand with clay (SP-SC), A-2-6; poorly graded sand with silt (SP-SM), A-3; and clayey sand (SC), A-2-6, A-6. The coarse-grained soils ranged from loose to dense conditions.

Creek Bed Material Mean Grain Size

We understand mean grain size (D_{50}) information is required for scour analyses to be performed by others. Grain size analyses were performed on select samples recovered in the upper 15 feet of the



borings drilled into the existing creek bed. The approximate mean grain size results are presented in Table 3.

Table 3. Approximate Mean Grain Size (D_{50}) of Creek Bed Material.

| Boring | Sample Depth (feet below ground surface) | Approximate Mean Grain Size, D_{50} (mm) |
|--------|---|--|
| B1-3 | 3.5 | 0.0058 |
| B1-3 | 6 | 0.01 |
| B1-3 | 13.5 | 0.35 |
| B1-4 | 10 | 0.25 |

Groundwater

Groundwater was encountered in Borings B1-1 and -4 through -6 at depths ranging from 10 to 30 feet below existing ground surface, correlating to groundwater elevations of El 185 to 195. Groundwater was interpreted at approximately 10 feet below existing ground surface, correlating to a groundwater elevation of approximately El 194 based on porewater pressure recordings. Groundwater levels were not encountered in Boring B1-2 and -3; however, groundwater levels may have been masked by the use of wet-rotary drilling methods in Boring B1-2.

Groundwater levels will vary over time due to the effects of seasonal variations in precipitation, the stage and recharge rate of Tyronza River, or other factors not evident at the time of exploration.

6.0 ENGINEERING EVALUATION, ANALYSIS, AND RECOMMENDATIONS

Site Preparation and Earthwork

The following procedures are recommended for site preparation in cut and fill areas. These recommendations do not supersede ARDOT standards and specifications. Site preparation and compaction requirements must conform to the latest ARDOT standards.

Site Preparation. In general, cut areas and areas to receive new fill should be stripped of existing pavements, topsoil, vegetation, and other deleterious materials. Topsoil should be placed in landscape areas or disposed of off-site. Vegetation and tree roots should be over-excavated.

The exposed subgrade should be proof-rolled using a tandem axle dump truck loaded to approximately 20,000 pounds per axle (or equivalent proof-rolling equipment). Soft areas that develop should be over-excavated and backfilled with select fill, which is defined as soil conforming to A-4 or better material, and compacted to the unit weights specified in subsequent paragraphs.

Side Slopes. Existing slopes steeper than 4H:1V should be benched prior to placing new fill. Slope ratios of 2H:1V are proposed for abutment slopes. Slope ratios of 3H:1V or flatter are recommended for all cut and fill side slopes along the proposed alignment.



Cut Areas. It is our understanding up to 8 feet of cut will be required at the abutments. Based on the stratigraphy, excavations for pile cap foundations will terminate in lean clay or fat clay. After excavation, the top 6 inches of the resulting subgrade should be compacted to a minimum of 95% of the maximum dry unit weight as determined by a standard Proctor test (ASTM D698/AASHTO T 99). Areas supporting pavement should be compacted to 98% of the maximum unit weight as determined by the standard Proctor test.

Fill Materials. It is our understanding up to 2 feet of fill will be required at the approach embankments. Fill material should consist of natural soils classifying as AASHTO A-6 or better², and should meet the minimum requirements set forth in ARDOT's Special Provision³ (SP) dated March 1, 2022. Soils classifying as AASHTO A-4 or better are considered to be select fill. Fine-grained "silt-clay" soils (A-4 through A-6) should have a maximum LL of 45 and a PI between 8 and 20 percent. Coarse-grained "sandy" soils used for embankment fills should have a minimum PI of 5 to lower potential for erosion. Fill materials should be free from organic matter, debris, or other deleterious materials, and have a maximum particle size of 2 inches.

Fill and Backfill Placement. Fill and backfill should be placed in level lifts, up to 8 inches in loose thickness. For fill and backfill exhibiting a well-defined moisture-density relationship, each lift should be moisture-conditioned to within $\pm 2\%$ of the optimum moisture content and compacted with a sheepsfoot roller or self-propelled compactor to a minimum of 98% of the maximum dry unit weight as determined by the standard Proctor test. Moisture-conditioning can include: aeration and drying of wetter soils; wetting drier soils; and/or mixing wetter and drier soils into a uniform blend. The upper 3 feet of soil beneath the base of pavement should be compacted to 98% of the maximum unit weight as determined by the standard Proctor test.

For fill and backfill that do not exhibit a well-defined moisture-density relationship, each lift should be compacted to a 70% of the minimum relative density as evaluated from the maximum and minimum index densities measured by ASTM D4253 and D4254, respectively. The upper 3 feet of soil beneath the base of pavement should be compacted to 75% of the minimum relative density.

Fill Placement on Slopes. In areas that require fill to be placed on slopes, benching of existing slopes should be performed during placement of new fill. Fill on the sloped areas should begin from the toe of the slope and proceed upward, placing new fill on horizontal benches. Bench shelves should be 8 to 10 feet wide, and bench faces should be 1 to 2 feet in height. Fill lifts should be keyed into the slope to reduce the potential of a slip plane between the new fill and existing soils. Fill slopes should be constructed by extending the compacted fill beyond the planned profile of the slope and then trimming the slope to the desired configuration.

Moisture Considerations. Maintaining the moisture content of bearing and subgrade soils within the acceptable range is important during and after construction. Silty and clayey subgrade soils should

² A-6 soils or better as determined by ARDOT.

³ Special Provision "Compacted Embankment", developed by ARDOT, dated March 1, 2022.



not be allowed to become wet or dry during or after construction, and measures should be taken to hinder water from ponding on these soils. Positive drainage should be established to promote drainage of surface water away from the roadway.

Seismic Considerations

Earthquake Risk. The project area is located in the vicinity of 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 earthquake occurred, which were centered near New Madrid, Missouri. Three strong earthquakes occurred over the next three months 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 the bridge and approaches will be designed in accordance with the AASHTO publication “LRFD Bridge Design Specifications”, eighth edition (2020).

AASHTO LRFD 2020 Seismic Site Classification and Seismic Design Parameters

Seismic Design Parameters. Seismic design parameters based on a seismic hazard with 7% probability of exceedance in 75 years and field and laboratory testing is presented in Table 4.

Table 4. Seismic Design Parameters (7% Probability of Exceedance in 75 years).

| Latitude 35.509645°N/Longitude 90.358202°W | | |
|--|-----------------------|---|
| Category/ Parameter | Designation/ Value | Reference |
| Seismic Zone | 4 | AASHTO LRFD 2020 Table 3.10.6-1 |
| Seismic Site Class | D | AASHTO LRFD 2020 Table 3.10.3.1-1 |
| S _s | 1.775g | Ground motion parameters obtained from a computer program supplied with the AASHTO Guideline for the Seismic Design of Highway Bridges (2020) using the indicated latitude and coordinates of the project site and the seismic site class based on boring data. |
| S ₁ | 0.477g | |
| F _a | 1.000 | |
| F _v | 1.523 | |
| F _{PGA} | 1.000 | |
| t _s | 0.409 | |
| t ₀ | 0.082 | |
| S _{DS} | 1.775g | |
| S _{D1} | 0.726g | |
| PGA | 1.003g | |
| A _s | 1.003g | |

Site-Specific Ground Motion Assessment

A site-specific study was performed for the project site to develop a site-specific seismic design response spectrum. The process included 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 CPT1-1 of 767 feet per second as shown on Figure 3 (Shear Wave Velocity Profile) in Appendix B.

The results of the shear wave velocity measurements indicate that the site is a Site Class D, “stiff soil”, profile based on a $V_{S,100}$ of 767 feet per second. According to the results of the site-specific seismic study, the recommended site-specific design accelerations are presented in Table 5.

Table 5. Design Acceleration Parameters (7% Probability of Exceedance in 75 Years).

| Parameter | Value |
|-----------|--------|
| S_{DS} | 1.183g |
| S_{D1} | 0.972g |
| MCE_G | 0.669g |

Liquefaction and Dynamic Settlement

A study was performed to evaluate the liquefaction and dynamic settlement potential at the site. Both field and laboratory data were used to perform the analysis. The field measurements included the depth of the water table, SPT N-values, and information collected in CPT1-1. The laboratory data included USCS classification and soil unit weight. An earthquake magnitude (M_w) of 7.7 with a probability of exceedance of 7% in 75 years was considered. A site peak ground acceleration of 0.669g was utilized as obtained from the site-specific seismic study. Groundwater was set at an elevation of approximately El 190 as indicated in the CPT and borings.

Subsurface conditions (as characterized by the field and laboratory data) and earthquake characteristics were used to estimate the safety factors against liquefaction in each soil layer, as well as the associated dynamic settlement during the design seismic event. Based on the analysis, there is liquefaction potential at the site. The analysis results are presented in Table 6.



Table 6. Results of Liquefaction Analyses.

| Location | Depth of Boring (feet) | Liquefiable Zones ^a (feet below ground surface) | Estimated Dynamic Settlement (inches) | |
|----------|------------------------|---|---------------------------------------|------------------------------------|
| | | | Upper 50 Feet | Total Depth of Boring/CPT Sounding |
| B1-1 | 50 | 33.5 – 38.5 | 1 | 1 |
| B1-2 | 100 | N/A ^b | N/A ^b | N/A ^b |
| B1-3 | 100 | 13.5 – 18.5 | 1 | 1 |
| B1-4 | 100 | 28.5 – 33.5 | 1½ | 1½ |
| B1-5 | 100 | 28.5 – 38.5 | 4 | 4 |
| B1-6 | 50 | 28.5 – 43.5 | 3 | 3 |
| CPT1-1 | 100 | 13 – 31 47 - 60 | 5 | 8½ |

^a Defined as zones with a factor of safety against liquefaction in the design earthquake of less than 1.0, as computed using the deterministic method by Idriss and Boulanger, 2014.

^b Liquefaction and dynamic settlement not anticipated in Boring B-2.

Please note that the current state of practice for liquefaction hazard assessment is based on what is known as “The Simplified Method” as introduced by Seed (1971) and subsequent modifications/revisions by others (Seed 1982, Idriss 1999, Youd 2001, and Idriss and Boulanger 2008, among others). The simplified method was based on observations and assessments of soil zones that either liquefied or did not liquefy in the upper 50 feet. Because of reported uncertainties in the literature, the occurrence of substantial liquefaction in relatively deep sand deposits below 50 feet is considered unlikely.

Lateral Spreading. Lateral spreading is triggered and sustained by earthquake ground motions. Based on our seismic slope stability analyses, it is our professional opinion the potential for lateral spreading is low at the site. However, in the event of an earthquake, the soils that liquefy will have residual strengths which can have potentially destabilizing effects on overlying slopes. Post-liquefaction residual shear strength parameters are presented in Appendix I.

Approach Embankment Settlement

Settlement analyses of natural soils were performed to assess fill-induced settlement for the approaches. Based on the provided preliminary plans, up to approximately 2 feet of fill will be required at the southern and northern approaches, to bring the site to design grade. For settlement analyses, we have assumed cohesive, engineered fill will be used for the fill material. The results of the settlement due to fill placement are shown in Table 7. If grade changes will require the placement of additional fill, Geotechnology should be contacted to perform additional settlement analyses for fill-induced settlement at the approaches.



Table 7. Summary of Estimated Settlement.

| Southern Abutment (Exterior Bent No. 1) | | | | Northern Abutment (Exterior Bent No. 4) | | | |
|--|----------------------------------|------------------------------|-------|--|----------------------------------|------------------------------|-------|
| Max Fill (feet) | Estimated Settlement (inches) | | | Max Fill (feet) | Estimated Settlement (inches) | | |
| | Immediate | Long-Term (Consolidation) | Total | | Immediate | Long-Term (Consolidation) | Total |
| 2 | ½ | ½ | 1 | 2 | ½ | ½ | 1 |

The bent numbers presented in Table 7 are in reference to the bent number designations presented on the provided preliminary plans. Based on review of the preliminary plans, the bents are numbered from 1 to 4 such that Bent No. 1 is at the southern abutment.

Discussion of Fill-Induced Settlement. The results of the settlement analyses indicate immediate and long-term (primary) consolidation settlement at the approaches. We anticipate the immediate settlement to occur shortly after fill placement and practical completion of consolidation to occur within 2 to 4 weeks after fill placement.

Global Stability

Geotechnology performed stability analyses for deep-seated, global failure of bridge abutment slopes using the computer program SLIDE2. Short-term, long-term, and seismic conditions were considered using the Spencer method to compute factors of safety for the proposed slopes.

Calculated minimum factors of safety are summarized in the following table. Minimum required factors of safety for the proposed bridge were based on the ARDOT Minimum Acceptable Factors of Safety as provided by ARDOT using a seismic operational class of “Other”. A pseudo-static seismic acceleration of 0.335g, corresponding to one-half the peak ground acceleration (per FHWA Publication HI-99-012) was utilized.

A groundwater elevation of approximately El 195 feet, as measured in the borings and interpreted in CPT1-1, was utilized for the short-term and seismic condition analyses and a design highwater elevation of El 214.1, as obtained from the plans provided by Garver, was used for the long-term condition analyses. Section profiles with critical slip surfaces and utilized soil parameters are presented in Appendix H for the selected analyses. The analysis models did not consider the effect of foundation piles driven at the abutments or riprap placed on the abutment slopes that would provide additional restraining force to stabilize the slopes.



Table 8. Results of Slope Stability Analyses.

| Location | Description | Slope Height (ft.) | Calculated Factor of Safety | | |
|------------------------------------|-------------|--------------------|----------------------------------|---------------------------------|------------------------|
| | | | Short-Term Static ^{a,c} | Long-Term Static ^{a,d} | Seismic ^{b,c} |
| Southern Abutment STA 150+66.42 | 2H:1V | 24 | 3.27 | 1.94 | 1.12 |
| | Cut Slope | | | | |
| Northern Abutment STA 153.98.58 | 2H:1V | 24 | 3.36 | 1.99 | 1.18 |
| | Cut Slope | | | | |

- ^a Target factor of safety = 1.3, approximately equivalent to a global stability resistance factor = 0.75, as provided by ARDOT.
- ^b Target factor of safety = 1.1, approximately equivalent to a global stability resistance factor = 0.9, as provided by ARDOT.
- ^c Based on a groundwater elevation of approximately El 195; approximately 25 feet below existing ground surface at the abutments.
- ^d Based on a Tyronza River design highwater elevation of El 214.1 as obtained from the preliminary plans provided by ARDOT.

Fill material used for construction of the embankments will be required to meet the criteria established in the Special Provision dated March 1, 2022 provided by ARDOT.

Deep Foundations

Foundation design recommendations are provided herein based on the AASHTO LRFD Bridge Design Specifications (2020).

It is our understanding the proposed foundation type for the bridge will be driven, 18-inch diameter, closed-ended pipe piles as provided by ARDOT. Geotechnology should be notified if other foundation types or sizes are to be considered. Based on the provided information, we have assumed existing ground surface elevations of approximately El 220 for the exterior bents and existing ground surface elevations of El 198 and El 204 for interior Bents 2 and 3, respectively. Soil parameters, including LPILE lateral load analysis parameters, for each bent are included in Appendix I.

Nominal resistance curves showing axial resistance from skin friction and total axial capacity (skin friction + end bearing) for Bents 1 through 4 are presented in Appendix J. Nominal static resistances at each bent for the driven, closed-ended pipe piles are presented in Table 9. Uplift (tension) capacities may be calculated using the resistance provided by skin friction.

Also presented in Table 9 are nominal post-liquefaction resistances and nominal drag loads on piles. For the purposes of the post-liquefaction pile resistance analyses, liquefaction was considered to be limited to the upper 50 feet of the soil profile at the bents. Nominal post-liquefaction resistance curves are presented in Appendix J.



Table 9. Nominal Axial Resistance of Driven Closed-Ended Pipe Piles.

| Location | Pile Diameter (inches) | Embedment Length (feet) | Nominal Static Resistance (tons) | | | Nominal Post-Liquefaction Resistance ^d (tons) | |
|--|------------------------|-------------------------|----------------------------------|-------------|-------------------|--|-------------------|
| | | | Skin Friction | End Bearing | Compression Total | Compression Total ^e | Nominal Drag Load |
| Exterior (Abutment) Bents 1 and 4 | | | | | | | |
| Bent 1 ^a (South Abutment) | 18 | 60 | 178 | 153 | 331 | N/A ^f | |
| | | 70 | 250 | 172 | 422 | | |
| | | 80 | 330 | 177 | 507 | | |
| Bent 4 ^a (North Abutment) | | 60 | 211 | 177 | 388 | 343 | 45 |
| | | 70 | 286 | 177 | 463 | 418 | |
| | | 80 | 370 | 177 | 547 | 502 | |
| Interior Bents 2 and 3 | | | | | | | |
| Bent 2 ^b | 18 | 60 | 187 | 125 | 312 | 290 | 9 |
| | | 70 | 251 | 176 | 427 | 406 | |
| | | 80 | 324 | 177 | 501 | 479 | |
| | 24 | 60 | 249 | 211 | 460 | 431 | 12 |
| | | 70 | 335 | 313 | 648 | 619 | |
| | | 80 | 432 | 314 | 746 | 717 | |
| | 30 | 60 | 311 | 319 | 630 | 486 | 15 |
| | | 70 | 419 | 488 | 907 | 695 | |
| | | 80 | 540 | 491 | 1031 | 818 | |
| Bent 3 ^c | 18 | 60 | 182 | 147 | 329 | 267 | 47 |
| | | 70 | 244 | 138 | 382 | 321 | |
| | | 80 | 310 | 142 | 452 | 392 | |
| | 24 | 60 | 242 | 260 | 502 | 421 | 63 |
| | | 70 | 326 | 249 | 575 | 493 | |
| | | 80 | 414 | 253 | 667 | 586 | |
| | 30 | 60 | 302 | 407 | 709 | 613 | 78 |
| | | 70 | 407 | 392 | 799 | 703 | |
| | | 80 | 507 | 395 | 912 | 816 | |

^a Embedment length referenced from approximate ground surface elevation of El 220.

^b Embedment length referenced from approximate ground surface elevation of El 198.

^c Embedment length referenced from approximate ground surface elevation of El 204.

^d For the purposes of downdrag analyses, liquefaction-induced downdrag has been limited to the upper 50 feet of embedment. Liquefiable layers below 50 feet not considered for downdrag analyses.

^e Nominal post-liquefaction resistance has not been reduced by the drag load.

^f Liquefaction not anticipated at Bent 1; downdrag not considered.



Resistance Factors. Resistance factors should be applied to the nominal resistances provided. Based solely on the static analysis methods used to calculate nominal pile resistances, the factors presented in Table 10 may be applied.

Table 10. Resistance Factors Based on Static Analysis Methods.

| Deep Foundation and Condition | Clay | | Sand | |
|---|-----------------|-------------|-----------------|-------------|
| | Side Resistance | End-Bearing | Side Resistance | End-Bearing |
| Nominal Compressive Resistance of Single Pile | 0.35 | 0.35 | 0.45 | 0.45 |
| Uplift Resistance of Single Pile | 0.25 | -- | 0.35 | -- |

Based on the AASHTO LRFD (2020) Table 10.5.5.2.3-1, a higher resistance factor can be used in accordance with the method of pile testing performed as indicated in Table 11.

Table 11. Resistance Factors for Driven Piles.

| Condition/Resistance Determination Method | | Resistance Factor |
|---|--|-------------------|
| Nominal Bearing Resistance of Single Pile – Dynamic Analysis and Static Load Test Methods | Driving criteria established by successful static load test of at least one pile per site condition and dynamic testing of at least two piles per site, but no less than 2% of the production piles* | 0.80 |
| | Driving criteria established by successful static load test of at least one pile per site condition without dynamic testing | 0.75 |
| | Driving criteria established by dynamic testing conducted on 100% of production piles* | 0.75 |
| | Driving criteria established by dynamic testing, quality control by dynamic testing of at least two piles per site condition, but no less than 2% of production piles* | 0.65 |
| | Wave equation analysis, without pile dynamic measurements or load test but with field confirmation of hammer performance | 0.50 |
| | FHWA-modified Gates dynamic pile formula (End of Drive condition only) | 0.40 |
| Uplift Resistance of Single Pile | Dynamic test with signal matching | 0.50 |

* Dynamic testing requires signal matching, and estimates of nominal resistance are made from a restrrike. Dynamic tests are calibrated to a static load test, when available.



Pile Group Considerations. The settlement of pile groups should be evaluated as per AASHTO LRFD (2020) section 10.7.2.3. Settlement analysis of the pile groups can be performed when the foundation configurations and service loads are available. AASHTO LRFD (2020) section 10.7.3.9 addresses pile group resistance. Group capacity considerations for different pile groups, center-to-center spacings, and other conditions (cap contact with ground, softness of surface soil etc.) are given in AASHTO LRFD (2020) sections 10.7.3.9 and 10.7.3.11.

Driven Pile Construction Considerations. Minimum hammer energies required to drive the piles were not evaluated for the proposed foundations. If minimum hammer energy evaluations are required, Geotechnology should be contacted to perform analyses for the required minimum hammer energies for driving piles.

Static Pile Load Testing. At least one static pile compression load test should be performed for each bent or abutment location. The testing should be performed in accordance with ASTM D 1143 using the quick loading procedure and AASHTO LRFD (2020) section 10.7.3.8.2. Please refer to the previous Resistance Factors table for additional guidance regarding the minimum number of tests and alternate resistance factors associated with other field methods for determining resistance.

If the piles are to support net uplift loads, at least one tension load test should be performed for each location. The test should be performed in accordance with ASTM D 3689. Piles should be tested to the required nominal uplift resistances.

Load tests are required to verify recommended nominal pile resistance and will not be used to increase the design pile resistance. The piles used in the load tests should not be used for support of any structures. Geotechnology should be consulted regarding the locations of the test piles.

Dynamic Testing of Driven Piles. As an alternative to static pile load testing, high-strain dynamic pile testing can be performed according to AASHTO LRFD (2020) section 10.7.3.8.3 and the procedures given in ASTM D4945. Different resistance factors correspond to different load testing combinations as illustrated in the previous table. We recommend that the test piles be identified according to AASHTO LRFD (2020) Table 10.5.5.2.3-1 or 2 percent of the production piles, whichever results in a larger number of tests. We recommend that the identified piles be tested at the end of initial drive (EOID) and a restrike performed at a minimum seven days after EOID.

Pile driving monitoring should be performed by an engineer with a minimum 3 years dynamic pile testing and analysis experience and who has achieved Basic or better certification under the High-Strain Dynamic Pile Testing Examination and Certification process of the Pile Driving Contractors Association and Foundation QA. Pile driving modeling and analyses should be performed by an engineer with a minimum five years dynamic pile testing and analysis experience and who has achieved Advanced or better certification under the High-Strain Dynamic Pile Testing Examination and Certification process of the Pile Driving Contractors Association and Foundation QA.



Dynamic tests are required to monitor hammer and drive system performance, assess driving stresses and structural integrity and to evaluate pile resistance, and should not be used to increase design pile resistance. Dynamic tests should be performed on production piles with the lowest driving resistance. Geotechnology will be available to assist with development of specifications for this program and should be on site to perform or observe the testing and establish the pile driving criteria.

Settlement. Settlement of pile foundations depends on the loads applied and the foundation configuration. In general, settlement of deep foundations designed in accordance with the recommendations provided in this report is expected to be less than 1-inch. However, a calculation of the expected settlement of the pile foundations can be performed when the applied service loads and foundation configuration are available.

Uplift Resistance. Uplift forces can be resisted by the effective weight of the piles and caps, and frictional resistance between the piles and surrounding soil. If the anticipated maximum level of groundwater is higher than the tip of the pile then the buoyant unit weight of the pile must be used in computing uplift resistance for pile lengths extending below the design groundwater level.

Lateral Resistance. The lateral resistance of pile foundations depends on the lengths and dimensions of the foundations and the soil characteristics. The lateral resistance of pile foundations can be computed using the computer program LPILE to model the behavior of a single pile or shaft. Soil parameters are provided in Appendix I for the various strata and soil strengths present at the site. Soil parameters are based on field and laboratory test results and empirical correlations with SPT N-values.

The effects of group interaction must be considered when evaluating pile/shaft group horizontal movement. The lateral resistance for individual piles calculated by LPILE must be reduced by the P-multipliers provided in Section 10.7.2.4 of the AASHTO LRFD (2020) to determine lateral resistance of a pile group. Alternatively, the GROUP software can be used to evaluate the lateral resistance of the pile/shaft groups. The resistance factor for lateral resistance of single pile or pile group is 1.0.

Downdrag

The AASHTO LRFD (2020) suggests that soil settlement relative to a pile of 0.4-inch or greater could produce downdrag on pile foundations. Downdrag occurs as the soil strata move downward relative to foundations due to settlement of the soil layers. The relative movement of the soil layers versus the shaft depends on the final foundation configuration.

Downdrag Due to Fill-Induced Settlement. Based on settlement analyses performed for the 7-foot maximum fill placement at the abutments, up to 1 inch of settlement is predicted. The settlement due to fill placement is estimated to occur immediately after fill placement. Pile driving should not begin or at least 4 weeks following completion of fill placement to allow time for settlement to be essentially complete. Piles driven prior to completion of settlement may be subject to drag loads as the soil below the fill consolidates due to the weight of the fill.



Downdrag Due to Dynamic Settlement. Based on the results of the liquefaction analyses, up to 5 inches of dynamic settlement in the upper 50 feet of soil is anticipated during the design earthquake event (7% exceedance in 75 years) at the exploration locations. Additional analyses were performed to evaluate post-liquefaction pile resistances. The nominal post-liquefaction resistances and resistances are provided in Table 9. Pre-drilling and/or applying viscous coatings are not recommended to reduce liquefaction-induced downdrag because such methods will reduce the nominal static compressive resistance of the piles.

Liquefaction-induced downdrag can be reduced by performing ground improvement on the potentially liquefiable soil layers. It is our professional opinion that driving the 18-inch closed-ended pipe piles through the liquefiable layers will densify the susceptible layers in the vicinity of the piles; however, the extent of the densification cannot be determined at this stage. The extent of densification can be estimated by advancing CPT soundings in the vicinity of the piles after they are driven. If more information is desired, please contact Geotechnology.

Corrosion Potential

In addition to laboratory soil classification and strength testing, soil resistivity testing was also conducted. The purpose of soil resistivity testing is to provide soil data for use by a structural engineer for analysis of any necessary protection of the piling, concrete, reinforcing steel, etc. Corrosion and deterioration protection requirements and guidelines for piling are set forth in Section 10.7.5 of the AASHTO LRFD Bridge Design Specifications. The corrosion and deterioration testing results are summarized in Table 12 and are included in Appendix E.

Table 12. Results of pH and Soil Resistivity Testing.

| Boring | Sample No. | Sample Depth (feet) | pH | Soil Resistivity (ohm-cm) |
|--------|------------|---------------------|------|---------------------------|
| B1-2 | ST-9 | 20.0 | 6.66 | 588.81 |
| | SS-10 | 23.5 | 6.40 | 759.81 |
| | SS-14 | 43.5 | 7.61 | 2,639.10 |
| B1-4 | SS-12 | 38.5 | 8.04 | 5,728.50 |
| B1-5 | ST-7 | 20.0 | 7.12 | 660.63 |
| | SS-14 | 53.5 | 8.07 | 769.50 |
| B1-6 | SS-5 | 13.5 | 7.29 | 628.71 |



The following soil conditions should be considered as indicative of a potential for steel pile deterioration or corrosion:

- Resistivity values less than 2,000 ohms-cm; or
- pH less than 5.5.

The following soil conditions should be considered as indicative of a potential for steel reinforcement corrosion or deterioration:

- Resistivity values less than 3,000 ohms-cm; or
- pH less than 5.5.

Interpretation of the data and corrosion protection of the bridge structural components should be performed by the design team.

7.0 RECOMMENDED ADDITIONAL SERVICES

The conclusions and recommendations given in this report are based on: Geotechnology's understanding of the proposed design and construction, as outlined in this report; site observations; interpretation of the exploration data; and our experience. Since the intent of the design recommendations is best understood by Geotechnology, we recommend Geotechnology be included in the final design and construction process, and be retained to review the project plans and specifications to confirm the recommendations given in this report have been correctly implemented. We recommend Geotechnology be retained to participate in pre-bid and preconstruction conferences to reduce the risk of misinterpretation of the conclusions and recommendations in this report relative to the proposed construction of the subject project.

Since actual subsurface conditions between boring locations could vary from those encountered in the borings, our design recommendations are subject to adjustment in the field based on the subsurface conditions encountered during construction. Therefore, we recommend Geotechnology be retained to provide construction observation services as a continuation of the design process to confirm the recommendations in this report and to revise them accordingly to accommodate differing subsurface conditions. Construction observation is intended to enhance compliance with project plans and specifications. It is not insurance, nor does it constitute a warranty or guarantee of any type. Regardless of construction observation, contractors, suppliers, and others are solely responsible for the quality of their work and for adhering to plans and specifications.

8.0 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.**



Telephone: 301/565-2733

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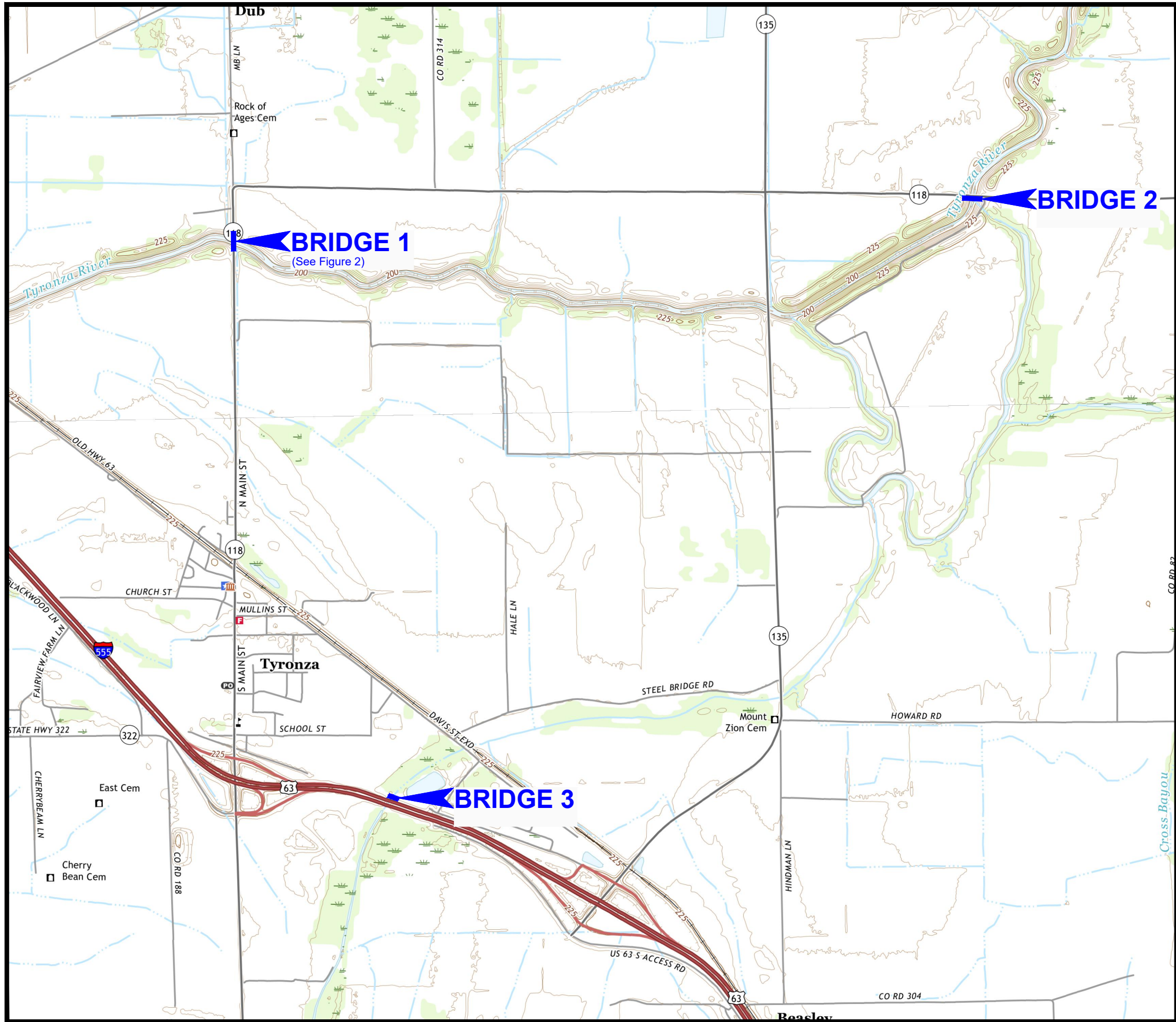
APPENDIX B – FIGURES

Figure 1 – Site Location and Topography

Figure 2 – Aerial Photograph of Site and Boring Locations

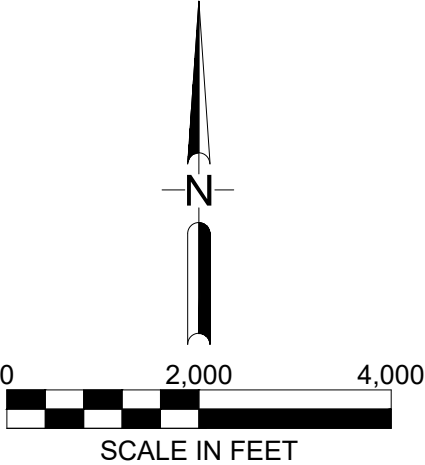
Figure 3 – Shear Wave Velocity Profile




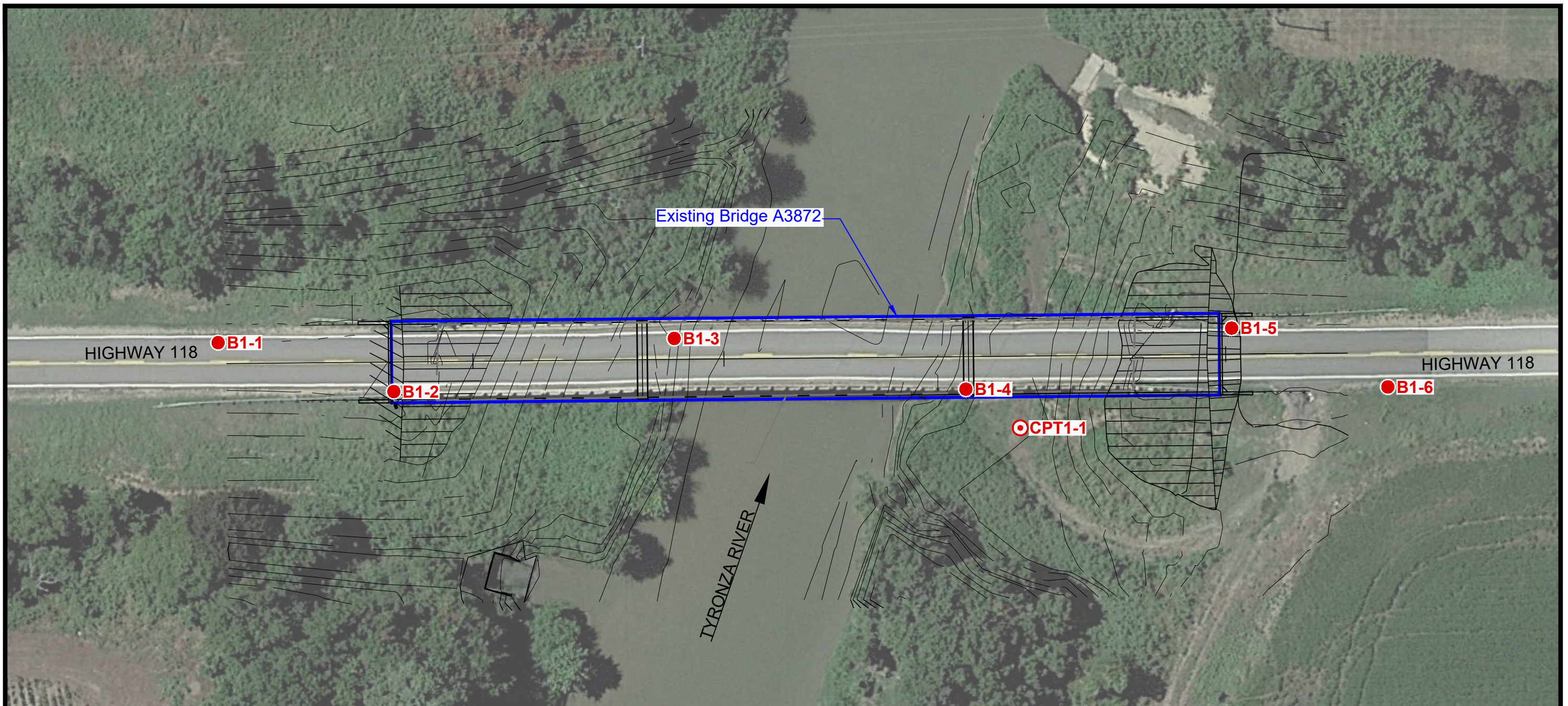


NOTES

1. Plan adapted from 7.5 minute U.S.G.S. maps for Lepanto and Tyronza, Arkansas quadrangles, last revised in 2020.



| | | |
|---|-----------------|----------------|
| Drawn By: WAH | Ck'd By: JDM | App'vd By: ASE |
| Date: 5-5-23 | Date: 2-8-24 | Date: 2-8-24 |
|  GEOTECHNOLOGY <small>A UES Company</small> | | |
| ARDOT 101017 Tyronza River & Ditch No. 6 Strs. & Apprs. (S) Poinsett County, Arkansas | | |
| SITE LOCATION AND TOPOGRAPHY | | |
| Project Number J042992.01 | FIGURE 1 | |

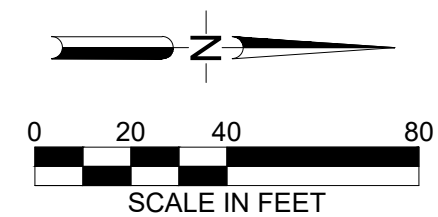


NOTES

1. Plan adapted from a September 5, 2021 aerial photograph courtesy of Google Earth and a drawing dated January 20, 2023, titled "Soil Boring Request Layout of Bridge Over Tyronza River", prepared by Arkansas State Highway Commission"
2. Borings and CPT were located in the field with reference to site features and are shown approximate only.

LEGEND

- Boring Location
- ⊙ CPT Sounding Location



| | | |
|---------------|--------------|----------------|
| Drawn By: WAH | Ck'd By: JDM | App'vd By: ASE |
| Date: 5-5-23 | Date: 2-8-24 | Date: 2-8-24 |



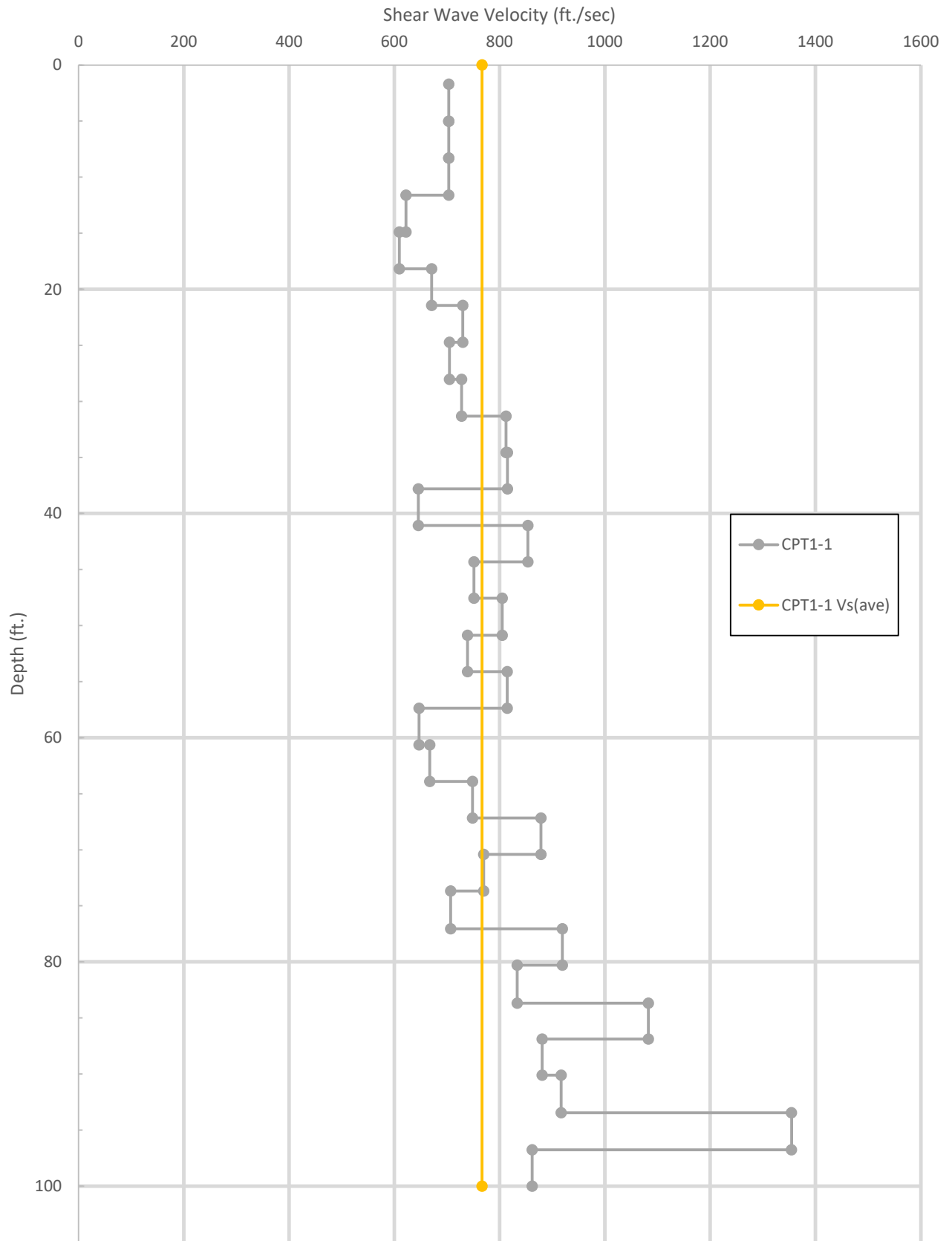
ARDOT 101017 Tyronza River &
Ditch No. 6 Strs. & Apprs. (S)
Poinsett County, Arkansas

**AERIAL PHOTOGRAPH OF BRIDGE 1
AND BORING AND CPT LOCATIONS**

Project Number
J042992.01

FIGURE 2

Shear Wave Velocity vs. Depth - CPT1-1





APPENDIX C – BORING INFORMATION

Boring Logs

Boring Log Terms and Symbols

Surface Elevation: 220

Completion Date: 5/19/23

Datum NAVD88

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 | 10 20 30 40 50 | LL

DEPTH
IN FEET

ELEVATION
IN FEET

DESCRIPTION OF MATERIAL

GRAPHIC LOG

DRY UNIT WEIGHT (pcf)
SPT BLOW COUNTS
CORE RECOVERY/RQD

SAMPLES

| | |
|-----|-----|
| 5 | 215 |
| 10 | 210 |
| 15 | 205 |
| 20 | 200 |
| 25 | 195 |
| 30 | 190 |
| 35 | 185 |
| 40 | 180 |
| 45 | 175 |
| 50 | 170 |
| 55 | 165 |
| 60 | 160 |
| 65 | 155 |
| 70 | 150 |
| 75 | 145 |
| 80 | 140 |
| 85 | 135 |
| 90 | 130 |
| 95 | 125 |
| 100 | 120 |

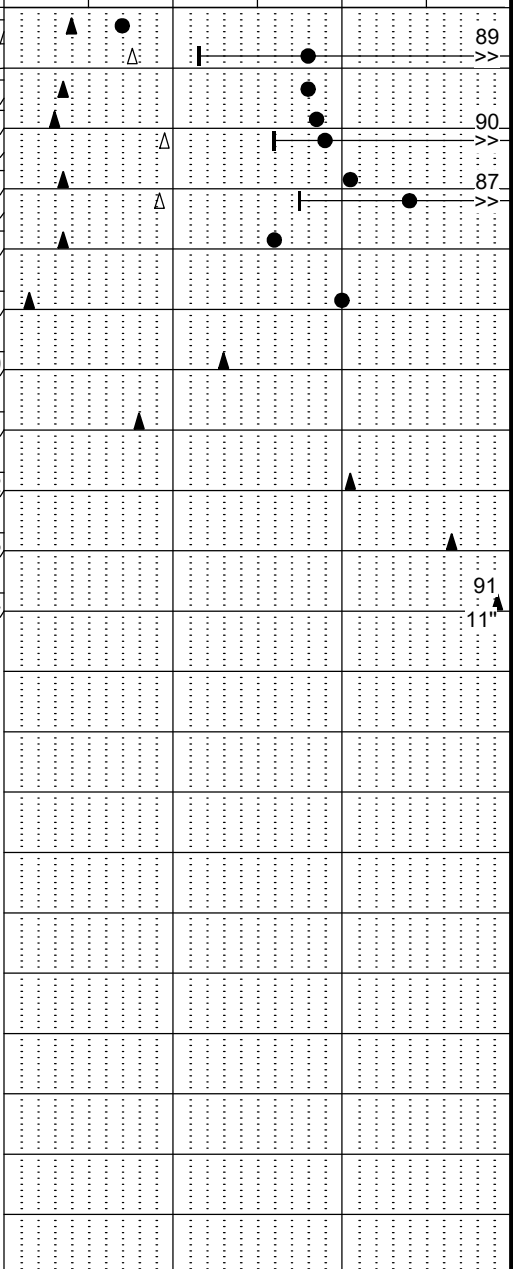
ASPHALT: 6 inches
Medium stiff to stiff, brown, sandy, FAT CLAY - (CH)

Stiff to soft, brown and gray to gray, FAT CLAY - (CH)
85.1% passing No. 200 sieve
trace sand

Medium dense to dense, brown and gray SAND - SP
trace clay

Boring terminated at 50 feet.

| | |
|-----------------|------|
| 3-4-4 | SS1 |
| 85 | ST2 |
| 3-3-4 | SS3 |
| 2-2-4 | SS4 |
| 84 | ST5 |
| 2-2-5 | SS6 |
| 77 | ST7 |
| 3-3-4 | SS8 |
| 2-1-2 | SS9 |
| 5-11-15 | SS10 |
| 7-6-10 | SS11 |
| 12-19-22 | SS12 |
| 19-21-32 | SS13 |
| 16-41 -50/5" | SS14 |



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES
LOG OF BORING 2020 JDM - ELEVATIONS .J042992.01.GPJ - 11/17/23 AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 30 FEET ∇

DRILLING DATA

___ AUGER 3 3/4" HOLLOW STEM
WASHBORING FROM ___ FEET
JCB DRILLER JRJ LOGGER
Diedrich D-50 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 93 %

REMARKS:

| | | |
|---------------|-----------------|-----------------|
| Drawn by: JSH | Checked by: JDM | App'vd. by: ASE |
| Date: 5/25/23 | Date: 11/17/23 | Date: 11/17/23 |



**Tyrnza River
& Ditch No.6 Strs. & Apprs. (S)
Poinsett County, Arkansas**

LOG OF BORING: B1-1

Project No. J042992.01

Surface Elevation: 220

Completion Date: 5/11/23

Datum NAVD88

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 | 10 20 30 40 50 | LL

DEPTH
IN FEET

ELEVATION
IN FEET

DESCRIPTION OF MATERIAL

GRAPHIC LOG

DRY UNIT WEIGHT (pcf)
SPT BLOW COUNTS
CORE RECOVERY/RQD

SAMPLES

| | | | | | | | |
|-----|-----|---|--|----------------------------------|-----|--|-----|
| 5 | 215 | ASPHALT: 7 inches Medium stiff, brown, sandy, LEAN CLAY, trace gravel - CL | | 6-4-4 SS1 2-3-3 SS2 | ▲ ● | | |
| 10 | 210 | Medium stiff, brown and gray, LEAN CLAY - CL Medium stiff to stiff, brown to brown and gray, FAT CLAY - (CH) | | 3-4-5 SS3 3-3-3 SS4 89 ST5 | ▲ ● | | 92 |
| 15 | 205 | Stiff to medium stiff, brown and gray, ELASTIC SILT - (MH) | | 3-4-5 SS6 72 ST7 | ▲ △ | | 102 |
| 20 | 200 | Soft to stiff, brown to gray, silty, FAT CLAY - (CH) | | 3-3-4 SS8 79 ST9 | ▲ △ | | 70 |
| 25 | 195 | | | 3-2-2 SS10 | ▲ ● | | |
| 30 | 190 | | | 2-2-1 SS11 | ▲ ● | | |
| 35 | 185 | Stiff, gray, sandy, LEAN CLAY - CL | | 2-2-7 SS12 | ▲ ● | | |
| 40 | 180 | Dense, brown and gray SAND with clay - SP-SC 10.3% passing No. 200 sieve | | 13-17-18 SS13 | ▲ | | |
| 45 | 175 | Medium dense to dense, brown to brown-gray SAND - SP | | 10-12-15 SS14 | ▲ | | |
| 50 | 170 | | | 12-17-21 SS15 | ▲ | | |
| 55 | 165 | | | 12-16-18 SS16 | ▲ | | |
| 60 | 160 | | | 12-13-20 SS17 | ▲ | | |
| 65 | 155 | | | | | | |
| 70 | 150 | | | 10-13-19 SS18 | ▲ | | |
| 75 | 145 | | | | | | |
| 80 | 140 | | | 10-13-18 SS19 | ▲ | | |
| 85 | 135 | | | | | | |
| 90 | 130 | | | 15-19-28 SS20 | ▲ | | |
| 95 | 125 | | | | | | |
| 100 | 120 | Boring terminated at 100 feet. | | 17-16-18 SS21 | ▲ | | |

NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES
LOG OF BORING 2020 JDM - ELEVATIONS .J042992.01.GPJ -GTINC 0638301.GPJ 11/17/23 AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

___ AUGER 3 3/4" HOLLOW STEM WASHBORING FROM 25 FEET
JCG DRILLER JWD LOGGER
Diedrich D-50 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 93 %

REMARKS:

| | | |
|---------------|-----------------|-----------------|
| Drawn by: JSH | Checked by: JDM | App'vd. by: ASE |
| Date: 5/15/23 | Date: 11/17/23 | Date: 11/17/23 |



Tyronza River & Ditch No.6 Strs. & Apprs. (S)
Poinsett County, Arkansas

LOG OF BORING: B1-2

Project No. J042992.01

Surface Elevation: 198

Completion Date: 7/11/23

Datum NAVD88

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 | 10 20 30 40 50 | LL

DEPTH
IN FEET

ELEVATION
IN FEET

DESCRIPTION OF MATERIAL

GRAPHIC LOG

DRY UNIT WEIGHT (pcf)
SPT BLOW COUNTS
CORE RECOVERY/RQD

SAMPLES

| | |
|-----|-----|
| 5 | 193 |
| 10 | 188 |
| 15 | 183 |
| 20 | 178 |
| 25 | 173 |
| 30 | 168 |
| 35 | 163 |
| 40 | 158 |
| 45 | 153 |
| 50 | 148 |
| 55 | 143 |
| 60 | 138 |
| 65 | 133 |
| 70 | 128 |
| 75 | 123 |
| 80 | 118 |
| 85 | 113 |
| 90 | 108 |
| 95 | 103 |
| 100 | 98 |

Soft to stiff, brown, silty, LEAN CLAY - CL

67.4% passing No. 200 sieve

72.6% passing No. 200 sieve

Medium dense, brown and gray SAND with clay - SP-SC

6.0% passing No. 200 sieve

Medium dense to dense, gray SAND - SP

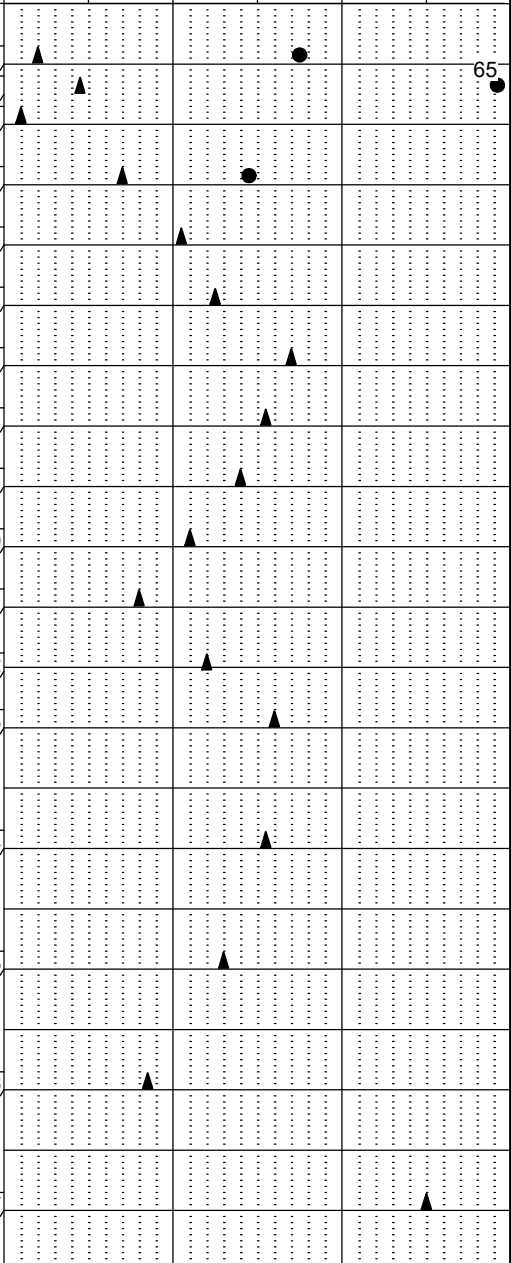
trace gravel

trace gravel

3.9% passing No. 200 sieve little gravel

Boring terminated at 100 feet.

| | |
|----------|------|
| 0-2-2 | SS1 |
| 1-3-6 | SS2 |
| 1-1-1 | SS3 |
| 4-8-6 | SS4 |
| 8-10-11 | SS5 |
| 7-10-15 | SS6 |
| 10-16-18 | SS7 |
| 10-15-16 | SS8 |
| 8-12-16 | SS9 |
| 8-12-10 | SS10 |
| 5-8-8 | SS11 |
| 9-11-13 | SS12 |
| 8-14-18 | SS13 |
| 12-16-15 | SS14 |
| 12-11-15 | SS15 |
| 9-8-9 | SS16 |
| 25-25-25 | SS17 |



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES LOG OF BORING 2020 JDM - ELEVATIONS .J042992.01.GPJ GTINC 0638301.GPJ 11/17/23 AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

AUGER 3 3/4 HOLLOW STEM WASHBORING FROM 0 FEET
BMF DRILLER JMP LOGGER
CME 550X DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 88 %

REMARKS: Mudline 20 feet below top of bridge deck. Casing seated approximately 3.5 feet into river bed.

| | | |
|----------------------|------------------------|------------------------|
| Drawn by: <u>JMP</u> | Checked by: <u>JDM</u> | App'vd. by: <u>ASE</u> |
| Date: <u>7/11/23</u> | Date: <u>11/17/23</u> | Date: <u>11/17/23</u> |



Tyrnza River & Ditch No.6 Strs. & Apprs. (S)
Poinsett County, Arkansas

LOG OF BORING: B1-3

Project No. J042992.01

LOG OF BORING 2020 JDM - ELEVATIONS J042992.01.GPJ GTINC 0638301.GPJ 11/17/23 AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

| Surface Elevation: <u>204</u> | | Completion Date: <u>5/22/23</u> | | GRAPHIC LOG | DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD | SAMPLES | SHEAR STRENGTH, tsf | | | | | | |
|-------------------------------|----------------------|---|------------------|-------------|---|---------|--------------------------|----------|--------|-----|-----|--|--|
| Datum <u>NAVD88</u> | | STANDARD PENETRATION RESISTANCE (ASTM D 1586) | | | | | | | | | | | |
| DEPTH IN FEET | ELEVATION IN FEET | DESCRIPTION OF MATERIAL | WATER CONTENT, % | | | | | | | | | | |
| | | | PL | | | | N-VALUE (BLOWS PER FOOT) | | | LL | | | |
| | | | | | | | △ - UU/2 | ○ - QU/2 | □ - SV | | | | |
| | | | | | | | 0.5 | 1.0 | 1.5 | 2.0 | 2.5 | | |
| | | Topsoil: 6 inches | | | 2-2-3 | SS1 | ▲ | | | | | | |
| | | Medium stiff to soft, brown, silty, LEAN CLAY - CL | | | 2-2-2 | SS2 | ▲ | | | | | | |
| 5 | 199 | trace sand | | | 2-1-3 | SS3 | ▲ | | | | | | |
| 10 | 194 | Medium dense, brown and gray SAND - SP | ∇ | | 4-7-11 | SS4 | ▲ | | | | | | |
| | | trace clay | | | | ST5 | ● | | | | | | |
| | | 1.9% passing No. 200 sieve | | | 4-8-10 | SS6 | ▲ | | | | | | |
| 15 | 189 | Medium dense, gray SAND with silt - SP-SM | | | 5-6-10 | SS7 | ▲ | | | | | | |
| | | 5.1% passing No. 200 sieve | | | 5-6-11 | SS8 | ▲ | | | | | | |
| 20 | 184 | Medium dense to loose, gray SAND - SP | | | 3-4-6 | SS9 | ▲ | | | | | | |
| | | 2.7% passing No. 200 sieve | | | 7-9-11 | SS11 | ▲ | | | | | | |
| 25 | 179 | | | | 14-17-21 | SS12 | ▲ | | | | | | |
| 30 | 174 | 1.7% passing No. 200 sieve | | | 9-11-9 | SS13 | ▲ | | | | | | |
| 35 | 169 | | | | 8-10-12 | SS14 | ▲ | | | | | | |
| 40 | 164 | Dense to medium dense, gray SAND with silt - SP-SM | | | 11-13-16 | SS15 | ▲ | | | | | | |
| | | 6.6% passing No. 200 sieve | | | 6-9-12 | SS16 | ▲ | | | | | | |
| 45 | 159 | | | | 12-16-21 | SS17 | ▲ | | | | | | |
| 50 | 154 | Medium dense to dense, gray SAND, trace gravel - SP | | | 9-8-9 | SS18 | ▲ | | | | | | |
| 55 | 149 | | | | 12-13-11 | SS19 | ▲ | | | | | | |
| 60 | 144 | | | | 10-13-15 | SS20 | ▲ | | | | | | |
| 65 | 139 | | | | | | | | | | | | |
| 70 | 134 | | | | | | | | | | | | |
| 75 | 129 | | | | | | | | | | | | |
| 80 | 124 | | | | | | | | | | | | |
| 85 | 119 | | | | | | | | | | | | |
| 90 | 114 | | | | | | | | | | | | |
| 95 | 109 | | | | | | | | | | | | |
| 100 | 104 | Boring terminated at 100 feet. | | | | | | | | | | | |

GROUNDWATER DATA

ENCOUNTERED AT 10 FEET ∇

DRILLING DATA

___ AUGER 3 3/4" HOLLOW STEM
WASHBORING FROM 20 FEET
AKM DRILLER JWD LOGGER
Diedrich D-50 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 93 %

REMARKS: Shelby tube ST-5 too sandy for strength testing.

| | | |
|---------------|-----------------|-----------------|
| Drawn by: JSH | Checked by: JDM | App'vd. by: ASE |
| Date: 5/25/23 | Date: 11/17/23 | Date: 11/17/23 |



**Tyrnza River
& Ditch No.6 Strs. & Apprs. (S)
Poinsett County, Arkansas**

LOG OF BORING: B1-4

Project No. J042992.01

LOG OF BORING 2020 JDM - ELEVATIONS J042992.01.GPJ GTINC 0638301.GPJ 11/17/23 AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

| Surface Elevation: <u>220</u> | | Completion Date: <u>5/11/23</u> | | GRAPHIC LOG | DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD | SAMPLES | SHEAR STRENGTH, tsf | | | | | |
|-------------------------------|----------------------|--|---|---|---|---------|--------------------------|--|--|----|--|----|
| Datum <u>NAVD88</u> | | STANDARD PENETRATION RESISTANCE (ASTM D 1586) | | | | | | | | | | |
| DEPTH IN FEET | ELEVATION IN FEET | DESCRIPTION OF MATERIAL | WATER CONTENT, % | | | | | | | | | |
| | | | PL | | | | N-VALUE (BLOWS PER FOOT) | | | LL | | |
| | | | | Δ - UU/2 ○ - QU/2 □ - SV 0.5 1.0 1.5 2.0 2.5 | | | | | | | | |
| | | ASPHALT: 7 inches | | | | | | | | | | |
| 5 | 215 | Medium stiff, brown and gray, sandy, FAT CLAY - CH | | 5-5-3 | SS1 | | | | | | | |
| | | | | | 1-3-4 | SS2 | | | | | | |
| | | | | | 3-4-4 | SS3 | | | | | | |
| 10 | 210 | | Stiff to medium stiff, brown and gray to brown, FAT CLAY - (CH) | | 95 | ST4 | | | | | | 66 |
| 15 | 205 | | | | 2-2-3 | SS5 | | | | | | |
| 20 | 200 | | | | 2-3-3 | SS6 | | | | | | |
| | | | | | 93 | ST7 | | | | | | |
| 25 | 195 | Loose, gray, CLAYEY SAND - SC 45.6% passing No. 200 sieve trace gravel | | 2-1-4 | SS8 | | | | | | | |
| 30 | 190 | | | | 2-2-3 | SS9 | | | | | | |
| 35 | 185 | | | | 2-2-3 | SS10 | | | | | | |
| 40 | 180 | | Medium dense to very dense, gray SAND - SP | | 6-11-19 | SS11 | | | | | | |
| 45 | 175 | | | | 12-8-10 | SS12 | | | | | | |
| 50 | 170 | | | | 12-21-39 | SS13 | | | | | | |
| 55 | 165 | | | | 16-18-21 | SS14 | | | | | | |
| 60 | 160 | Boring terminated at 100 feet. | | 9-13-16 | SS15 | | | | | | | |
| 65 | 155 | | | | | | | | | | | |
| 70 | 150 | | | | 15-19-23 | SS16 | | | | | | |
| 75 | 145 | | | | | | | | | | | |
| 80 | 140 | Boring terminated at 100 feet. | | 9-15-17 | SS17 | | | | | | | |
| 85 | 135 | | | | | | | | | | | |
| 90 | 130 | | | | 17-19-30 | SS18 | | | | | | |
| 95 | 125 | | | | | | | | | | | |
| 100 | 120 | | | 11-8-8 | SS19 | | | | | | | |

GROUNDWATER DATA

ENCOUNTERED AT 25 FEET ∇

DRILLING DATA

___ AUGER 3 3/4" HOLLOW STEM
WASHBORING FROM 25 FEET
JCG DRILLER JWD LOGGER
Diedrich D-50 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 93 %

REMARKS:

| | | |
|---------------|-----------------|-----------------|
| Drawn by: JSH | Checked by: JDM | App'vd. by: ASE |
| Date: 5/15/23 | Date: 11/17/23 | Date: 11/17/23 |



**Tyrnza River
& Ditch No.6 Strs. & Apprs. (S)
Poinsett County, Arkansas**

LOG OF BORING: B1-5

Project No. J042992.01

Surface Elevation: 220

Completion Date: 5/10/23

Datum NAVD88

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 | 10 20 30 40 50 | LL

DEPTH
IN FEET

ELEVATION
IN FEET

DESCRIPTION OF MATERIAL

GRAPHIC LOG

DRY UNIT WEIGHT (pcf)
SPT BLOW COUNTS
CORE RECOVERY/RQD

SAMPLES

| | | | | | | | | | |
|-----|-----|--|---|----------|------|---|---|--|----|
| 5 | 215 | ASPHALT: 7 inches Base Material: 6 inches | | 2-2-3 | SS1 | ▲ | | | |
| | | Medium stiff, gray to gray and brown, sandy, LEAN CLAY - (CL) | | 3-2-3 | SS2 | ▲ | | | |
| | | trace gravel | | 3-4-4 | SS3 | ▲ | | | |
| 10 | 210 | 60.4% passing No. 200 sieve | | 2-2-3 | SS4 | ▲ | | | 72 |
| | | Stiff to medium stiff, gray and brown, FAT CLAY, trace sand - (CH) | | 92 | ST5 | ▲ | Δ | | |
| 15 | 205 | | | 3-4-5 | SS6 | ▲ | | | 80 |
| | | | | 87 | ST7 | ▲ | Δ | | |
| 20 | 200 | | | 2-3-4 | SS8 | ▲ | | | |
| 25 | 195 | Loose, gray and brown to gray, CLAYEY SAND - SC 27.2% passing No. 200 sieve | | 4-4-5 | SS9 | ▲ | | | |
| 30 | 190 | Loose to medium dense, gray, SILTY SAND - SM 20.7% passing No. 200 sieve | ▽ | 3-4-4 | SS10 | ▲ | | | |
| | | 17.7% passing No. 200 sieve | | 5-7-9 | SS11 | ▲ | | | |
| 35 | 185 | | | 7-9-8 | SS12 | ▲ | | | |
| 40 | 180 | 13.8% passing No. 200 sieve | | 9-12-11 | SS13 | ▲ | | | |
| 45 | 175 | | | 11-15-14 | SS14 | ▲ | | | |
| 50 | 170 | Boring terminated at 50 feet. | | | | | | | |
| 55 | 165 | | | | | | | | |
| 60 | 160 | | | | | | | | |
| 65 | 155 | | | | | | | | |
| 70 | 150 | | | | | | | | |
| 75 | 145 | | | | | | | | |
| 80 | 140 | | | | | | | | |
| 85 | 135 | | | | | | | | |
| 90 | 130 | | | | | | | | |
| 95 | 125 | | | | | | | | |
| 100 | 120 | | | | | | | | |

NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES
LOG OF BORING 2020 JDM - ELEVATIONS .J042992.01.GPJ GTINC 0638301.GPJ 11/17/23 AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 30 FEET ▽

DRILLING DATA

___ AUGER 3 3/4" HOLLOW STEM
WASHBORING FROM 30 FEET
JCG DRILLER 00M/JWD LOGGER
Diedrich D-50 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 93 %

REMARKS:

Drawn by: LLP Checked by: JDM App'vd. by: ASE
Date: 5/11/23 Date: 11/17/23 Date: 11/17/23



**Tyrnza River
& Ditch No.6 Strs. & Apprs. (S)
Poinsett County, Arkansas**

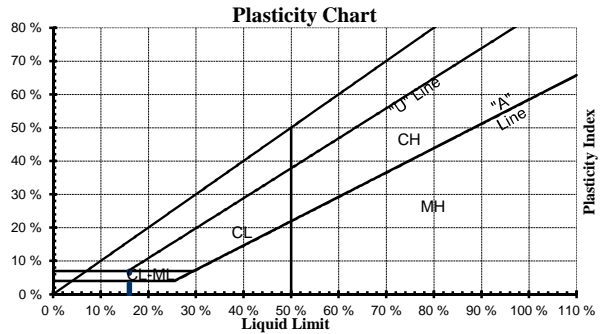
LOG OF BORING: B1-6

Project No. J042992.01

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

STRENGTH OF COHESIVE SOILS

DENSITY OF GRANULAR SOILS

| Consistency | Undrained Shear Strength (tsf) | Unconfined Comp. Strength (tsf) | Descriptive Term | Approximate N_{60} -Value Range |
|--------------|--------------------------------|---------------------------------|------------------|-----------------------------------|
| Very Soft | less than 0.125 | less than 0.25 | Very Loose | 0 to 4 |
| Soft | 0.125 to 0.25 | 0.25 to 0.5 | Loose | 5 to 10 |
| Medium Stiff | 0.25 to 0.5 | 0.5 to 1.0 | Medium Dense | 11 to 30 |
| Stiff | 0.5 to 1.0 | 1.0 to 2.0 | Dense | 31 to 50 |
| Very Stiff | 1.0 to 2.0 | 2.0 to 3.0 | Very Dense | >50 |
| Hard | greater than 2.0 | greater than 4.0 | | |

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

OTHER TERMS

| | | |
|--------|-----------|---|
| Trace | 0 to 10% | Layer - Inclusion greater than 3 inches thick. |
| Little | 10 to 20% | Seam - Inclusion 1/8-inch to 3 inches thick |
| Some | 20 to 35% | Parting - Inclusion less than 1/8-inch thick |
| And | 35 to 50% | Pocket - Inclusion of material that is smaller than sample diameter |



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 – CPT1-1 SOUNDING PLOT



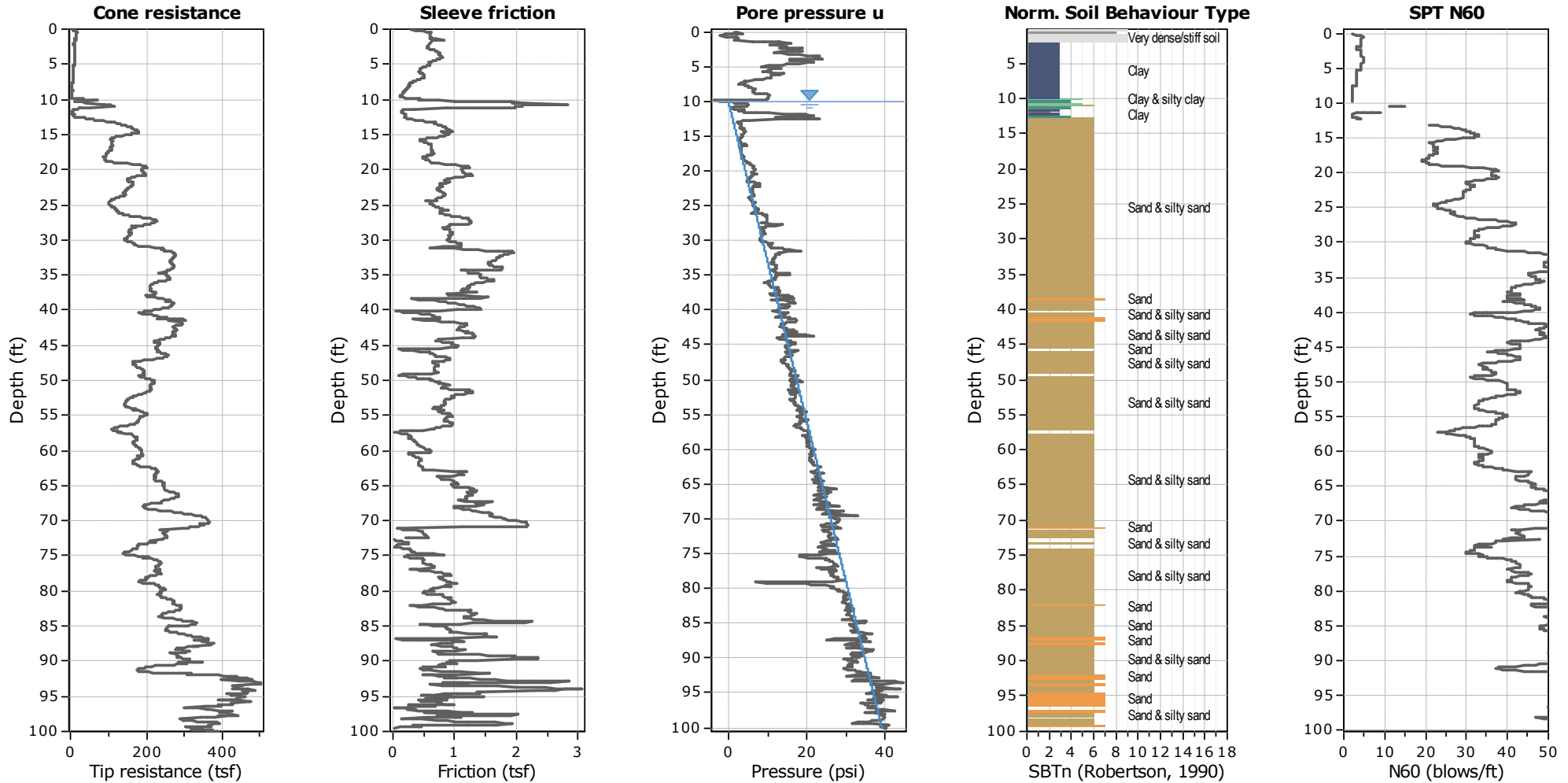


Geotechnology, LLC
 3312 Winbrook Dr
 Memphis, TN 38116
 (901) 353-1981

CPT: CPT1-1

Total depth: 100.09 ft, Date: 6/12/2023
 Coords: lat 35.509822° lon -90.358147°
 Cone Type: 15 Square-Centimeter
 Cone Operator: DWJ

Project: ARDOT G003 101017 Tyronza River & Ditch No. 6 Strs. & Apprs. (S)
Location: Poinsett County, Arkansas



SBTn legend

| | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravelly sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |



APPENDIX E – LABORATORY TEST DATA

Atterberg Limits

Grain Size Distributions

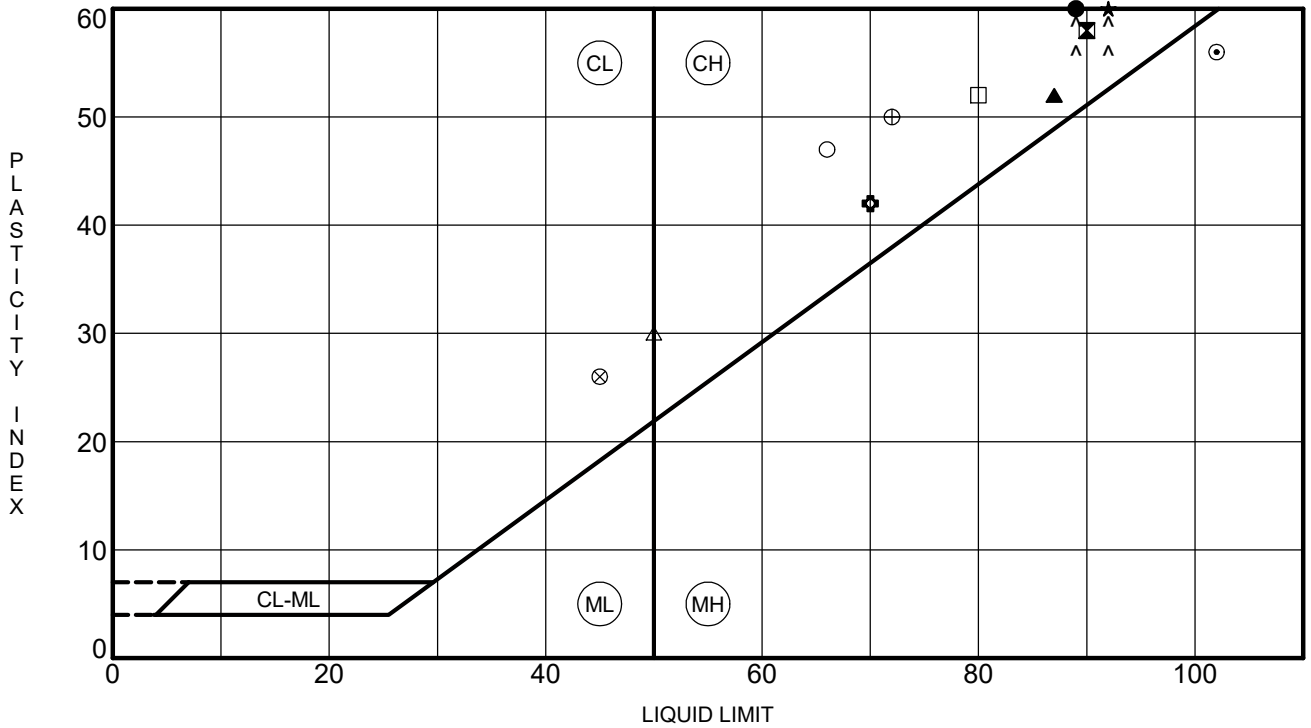
Unconsolidated-Undrained Triaxial Compression

One-Dimensional Consolidation

Consolidated-Undrained Triaxial Compression

Resistivity

pH

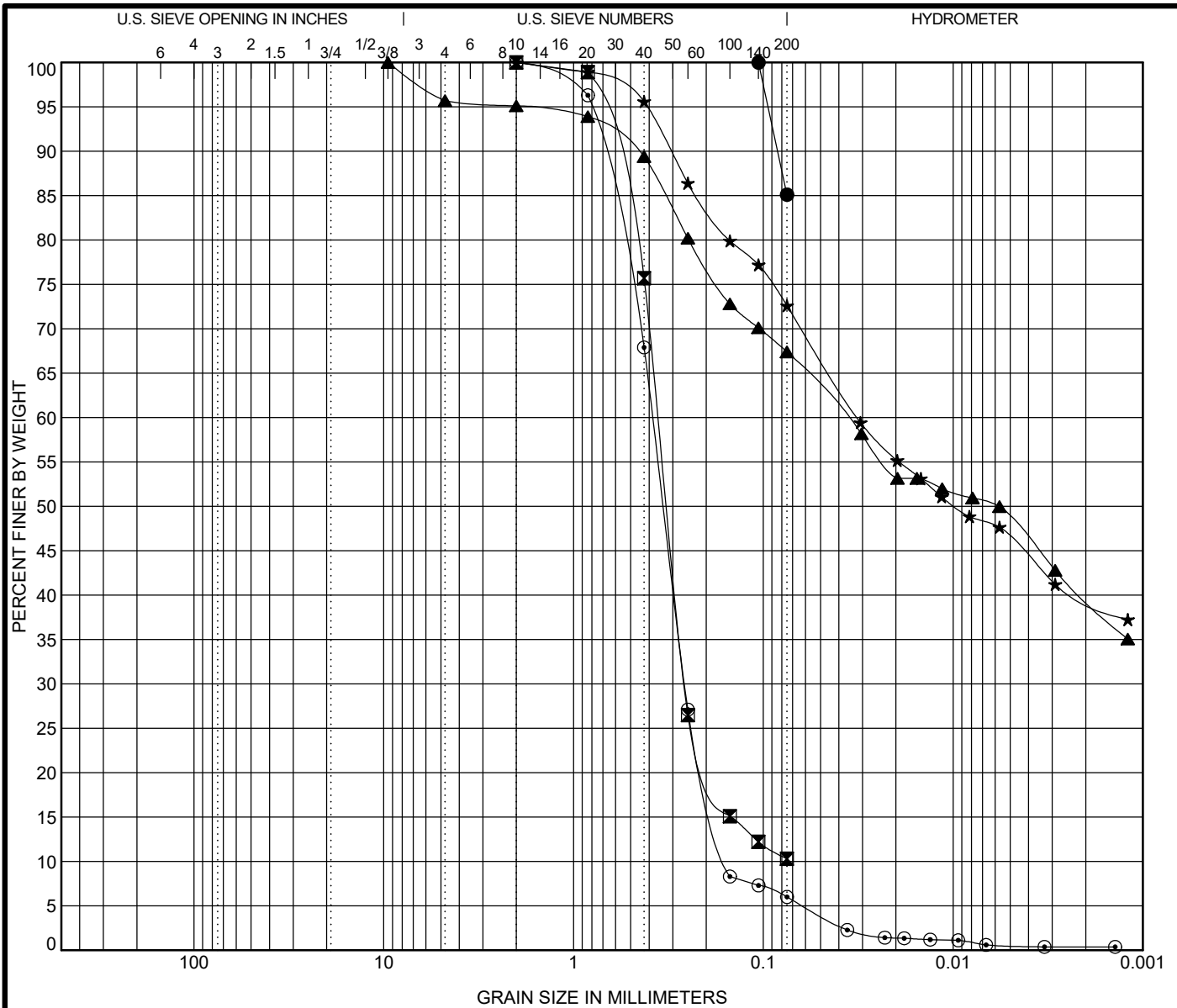


| Specimen Identification | LL | PL | PI | Fines | Classification |
|-------------------------|------|-----|----|-------|------------------------|
| ● B1-1 | 3.0 | 89 | 23 | 66 | FAT CLAY(CH) |
| ⊠ B1-1 | 10.0 | 90 | 32 | 58 | FAT CLAY(CH) |
| ▲ B1-1 | 15.0 | 87 | 35 | 52 | FAT CLAY(CH) |
| ★ B1-2 | 10.0 | 92 | 30 | 62 | FAT CLAY(CH) |
| ⊙ B1-2 | 15.0 | 102 | 46 | 56 | ELASTIC SILT(MH) |
| ⊕ B1-2 | 20.0 | 70 | 28 | 42 | FAT CLAY(CH) |
| ○ B1-5 | 8.0 | 66 | 19 | 47 | FAT CLAY(CH) |
| △ B1-5 | 20.0 | 50 | 20 | 30 | FAT CLAY(CH) |
| ⊗ B1-6 | 3.5 | 45 | 19 | 26 | 60 SANDY LEAN CLAY(CL) |
| ⊕ B1-6 | 10.0 | 72 | 22 | 50 | FAT CLAY(CH) |
| □ B1-6 | 15.0 | 80 | 28 | 52 | FAT CLAY(CH) |
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US ATTERBERG LIMITS J042992.01.GPJ US LAB.GDT 11/14/23



ATTERBERG LIMITS RESULTS
 Tyronza River
 & Ditch No.6 Strs. & Apprs. (S)
 Poinsett County, Arkansas
 J042992.01



| COBBLES | GRAVEL | | SAND | | | SILT OR CLAY |
|---------|--------|------|--------|--------|------|--------------|
| | coarse | fine | coarse | medium | fine | |

| Specimen Identification | Classification | LL | PL | PI | Cc | Cu |
|-------------------------|-------------------------------------|----|----|----|------|------|
| ● B1-1 6.0 | FAT CLAY(CH) | | | | | |
| ■ B1-2 38.5 | POORLY GRADED SAND with CLAY(SP-SC) | | | | 2.65 | 5.05 |
| ▲ B1-3 3.5 | LEAN CLAY(CL) | | | | | |
| ★ B1-3 6.0 | SANDY LEAN CLAY(CL) | | | | | |
| ⊙ B1-3 13.5 | POORLY GRADED SAND with CLAY(SP-SC) | | | | 1.12 | 2.44 |

| Specimen Identification | D100 | D60 | D30 | D10 | %Gravel | %Sand | %Silt | %Clay |
|-------------------------|-------|-------|------|-------|---------|-------|-------|-------|
| ● B1-1 6.0 | 0.106 | | | | 0.0 | 14.9 | 85.1 | |
| ■ B1-2 38.5 | 2 | 0.359 | 0.26 | | 0.0 | 89.7 | 10.3 | |
| ▲ B1-3 3.5 | 9.5 | 0.036 | | | 4.3 | 28.3 | 18.8 | 48.6 |
| ★ B1-3 6.0 | 2 | 0.032 | | | 0.0 | 27.4 | 26.2 | 46.4 |
| ⊙ B1-3 13.5 | 2 | 0.384 | 0.26 | 0.157 | 0.0 | 94.0 | 5.5 | 0.5 |



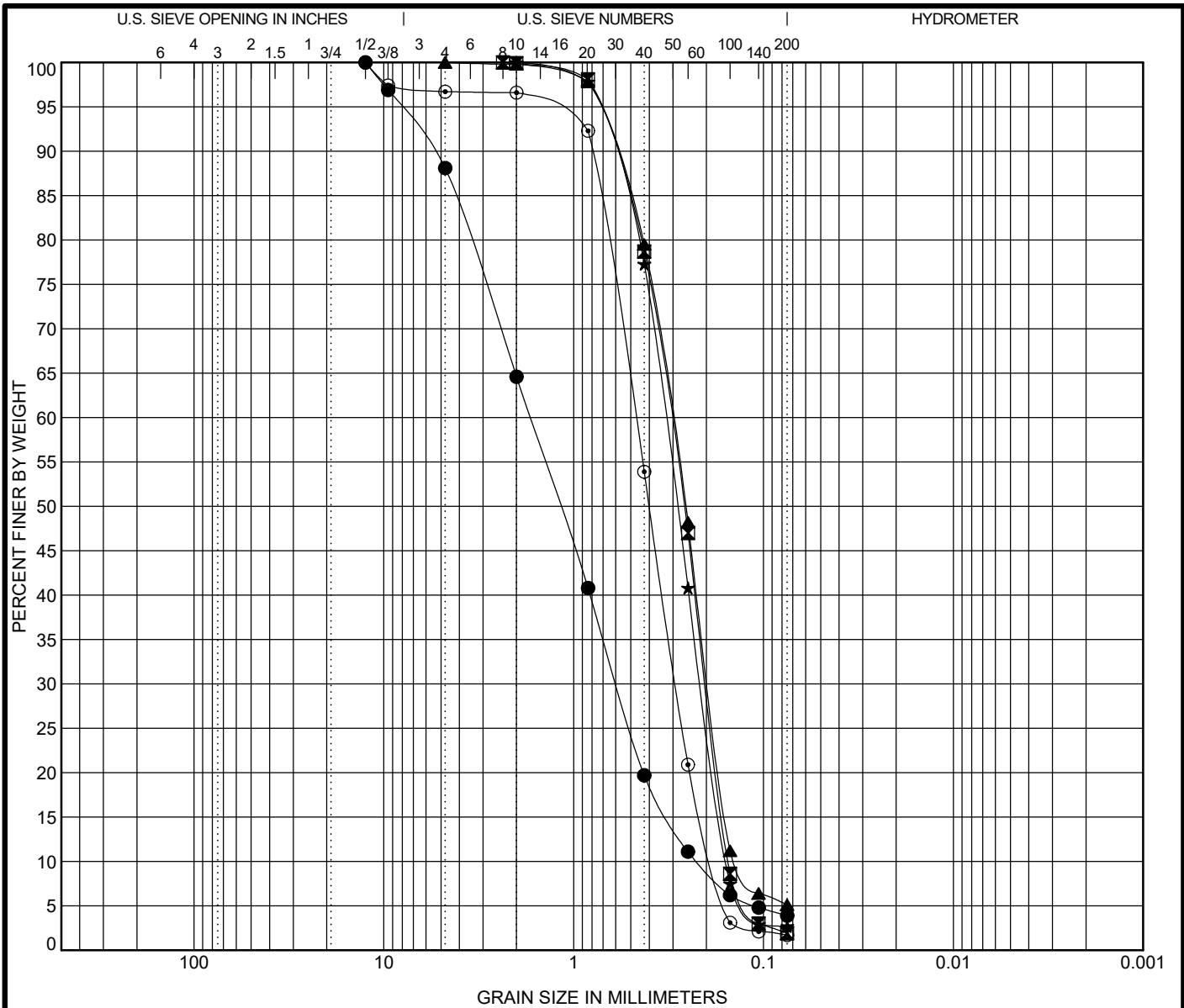
GEOTECHNOLOGY

A UES Company

GRAIN SIZE DISTRIBUTION

Tyrnza River
 & Ditch No.6 Strs. & Apprs. (S)
 Poinsett County, Arkansas
 J042992.01

U.S. GRAIN SIZE J042992.01.GPJ US LAB.GDT 11/14/23



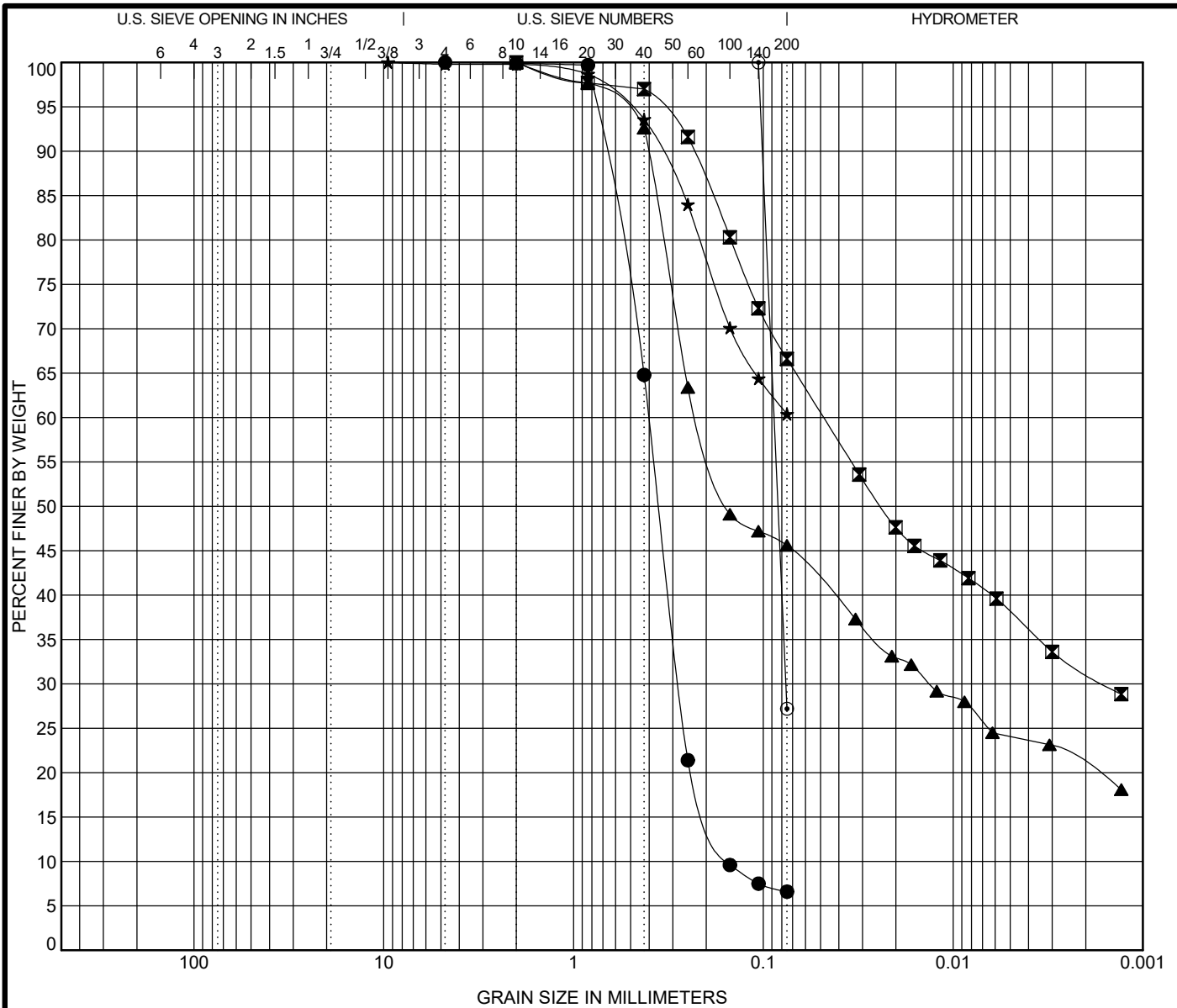
| COBBLES | GRAVEL | | SAND | | | SILT OR CLAY |
|---------|--------|------|--------|--------|------|--------------|
| | coarse | fine | coarse | medium | fine | |

| Specimen Identification | Classification | LL | PL | PI | Cc | Cu | | |
|-------------------------|-------------------------------------|-------|-------|-------|---------|-------|-------|-------|
| ● B1-3 88.5 | POORLY GRADED SAND(SP) | | | | 0.93 | 7.59 | | |
| ☒ B1-4 10.0 | POORLY GRADED SAND(SP) | | | | 0.84 | 2.03 | | |
| ▲ B1-4 13.5 | POORLY GRADED SAND with SILT(SP-SM) | | | | 0.90 | 2.22 | | |
| ★ B1-4 18.5 | POORLY GRADED SAND(SP) | | | | 0.87 | 2.12 | | |
| ⊙ B1-4 28.5 | POORLY GRADED SAND(SP) | | | | 0.97 | 2.59 | | |
| Specimen Identification | D100 | D60 | D30 | D10 | %Gravel | %Sand | %Silt | %Clay |
| ● B1-3 88.5 | 12.5 | 1.691 | 0.593 | 0.223 | 11.9 | 84.2 | 3.9 | |
| ☒ B1-4 10.0 | 2.36 | 0.311 | 0.199 | 0.153 | 0.0 | 98.1 | 1.9 | |
| ▲ B1-4 13.5 | 4.75 | 0.305 | 0.194 | 0.138 | 0.0 | 94.9 | 5.1 | |
| ★ B1-4 18.5 | 2 | 0.33 | 0.212 | 0.156 | 0.0 | 97.3 | 2.7 | |
| ⊙ B1-4 28.5 | 12.5 | 0.474 | 0.289 | 0.183 | 3.3 | 95.0 | 1.7 | |

US GRAIN SIZE J042992.01.GPJ US LAB.GDT 11/14/23



GRAIN SIZE DISTRIBUTION
 Tyrnza River
 & Ditch No.6 Strs. & Apprs. (S)
 Poinsett County, Arkansas
 J042992.01



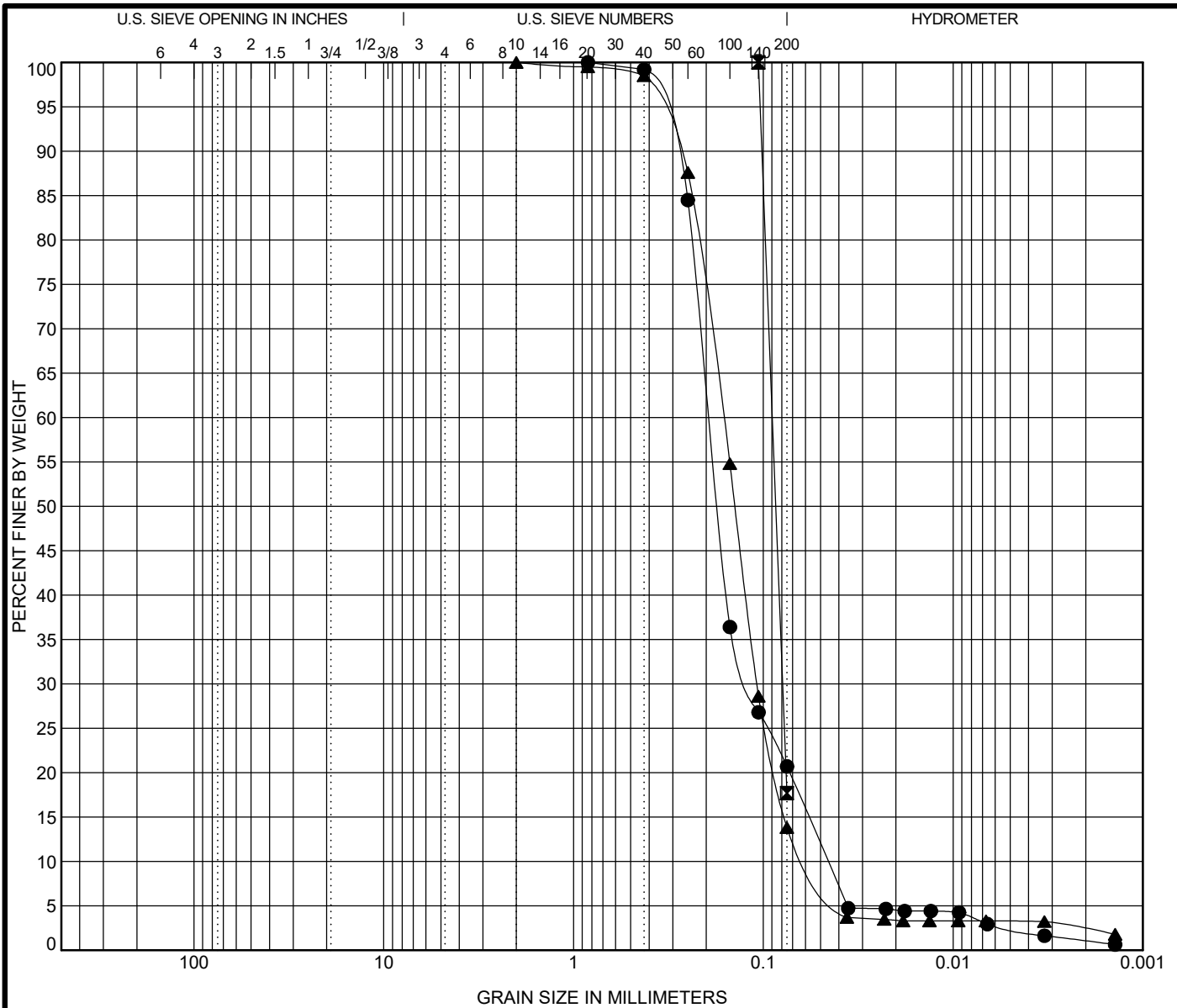
| COBBLES | GRAVEL | | SAND | | | SILT OR CLAY |
|---------|--------|------|--------|--------|------|--------------|
| | coarse | fine | coarse | medium | fine | |

| Specimen Identification | Classification | LL | PL | PI | Cc | Cu | | |
|-------------------------|-------------------------------------|-------|-------|-------|---------|-------|-------|-------|
| ● B1-4 38.5 | POORLY GRADED SAND with SILT(SP-SM) | | | | 1.26 | 2.63 | | |
| ☒ B1-5 6.0 | SANDY FAT CLAY(CH) | | | | | | | |
| ▲ B1-5 28.5 | CLAYEY SAND(SC) | | | | | | | |
| ★ B1-6 3.5 | SANDY LEAN CLAY(CL) | 45 | 19 | 26 | | | | |
| ⊙ B1-6 23.5 | CLAYEY SAND(SC) | | | | | | | |
| Specimen Identification | D100 | D60 | D30 | D10 | %Gravel | %Sand | %Silt | %Clay |
| ● B1-4 38.5 | 4.75 | 0.401 | 0.278 | 0.153 | 0.0 | 93.4 | 6.6 | |
| ☒ B1-5 6.0 | 2 | 0.048 | 0.002 | | 0.0 | 33.4 | 28.5 | 38.1 |
| ▲ B1-5 28.5 | 2 | 0.221 | 0.013 | | 0.0 | 54.4 | 21.5 | 24.1 |
| ★ B1-6 3.5 | 9.5 | | | | 0.2 | 39.4 | | 60.4 |
| ⊙ B1-6 23.5 | 0.106 | 0.088 | 0.076 | | 0.0 | 72.8 | | 27.2 |

U.S. GRAIN SIZE J042992.01.GPJ US LAB.GDT 11/14/23



GRAIN SIZE DISTRIBUTION
 Tyrnza River
 & Ditch No.6 Strs. & Apprs. (S)
 Poinsett County, Arkansas
 J042992.01



| COBBLES | GRAVEL | | SAND | | | SILT OR CLAY |
|---------|--------|------|--------|--------|------|--------------|
| | coarse | fine | coarse | medium | fine | |

| Specimen Identification | Classification | LL | PL | PI | Cc | Cu |
|-------------------------|----------------|----|----|----|------|------|
| ● B1-6 28.5 | SILTY SAND(SM) | | | | 1.61 | 4.23 |
| ☒ B1-6 33.5 | SILTY SAND(SM) | | | | | |
| ▲ B1-6 38.5 | SILTY SAND(SM) | | | | 1.26 | 2.85 |

| Specimen Identification | D100 | D60 | D30 | D10 | %Gravel | %Sand | %Silt | %Clay |
|-------------------------|-------|-------|-------|-------|---------|-------|-------|-------|
| ● B1-6 28.5 | 0.84 | 0.193 | 0.119 | 0.046 | 0.0 | 79.3 | 18.3 | 2.4 |
| ☒ B1-6 33.5 | 0.106 | 0.09 | 0.079 | | 0.0 | 82.3 | 17.7 | |
| ▲ B1-6 38.5 | 2 | 0.163 | 0.108 | 0.057 | 0.0 | 86.2 | 10.5 | 3.3 |

US GRAIN SIZE J042992.01.GPJ US LAB.GDT 11/14/23



GRAIN SIZE DISTRIBUTION
 Tyrnza River
 & Ditch No.6 Strs. & Apprs. (S)
 Poinsett County, Arkansas
 J042992.01

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS ASTM D2850

CLIENT : Arkansas State Highway and Transportation Department
 PROJECT NO.: J042992.01
 PROJECT: ARDOT 2023-2027 Contract TO G003 Tyrnza River & Ditch No. 6 Strs & App. Poinsett County
 LOCATION: Poinsett County, Arkansas

DATE: 7/24/2023

BORING NO.: B1-1
 SAMPLE OBTAINED BY: Shelby Tube
 SAMPLE DESCRIPTION: Brown/Gray Fat Clay

SAMPLE NO.: ST-2
 CONDITION: Undisturbed

DEPTH (ft.): 3.0-5.0

LIQUID LIMIT (%): 89 PLASTIC LIMIT (%): 23 PLASTICITY INDEX (%): 66 USCS: CH

SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)

LOAD CELL NO.:

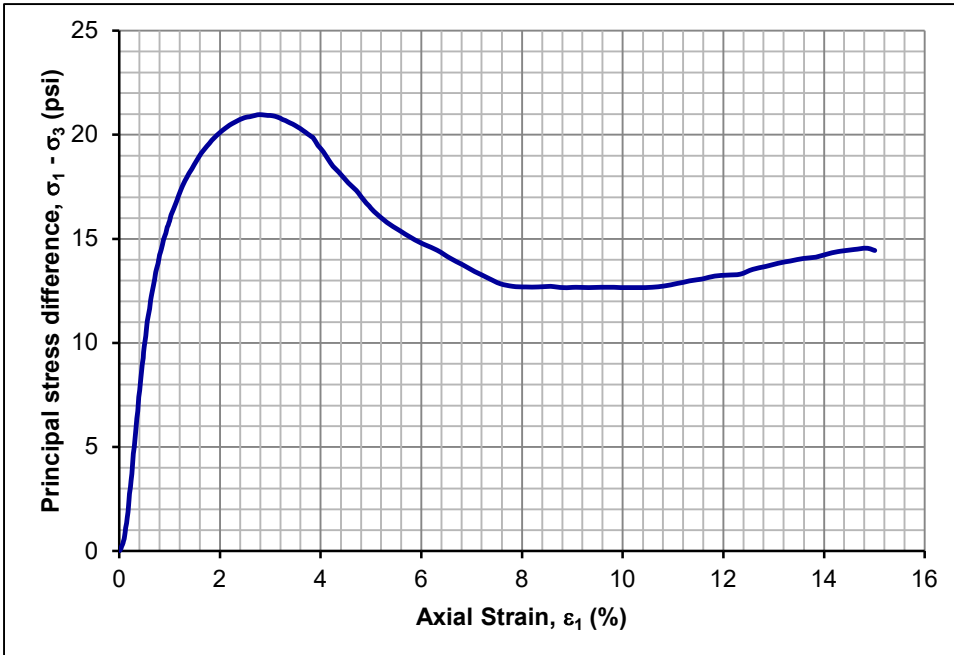
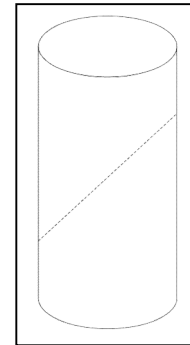
INITIAL SAMPLE DATA

FAILURE DATA***

| | |
|---------------------------|-------|
| AVERAGE DIAMETER (in.): | 2.85 |
| HEIGHT (in.): | 6.00 |
| HEIGHT TO DIAMETER RATIO: | 2.11 |
| WET UNIT WEIGHT (pcf): | 116.8 |
| DRY UNIT WEIGHT (pcf): | 84.6 |
| VOID RATIO: | 1.03 |
| MOISTURE CONTENT (%)*: | 38.1 |
| DEGREE OF SATURATION (%): | 100.0 |

| | |
|--|--------------|
| MOISTURE CONTENT AFTER FAILURE (%)**: | 35.7 |
| AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.): | 1.0 |
| AXIAL STRAIN AT FAILURE (%): | 2.7 |
| PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi): | 21.0 |
| MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi): | 2.3 |
| MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi): | 23.3 |
| UNDRAINED COMPRESSIVE STRENGTH, U_u (psf): | 3,020 |
| UNDRAINED SHEAR STRENGTH, s_u (psf): | 1,510 |
| LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf): | N/A |

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 : Arkansas State Highway and Transportation Department
 PROJECT NO.: J042992.01
 PROJECT: ARDOT 2023-2027 Contract TO G003 Tyrnza River & Ditch No. 6 Strs & App. Poinsett County
 LOCATION: Poinsett County, Arkansas

DATE: 7/24/2023

BORING NO.: B1-1
 SAMPLE OBTAINED BY: Shelby Tube
 SAMPLE DESCRIPTION: Brown Fat Clay

SAMPLE NO.: ST-5
 CONDITION: Undisturbed

DEPTH (ft.): 10.0-12.0

LIQUID LIMIT (%): 90 PLASTIC LIMIT (%): 32 PLASTICITY INDEX (%): 58 USCS: CH

SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)

LOAD CELL NO.:

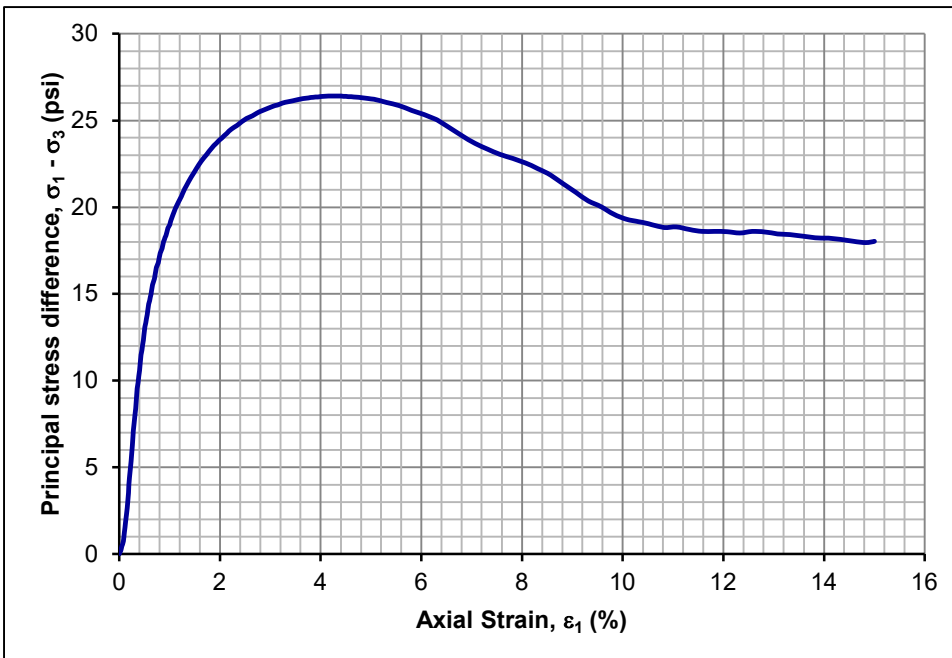
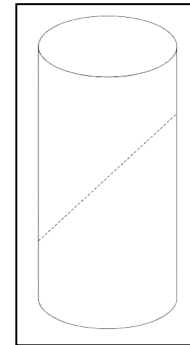
INITIAL SAMPLE DATA

FAILURE DATA***

| | |
|---------------------------|-------|
| AVERAGE DIAMETER (in.): | 2.85 |
| HEIGHT (in.): | 5.96 |
| HEIGHT TO DIAMETER RATIO: | 2.10 |
| WET UNIT WEIGHT (pcf): | 114.6 |
| DRY UNIT WEIGHT (pcf): | 83.9 |
| VOID RATIO: | 1.05 |
| MOISTURE CONTENT (%)*: | 36.7 |
| DEGREE OF SATURATION (%): | 96.4 |

| | |
|--|-------|
| MOISTURE CONTENT AFTER FAILURE (%)**: | 37.8 |
| AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.): | 1.0 |
| AXIAL STRAIN AT FAILURE (%): | 4.4 |
| PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi): | 26.4 |
| MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi): | 6.4 |
| MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi): | 32.8 |
| UNDRAINED COMPRESSIVE STRENGTH, U_u (psf): | 3,800 |
| UNDRAINED SHEAR STRENGTH, s_u (psf): | 1,900 |
| LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf): | N/A |

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 : Arkansas State Highway and Transportation Department
 PROJECT NO.: J042992.01
 PROJECT: ARDOT 2023-2027 Contract TO G003 Tyrnza River & Ditch No. 6 Strs & App. Poinsett County
 LOCATION: Poinsett County, Arkansas

DATE: 7/24/2023

BORING NO.: B1-1
 SAMPLE OBTAINED BY: Shelby Tube
 SAMPLE DESCRIPTION: Brown/Black Fat Clay

SAMPLE NO.: ST-7
 CONDITION: Undisturbed

DEPTH (ft.): 15.0-17.0

LIQUID LIMIT (%): 87 PLASTIC LIMIT (%): 35 PLASTICITY INDEX (%): 52 USCS: CH

SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)

LOAD CELL NO.:

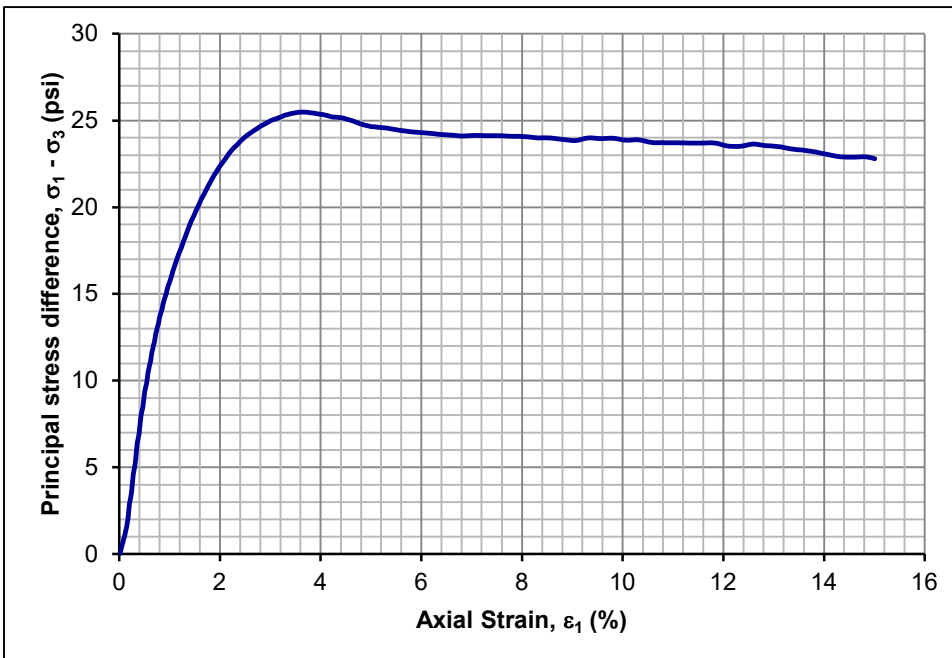
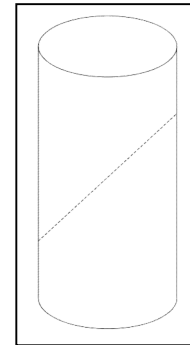
INITIAL SAMPLE DATA

| | |
|---------------------------|-------|
| AVERAGE DIAMETER (in.): | 2.85 |
| HEIGHT (in.): | 6.00 |
| HEIGHT TO DIAMETER RATIO: | 2.11 |
| WET UNIT WEIGHT (pcf): | 111.3 |
| DRY UNIT WEIGHT (pcf): | 76.6 |
| VOID RATIO: | 1.24 |
| MOISTURE CONTENT (%)*: | 45.4 |
| DEGREE OF SATURATION (%): | 100.0 |

FAILURE DATA***

| | |
|--|--------------|
| MOISTURE CONTENT AFTER FAILURE (%)**: | 48.2 |
| AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.): | 1.0 |
| AXIAL STRAIN AT FAILURE (%): | 3.6 |
| PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi): | 25.5 |
| MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi): | 9.3 |
| MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi): | 34.8 |
| UNDRAINED COMPRESSIVE STRENGTH, U_u (psf): | 3,670 |
| UNDRAINED SHEAR STRENGTH, s_u (psf): | 1,835 |
| LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf): | N/A |

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 : Arkansas State Highway and Transportation Department
 PROJECT NO.: J042992.01
 PROJECT: ARDOT 2023-2027 Contract TO G003 Tyrnza River & Ditch No. 6 Strs & App. Poinsett County
 LOCATION: Poinsett County, Arkansas

DATE: 7/26/2023

BORING NO.: B1-2
 SAMPLE OBTAINED BY: Shelby Tube
 SAMPLE DESCRIPTION: Brown Fat Clay

SAMPLE NO.: ST-7
 CONDITION: Undisturbed

DEPTH (ft.): 15.0-17.0

LIQUID LIMIT (%): 106 PLASTIC LIMIT (%): 46 PLASTICITY INDEX (%): 60 USCS: CH

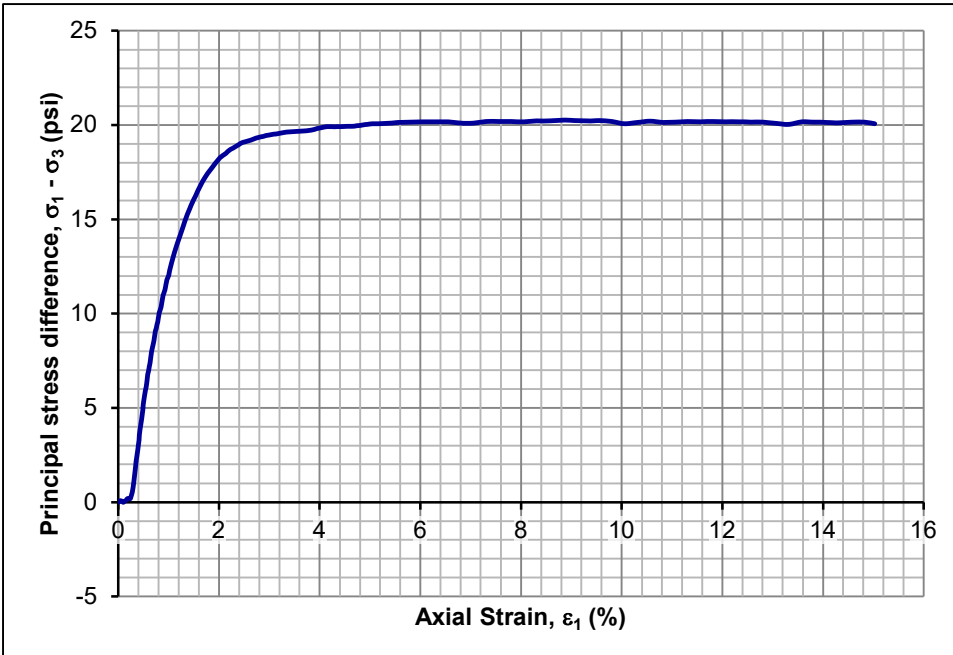
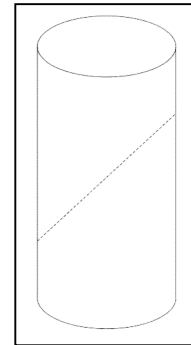
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)

LOAD CELL NO.:

| INITIAL SAMPLE DATA | |
|---------------------------|-------|
| AVERAGE DIAMETER (in.): | 2.84 |
| HEIGHT (in.): | 5.97 |
| HEIGHT TO DIAMETER RATIO: | 2.10 |
| WET UNIT WEIGHT (pcf): | 109.1 |
| DRY UNIT WEIGHT (pcf): | 72.2 |
| VOID RATIO: | 1.38 |
| MOISTURE CONTENT (%)*: | 51.1 |
| DEGREE OF SATURATION (%): | 100.0 |

| FAILURE DATA*** | |
|--|--------------|
| MOISTURE CONTENT AFTER FAILURE (%)**: | 50.1 |
| AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.): | 1.0 |
| AXIAL STRAIN AT FAILURE (%): | 8.8 |
| PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi): | 20.3 |
| MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi): | 9.3 |
| MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi): | 29.6 |
| UNDRAINED COMPRESSIVE STRENGTH, U_u (psf): | 2,920 |
| UNDRAINED SHEAR STRENGTH, s_u (psf): | 1,460 |
| LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf): | N/A |

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 : Arkansas State Highway and Transportation Department
 PROJECT NO.: J042992.01
 PROJECT: ARDOT 2023-2027 Contract TO G003 Tyrnza River & Ditch No. 6 Strs & App. Poinsett County
 LOCATION: Poinsett County, Arkansas

DATE: 8/1/2023

BORING NO.: B1-2
 SAMPLE OBTAINED BY: Shelby Tube
 SAMPLE DESCRIPTION: Brown Fat Clay

SAMPLE NO.: ST-9
 CONDITION: Undisturbed

DEPTH (ft.): 20.0-22.0

LIQUID LIMIT (%): 70 PLASTIC LIMIT (%): 28 PLASTICITY INDEX (%): 42 USCS: CH

SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)

LOAD CELL NO.:

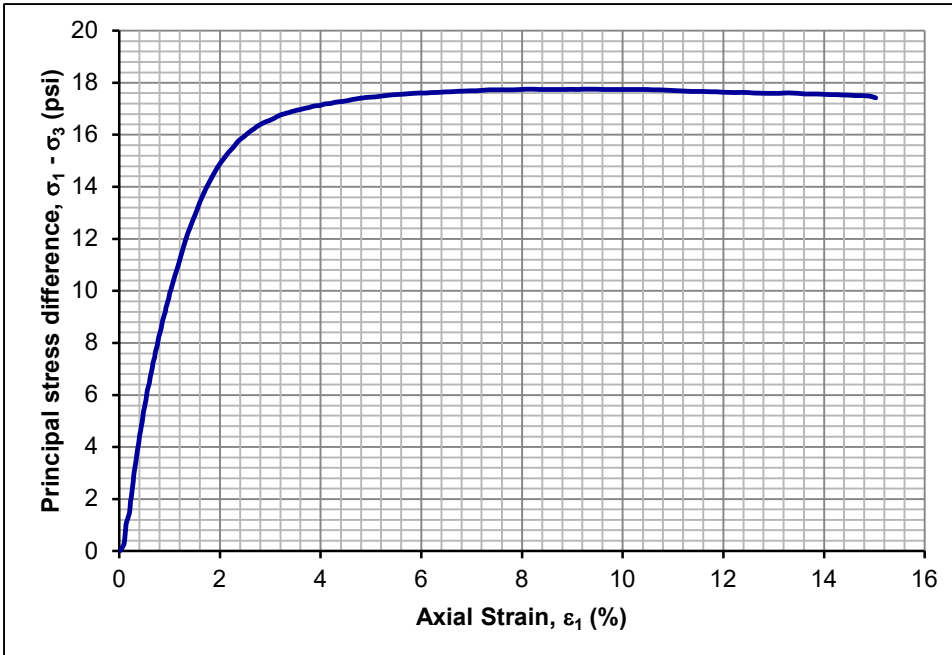
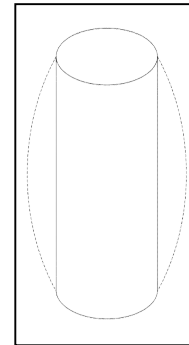
INITIAL SAMPLE DATA

| | |
|---------------------------|-------|
| AVERAGE DIAMETER (in.): | 2.81 |
| HEIGHT (in.): | 5.97 |
| HEIGHT TO DIAMETER RATIO: | 2.12 |
| WET UNIT WEIGHT (pcf): | 114.9 |
| DRY UNIT WEIGHT (pcf): | 78.8 |
| VOID RATIO: | 1.18 |
| MOISTURE CONTENT (%)*: | 45.7 |
| DEGREE OF SATURATION (%): | 100.0 |

FAILURE DATA***

| | |
|--|--------------|
| MOISTURE CONTENT AFTER FAILURE (%)**: | 43.6 |
| AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.): | 1.0 |
| AXIAL STRAIN AT FAILURE (%): | 9.4 |
| PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi): | 17.8 |
| MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi): | 12.3 |
| MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi): | 30.0 |
| UNDRAINED COMPRESSIVE STRENGTH, U_u (psf): | 2,560 |
| UNDRAINED SHEAR STRENGTH, s_u (psf): | 1,280 |
| LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf): | N/A |

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 : Arkansas State Highway and Transportation Department
 PROJECT NO.: J042992.01
 PROJECT: ARDOT 2023-2027 Contract TO G003 Tyrnza River & Ditch No. 6 Strs & App. Poinsett County
 LOCATION: Poinsett County, Arkansas

DATE: 7/27/2023

BORING NO.: B1-5
 SAMPLE OBTAINED BY: Shelby Tube
 SAMPLE DESCRIPTION: Brown Fat Clay

SAMPLE NO.: ST-4
 CONDITION: Undisturbed

DEPTH (ft.): 8.0-10.0

LIQUID LIMIT (%): 66 PLASTIC LIMIT (%): 19 PLASTICITY INDEX (%): 47 USCS: CH

SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)

LOAD CELL NO.:

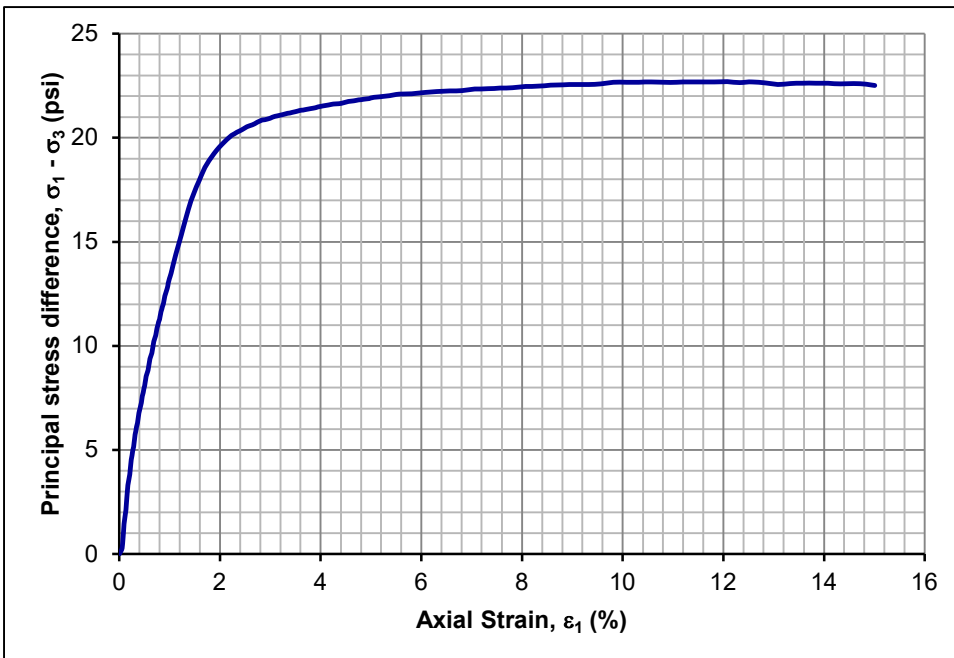
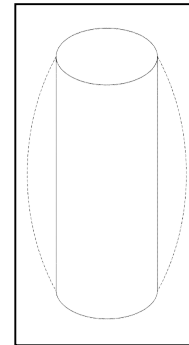
INITIAL SAMPLE DATA

FAILURE DATA***

| | |
|---------------------------|-------|
| AVERAGE DIAMETER (in.): | 2.84 |
| HEIGHT (in.): | 5.95 |
| HEIGHT TO DIAMETER RATIO: | 2.09 |
| WET UNIT WEIGHT (pcf): | 121.5 |
| DRY UNIT WEIGHT (pcf): | 95.4 |
| VOID RATIO: | 0.80 |
| MOISTURE CONTENT (%)*: | 27.4 |
| DEGREE OF SATURATION (%): | 94.3 |

| | |
|--|--------------|
| MOISTURE CONTENT AFTER FAILURE (%)**: | 26.3 |
| AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.): | 1.0 |
| AXIAL STRAIN AT FAILURE (%): | 12.1 |
| PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi): | 22.7 |
| MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi): | 4.1 |
| MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi): | 26.8 |
| UNDRAINED COMPRESSIVE STRENGTH, U_u (psf): | 3,270 |
| UNDRAINED SHEAR STRENGTH, s_u (psf): | 1,635 |
| LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf): | 3,265 |

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 : Arkansas State Highway and Transportation Department
 PROJECT NO.: J042992.01
 PROJECT: ARDOT 2023-2027 Contract TO G003 Tyrnza River & Ditch No. 6 Strs & App. Poinsett County
 LOCATION: Poinsett County, Arkansas

DATE: 7/27/2023

BORING NO.: B1-5
 SAMPLE OBTAINED BY: Shelby Tube
 SAMPLE DESCRIPTION: Gray Fat Clay

SAMPLE NO.: ST-7
 CONDITION: Undisturbed

DEPTH (ft.): 20.0-22.0

LIQUID LIMIT (%): 50 PLASTIC LIMIT (%): 20 PLASTICITY INDEX (%): 30 USCS: CH

SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)

LOAD CELL NO.:

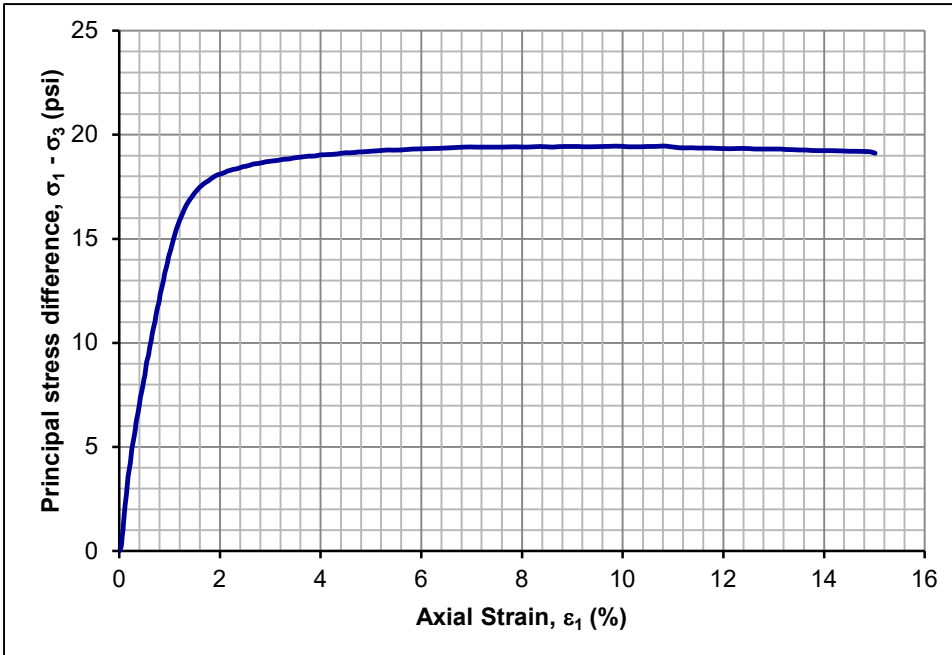
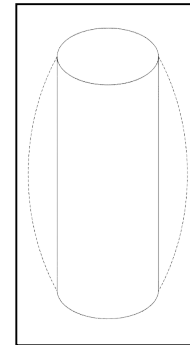
INITIAL SAMPLE DATA

FAILURE DATA***

| | |
|---------------------------|-------|
| AVERAGE DIAMETER (in.): | 2.85 |
| HEIGHT (in.): | 5.91 |
| HEIGHT TO DIAMETER RATIO: | 2.07 |
| WET UNIT WEIGHT (pcf): | 118.8 |
| DRY UNIT WEIGHT (pcf): | 92.8 |
| VOID RATIO: | 0.85 |
| MOISTURE CONTENT (%)*: | 28.1 |
| DEGREE OF SATURATION (%): | 91.0 |

| | |
|--|--------------|
| MOISTURE CONTENT AFTER FAILURE (%)**: | 26.0 |
| AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.): | 1.0 |
| AXIAL STRAIN AT FAILURE (%): | 10.9 |
| PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi): | 19.5 |
| MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi): | 12.3 |
| MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi): | 31.7 |
| UNDRAINED COMPRESSIVE STRENGTH, U_u (psf): | 2,800 |
| UNDRAINED SHEAR STRENGTH, s_u (psf): | 1,400 |
| LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf): | 2,800 |

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 : Arkansas State Highway and Transportation Department
 PROJECT NO.: J042992.01
 PROJECT: ARDOT 2023-2027 Contract TO G003 Tyrnza River & Ditch No. 6 Strs & App. Poinsett County
 LOCATION: Poinsett County, Arkansas

DATE: 7/24/2023

BORING NO.: B1-6
 SAMPLE OBTAINED BY: Shelby Tube
 SAMPLE DESCRIPTION: Gray/Brown FAT Clay

SAMPLE NO.: ST-5
 CONDITION: Undisturbed

DEPTH (ft.): 10.0-12.0

LIQUID LIMIT (%): 72 PLASTIC LIMIT (%): 22 PLASTICITY INDEX (%): 50 USCS: CH

SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)

LOAD CELL NO.:

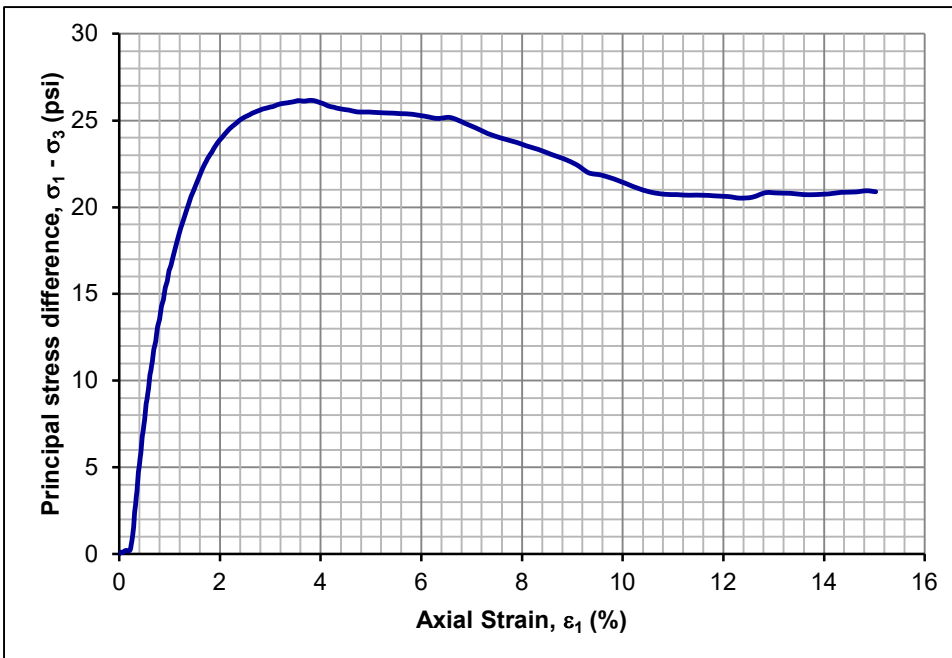
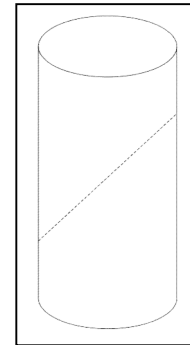
INITIAL SAMPLE DATA

FAILURE DATA***

| | |
|---------------------------|-------|
| AVERAGE DIAMETER (in.): | 2.85 |
| HEIGHT (in.): | 6.04 |
| HEIGHT TO DIAMETER RATIO: | 2.12 |
| WET UNIT WEIGHT (pcf): | 119.8 |
| DRY UNIT WEIGHT (pcf): | 92.1 |
| VOID RATIO: | 0.86 |
| MOISTURE CONTENT (%)*: | 30.1 |
| DEGREE OF SATURATION (%): | 95.7 |

| | |
|--|-------|
| MOISTURE CONTENT AFTER FAILURE (%)**: | 31.0 |
| AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.): | 1.0 |
| AXIAL STRAIN AT FAILURE (%): | 3.9 |
| PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi): | 26.2 |
| MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi): | 6.4 |
| MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi): | 32.6 |
| UNDRAINED COMPRESSIVE STRENGTH, U_u (psf): | 3,770 |
| UNDRAINED SHEAR STRENGTH, s_u (psf): | 1,885 |
| LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf): | N/A |

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 : Arkansas State Highway and Transportation Department
 PROJECT NO.: J042992.01
 PROJECT: ARDOT 2023-2027 Contract TO G003 Tyronza River & Ditch No. 6 Strs & App. Poinsett County
 LOCATION: Poinsett County, Arkansas

DATE: 7/26/2023

BORING NO.: B1-6
 SAMPLE OBTAINED BY: Shelby Tube
 SAMPLE DESCRIPTION: Gray Fat Clay

SAMPLE NO.: ST-7
 CONDITION: Undisturbed

DEPTH (ft.): 15.0-17.0

LIQUID LIMIT (%): 80 PLASTIC LIMIT (%): 28 PLASTICITY INDEX (%): 52 USCS: CH

SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)

LOAD CELL NO.:

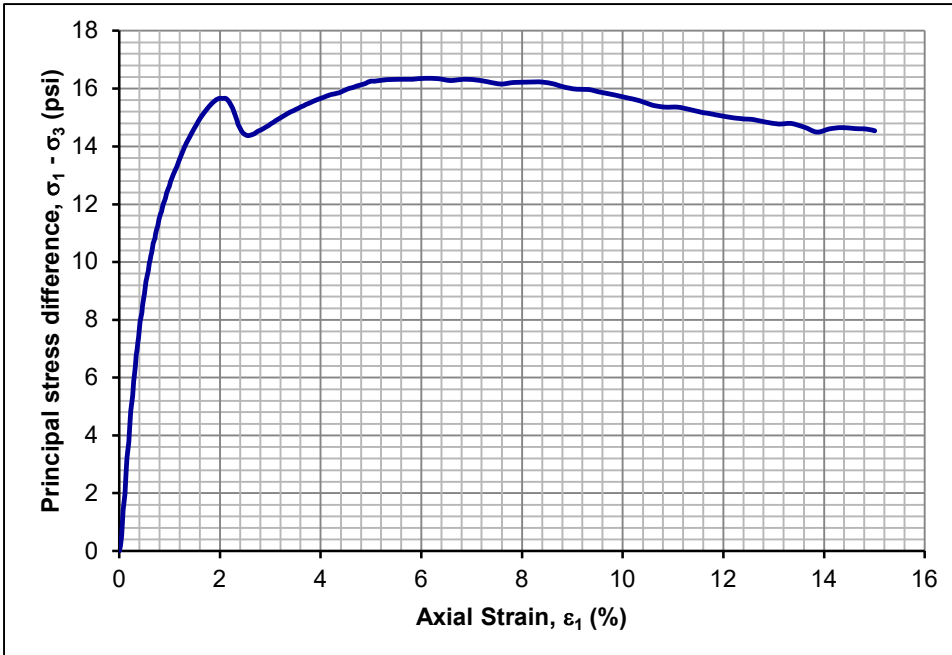
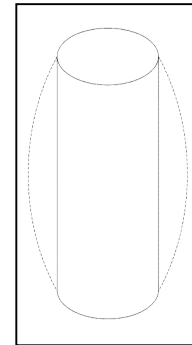
INITIAL SAMPLE DATA

FAILURE DATA***

| | |
|---------------------------|-------|
| AVERAGE DIAMETER (in.): | 2.85 |
| HEIGHT (in.): | 5.96 |
| HEIGHT TO DIAMETER RATIO: | 2.09 |
| WET UNIT WEIGHT (pcf): | 116.1 |
| DRY UNIT WEIGHT (pcf): | 87.2 |
| VOID RATIO: | 0.97 |
| MOISTURE CONTENT (%)*: | 33.1 |
| DEGREE OF SATURATION (%): | 94.1 |

| | |
|--|--------------|
| MOISTURE CONTENT AFTER FAILURE (%)**: | 35.8 |
| AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.): | 1.0 |
| AXIAL STRAIN AT FAILURE (%): | 6.1 |
| PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi): | 16.4 |
| MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi): | 9.3 |
| MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi): | 25.7 |
| UNDRAINED COMPRESSIVE STRENGTH, U_u (psf): | 2,350 |
| UNDRAINED SHEAR STRENGTH, s_u (psf): | 1,175 |
| LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf): | N/A |

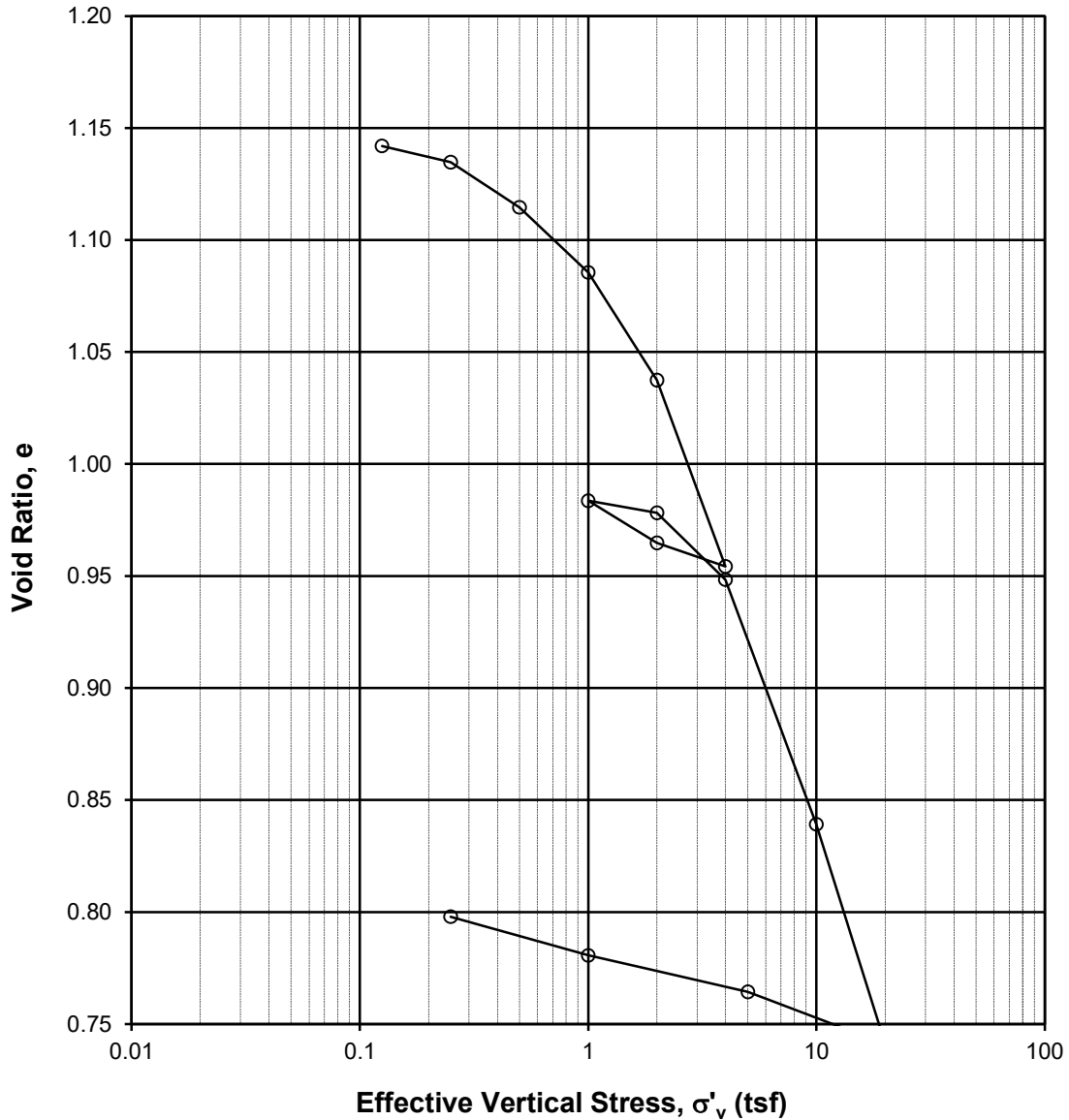
FAILURE SHAPES



REMARKS :

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

| | | | |
|--|---|------------------------------|-----------------|
| Liquid Limit= <u>90</u> | Plastic Limit= <u>32</u> | Plasticity Index = <u>58</u> | USCS: <u>CH</u> |
| Compression Index, $C_c =$ <u>0.30</u> | Void Ratio, $e_o =$ <u>1.142</u> | | |
| Recompression Index, $C_r =$ <u>0.05</u> | Preconsolidation Pressure = <u>1.50</u> tsf | | |



1-D CONSOLIDATION TEST: INCREMENTAL

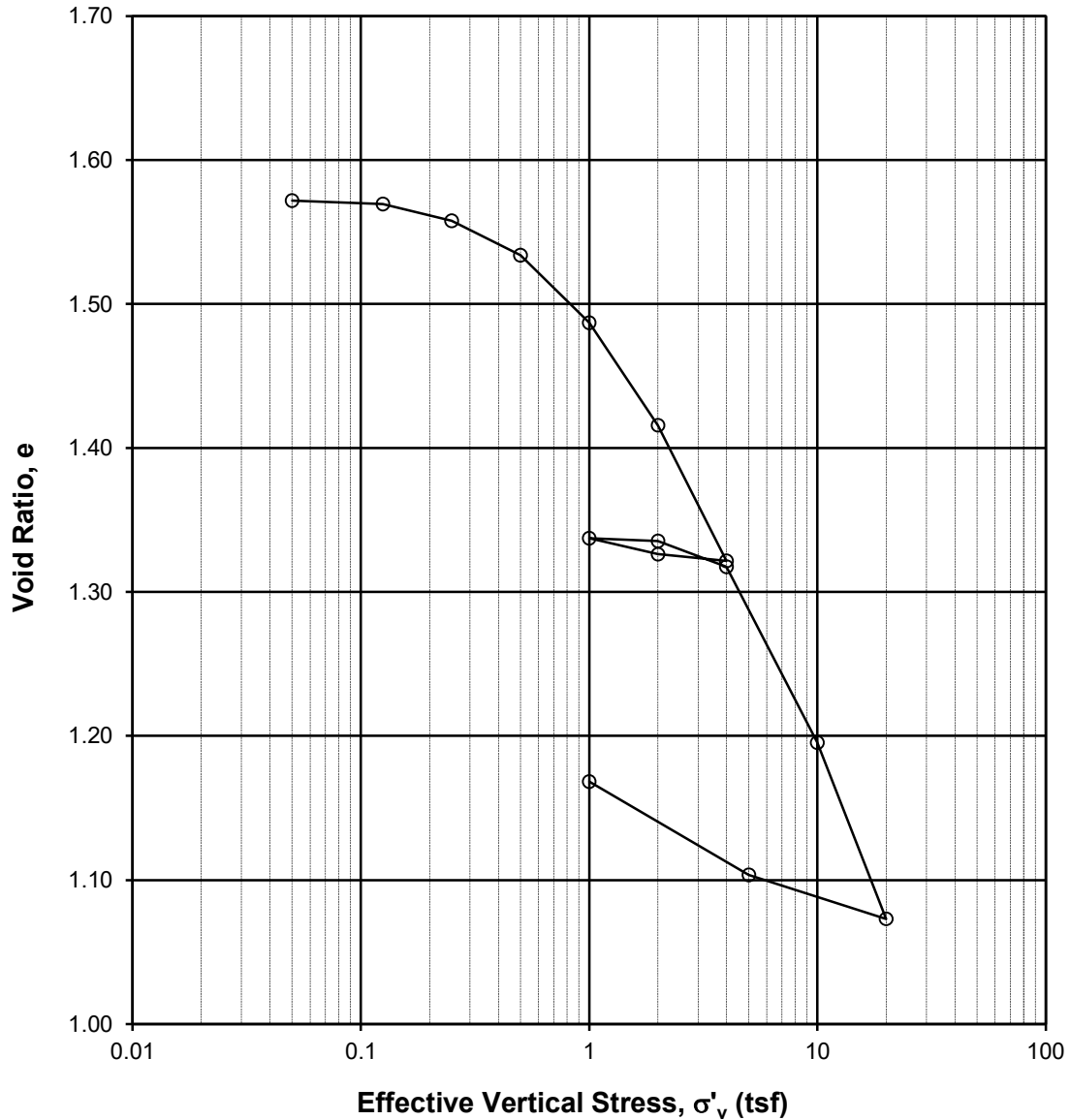
ASTM D 2435

Project No.: J042992.01

Boring: B1-1

Sample: ST-5 - Depth: 10.0

| | | | |
|--|---|------------------------------|-----------------|
| Liquid Limit= <u>102</u> | Plastic Limit= <u>46</u> | Plasticity Index = <u>56</u> | USCS: <u>CH</u> |
| Compression Index, C_c = <u>0.35</u> | Void Ratio, e_o = <u>1.572</u> | | |
| Recompression Index, C_r = <u>0.06</u> | Preconsolidation Pressure = <u>1.20</u> tsf | | |



1-D CONSOLIDATION TEST: INCREMENTAL

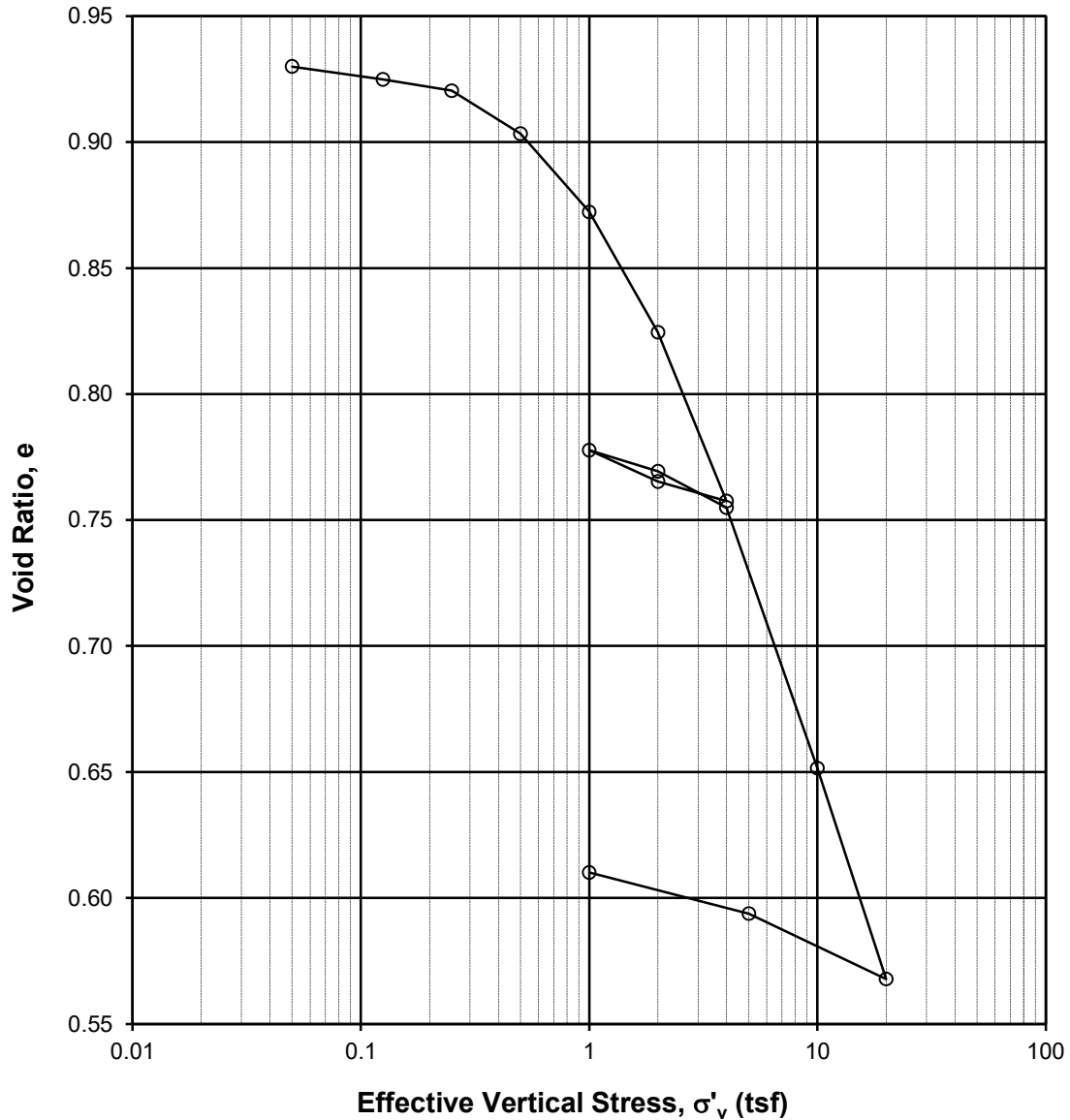
ASTM D 2435

Project No.: J042992.01

Boring: B1-2

Sample: ST-7 - Depth: 15.0

| | | | |
|--|---|------------------------------|-----------------|
| Liquid Limit= <u>66</u> | Plastic Limit= <u>19</u> | Plasticity Index = <u>47</u> | USCS: <u>CH</u> |
| Compression Index, $C_c =$ <u>0.27</u> | Void Ratio, $e_o =$ <u>0.93</u> | | |
| Recompression Index, $C_r =$ <u>0.04</u> | Preconsolidation Pressure = <u>1.20</u> tsf | | |



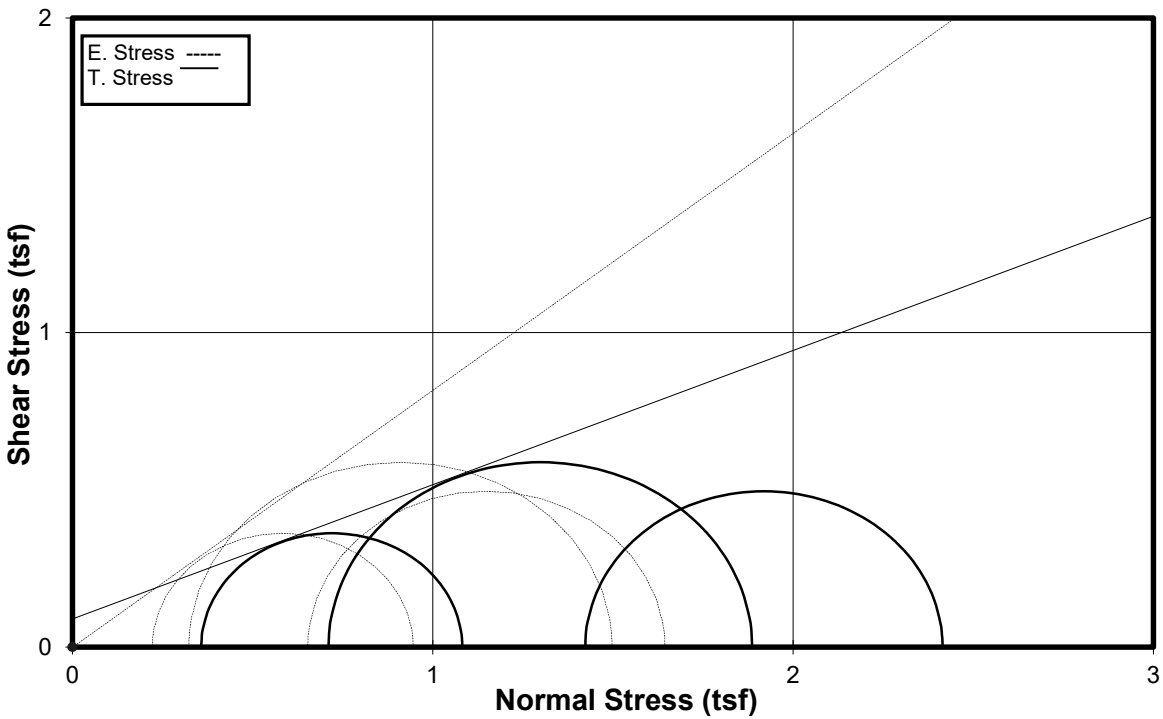
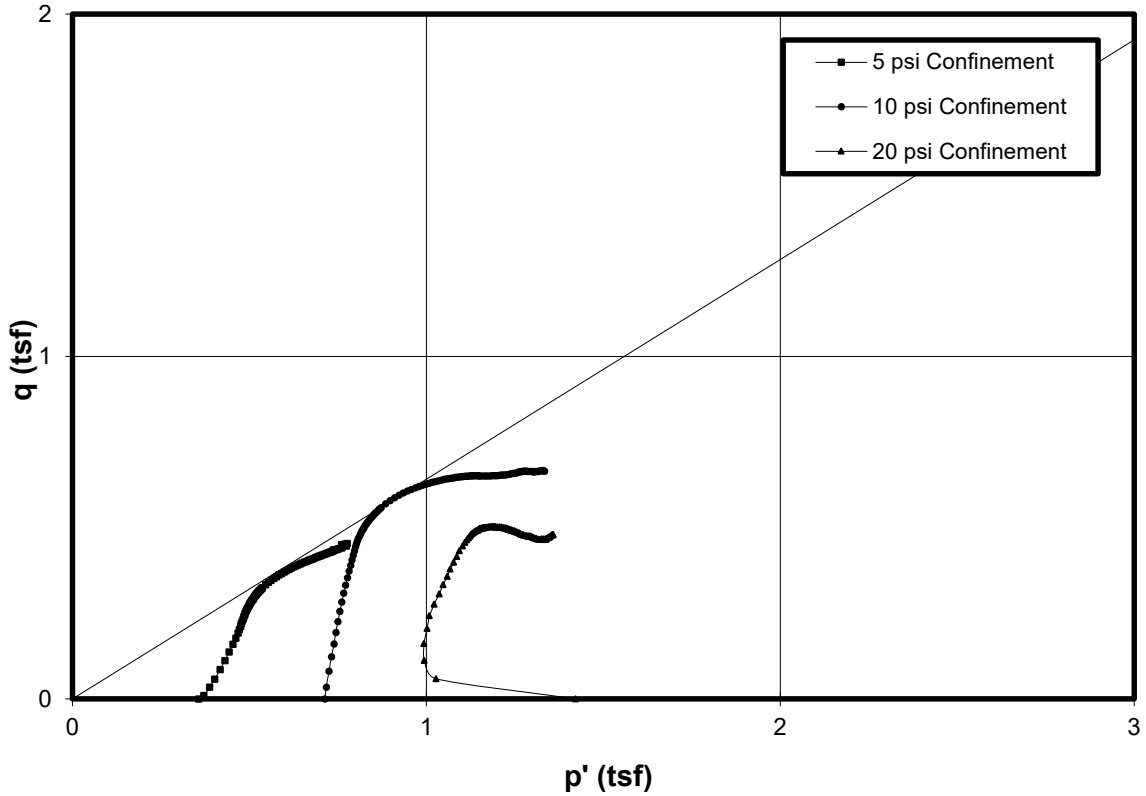
1-D CONSOLIDATION TEST: INCREMENTAL

ASTM D 2435

Project No.: J042992.01

Boring: B1-5

Sample: ST-4 - Depth: 8.0



CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 4767

Project No.: J042992.01

Boring: B1-2

Sample: ST-5 - Depth: 10.0



SOIL RESISTIVITY TEST REPORT

| | | |
|----------------|---|----------------------|
| Project No.: | J042992.01 | September 20th, 2023 |
| Project Name: | ARDOT 101017 Poinsett County – Bridge 1 | Page 1 of 1 |
| Boring Number: | B1-2 | |
| Sample ID: | SS10 | |
| Depth (ft): | 23.5 | |

MINIMUM LABORATORY SOIL RESISTIVITY AASHTO T288

| <u>Reading</u> | <u>Resistance Measurement</u> | <u>Soil Box Factor (cm)</u> | <u>Soil Resistivity (ohms-cm)</u> | <u>Moisture Content (%)</u> |
|----------------|-------------------------------|-----------------------------|-----------------------------------|-----------------------------|
| #1 | 35,500 | 0.57 | 20,235.00 | 19.8 |
| #2 | 1,333 | 0.57 | 759.81 | 24.0 |
| #3 | 1,455 | 0.57 | 829.35 | 30.2 |

Minimum Soil Resistivity **759.81**



SOIL RESISTIVITY TEST REPORT

| | | |
|----------------|---|----------------------|
| Project No.: | J042992.01 | September 21st, 2023 |
| Project Name: | ARDOT 101017 Poinsett County – Bridge 1 | Page 1 of 1 |
| Boring Number: | B1-2 | |
| Sample ID: | SS14 | |
| Depth (ft): | 43.5 | |

MINIMUM LABORATORY SOIL RESISTIVITY AASHTO T288

| <u>Reading</u> | <u>Resistance Measurement</u> | <u>Soil Box Factor (cm)</u> | <u>Soil Resistivity (ohms-cm)</u> | <u>Moisture Content (%)</u> |
|----------------|-------------------------------|-----------------------------|-----------------------------------|-----------------------------|
| #1 | 11,830 | 0.57 | 6,743.10 | 10.2 |
| #2 | 8,050 | 0.57 | 4,588.50 | 17.2 |
| #3 | 4,630 | 0.57 | 2,639.10 | 25.4 |
| #4 | 5,450 | 0.57 | 3,106.50 | 29.1 |

Minimum Soil Resistivity **2,639.10**



SOIL RESISTIVITY TEST REPORT

| | | |
|-----------------------|--|--------------------------|
| Project No.: | J042992.01 | August 15th, 2023 |
| Project Name: | ARDOT 101017 Poinsett County – Bridge 1 | Page 1 of 1 |
| Boring Number: | B1-2 | |
| Sample ID: | ST-9 | |
| Depth (ft): | 20.0 | |

MINIMUM LABORATORY SOIL RESISTIVITY AASHTO T288

| <u>Reading</u> | <u>Resistance Measurement</u> | <u>Soil Box Factor (cm)</u> | <u>Soil Resistivity (ohms-cm)</u> | <u>Moisture Content (%)</u> |
|----------------|-------------------------------|-----------------------------|-----------------------------------|-----------------------------|
| #1 | 7,213 | 0.57 | 4,111.41 | 19.8 |
| #2 | 2,873 | 0.57 | 1,637.61 | 24.0 |
| #3 | 1,283 | 0.57 | 731.31 | 30.2 |
| #4 | 1,033 | 0.57 | 588.81 | 40.4 |
| #5 | 1,076 | 0.57 | 613.32 | 47.6 |

Minimum Soil Resistivity **588.81**



SOIL RESISTIVITY TEST REPORT

| | | |
|----------------|---|----------------------|
| Project No.: | J042992.01 | September 21st, 2023 |
| Project Name: | ARDOT 101017 Poinsett County – Bridge 1 | Page 1 of 1 |
| Boring Number: | B1-4 | |
| Sample ID: | SS12 | |
| Depth (ft): | 38.5 | |

MINIMUM LABORATORY SOIL RESISTIVITY AASHTO T288

| <u>Reading</u> | <u>Resistance Measurement</u> | <u>Soil Box Factor (cm)</u> | <u>Soil Resistivity (ohms-cm)</u> | <u>Moisture Content (%)</u> |
|----------------|-------------------------------|-----------------------------|-----------------------------------|-----------------------------|
| #1 | 24,290 | 0.57 | 13,845.30 | 9.9 |
| #2 | 10,630 | 0.57 | 6,059.10 | 16.5 |
| #3 | 10,050 | 0.57 | 5,728.50 | 23.5 |
| #4 | 11,390 | 0.57 | 6,492.30 | 26.0 |

Minimum Soil Resistivity **5,728.50**



SOIL RESISTIVITY TEST REPORT

| | | |
|-----------------------|--|-----------------------------|
| Project No.: | J042992.01 | September 21st, 2023 |
| Project Name: | ARDOT 101017 Poinsett County – Bridge 1 | Page 1 of 1 |
| Boring Number: | B1-5 | |
| Sample ID: | SS14 | |
| Depth (ft): | 53.5 | |

MINIMUM LABORATORY SOIL RESISTIVITY AASHTO T288

| <u>Reading</u> | <u>Resistance Measurement</u> | <u>Soil Box Factor (cm)</u> | <u>Soil Resistivity (ohms-cm)</u> | <u>Moisture Content (%)</u> |
|----------------|-------------------------------|-----------------------------|-----------------------------------|-----------------------------|
| #1 | 12,330 | 0.57 | 7,028.10 | 13.5 |
| #2 | 8,230 | 0.57 | 4,691.10 | 20.4 |
| #3 | 1,350 | 0.57 | 769.50 | 25.4 |
| #4 | 10,830 | 0.57 | 6,173.10 | 25.3 |

Minimum Soil Resistivity **769.50**



SOIL RESISTIVITY TEST REPORT

| | | |
|-----------------------|--|--------------------------|
| Project No.: | J042992.01 | August 15th, 2023 |
| Project Name: | ARDOT 101017 Poinsett County – Bridge 1 | Page 1 of 1 |
| Boring Number: | B1-5 | |
| Sample ID: | ST-7 | |
| Depth (ft): | 20.0 | |

MINIMUM LABORATORY SOIL RESISTIVITY AASHTO T288

| <u>Reading</u> | <u>Resistance Measurement</u> | <u>Soil Box Factor (cm)</u> | <u>Soil Resistivity (ohms-cm)</u> | <u>Moisture Content (%)</u> |
|----------------|-------------------------------|-----------------------------|-----------------------------------|-----------------------------|
| #1 | 2,385 | 0.57 | 1,359.45 | 15.6 |
| #2 | 1,159 | 0.57 | 660.63 | 22.5 |
| #3 | 1,385 | 0.57 | 789.45 | 27.4 |

Minimum Soil Resistivity 660.63



SOIL RESISTIVITY TEST REPORT

| | | |
|-----------------------|--|-----------------------------|
| Project No.: | J042992.01 | September 21st, 2023 |
| Project Name: | ARDOT 101017 Poinsett County – Bridge 1 | Page 1 of 1 |
| Boring Number: | B1-6 | |
| Sample ID: | SS5 | |
| Depth (ft): | 13.5 | |

MINIMUM LABORATORY SOIL RESISTIVITY AASHTO T288

| <u>Reading</u> | <u>Resistance Measurement</u> | <u>Soil Box Factor (cm)</u> | <u>Soil Resistivity (ohms-cm)</u> | <u>Moisture Content (%)</u> |
|----------------|-------------------------------|-----------------------------|-----------------------------------|-----------------------------|
| #1 | 5,523 | 0.57 | 3,148.11 | 17.2 |
| #2 | 1,145 | 0.57 | 652.65 | 25.6 |
| #3 | 1,103 | 0.57 | 628.71 | 40.8 |
| #4 | 1,155 | 0.57 | 658.35 | 42.2 |

Minimum Soil Resistivity **628.71**

pH TESTS (ASTM D 4972 or AASHTO T-289)



| | | |
|------------------|---|---------------------------|
| DATE 8/1/2023 | PROJECT NAME ARDOT 101017 Poinsett County - Bridge 1 | PROJECT NO. J042992.01 |
|------------------|---|---------------------------|

General Test Information: pH Meter: Humboldt Ph Testr H-4371 or _____
 Distilled Water: required pH=5.5 to 7.5 Measured value: _____
 Soil/Water Ratio: Typically 1/1 or 1/2, but 1/5 for lime stabilized soils

| Boring No. | Sample No. | Depth (ft) | Visual Identification (Color, Group Name & Symbol) | Soil : Water Ratio (g/g) or (g/mL) | pH of Solution (Meter/ Temp.) | Tare No. Air Drying | Jar Number | Remarks |
|------------|------------|------------|---|---|--|---------------------------|---------------|---------|
| B1-2 | ST-9 | 20.00 | | 1:1 | 6.66 ----- 23.3 | | | |
| B1-2 | SS-10 | 23.50 | | 1:1 | 6.40 ----- 22.4 | | | |
| B1-2 | SS-14 | 43.50 | | 1:1 | 7.61 ----- 22.1 | | | |
| B1-4 | SS-12 | 38.50 | | 1:1 | 8.04 ----- 22.6 | | | |
| B1-5 | ST-7 | 20.00 | | 1:1 | 7.12 ----- 21.7 | | | |
| B1-5 | SS-14 | 53.50 | | 1:1 | 8.07 ----- 22.6 | | | |
| B1-6 | SS-5 | 13.50 | | 1:1 | 7.29 ----- 22.5 | | | |
| | | | | | ----- | | | |
| | | | | | ----- | | | |
| | | | | | ----- | | | |
| | | | | | ----- | | | |
| | | | | | ----- | | | |
| | | | | | ----- | | | |
| | | | | | ----- | | | |
| | | | | | ----- | | | |
| | | | | | ----- | | | |

pH by Meter is Method A; pH by Paper is Method B

Tested By: MM
Date: 08/01/23

Calculated By: MM
Date: 08/15/23

Checked By: JDM
Date: 09/19/23



APPENDIX F – SITE-SPECIFIC SEISMIC STUDY

**Site-Specific Seismic Study
ARDOT G003 101017 Tyronza River &
Ditch No. 6 Strs. & Apprs
Bridge 1
Pointsett County, Arkansas**

By

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

October 7, 2023

TABLE OF CONTENTS

| | <u>Page</u> |
|---|-------------|
| 1.0. EXECUTIVE SUMMARY | 1 |
| 2.0. SCOPE OF WORK..... | 2 |
| 3.0. SUBSURFACE CONDITIONS | 2 |
| 4.0. SHEAR-WAVE VELOCITY PROFILE..... | 3 |
| 5.0. GENERAL INFORMATION..... | 5 |
| 6.0. REGIONAL SEISMICITY..... | 5 |
| 7.0. SEISMIC HAZARD ANALYSIS | 6 |
| 7.1. SEISMIC SOURCE MODELS | 6 |
| 7.2. GROUND MOTION MODELS..... | 6 |
| 7.3. TREATMENT OF UNCERTAINTIES..... | 8 |
| 8.0. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2 nd Edition, 2022 Interim Revisions | 8 |
| 8.1. Dynamic Soil Properties..... | 8 |
| 9.0. CODE-BASED DESIGN APPROACH | 9 |
| 9.1. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2 nd Edition, 2022 Interim Revisions | 9 |
| 10.0. SITE-SPECIFIC PROCEDURE..... | 10 |
| 10.1. Seismic Hazard Analysis | 12 |
| 10.2. Variability in Soil's Shear-Wave and Thickness Profile..... | 12 |
| 10.3. Site-Specific Results..... | 13 |
| 11.0. DESIGN RESPONSE SPECTRAL PARAMETERS | 13 |
| 12.0. LIMITATIONS OF THE REPORT..... | 17 |
| 13.0. REFERENCES | 17 |
| APPENDIX A. Site Location | 20 |

Site-Specific Seismic Study ARDOT G003 101017 Tyronza River & Ditch No. 6 Strs. & Apprs Bridge 1 Pointsett County, Arkansas

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 $35.5098215^{\circ}\text{N}$ and $90.3581467^{\circ}\text{W}$ (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_{DI} (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). The shear-wave velocity profiles obtained from SCPT are provide Table 1 and are shown in Figure 1.

Table 1. Shear-Wave Velocities Measured.

| CPT1 | | |
|------------|-------------|----------------|
| Depth1(ft) | Depth2(ft.) | velocity(ft/s) |
| 1.71 | 5.02 | 703.59 |
| 5.02 | 8.30 | 703.59 |
| 8.30 | 11.61 | 703.59 |
| 11.61 | 14.89 | 622.02 |
| 14.89 | 18.17 | 609.39 |
| 18.17 | 21.45 | 670.86 |
| 21.45 | 24.73 | 730.23 |
| 24.73 | 28.04 | 704.97 |
| 28.04 | 31.32 | 727.83 |
| 31.32 | 34.57 | 812.39 |
| 34.57 | 37.82 | 815.05 |
| 37.82 | 41.07 | 645.80 |
| 41.07 | 44.31 | 853.59 |
| 44.31 | 47.56 | 751.05 |
| 47.56 | 50.87 | 804.85 |
| 50.87 | 54.12 | 739.02 |
| 54.12 | 57.37 | 814.39 |
| 57.37 | 60.65 | 646.82 |
| 60.65 | 63.89 | 667.22 |
| 63.89 | 67.17 | 748.56 |
| 67.17 | 70.42 | 878.42 |
| 70.42 | 73.67 | 769.95 |
| 73.67 | 77.05 | 707.07 |
| 77.05 | 80.29 | 919.32 |
| 80.29 | 83.67 | 833.58 |
| 83.67 | 86.89 | 1082.73 |
| 86.89 | 90.10 | 880.65 |
| 90.10 | 93.45 | 916.79 |
| 93.45 | 96.76 | 1354.48 |
| 96.76 | 100.01 | 861.69 |

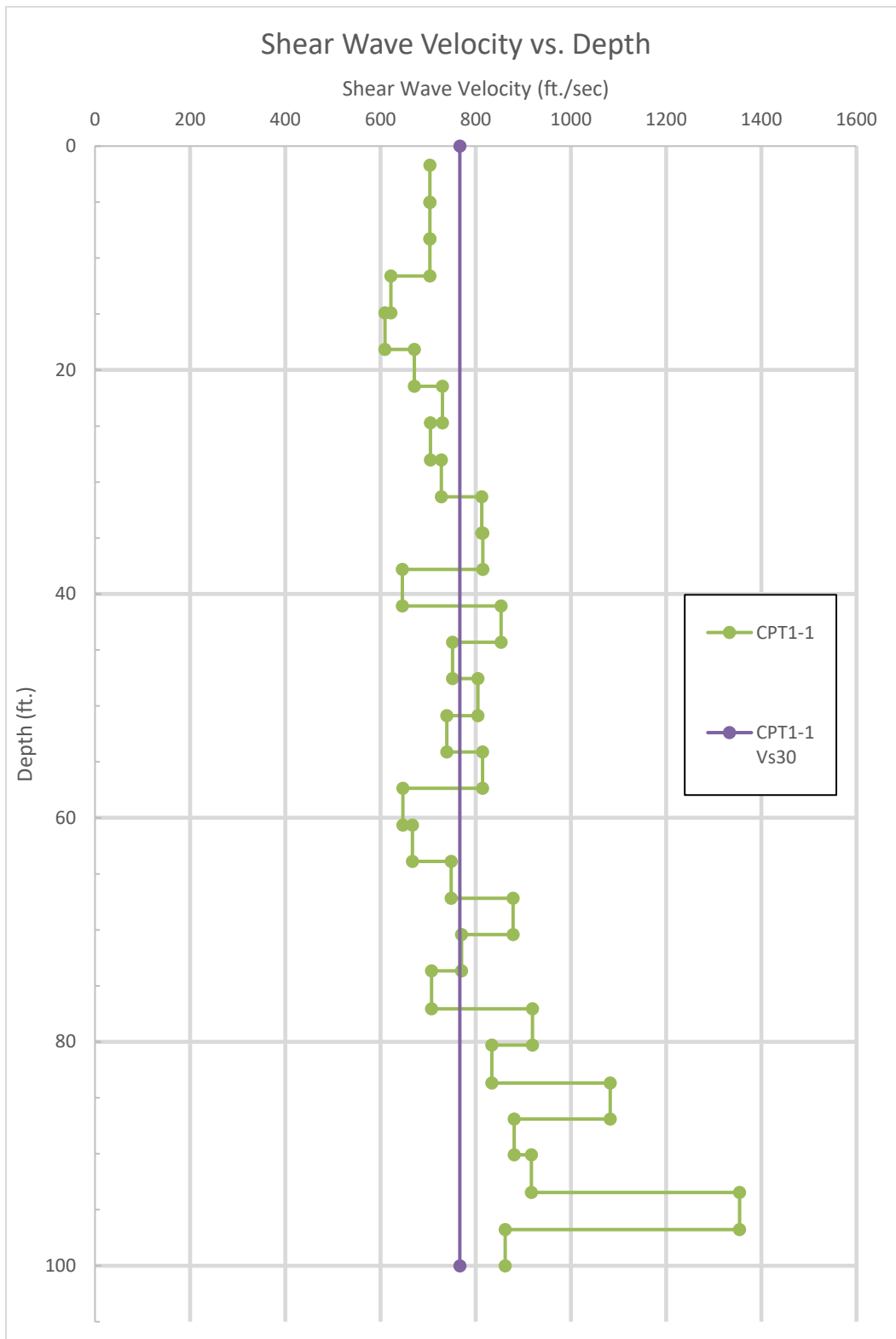


Figure 1. Shear-Wave Velocities Measured by Geotechnology.

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 2 shows the USGS 2018 declustered catalog for CEUS.

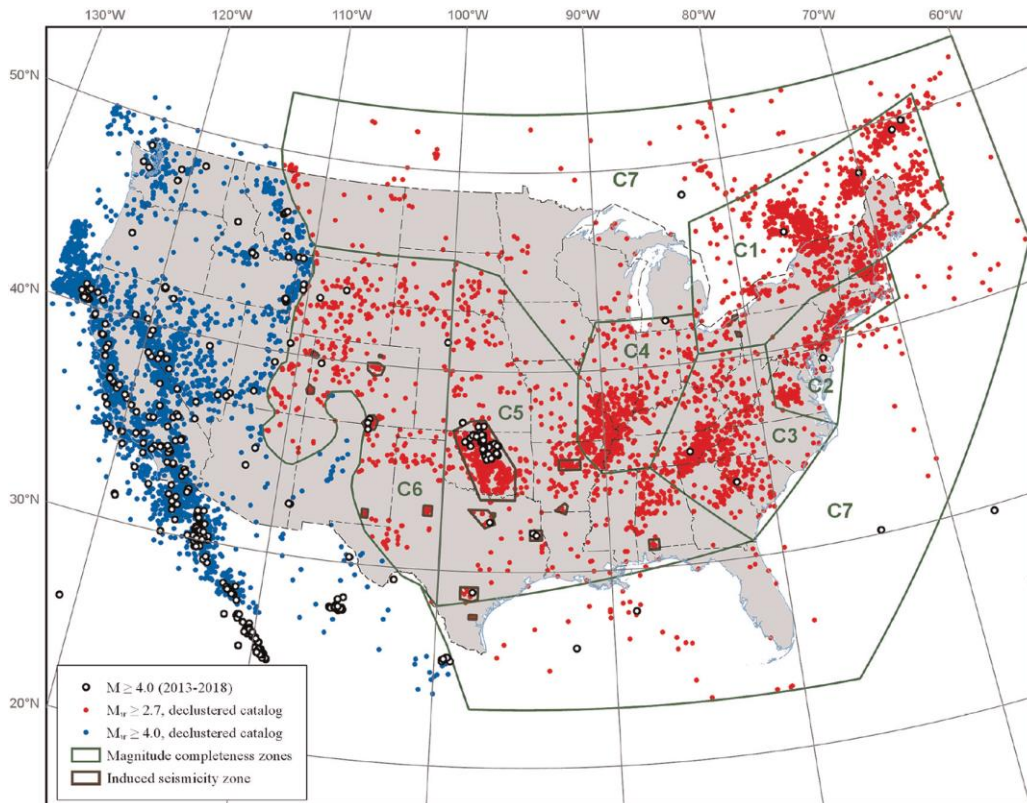


Figure 2. The 2018 NSHM Declustered Catalog for Central and Eastern United States (red) and Western United States (blue).

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).[Source Rezaeian et al. (2021)].

| 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 | Shajouei-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) | Shajouei-Pezeshk | 0 |
| Graizer (replaced with Graizer16 & Graizer17 by USGS) | Graizer | 0 |
| PEER-EX | Hollenback-Kuehn-Goulet-Abrahamson | 0 |
| 17 NGA-East GMMs (used by USGS in 2018 NSHM) | | 0.667 |
| Models 1 to 17 | NGA-East Project | Period-dependen ^a |

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 766 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 | 1.000 |
| F_v | 1.523 |
| F_{PGA} | 1.000 |
| S_s | 1.775 |
| S_1 | 0.477 |
| S_{DS} | 1.775 |
| S_{D1} | 0.726 |
| PGA | 1.003 |
| A_s | 1.003 |

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 3. The same process was repeated for all eleven earthquakes for both components.

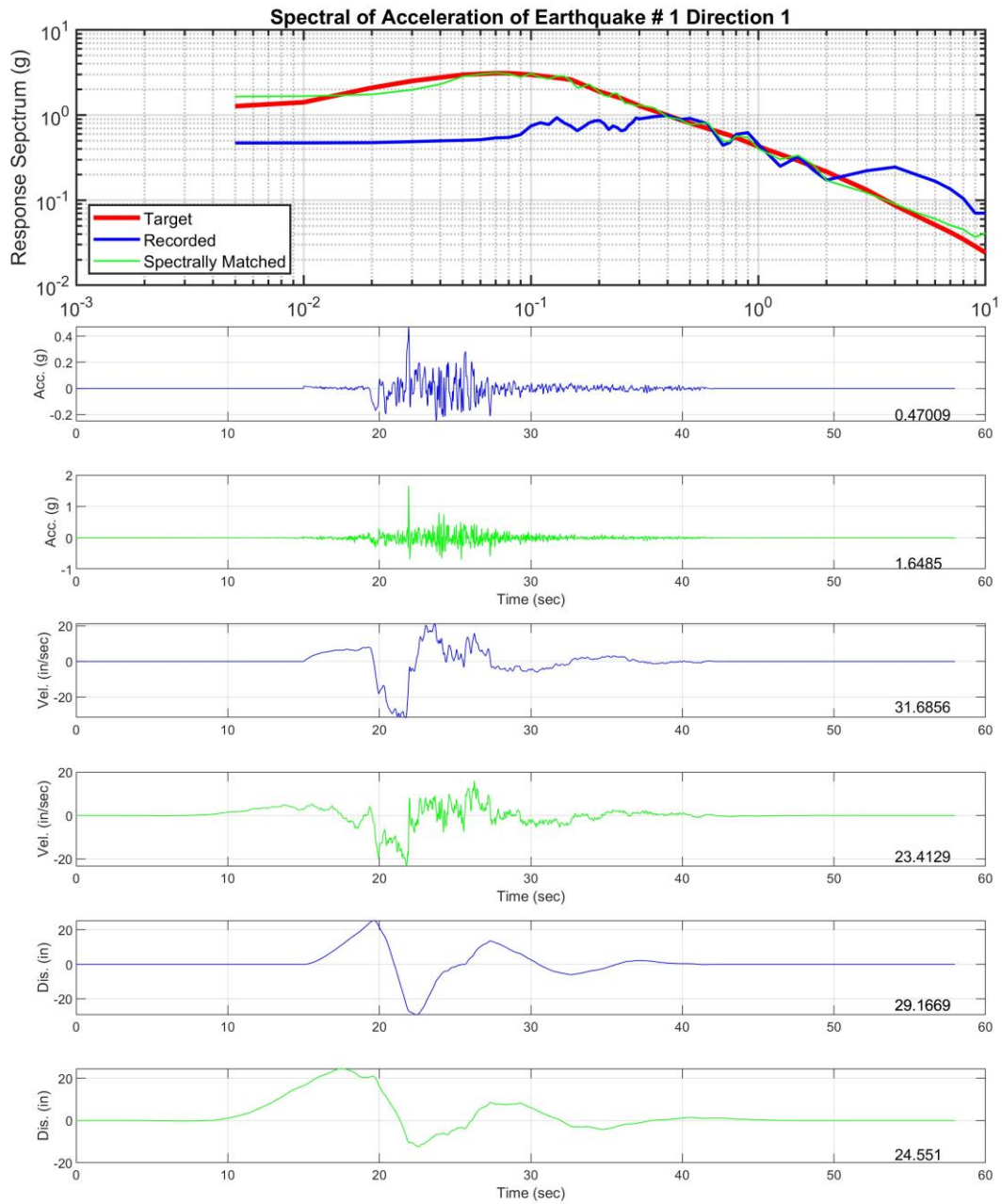


Figure 3. Time Histories Before and After the Spectral Matching Process for Earthquake #1. The numbers Shown in the Bottom right of Each Figure Represent the Absolute Maximum Value of the Graph.

Table 4. Deaggregation.

| Mean and Mode Deaggregation Parameter at 1,033 Years | | | | | |
|--|------|--------|--------|------|--------|
| Mean | | | Mode | | |
| Period | M | R (km) | Period | M | R (km) |
| PGA | 7.36 | 12.49 | PGA | 7.52 | 12.59 |
| 0.01 | 7.36 | 12.55 | 0.01 | 7.55 | 12.04 |
| 0.02 | 7.34 | 12.78 | 0.02 | 7.52 | 12.55 |
| 0.03 | 7.35 | 12.88 | 0.03 | 7.52 | 12.56 |
| 0.05 | 7.36 | 13.24 | 0.05 | 7.52 | 12.55 |
| 0.08 | 7.37 | 13.56 | 0.08 | 7.52 | 12.54 |
| 0.10 | 7.39 | 14.00 | 0.10 | 7.52 | 12.55 |
| 0.15 | 7.42 | 14.40 | 0.15 | 7.55 | 12.04 |
| 0.20 | 7.43 | 14.72 | 0.20 | 7.55 | 12.04 |
| 0.25 | 7.46 | 14.97 | 0.25 | 7.55 | 12.04 |
| 0.30 | 7.46 | 15.42 | 0.30 | 7.55 | 12.04 |
| 0.40 | 7.48 | 15.78 | 0.40 | 7.55 | 12.04 |
| 0.50 | 7.48 | 16.31 | 0.50 | 7.55 | 12.04 |
| 0.75 | 7.50 | 17.53 | 0.75 | 7.55 | 12.05 |
| 1.00 | 7.52 | 18.58 | 1.00 | 7.55 | 12.05 |
| 1.50 | 7.54 | 19.83 | 1.50 | 7.55 | 12.06 |
| 2.00 | 7.56 | 21.12 | 2.00 | 7.55 | 12.06 |
| 3.00 | 7.59 | 22.02 | 3.00 | 7.76 | 11.88 |
| 4.00 | 7.61 | 22.73 | 4.00 | 7.76 | 11.88 |
| 5.00 | 7.62 | 23.40 | 5.00 | 7.76 | 11.88 |
| 7.50 | 7.64 | 24.61 | 7.50 | 7.76 | 11.88 |
| 10.00 | 7.66 | 25.11 | 10.00 | 7.93 | 11.69 |

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 4 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 S_a values provided in Table 5. Figure 5 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 5, 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 | 0.669 |
| 0.030 | 0.857 |
| 0.040 | 0.920 |
| 0.050 | 0.983 |
| 0.070 | 1.109 |
| 0.100 | 1.183 |
| 0.150 | 1.183 |
| 0.200 | 1.183 |
| 0.250 | 1.183 |
| 0.300 | 1.183 |
| 0.400 | 1.183 |
| 0.500 | 1.080 |
| 0.750 | 0.849 |
| 1.000 | 0.972 |
| 1.500 | 0.621 |
| 2.000 | 0.486 |
| 3.000 | 0.256 |
| 4.000 | 0.143 |
| 5.000 | 0.097 |
| 7.500 | 0.065 |
| 10.000 | 0.073 |

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

| PARAMETER | DESIGN ACCELERATION PARAMETERS (g) |
|-----------|------------------------------------|
| S_{DS} | 1.183 |
| S_{DI} | 0.972 |
| S_{MS} | 1.183 |
| S_{MI} | 0.972 |
| MCE_G | 0.669 |

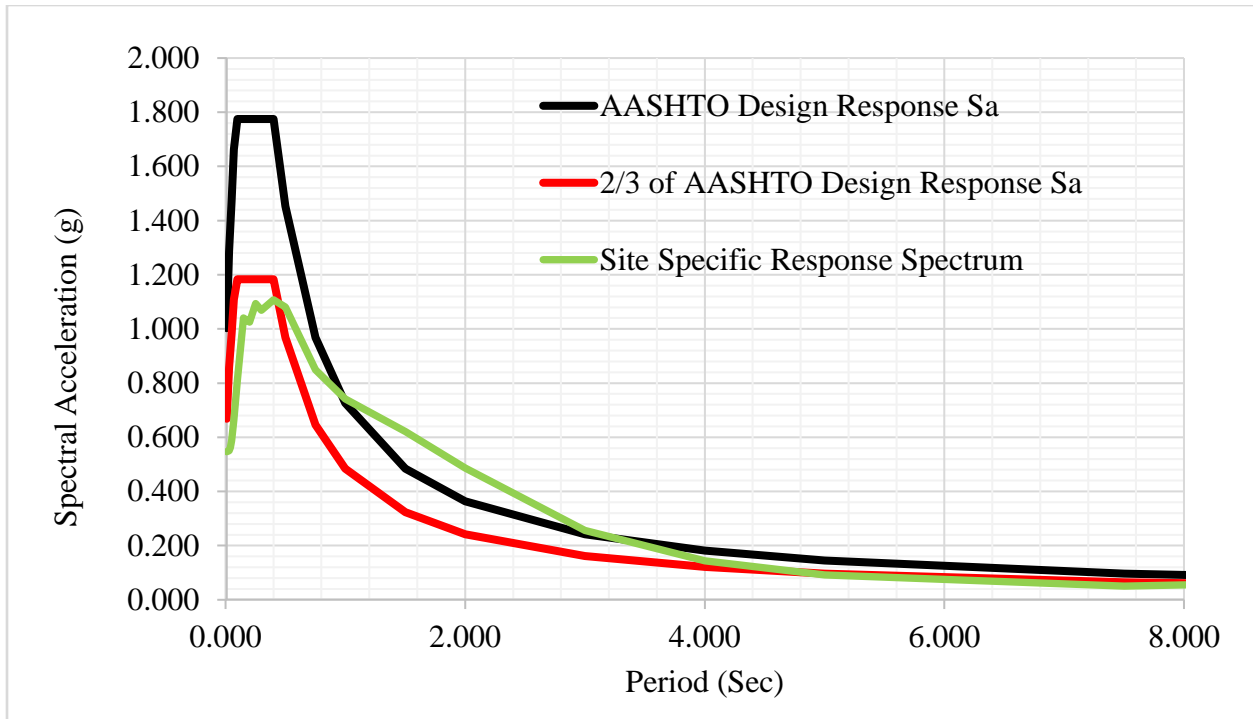


Figure 4. Site-Specific Design Response Spectrum, AASHTO Guide Specifications Design Response Spectrum, and 2/3 of the AASHTO Guide Specifications Design Response Spectrum.

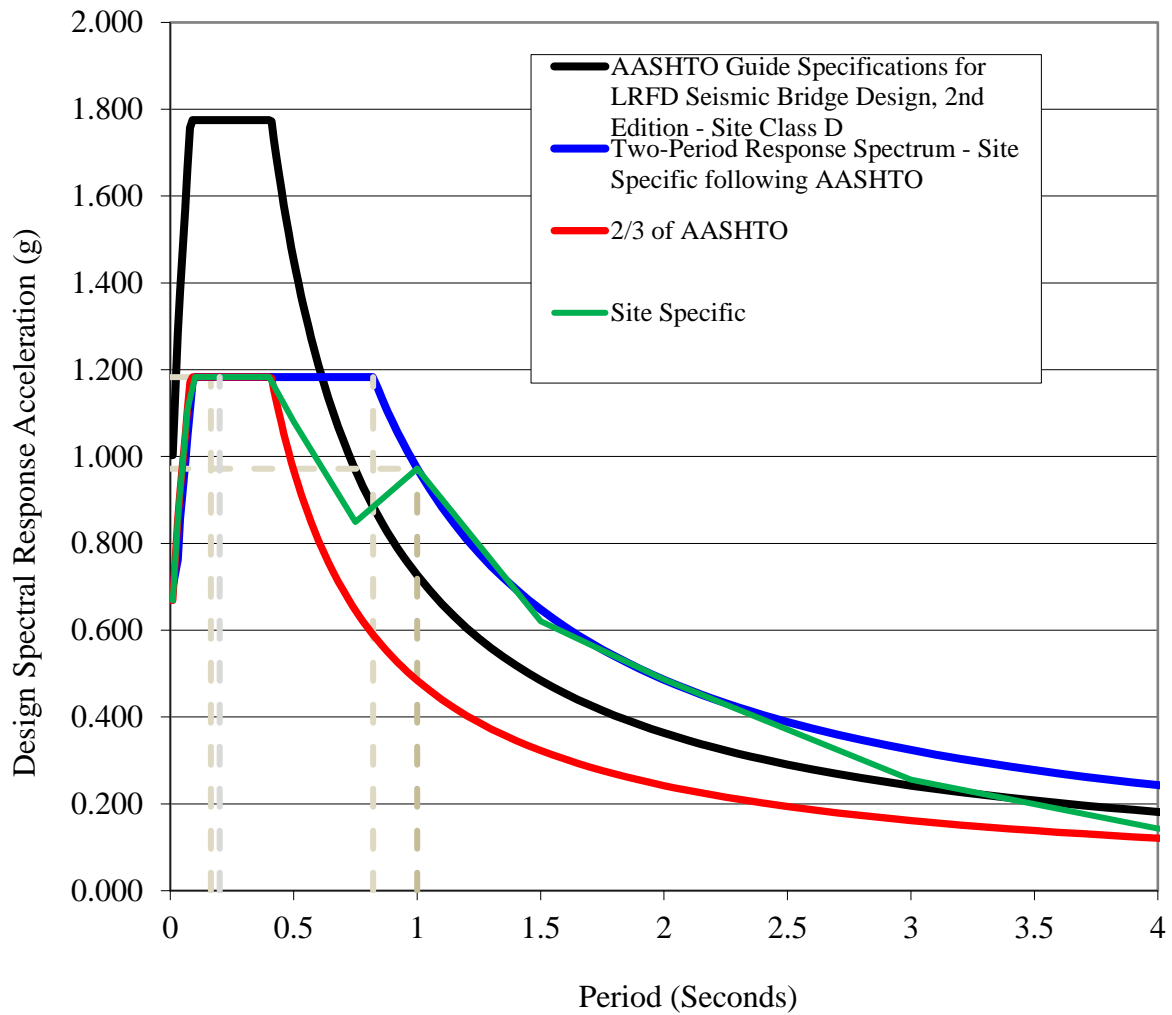


Figure 5. Design Response Spectrum based on AASHTO Guide Specifications, 2/3 of the AASHTO Guide Specifications Site-Specific, and Design Response Spectrum Based on PGA, 0.2, and 1 Second.

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

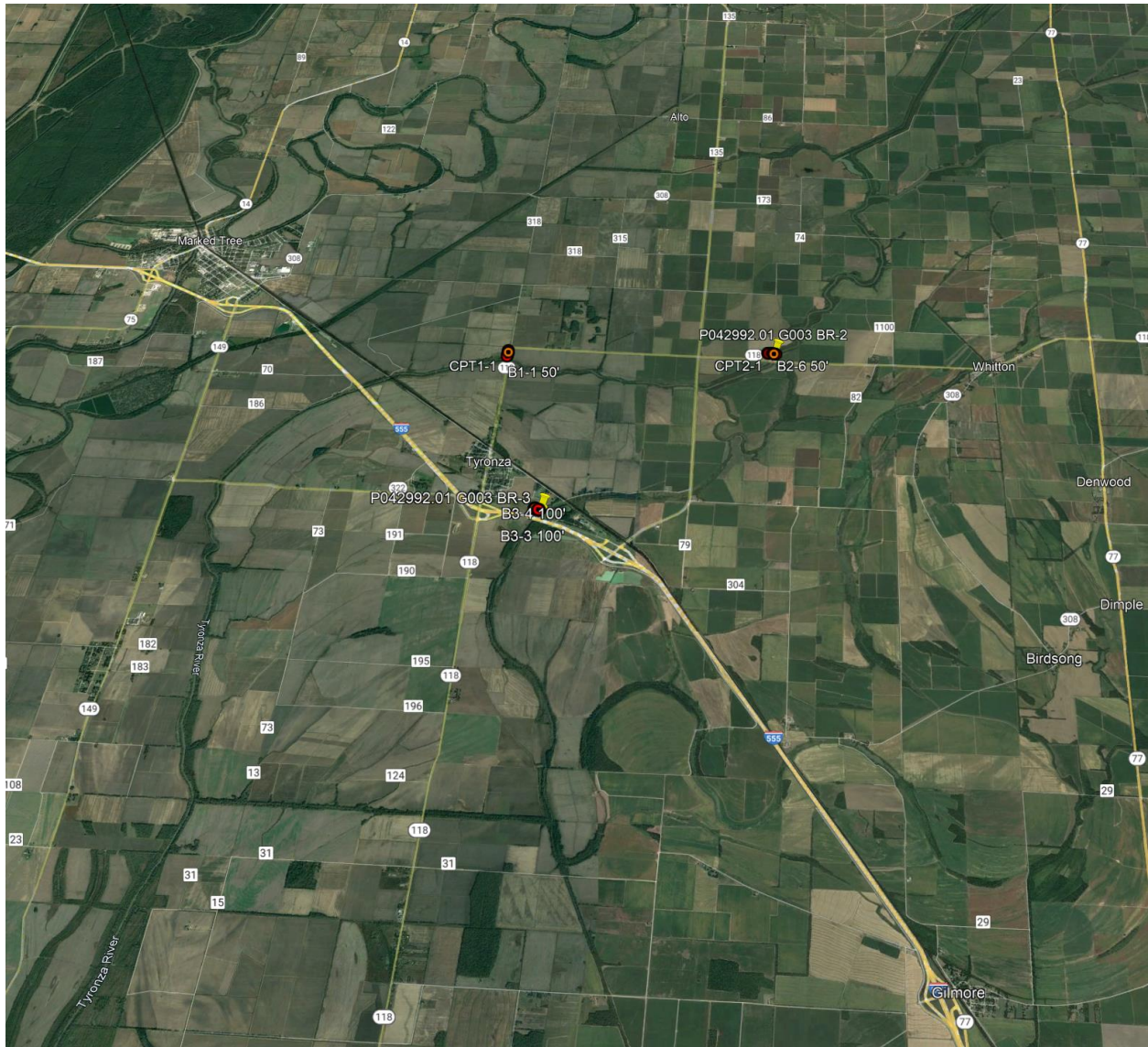


Figure A.1. The Location of the Study Site.



APPENDIX G – AASHTO AND USCS CLASSIFICATIONS



Project: ARDOT 101017
 Tyronza River & Ditch No. 6 Strs. & Apprs. (S)
 Bridge 1 Over Tyronza River
 Number: J042992.01

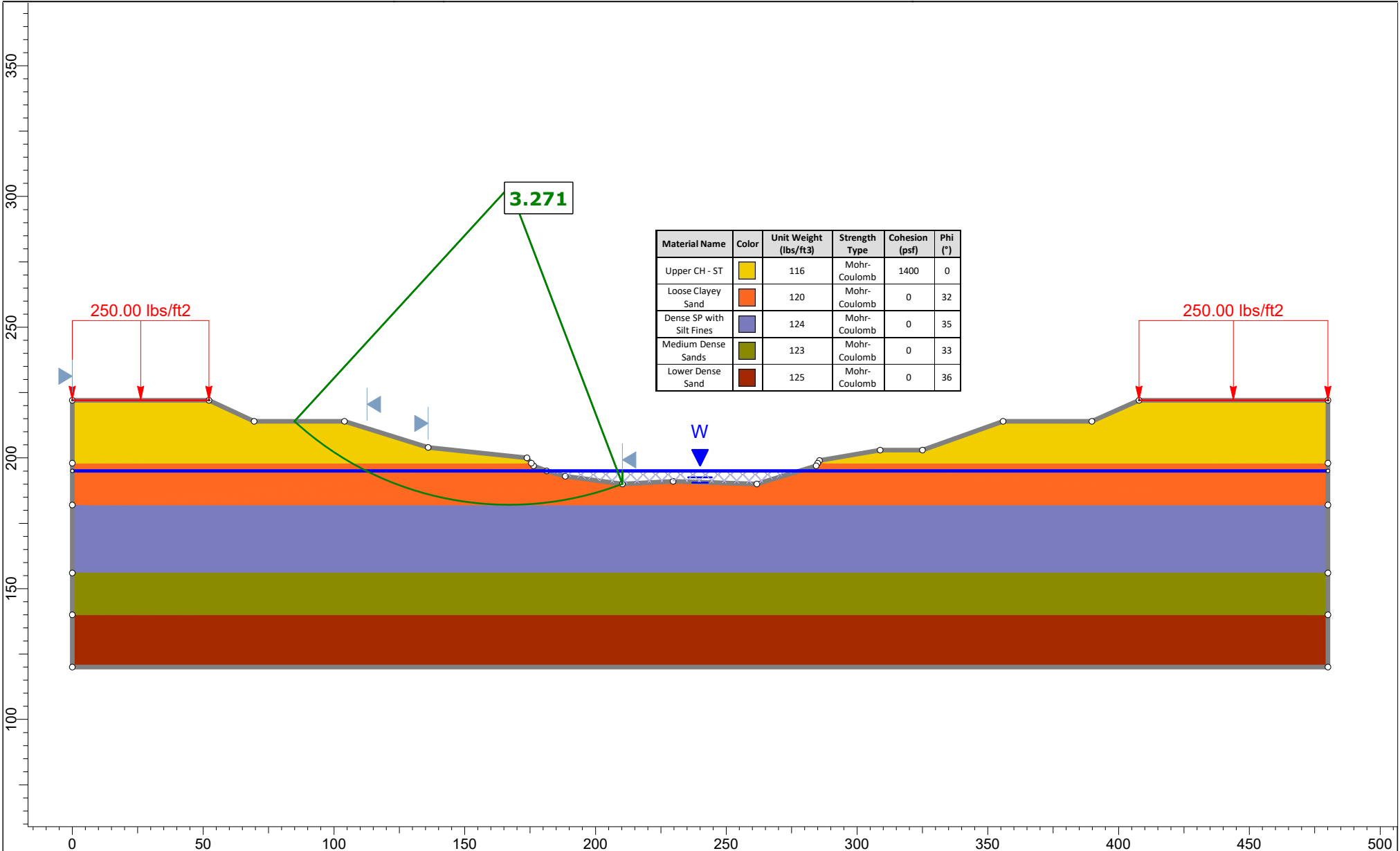
| Borehole | Depth (feet) | Liquid Limit (LL) (percent) | Plastic Limit (PL) (percent) | Plasticity Index (PI) (percent) | %<#10 Sieve | %<#40 Sieve | %<#200 Sieve | GI | AASHTO CLASS. | USCS CLASS. |
|----------|--------------|-----------------------------|------------------------------|---------------------------------|-------------|-------------|--------------|----|---------------|-------------|
| B1-1 | 3 | 89 | 23 | 66 | -- | -- | -- | -- | A-7-6 | CH |
| | 6 | -- | -- | -- | 100 | 100 | 85.1 | -- | A-7-6 | CH |
| | 10 | 90 | 32 | 58 | -- | -- | -- | -- | A-7-6 | CH |
| | 15 | 87 | 35 | 52 | -- | -- | -- | -- | A-7-6 | CH |
| B1-2 | 10 | 92 | 30 | 62 | -- | -- | -- | -- | A-7-6 | CH |
| | 15 | 102 | 46 | 56 | -- | -- | -- | -- | A-7-5 | MH |
| | 20 | 70 | 28 | 42 | -- | -- | -- | -- | A-7-6 | CH |
| | 38.5 | -- | -- | -- | 100.0 | 75.7 | 10.3 | -- | A-2-6 | SP-SC |
| B1-3 | 3.5 | -- | -- | -- | 99.3 | 95.4 | 67.4 | -- | A-6 | CL |
| | 6 | -- | -- | -- | 99.2 | 90.3 | 72.6 | -- | A-6 | CL |
| | 13.5 | -- | -- | -- | 100 | 67.9 | 6.0 | -- | A-2-6 | SP-SC |
| | 88.5 | -- | -- | -- | 64.6 | 19.7 | 3.9 | -- | A-1-b | SP |
| B1-4 | 10 | -- | -- | -- | 99.9 | 78.7 | 1.9 | -- | A-3 | SP |
| | 13.5 | -- | -- | -- | 99.8 | 79.5 | 5.1 | -- | A-3 | SP-SM |
| | 18.5 | -- | -- | -- | 100.0 | 77.3 | 2.7 | -- | A-3 | SP |
| | 28.5 | -- | -- | -- | 96.6 | 53.9 | 1.7 | -- | A-3 | SP |
| | 38.5 | -- | -- | -- | 100 | 64.8 | 6.6 | -- | A-3 | SP-SM |
| B1-5 | 6 | -- | -- | -- | 100 | 97.0 | 66.6 | -- | A-7-6 | CH |
| | 8 | 66 | 19 | 47 | -- | -- | -- | -- | A-7-6 | CH |
| | 20 | 50 | 20 | 30 | -- | -- | -- | -- | A-7-6 | CH |
| | 28.5 | -- | -- | -- | 100 | 92.6 | 45.6 | -- | A-6 | SC |
| B1-6 | 3.5 | 45 | 19 | 26 | 99.8 | 93.6 | 60.4 | 13 | A-7-6 (13) | CL |
| | 10 | 72 | 22 | 50 | -- | -- | -- | -- | A-7-6 | CH |
| | 15 | 80 | 28 | 52 | -- | -- | -- | -- | A-7-6 | CH |
| | 23.5 | -- | -- | -- | 100 | 100 | 27.2 | -- | A-2-6 | SC |
| | 28.5 | -- | -- | -- | 100 | 99.2 | 20.7 | -- | A-2-4 | SM |
| | 33.5 | -- | -- | -- | 100 | 100 | 17.7 | -- | A-2-4 | SM |
| | 38.5 | -- | -- | -- | 100 | 98.5 | 13.8 | -- | A-2-4 | SM |



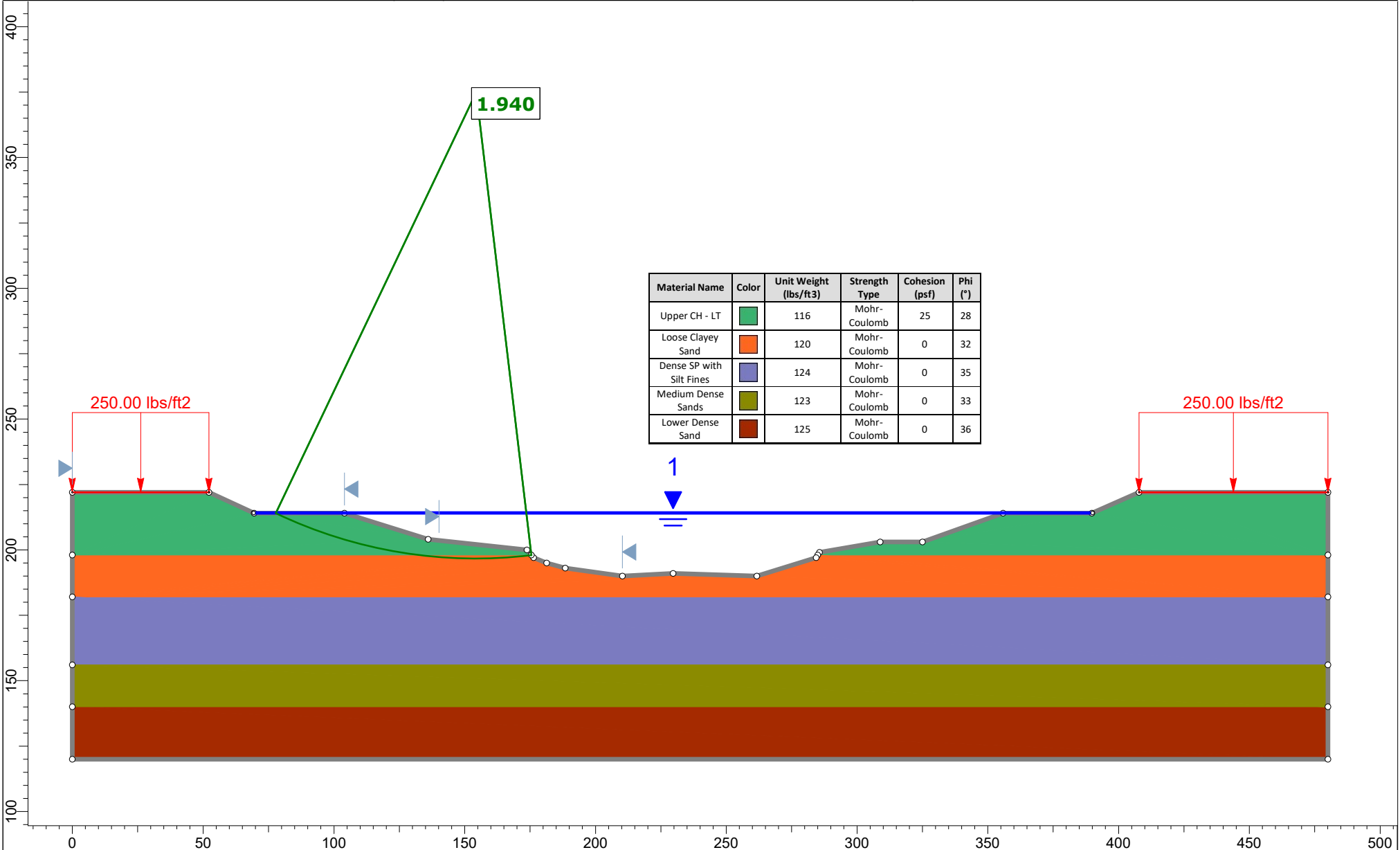
APPENDIX H – GLOBAL STABILITY ANALYSES



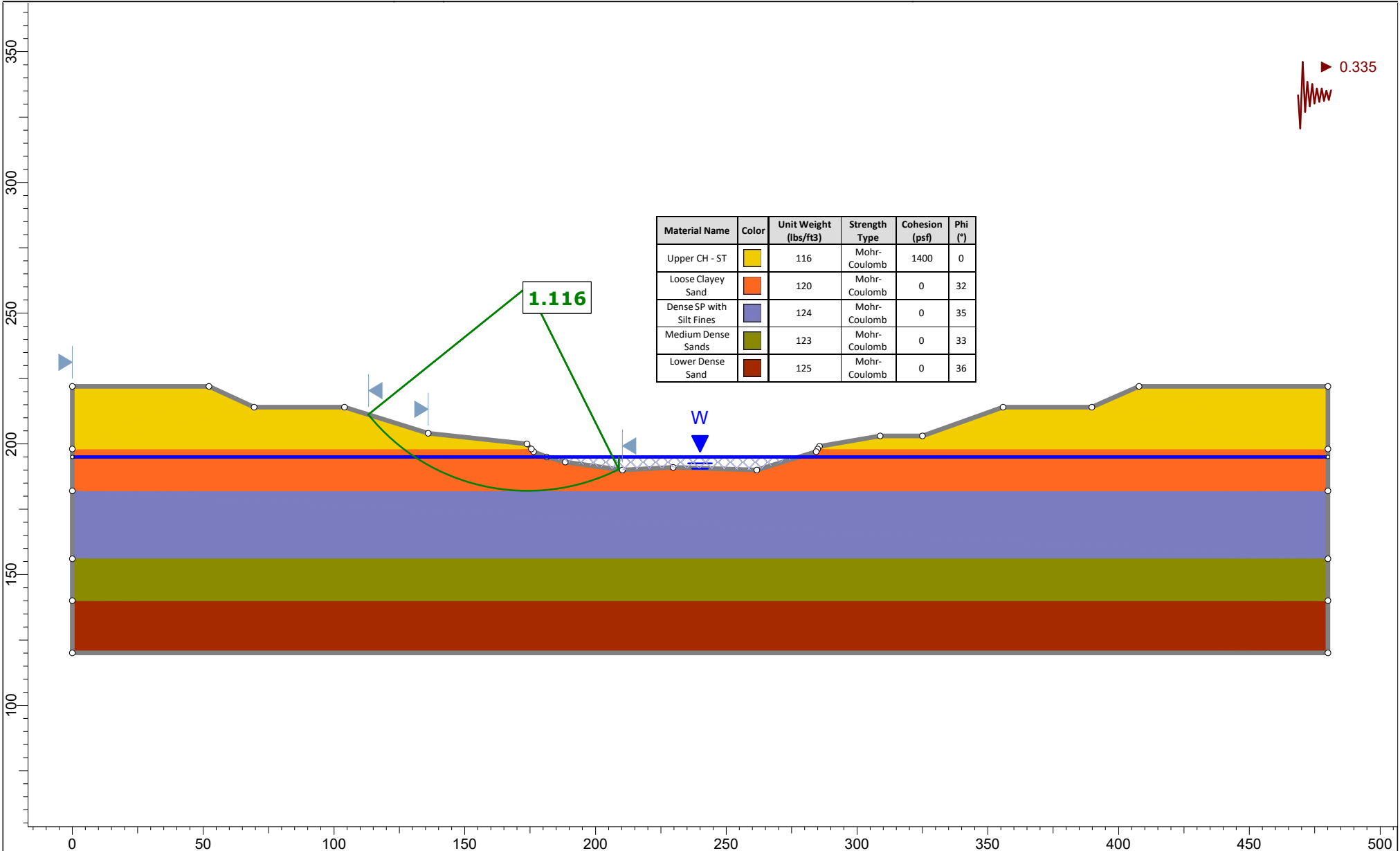
SLIDEINTERPRET 9.029



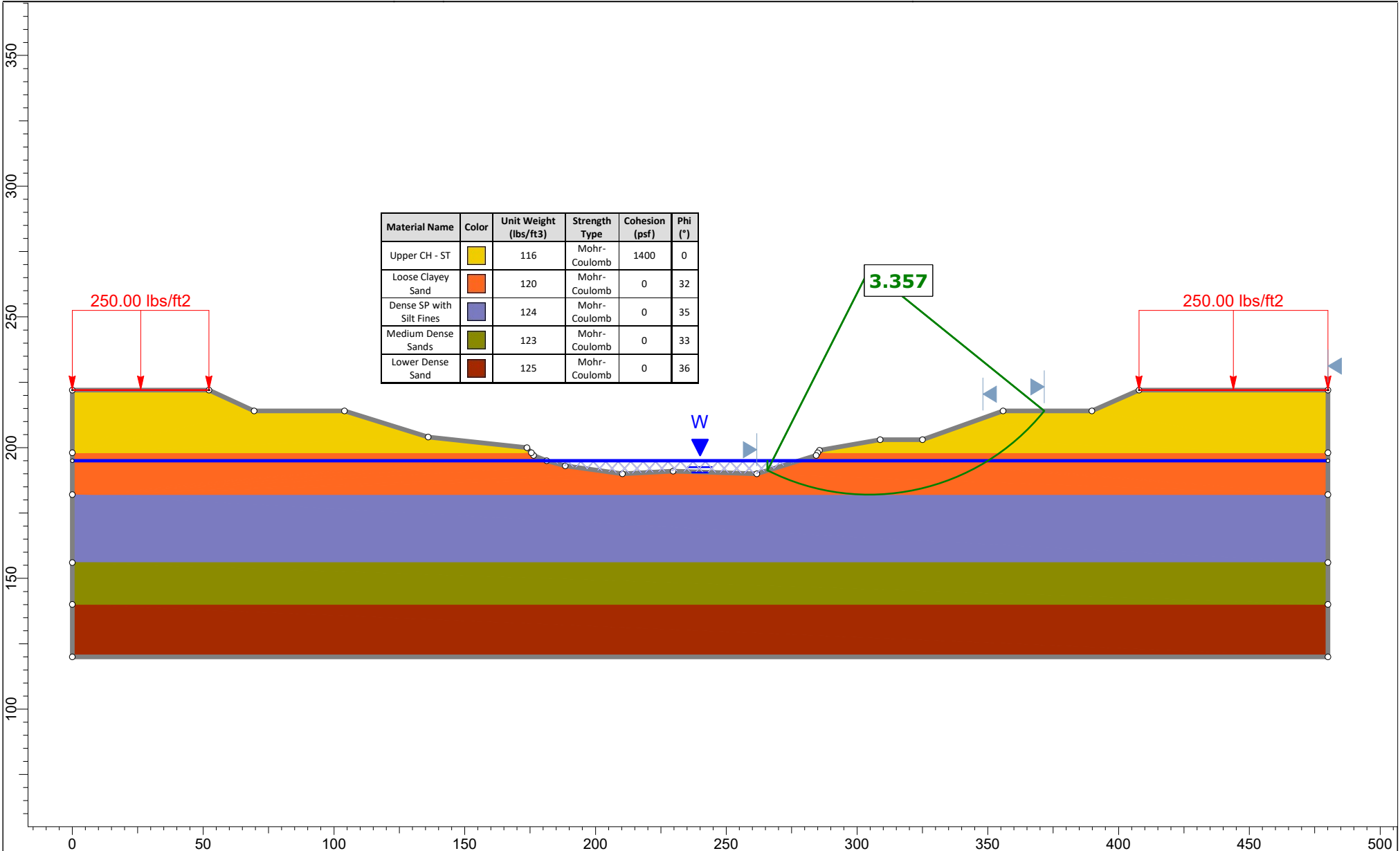
SLIDEINTERPRET 9.029



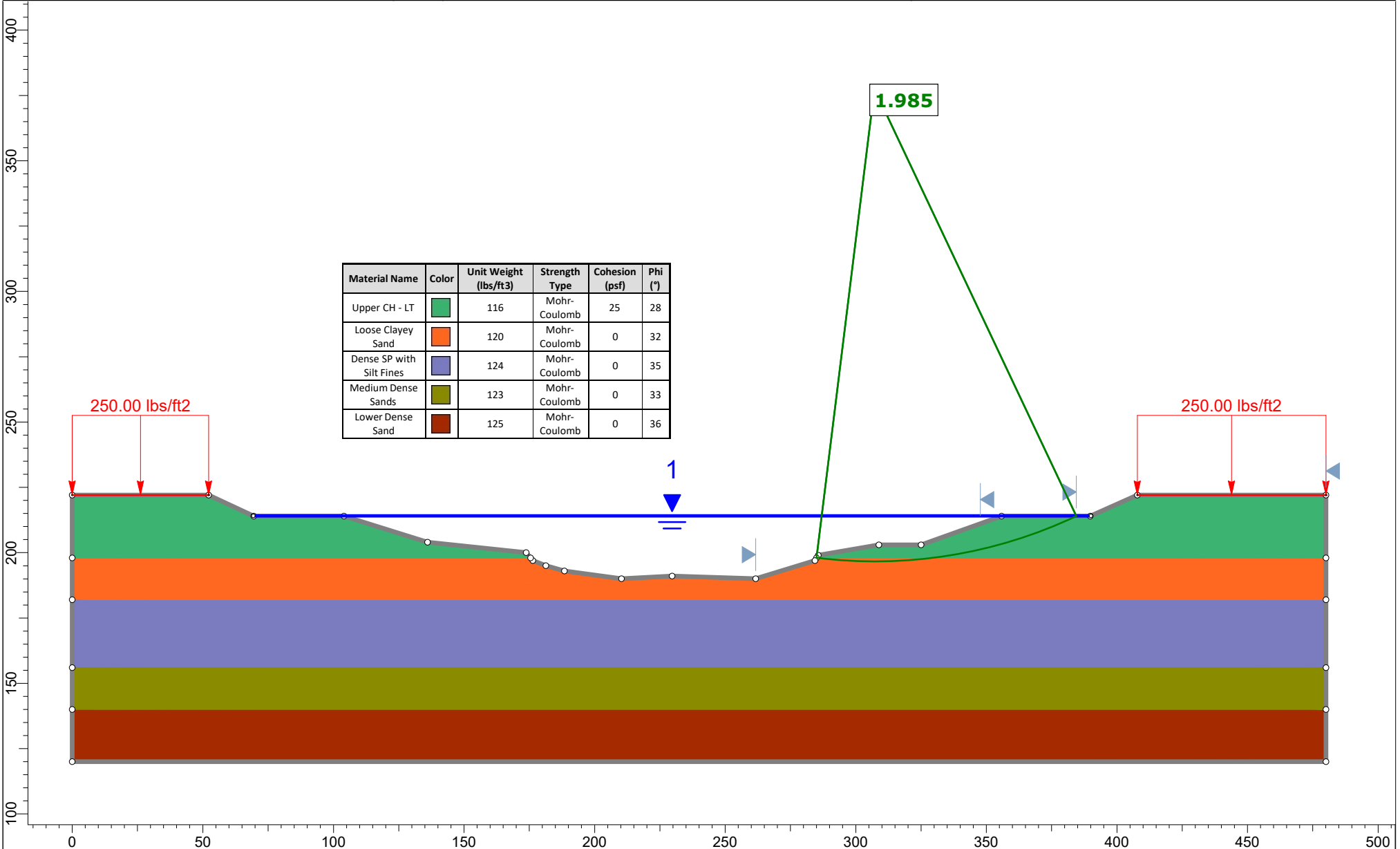
SLIDEINTERPRET 9.029



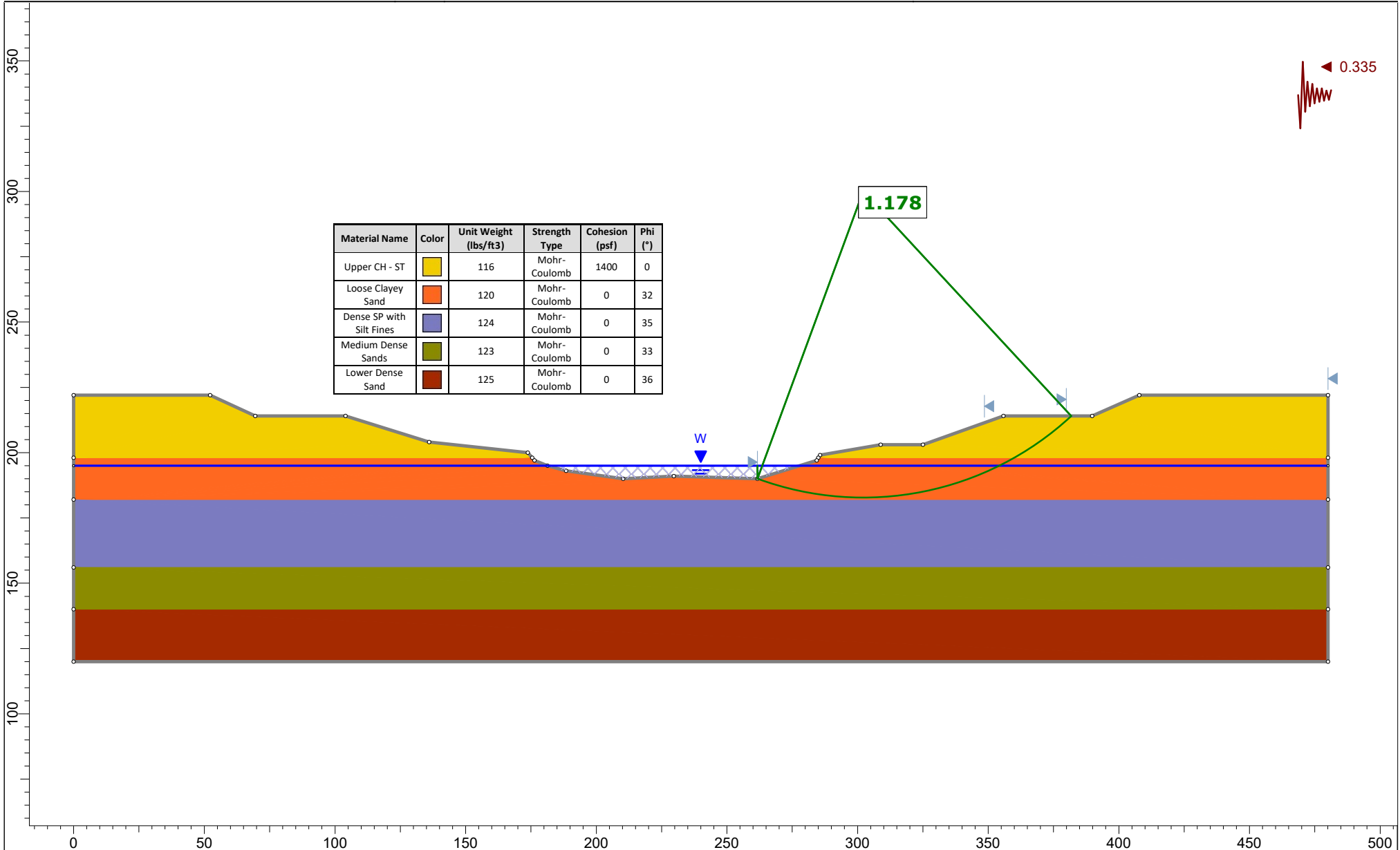
SLIDEINTERPRET 9.029



SLIDEINTERPRET 9.029



SLIDEINTERPRET 9.029





APPENDIX I – SOIL PARAMETERS FOR SYNTHETIC PROFILES



| BRIDGE 1 OVER TYRONZA RIVER – BENT 1 (SOUTH ABUTMENT; BORING B1-2) | | | | | | | | | | | | | |
|--|-------------------------------|--|------|-------------------------|---------------------------|-----------------|--------------------------|------------------|--------------------------------------|--|-------------------------------|--|--------------------------|
| APPROXIMATE GROUND SURFACE ELEVATION = EL 220 | | | | | | | | | | | | | |
| ZONE | SOIL TYPES | DEPTH ^a (feet from ground surface) | | TOTAL UNIT WEIGHT (PCF) | SHEAR STRENGTH PARAMETERS | | | | LATERAL LOAD PARAMETERS ^d | | | LIQUEFACTION SHEAR STRENGTH PARAMETERS | |
| | | FROM | TO | | UNDRAINED (SHORT TERM) | | DRAINED (LONG TERM) | | SOIL STRAIN, E ₅₀ | STATIC SOIL MODULUS (PCI) ^c | LPILE SOIL MODEL | RESIDUAL COHESION (PSF) | RESIDUAL ϕ (DEGREE) |
| | | | | | COHESION (PSF) | ϕ (DEGREE) | EFFECTIVE COHESION (PSF) | ϕ' (DEGREE) | | | | | |
| 1 | Stiff Fat Clay | 0 ^b | 23.5 | 116 | 1,400 | -- | -- | 26 | 0.007 | 500 | Soft Clay | N/A ^e | N/A ^e |
| 2 | Medium Stiff Lean Clay / Silt | 23.5 | 38.5 | 120 | 800 | -- | -- | 28 | 0.01 | 100 | Stiff Clay without Free Water | N/A ^e | N/A ^e |
| 3 | Dense Sands | 38.5 | 100 | 124 | -- | 35 | -- | 35 | -- | 125 | Sand | N/A ^e | N/A ^e |

Note: Groundwater elevation assumed at El. 195 based on the water levels encountered in the borings. The effective unit weight should be used below the groundwater level.

Subtract the density of water (62.4 pounds per cubic foot) from the total unit weight to calculate the effective unit weight.

^a Depth in reference to ground surface at boring locations.

^b Zero depth as measured at top of boring.

^c Pounds per cubic inch.

^d For lateral load analysis only.

^e Liquefaction not anticipated in boring B1-2.

| BRIDGE 1 OVER TYRONZA RIVER – BENT 2 (BORING B1-3) | | | | | | | | | | | | | |
|---|------------------------|--|------|-------------------------|---------------------------|-----------------|--------------------------|------------------|--------------------------------------|--|------------------|--|--------------------------|
| APPROXIMATE AVERAGE GROUND SURFACE ELEVATION = EL 198 | | | | | | | | | | | | | |
| ZONE | SOIL TYPES | DEPTH ^a (feet from ground surface) | | TOTAL UNIT WEIGHT (PCF) | SHEAR STRENGTH PARAMETERS | | | | LATERAL LOAD PARAMETERS ^d | | | LIQUEFACTION SHEAR STRENGTH PARAMETERS | |
| | | FROM | TO | | UNDRAINED (SHORT TERM) | | DRAINED (LONG TERM) | | SOIL STRAIN, E ₅₀ | STATIC SOIL MODULUS (PCI) ^c | LPILE SOIL MODEL | RESIDUAL COHESION (PSF) | RESIDUAL ϕ (DEGREE) |
| | | | | | COHESION (PSF) | ϕ (DEGREE) | EFFECTIVE COHESION (PSF) | ϕ' (DEGREE) | | | | | |
| 1 | Medium Stiff Lean Clay | 0 ^b | 13.5 | 118 | 800 | -- | -- | 26 | 0.01 | 100 | Soft Clay | 700 | -- |
| 2 | Dense Sands | 13.5 | 18.5 | 123 | -- | 34 | -- | 34 | -- | 90 | Sand | -- | 9 |
| 3 | Dense Sands | 18.5 | 48.5 | 123 | -- | 34 | -- | 34 | -- | 90 | | -- | N/A ^e |
| 4 | Medium Dense Sands | 48.5 | 53.5 | 122 | -- | 32 | -- | 32 | -- | 25 | | -- | N/A ^e |
| 5 | Dense/Very Dense Sands | 53.5 | 100 | 124 | -- | 35 | -- | 35 | -- | 125 | | -- | N/A ^e |

Note: Groundwater elevation assumed at El. 195 based on the water levels encountered in the borings. The effective unit weight should be used below the groundwater level. Subtract the density of water (62.4 pounds per cubic foot) from the total unit weight to calculate the effective unit weight.

^a Depth in reference to ground surface at boring locations.

^b Zero depth as measured at top of boring.

^c Pounds per cubic inch.

^d For lateral load analysis only.

^e Liquefaction not anticipated in these layers.

| BRIDGE 1 OVER TYRONZA RIVER – BENT 3 (BORING B1-4) | | | | | | | | | | | | | |
|--|--------------------------------|--|------|-------------------------|---------------------------|-----------------|--------------------------|------------------|--------------------------------------|--|------------------|--|--------------------------|
| APPROXIMATE GROUND SURFACE ELEVATION = EL 204 | | | | | | | | | | | | | |
| ZONE | SOIL TYPES | DEPTH ^a (feet from ground surface) | | TOTAL UNIT WEIGHT (PCF) | SHEAR STRENGTH PARAMETERS | | | | LATERAL LOAD PARAMETERS ^d | | | LIQUEFACTION SHEAR STRENGTH PARAMETERS | |
| | | FROM | TO | | UNDRAINED (SHORT TERM) | | DRAINED (LONG TERM) | | SOIL STRAIN, E ₅₀ | STATIC SOIL MODULUS (PCI) ^c | LPILE SOIL MODEL | RESIDUAL COHESION (PSF) | RESIDUAL ϕ (DEGREE) |
| | | | | | COHESION (PSF) | ϕ (DEGREE) | EFFECTIVE COHESION (PSF) | ϕ' (DEGREE) | | | | | |
| 1 | Soft Lean Clay | 0 ^b | 8.5 | 118 | 800 | -- | -- | 26 | 0.01 | 100 | Soft Clay | 700 | -- |
| 2 | Loose/Medium Dense Silty Sands | 8.5 | 28.5 | 122 | -- | 33 | -- | 33 | -- | 40 | Sand | -- | N/A ^e |
| 3 | Loose Silty Sands | 28.5 | 38.5 | 122 | -- | 33 | -- | 33 | -- | 40 | | -- | 8 |
| 4 | Dense Sands | 38.5 | 68.5 | 124 | -- | 35 | -- | 35 | -- | 125 | | -- | N/A ^e |
| 5 | Medium Dense Sands | 68.5 | 100 | 123 | -- | 34 | -- | 34 | -- | 90 | | -- | N/A ^e |

Note: Groundwater elevation assumed at El. 195 based on the water levels encountered in the borings. The effective unit weight should be used below the groundwater level. Subtract the density of water (62.4 pounds per cubic foot) from the total unit weight to calculate the effective unit weight.

^a Depth in reference to ground surface at boring locations.

^b Zero depth as measured at top of boring.

^c Pounds per cubic inch.

^d For lateral load analysis only.

^e Liquefaction not anticipated in these layers.

| BRIDGE 1 OVER TYRONZA RIVER – BENT 4 (NORTH ABUTMENT; BORING B1-5) | | | | | | | | | | | | | |
|--|-----------------------------|--|------|-------------------------|---------------------------|-----------------|--------------------------|------------------|--------------------------------------|--|------------------|--|--------------------------|
| APPROXIMATE GROUND SURFACE ELEVATION = EL 220 | | | | | | | | | | | | | |
| ZONE | SOIL TYPES | DEPTH ^a (feet from ground surface) | | TOTAL UNIT WEIGHT (PCF) | SHEAR STRENGTH PARAMETERS | | | | LATERAL LOAD PARAMETERS ^d | | | LIQUEFACTION SHEAR STRENGTH PARAMETERS | |
| | | FROM | TO | | UNDRAINED (SHORT TERM) | | DRAINED (LONG TERM) | | SOIL STRAIN, E ₅₀ | STATIC SOIL MODULUS (PCI) ^c | LPILE SOIL MODEL | RESIDUAL COHESION (PSF) | RESIDUAL ϕ (DEGREE) |
| | | | | | COHESION (PSF) | ϕ (DEGREE) | EFFECTIVE COHESION (PSF) | ϕ' (DEGREE) | | | | | |
| 1 | Stiff/Medium Stiff Fat Clay | 0 ^b | 28.5 | 116 | 1,400 | -- | -- | 26 | 0.007 | 500 | Soft Clay | 700 | -- |
| 2 | Loose Clayey Sand | 28.5 | 38.5 | 120 | -- | 32 | -- | 32 | -- | 25 | Sand | -- | 8 |
| 3 | Medium Dense Sand | 38.5 | 48.5 | 123 | -- | 34 | -- | 34 | -- | 90 | | -- | N/A ^e |
| 4 | Dense/Very Dense Sand | 48.5 | 100 | 124 | -- | 35 | -- | 35 | -- | 125 | | -- | N/A ^e |

Note: Groundwater elevation assumed at El. 195 based on the water levels encountered in the borings. The effective unit weight should be used below the groundwater level.

Subtract the density of water (62.4 pounds per cubic foot) from the total unit weight to calculate the effective unit weight.

^a Depth in reference to ground surface at boring locations.

^b Zero depth as measured at top of boring.

^c Pounds per cubic inch.

^d For lateral load analysis only.

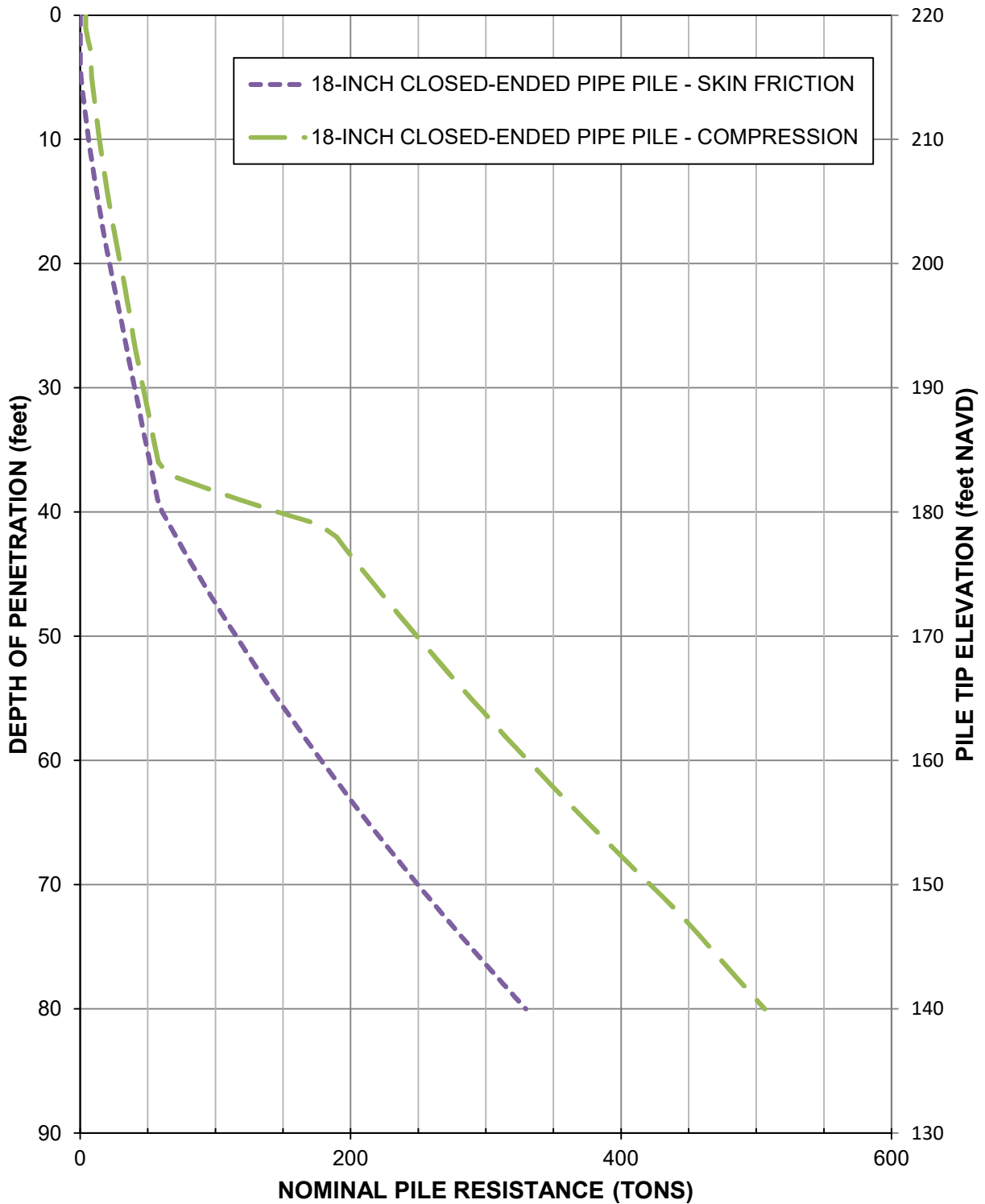
^e Liquefaction not anticipated below 50 feet.



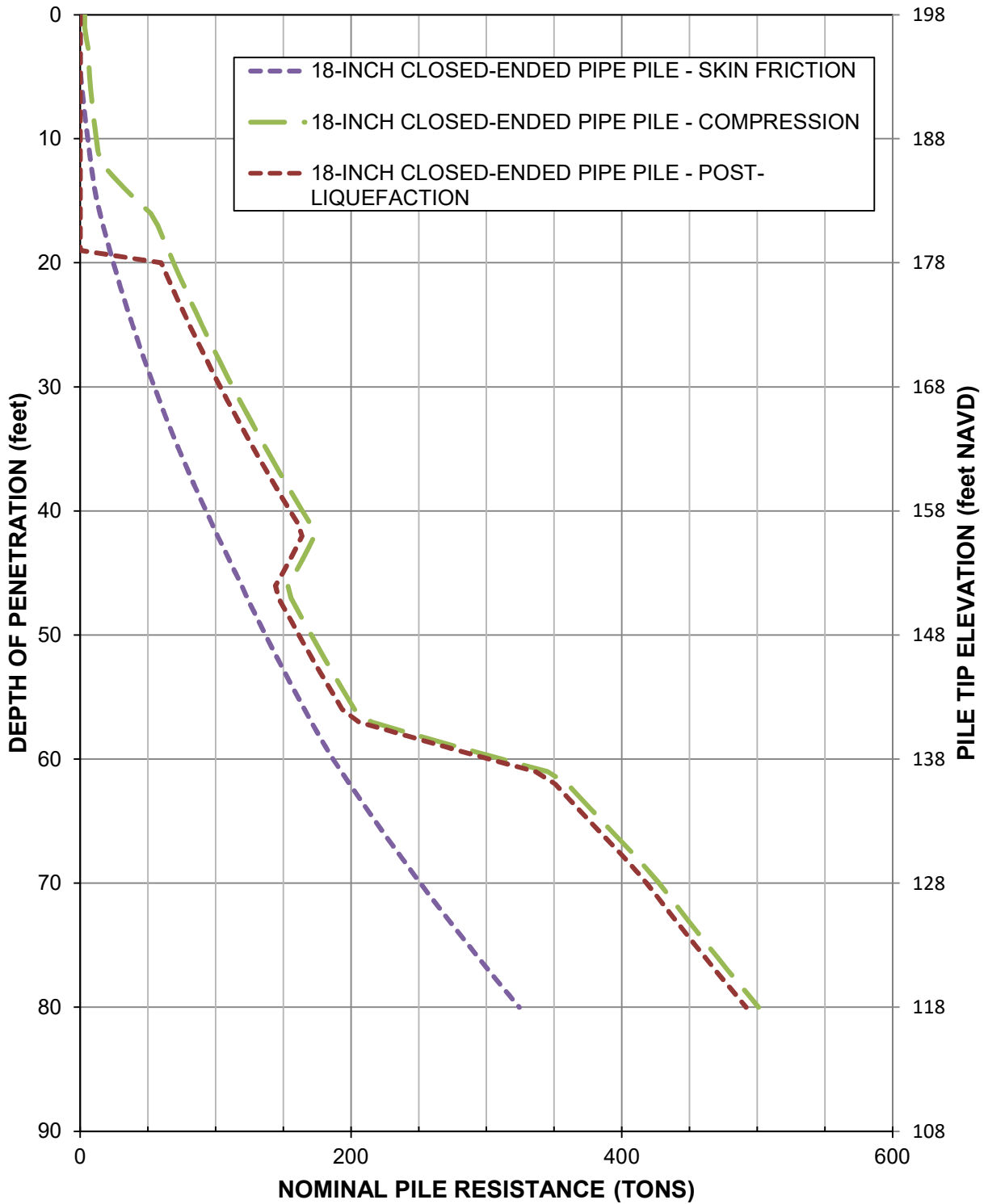
APPENDIX J – NOMINAL RESISTANCE CURVES



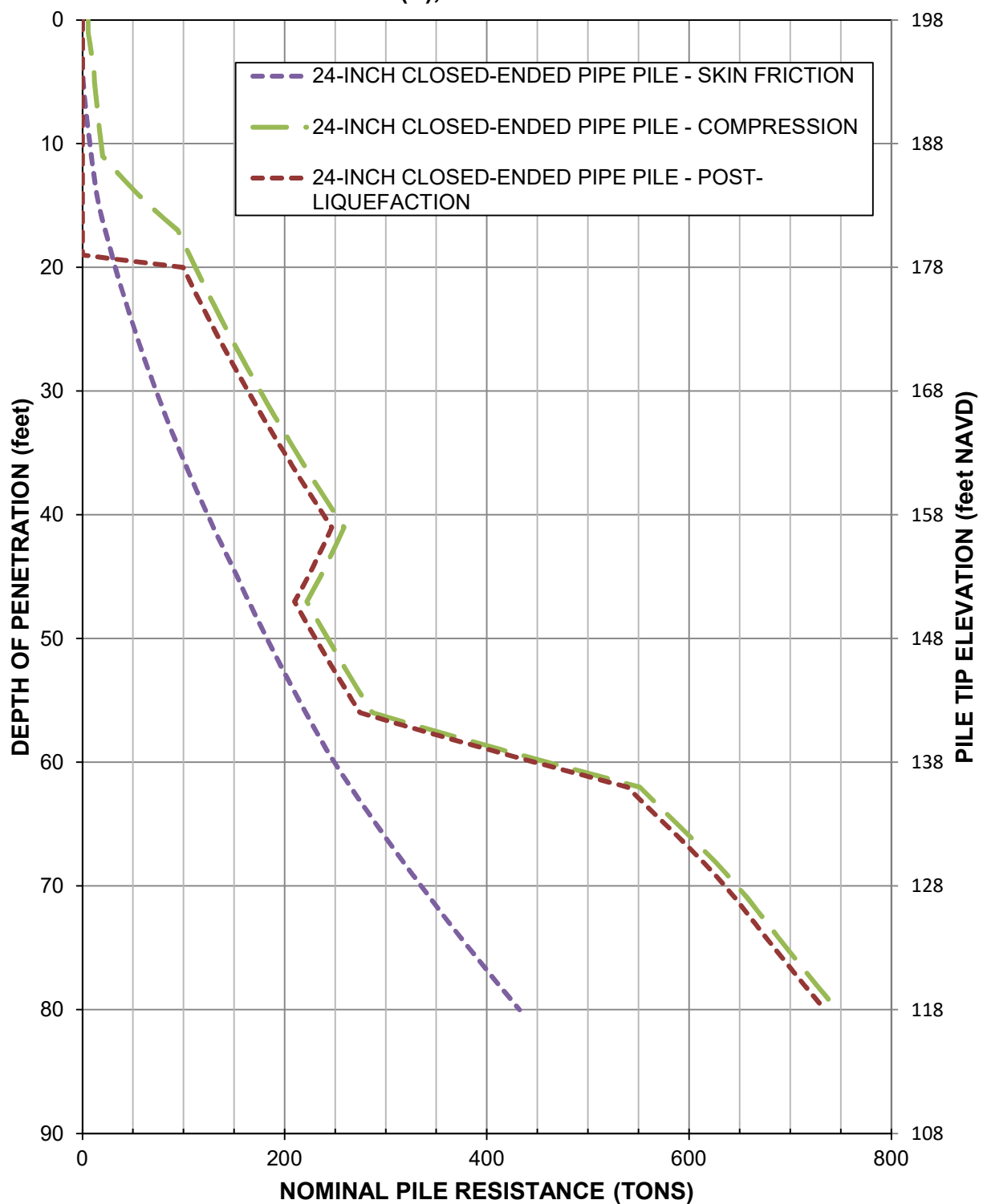
**NOMINAL RESISTANCE CURVES
 18-INCH CLOSED-ENDED PIPE PILE
 ARDOT TYRONZA RIVER & DITCH NO. 6 - BRIDGE 1
 STR. & APPRS. (S), POINSETT COUNTY - BENT 1**



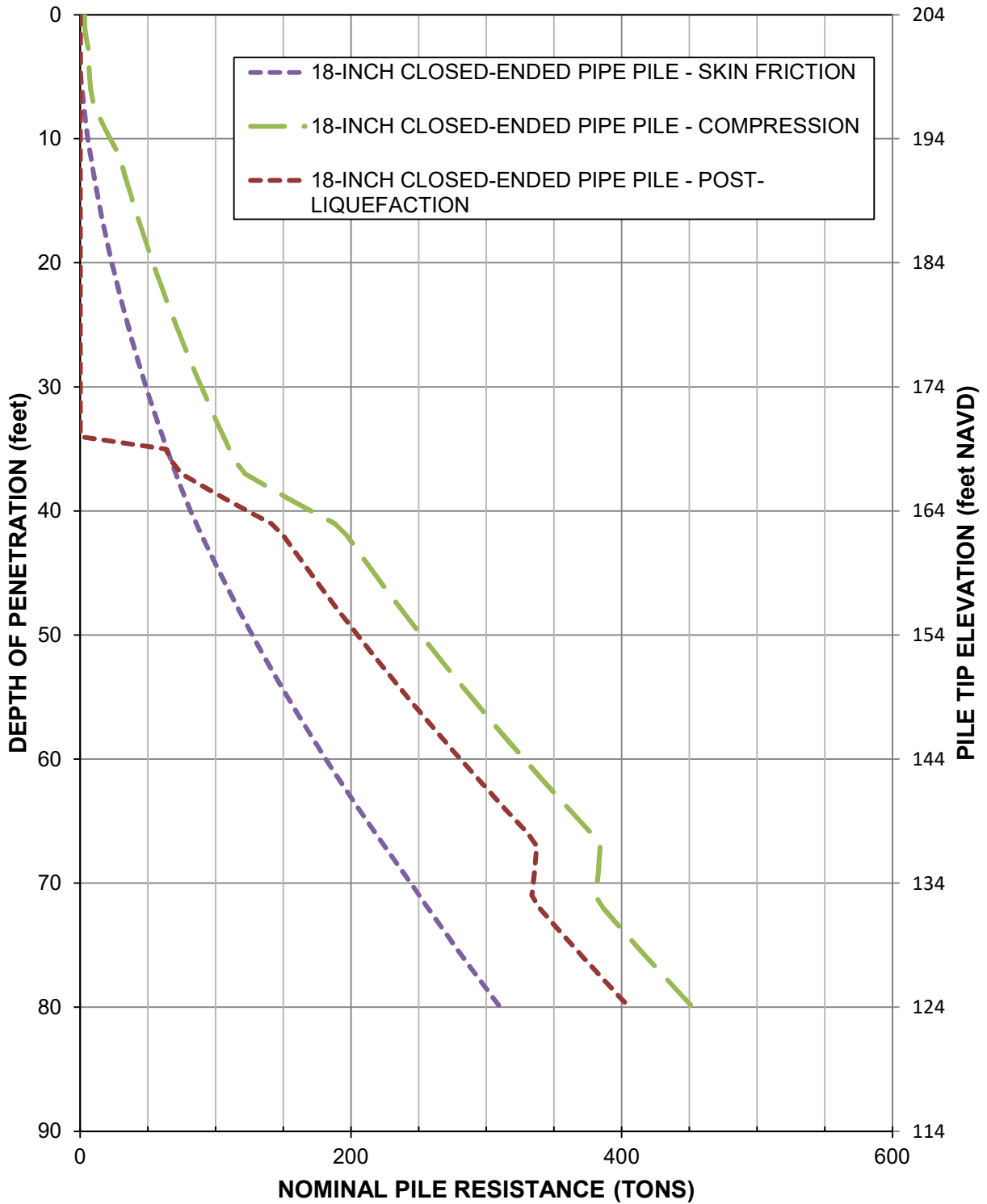
**NOMINAL RESISTANCE CURVES
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 ARDOT TYRONZA RIVER & DITCH NO. 6 - BRIDGE 1
 STR. & APPRS. (S), POINSETT COUNTY - BENT 2**



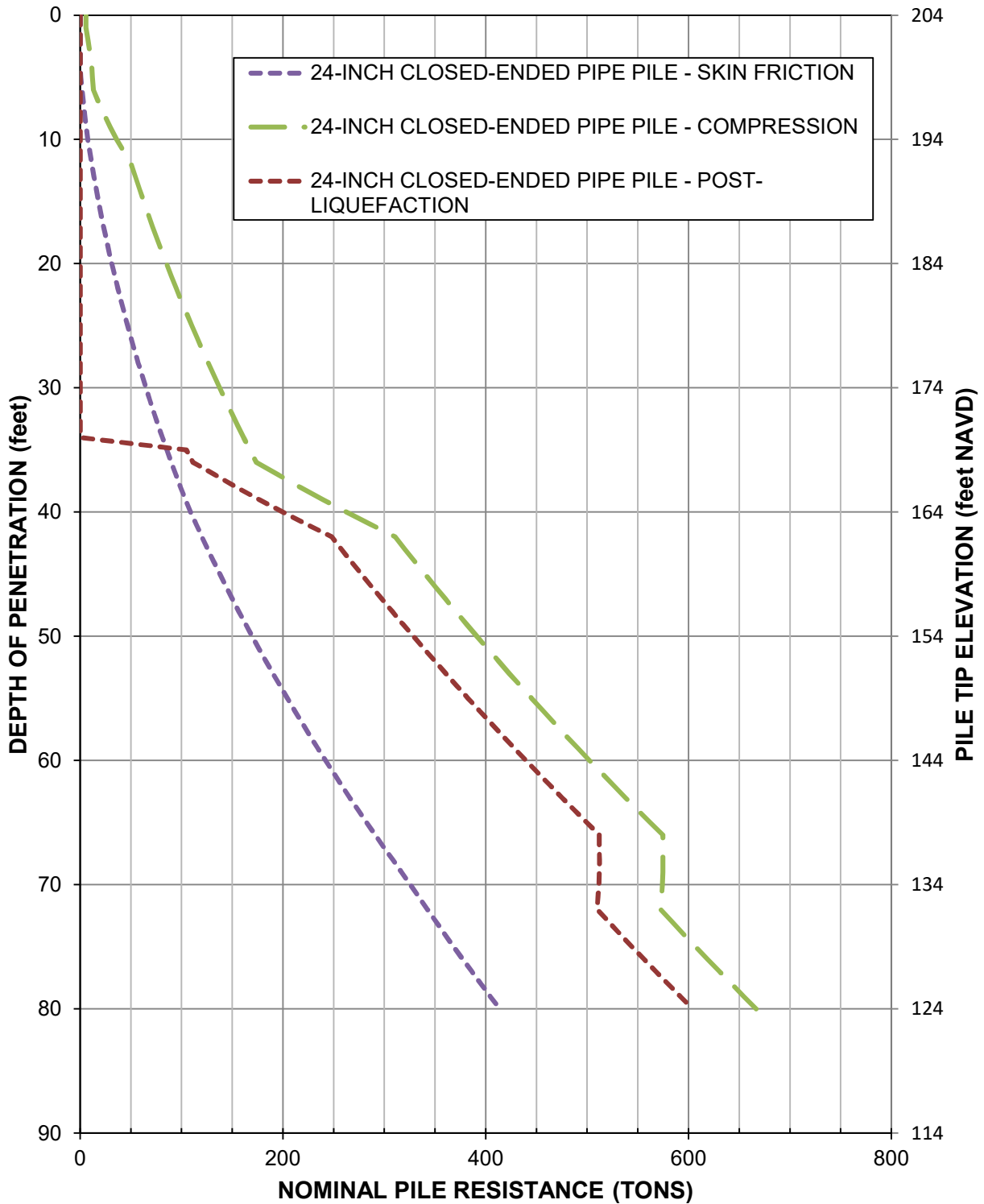
**NOMINAL RESISTANCE CURVES
 24-INCH CLOSED-ENDED PIPE PILE
 ARDOT TYRONZA RIVER & DITCH NO. 6 - BRIDGE 1
 STR. & APPRS. (S), POINSETT COUNTY - BENT 2**



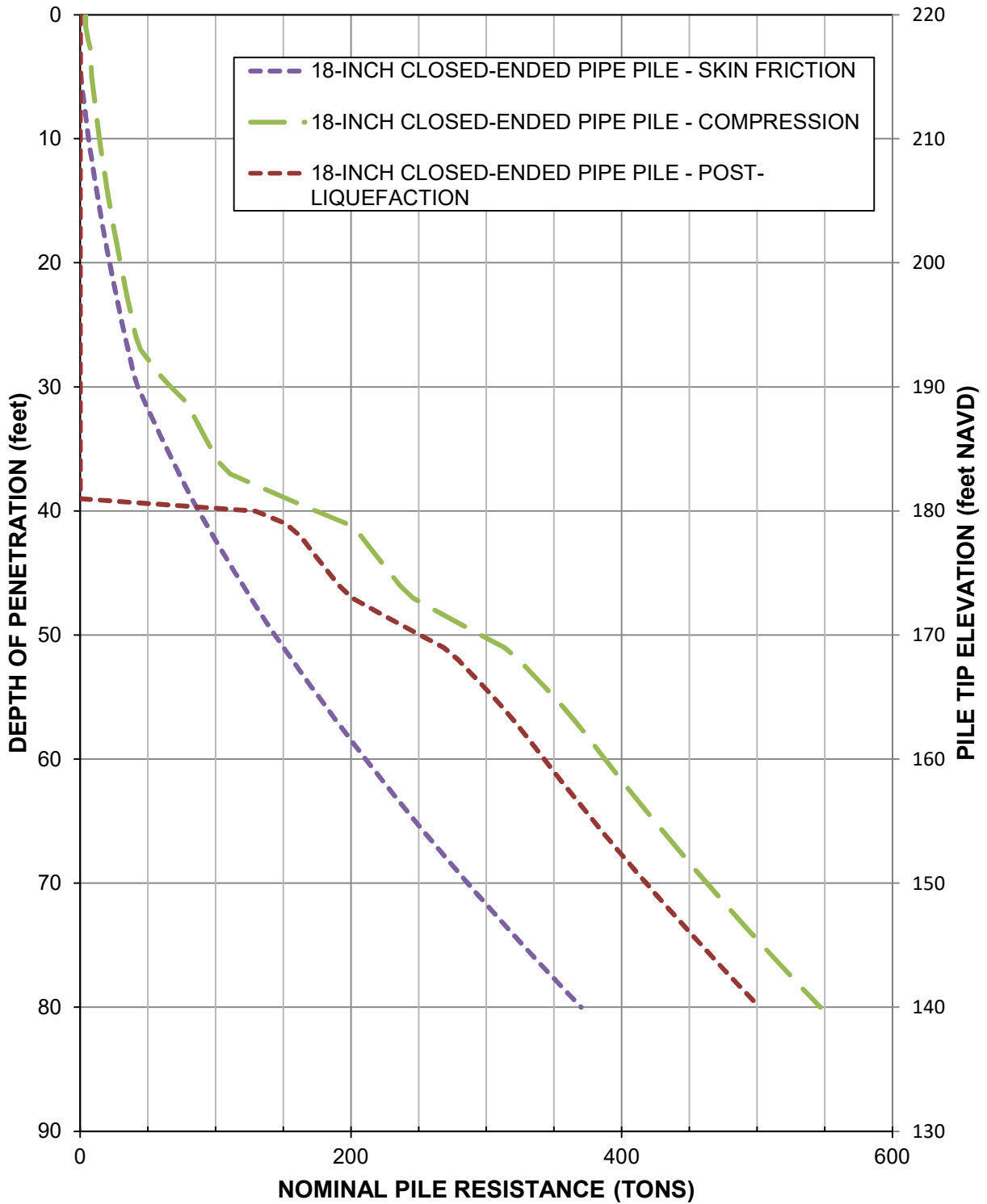
**NOMINAL RESISTANCE CURVES
 18-INCH CLOSED-ENDED PIPE PILE
 ARDOT TYRONZA RIVER & DITCH NO. 6 - BRIDGE 1
 STR. & APPRS. (S), POINSETT COUNTY - BENT 3**



**NOMINAL RESISTANCE CURVES
 24-INCH CLOSED-ENDED PIPE PILE
 ARDOT TYRONZA RIVER & DITCH NO. 6 - BRIDGE 1
 STR. & APPRS. (S), POINSETT COUNTY - BENT 3**



**NOMINAL RESISTANCE CURVES
 18-INCH CLOSED-ENDED PIPE PILE
 ARDOT TYRONZA RIVER & DITCH NO. 6 - BRIDGE 1
 STR. & APPRS. (S), POINSETT COUNTY - BENT 4**



GEOTECHNOLOGY

A UES Company



**GEOTECHNICAL REPORT
TYRONZA RIVER & DITCH No. 6
STRS. & APPRS. (S)
BRIDGE 2 – HWY. 118 OVER
TYRONZA RIVER
POINSETT COUNTY, ARKANSAS**

**ARKANSAS DEPARTMENT OF TRANSPORTATION
STATE PROJECT No. 101017**

Prepared for:
**ARKANSAS DEPARTMENT OF TRANSPORTATION (ARDOT)
LITTLE ROCK, ARKANSAS**

Prepared by:
**GEOTECHNOLOGY, LLC
MEMPHIS, TENNESSEE**

Date:
FEBRUARY 9, 2024

Geotechnology Project No.:
J042992.01

**SAFETY
QUALITY
INTEGRITY
PARTNERSHIP
OPPORTUNITY
RESPONSIVENESS**



February 9, 2024

Mr. Paul Tierney
Geotechnical Engineer
Arkansas Department of Transportation (ARDOT)
PO Box 2261
Little Rock, Arkansas 72203

Re: Geotechnical Report
Tyronza River & Ditch No. 6 Strs. & Apprs. (S)
Bridge 2 - Hwy. 118 Over Tyronza River
Poinsett County, Arkansas
ARDOT Project No. 101017
Geotechnology Project No. J042992.01

Dear Mr. Tierney:

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, conclusions and/or recommendations, and support 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



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

Jacob Monroe, P.E.
Project Engineer

JDM/ASE/DBA:jdm

Copies submitted: Client (email)



TABLE OF CONTENTS

| | |
|---|----|
| 1.0 Scope of Services..... | 1 |
| 2.0 General Information | 1 |
| Planned Modifications..... | 1 |
| Topography | 2 |
| Drainage | 2 |
| Geology | 2 |
| 3.0 Geotechnical Exploration | 2 |
| Cone Penetration Testing | 2 |
| Drilling and Sampling..... | 2 |
| 4.0 Laboratory Review and Testing | 3 |
| 5.0 Subsurface Conditions..... | 4 |
| Subgrade Materials..... | 4 |
| Creek Bed Material Mean Grain Size | 5 |
| Groundwater..... | 5 |
| 6.0 Engineering Evaluation, Analysis, and Recommendations | 5 |
| Site Preparation and Earthwork | 5 |
| Seismic Considerations | 7 |
| AASHTO LRFD 2020 Seismic Site Classification and Seismic Design Parameters | 7 |
| Site-Specific Ground Motion Assessment..... | 8 |
| Liquefaction and Dynamic Settlement..... | 8 |
| Approach Embankment Settlement | 10 |
| Global Stability..... | 10 |
| Deep Foundations | 11 |
| Downdrag | 15 |
| Corrosion Potential | 16 |
| 7.0 Recommended Additional Services | 17 |
| 8.0 Limitations..... | 17 |

Appendices

- Appendix A – Important Information about This Geotechnical-Engineering Report
- Appendix B – Figures
- Appendix C – Boring Information
- Appendix D – CPT-1 Sounding Plot
- Appendix E – Laboratory Test Data
- Appendix F – Site-Specific Seismic Study
- Appendix G – AASHTO and USCS Classifications
- Appendix H – Global Stability Analyses
- Appendix I – Soil Parameters for Synthetic Profiles
- Appendix J – Nominal Resistance Curves



LIST OF TABLES

| | |
|---|----|
| Table 1. Field Tests and Measurements..... | 3 |
| Table 2. Summary of Laboratory Tests and Methods. | 4 |
| Table 3. Approximate Mean Grain Size (D_{50}) of Tyronza River Bed Material..... | 5 |
| Table 4. Seismic Design Parameters (7% Probability of Exceedance in 75 years)..... | 8 |
| Table 5. Design Acceleration Parameters (7% Probability of Exceedance in 75 Years)..... | 8 |
| Table 6. Results of Liquefaction Analyses. | 9 |
| Table 7. Summary of Estimated Settlement..... | 10 |
| Table 8. Results of Slope Stability Analyses. | 11 |
| Table 9. Nominal Axial Resistance of Driven Closed-Ended Pipe Piles..... | 12 |
| Table 10. Resistance Factors Based on Static Analysis Methods. | 13 |
| Table 11. Resistance Factors for Driven Piles. | 13 |
| Table 12. Results of pH and Soil Resistivity Testing. | 16 |

GEOTECHNICAL REPORT
TYRONZA RIVER & DITCH NO. 6 STRS. & APPRS. (S)
BRIDGE 2 - HWY. 118 OVER TYRONZA RIVER
POINSETT COUNTY, ARKANSAS
February 9, 2024 | Geotechnology Project No. J042992.01

1.0 SCOPE OF SERVICES

Presented in this report are the results of the geotechnical exploration and recommendations for design and construction of the proposed replacement of the existing Bridge No. A3311 over Tyronza River along Highway 118 (Hwy. 118) in Poinsett County, Arkansas. The referenced project includes the construction of a new bridge to replace the existing Bridge No. A3311. It is our understanding the anticipated foundation type for support of the new bridge will be driven 18-inch-diameter, closed-ended pipe piles at the external (abutment) bents and 24-inch-diameter, closed-ended pipe piles at the interior bents as provided by ARDOT. The project location is shown on Figure 1 included in Appendix B.

The recommendations presented in this report are based on the geology, provided plans and project information, and the results of the geotechnical exploration. Results of the borings, Cone Penetration Test (CPT) sounding, in-situ testing, sampling, and laboratory testing are included in the report. A total of six borings and one seismic CPT sounding were performed in the vicinity of the site as shown on Figure 2 included in Appendix B. The boring logs and CPT sounding plot, along with field and laboratory test results, are enclosed. The collected data have been analyzed and the physical properties of the in-situ soils summarized. General site conditions are discussed, along with recommendations for subgrade preparation. 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

Planned Modifications

The existing two-lane, approximately 244-foot-long, 25-foot-wide, five-span Hwy. 118 Bridge No. A3311 over Tyronza River will be replaced by a two-lane, 332-foot-long, 32.5-foot-wide, three-span bridge. It is our understanding the proposed construction will be performed in two phases: in Phase 1 the existing Bridge No. A3311 will remain in use during construction of the replacement bridge; in Phase 2 the existing bridge will be demolished, and traffic will be facilitated to the new bridge.

The abutment slopes are anticipated to be two horizontal units for every vertical unit (2H:1V) and side slopes are anticipated to be 3H:1V. Up to 10 feet of cut and 8 feet of fill will be required at the bridge abutments to achieve design grades.



Topography

The proposed Hwy. 118 bridge replacement is located in Poinsett County, Arkansas. According to the provided plans¹, existing elevations at the west and east abutments of the proposed bridge location are approximately El 224 and El 220, respectively. which slope down to the bottom of Tyronza River at approximately El 190.

Drainage

Drainage at the site consists of Tyronza River which is aligned north-south. The drainage system in the project area consists of the Lower St. Francis Watershed. The Lower-St. Francis Watershed, in turn, is part of the overall drainage system of the St. Francis River Basin.

Geology

Poinsett County is located in northeast Arkansas in the Mississippi Embayment. The Mississippi Embayment is a trough-like depression dipping southward along an axis approximately following the Mississippi River. Site geology generally consists of Holocene-aged alluvial deposits of clay and silt underlain by fine-grained sand.

3.0 GEOTECHNICAL EXPLORATION

Cone Penetration Testing

One seismic cone penetration testing (CPT) sounding was performed in the proposed bridge eastern approach for continuous soil data collection and to measure the average shear wave velocity. The CPT sounding was performed to a depth of approximately 100 feet.

CPT2-1 was advanced using a 20-ton, track-mounted Vertek direct-push rig on June 13, 2023. The data were collected using a Vertek 15 square-centimeter end area, seismic piezometric cone with a u_2 pore pressure location (behind the cone) following the procedures outlined in ASTM D3441 and D5778. A plot of the CPT measurements is presented in Appendix D along with interpreted soil behavior types. Seismic CPT tests were performed in CPT2-1 at approximately 1-meter intervals to collect shear wave velocity data. A plot of the shear wave velocity profile is presented in Figure 3 in Appendix B.

Drilling and Sampling

A total of six borings were drilled to approximate depths of 50 to 100 feet at select locations in the proposed bridge approaches and through the existing bridge deck. The borings were drilled during the period of June 13 through July 19, 2023 using rotary drill rigs (CME 550X and Diedrich D-50), hollow-stem augers, and wet rotary methods. Sampling procedures included Standard Penetration Test (SPT) and thin-wall (Shelby) tube methods. SPT's were conducted at 2.5, 5, and 10-foot depth

¹ *Arkansas Department of Transportation, Construction Plans for State Highway, Tyronza River & Ditch No. 6 Strs. & Apprs. (S), Poinsett County, Route 118, Section 2, Job 101017, Fed. Aid Proj. NHPP-0056(57). Provided by Garver USA, dated September 14, 2023.*



intervals using an automatic hammer. Thin-walled Shelby tube samples were collected in cohesive soils at selected depths. Groundwater observations were made during drilling operations.

The collected samples were visually examined by field staff and transported to our laboratory for further evaluation and testing. The samples were examined in the laboratory by a geotechnical professional who prepared descriptive logs of the materials encountered. The boring logs are presented in Appendix C along with an explanation of the terms and symbols used on the boring logs. Included on each boring log are elevation data estimated from the provided plans. Included 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 |

The boring logs and CPT sounding plot represent conditions observed at the time of exploration and have been edited to incorporate results of the laboratory tests. Unless noted on the boring logs, the lines designating the changes between various strata represent approximate boundaries. The transition between materials could be gradual or 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.

The boring logs and CPT sounding plot 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. Most of the laboratory test results are presented on the boring logs in Appendix C. The Atterberg limits, grain size analyses, unconsolidated-undrained triaxial compression (UU), one-dimension consolidation, direct shear, pH, and soil resistivity test results are also provided in Appendix E. The laboratory tests and corresponding test method standards are presented in Table 2.



Table 2. Summary of Laboratory Tests and Methods.

| Laboratory Test | ASTM | AASHTO |
|---|--------|--------|
| Moisture Content | D 2216 | T 265 |
| Atterberg Limits | D 4318 | T 98 |
| Grain Size Analysis | D 6913 | T 88 |
| Percent Finer Than No. 200 Sieve | D 1140 | T 11 |
| Unconsolidated-Undrained Triaxial Compression | D 2850 | T 296 |
| One-Dimensional Consolidation | D 2435 | T 216 |
| Direct Shear | D 3080 | T 236 |
| pH of Soil | D 4972 | T 289 |
| Soil Electrical Resistivity | G 57 | T 288 |

The boring logs and CPT plot were prepared by a project geotechnical engineer from the field logs, visual classification of the soil samples in the laboratory, 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 strata could be abrupt or gradual.

5.0 SUBSURFACE CONDITIONS

Subgrade Materials

Borings B2-1, -2, -5, and -6 and CPT2-1 were performed in the proposed alignment of the replacement bridge. Borings B2-3 and -4 were drilled through the existing bridge deck and mudline of Tyronza River. Surficial material at Borings B2-1, -2, -5, and -6 consisted of approximately 6 to 12 inches of topsoil. Below the topsoil at Borings B2-1, -2, -5, and -6, at the ground surface at CPT2-1, and below the mudline at Borings B2-3 and -4, the soils generally consisted of predominately fine-grained soils overlying coarse-grained soils that extended to the boring and CPT termination depths. The boring logs and CPT sounding plot, with more detailed descriptions, are included in Appendix C and D, respectively. Laboratory testing was used to determine the AASHTO classifications as presented in Appendix G.

The upper, fine-grained soils were classified as high plasticity “fat” clay (CH), A-7-6; and low plasticity “lean” clay (CL), A-7-6, A-4. The fine-grained soils ranged from soft to very stiff in consistency.

The lower, coarse-grained soils were classified as poorly graded sand (SP), A-1-b, A-3; poorly graded sand with clay (SP-SC), A-2-7; poorly graded sand with silt (SP-SM), A-2-4; clayey sand (SC), A-2-6, A-7-6; and silty sand (SM), A-2-4. The coarse-grained soils ranged from loose to very dense conditions.



Creek Bed Material Mean Grain Size

We understand mean grain size (D_{50}) information is required for scour analyses to be performed by others. Grain size analyses were performed on select samples recovered in the upper 20 feet of the borings drilled into the existing Tyronza River bed. The approximate mean grain size results are presented in Table 3.

Table 3. Approximate Mean Grain Size (D_{50}) of Tyronza River Bed Material.

| Boring | Sample Depth (feet below ground surface) | Approximate Mean Grain Size, D_{50} (mm) |
|--------|---|--|
| B2-3 | 3.5 | 0.19 |
| B2-3 | 13.5 | 0.60 |
| B2-4 | 3.5 | 0.011 |
| B2-4 | 18.5 | 0.012 |

Groundwater

Groundwater was encountered in Borings B2-2 and 2-5 at depths of 35 and 33.5 feet below existing ground surface, respectively, correlating to groundwater elevations of El 185 and 188.5, respectfully. At CPT2-1, groundwater was interpreted at approximately 10 feet below existing ground surface, correlating to a groundwater elevation of approximately El 194 based on porewater pressure recordings. Groundwater levels were not encountered in Borings B2-1, -3, -4, and -6; however, groundwater levels may have been masked by the use of wet-rotary drilling methods or the water in the Tyronza River.

Groundwater levels will vary over time due to the effects of seasonal variations in precipitation, the stage and recharge rate of Tyronza River, or other factors not evident at the time of exploration.

6.0 ENGINEERING EVALUATION, ANALYSIS, AND RECOMMENDATIONS

Site Preparation and Earthwork

The following procedures are recommended for site preparation in cut and fill areas. These recommendations do not supersede ARDOT standards and specifications. Site preparation and compaction requirements must conform to the latest ARDOT standards.

Site Preparation. In general, cut areas and areas to receive new fill should be stripped of existing pavements, topsoil, vegetation, and other deleterious materials. Topsoil should be placed in landscape areas or disposed of off-site. Vegetation and tree roots should be over-excavated.

The exposed subgrade should be proof-rolled using a tandem axle dump truck loaded to approximately 20,000 pounds per axle (or equivalent proof-rolling equipment). Soft areas that develop should be over-excavated and backfilled with select fill, which is defined as soil conforming to A-4 or better material, and compacted to the unit weights specified in subsequent paragraphs.



Side Slopes. Existing slopes steeper than 4H:1V should be benched prior to placing new fill. Slope ratios of 2H:1V are proposed for abutment slopes. Slope ratios of 3H:1V or flatter are recommended for all cut and fill side slopes along the proposed alignment.

Cut Areas. It is our understanding up to 10 feet of cut will be required at the abutments. Based on the stratigraphy, excavations for pile cap foundations will terminate in fat clay. After excavation, the top 6 inches of the resulting subgrade should be compacted to a minimum of 95% of the maximum dry unit weight as determined by a standard Proctor test (ASTM D698/AASHTO T 99). Areas supporting pavement should be compacted to 98% of the maximum unit weight as determined by the standard Proctor test.

Fill Materials. It is our understanding up to 8 feet of fill will be required at the approach embankments. Fill material should consist of natural soils classifying as AASHTO A-6 or better², and should meet the minimum requirements set forth in ARDOT's Special Provision³ (SP) dated March 1, 2022. Soils classifying as AASHTO A-4 or better are considered to be select fill. Fine-grained "silt-clay" soils (A-4 through A-6) should have a maximum LL of 45 and a PI between 8 and 20 percent. Coarse-grained "sandy" soils used for embankment fills should have a minimum PI of 5 to lower potential for erosion. Fill materials should be free from organic matter, debris, or other deleterious materials, and have a maximum particle size of 2 inches.

Fill and Backfill Placement. Fill and backfill should be placed in level lifts, up to 8 inches in loose thickness. For fill and backfill exhibiting a well-defined moisture-density relationship, each lift should be moisture-conditioned to within $\pm 2\%$ of the optimum moisture content and compacted with a sheepfoot roller or self-propelled compactor to a minimum of 98% of the maximum dry unit weight as determined by the standard Proctor test. Moisture-conditioning can include: aeration and drying of wetter soils; wetting drier soils; and/or mixing wetter and drier soils into a uniform blend. The upper 3 feet of soil beneath the base of pavement should be compacted to 98% of the maximum unit weight as determined by the standard Proctor test.

For fill and backfill that do not exhibit a well-defined moisture-density relationship, each lift should be compacted to a 70% of the minimum relative density as evaluated from the maximum and minimum index densities measured by ASTM D4253 and D4254, respectively. The upper 3 feet of soil beneath the base of pavement should be compacted to 75% of the minimum relative density.

Fill Placement on Slopes. In areas that require fill to be placed on slopes, benching of existing slopes should be performed during placement of new fill. Fill on the sloped areas should begin from the toe of the slope and proceed upward, placing new fill on horizontal benches. Bench shelves should be 8 to 10 feet wide, and bench faces should be 1 to 2 feet in height. Fill lifts should be keyed into the slope to reduce the potential of a slip plane between the new fill and existing soils. Fill slopes should

² A-6 soils or better as determined by ARDOT.

³ Special Provision "Compacted Embankment", developed by ARDOT, dated March 1, 2022.



be constructed by extending the compacted fill beyond the planned profile of the slope and then trimming the slope to the desired configuration.

Moisture Considerations. Maintaining the moisture content of bearing and subgrade soils within the acceptable range is important during and after construction. Silty and clayey subgrade soils should not be allowed to become wet or dry during or after construction, and measures should be taken to hinder water from ponding on these soils. Positive drainage should be established to promote drainage of surface water away from the roadway.

Seismic Considerations

Earthquake Risk. The project area is located in the vicinity of 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 earthquake occurred, which were centered near New Madrid, Missouri. Three strong earthquakes occurred over the next three months 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 the bridge and approaches will be designed in accordance with the AASHTO publication “LRFD Bridge Design Specifications”, eighth edition (2020).

AASHTO LRFD 2020 Seismic Site Classification and Seismic Design Parameters

Seismic Design Parameters. Seismic design parameters based on a seismic hazard with 7% probability of exceedance in 75 years and field and laboratory testing is presented in Table 4.



Table 4. Seismic Design Parameters (7% Probability of Exceedance in 75 years).

| Latitude 35.511323°N/Longitude 90.309347°W | | |
|--|-----------------------|---|
| Category/ Parameter | Designation/ Value | Reference |
| Seismic Zone | 4 | AASHTO LRFD 2020 Table 3.10.6-1 |
| Seismic Site Class | D | AASHTO LRFD 2020 Table 3.10.3.1-1 |
| S_S | 1.720g | Ground motion parameters obtained from a computer program supplied with the AASHTO Guideline for the Seismic Design of Highway Bridges (2020) using the indicated latitude and coordinates of the project site and the seismic site class based on boring data. |
| S_1 | 0.455g | |
| F_a | 1.000 | |
| F_v | 1.545 | |
| F_{PGA} | 1.000 | |
| t_s | | |
| t_0 | | |
| S_{DS} | 1.720g | |
| S_{D1} | 0.703g | |
| PGA | 0.973g | |
| A_s | 0.973g | |

Site-Specific Ground Motion Assessment

A site-specific study was performed for the project site to develop a site-specific seismic design response spectrum. The process included 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 CPT1-1 of 767 feet per second as shown on Figure 3 (Shear Wave Velocity Profile) in Appendix B.

The results of the shear wave velocity measurements indicate that the site is a Site Class D, “stiff soil”, profile based on a $V_{S,100}$ of 767 feet per second. According to the results of the site-specific seismic study, the recommended site-specific design accelerations are presented in Table 5.

Table 5. Design Acceleration Parameters (7% Probability of Exceedance in 75 Years).

| Parameter | Value |
|-----------|--------|
| S_{DS} | 1.284g |
| S_{D1} | 1.017g |
| MCE_G | 0.649g |

Liquefaction and Dynamic Settlement

A study was performed to evaluate the liquefaction and dynamic settlement potential at the site. Both field and laboratory data were used to perform the analysis. The field measurements included the



depth of the water table, SPT N-values, and information collected in CPT1-1. The laboratory data included USCS classification and soil unit weight. An earthquake magnitude (M_w) of 7.7 with a probability of exceedance of 7% in 75 years was considered. A site peak ground acceleration of 0.649g was utilized as obtained from the site-specific seismic study. Groundwater was set at an elevation of approximately El 188 as indicated in the CPT and borings.

Subsurface conditions (as characterized by the field and laboratory data) and earthquake characteristics were used to estimate the safety factors against liquefaction in each soil layer, as well as the associated dynamic settlement during the design seismic event. Based on the analysis, there is liquefaction potential at the site. The analysis results are presented in Table 6.

Table 6. Results of Liquefaction Analyses.

| Location | Depth of Boring (feet) | Liquefiable Zones ^a (feet below ground surface) | Estimated Dynamic Settlement (inches) | |
|----------|------------------------|---|---------------------------------------|------------------------------------|
| | | | Upper 50 Feet | Total Depth of Boring/CPT Sounding |
| B2-1 | 50 | N/A ^b | N/A ^b | N/A ^b |
| B2-2 | 100 | 33.5 – 43.5 68.5 – 83.5 | 2½ | 4½ |
| B2-3 | 100 | 38.5 – 43.5 | 1½ | 1½ |
| B2-4 | 100 | 18.5 – 23.5 | ½ | ½ |
| B2-5 | 100 | 53.5 – 58.5 | 1½ | 1½ |
| B2-6 | 50 | N/A ^b | N/A ^b | N/A ^b |
| CPT2-1 | 100 | 52 – 54 | ½ | 1¼ |

^a Defined as zones with a factor of safety against liquefaction in the design earthquake of less than 1.0, as computed using the deterministic method by Idriss and Boulanger, 2014.

^b Liquefaction and dynamic settlement not anticipated in boring.

Please note that the current state of practice for liquefaction hazard assessment is based on what is known as “The Simplified Method” as introduced by Seed (1971) and subsequent modifications/revisions by others (Seed 1982, Idriss 1999, Youd 2001, and Idriss and Boulanger 2014, among others). The simplified method was based on observations and assessments of soil zones that either liquefied or did not liquefy in the upper 50 feet. Because of reported uncertainties in the literature, the occurrence of substantial liquefaction in relatively deep sand deposits below 50 feet is considered unlikely.

Lateral Spreading. Lateral spreading is triggered and sustained by earthquake ground motions. Based on our seismic slope stability analyses, it is our professional opinion that the potential for lateral spreading is low at the site. However, in the event of an earthquake, the soils that liquefy will have residual strengths which can have potentially destabilizing effects on overlying slopes. Post-liquefaction residual shear strength parameters are presented in Appendix I.



Approach Embankment Settlement

Settlement analyses of natural soils were performed to assess fill-induced settlement for the approaches. Based on the provided preliminary plans, up to approximately 8 and feet of fill will be required at the western and eastern approaches, respectively, to bring the site to design grade. For settlement analyses, we have assumed cohesive, engineered fill will be used for the fill material. The results of the settlement due to fill placement are shown in Table 7. If grade changes will require the placement of additional fill, Geotechnology should be contacted to perform additional settlement analyses for fill-induced settlement at the approaches.

Table 7. Summary of Estimated Settlement.

| Southern Abutment (Exterior Bent No. 1) | | | | Northern Abutment (Exterior Bent No. 4) | | | |
|--|----------------------------------|------------------------------|-------|--|----------------------------------|------------------------------|-------|
| Max Fill (feet) | Estimated Settlement (inches) | | | Max Fill (feet) | Estimated Settlement (inches) | | |
| | Immediate | Long-Term (Consolidation) | Total | | Immediate | Long-Term (Consolidation) | Total |
| 8 | 1¼ | 2½ | 3¾ | 5 | ¾ | 1¼ | 2 |

The bent numbers presented in Table 7 are in reference to the bent number designations presented on the provided preliminary plans. Based on review of the preliminary plans, the bents are numbered from 1 to 4 such that Bent No. 1 is at the western abutment.

Discussion of Fill-Induced Settlement. The results of the settlement analyses indicate immediate and long-term (primary) consolidation settlement at the approaches. We anticipate the immediate settlement to occur shortly after fill placement and practical completion of consolidation to occur within 2 to 4 weeks after fill placement.

Global Stability

Geotechnology performed stability analyses for deep-seated, global failure of bridge abutment slopes using the computer program SLIDE2. Short-term, long-term, and seismic conditions were considered using the Spencer method to compute factors of safety for the proposed slopes.

Calculated minimum factors of safety are summarized in the following table. Minimum required factors of safety for the proposed bridge were based on the ARDOT Minimum Acceptable Factors of Safety as provided by ARDOT using a seismic operational class of “Other”. A pseudo-static seismic acceleration of 0.3245g, corresponding to one-half the peak ground acceleration (per FHWA Publication HI-99-012) was utilized.

A groundwater elevation of approximately El 188 feet, as measured in the borings and interpreted in CPT2-1, was utilized for the short-term and seismic condition analyses and a design highwater elevation of El 216.6, as obtained from the plans provided by Garver, was used for the long-term condition analyses. Section profiles with critical slip surfaces and utilized soil parameters are



presented in Appendix H for the selected analyses. The analysis models did not consider the effect of foundation piles driven at the abutments or riprap placed on the abutment slopes that would provide additional restraining force to stabilize the slopes.

Table 8. Results of Slope Stability Analyses.

| Location | Description | Slope Height (ft.) | Calculated Factor of Safety | | |
|-----------------------------------|-------------|--------------------|----------------------------------|---------------------------------|------------------------|
| | | | Short-Term Static ^{a,c} | Long-Term Static ^{a,d} | Seismic ^{b,c} |
| Western Abutment STA 150+66.42 | 2H:1V | 14 | 5.01 | 1.44 | 1.91 |
| | Cut Slope | | | | |
| Eastern Abutment STA 153.98.58 | 2H:1V | 11 | 5.33 | 2.84 | 1.94 |
| | Cut Slope | | | | |

- ^a Target factor of safety = 1.3, approximately equivalent to a global stability resistance factor = 0.75, as provided by ARDOT.
- ^b Target factor of safety = 1.1, approximately equivalent to a global stability resistance factor = 0.9, as provided by ARDOT.
- ^c Based on a groundwater elevation of approximately El 185; approximately 35 feet below existing ground surface at the abutments.
- ^d Based on a Tyronza River design highwater elevation of El 216.6 as obtained from the preliminary plans provided by ARDOT.

Fill material used for construction of the embankments will be required to meet the criteria established in the Special Provision dated March 1, 2022 provided by ARDOT.

Deep Foundations

Foundation design recommendations are provided herein based on the AASHTO LRFD Bridge Design Specifications (2020).

It is our understanding the proposed foundation type for the bridge will be driven, closed-ended pipe piles as provided by ARDOT. A closed-ended pipe pile diameter of 18-inches was considered for the abutment and intermediate bents; additionally, a closed-ended pipe pile diameter of 24-inches was considered for intermediate bents. Geotechnology should be notified if other foundation types or sizes are to be considered. Based on the provided information, we have assumed existing ground surface elevations of approximately El 220 and El 222 for exterior bents 1 and 2, respectively, and El 193 and El 204 for interior bents 2 and 3, respectively. Soil parameters, including LPILE lateral load analysis parameters, for each bent are included in Appendix I.

Nominal resistance curves showing axial resistance from skin friction and total axial capacity (skin friction + end bearing) for Bents 1 through 4 are presented in Appendix J. Nominal static resistances at each bent for the driven, closed-ended pipe piles are presented in Table 9. Uplift (tension) capacities may be calculated using the resistance provided by skin friction.



Also presented in Table 9 are nominal post-liquefaction resistances and nominal drag loads on piles. For the purposes of the post-liquefaction pile resistance analyses, liquefaction was considered to be limited to the upper 50 feet of the soil profile at the bents. Nominal post-liquefaction resistance curves are presented in Appendix J.

Table 9. Nominal Axial Resistance of Driven Closed-Ended Pipe Piles.

| Location | Pile Diameter (inches) | Embedment Length (feet) | Nominal Static Resistance (tons) | | | Nominal Post-Liquefaction Resistance ^e (tons) | |
|--|------------------------|-------------------------|----------------------------------|-------------|-------------------|--|-------------------|
| | | | Skin Friction | End Bearing | Compression Total | Compression Total ^f | Nominal Drag Load |
| Exterior (Abutment) Bents 1 and 4 | | | | | | | |
| Bent 1 ^a (West Abutment) | 18 | 60 | 232 | 172 | 404 | 282 | 61 |
| | | 70 | 311 | 128 | 439 | 318 | |
| | | 80 | 392 | 119 | 511 | 390 | |
| Bent 4 ^b (East Abutment) | | 60 | 228 | 146 | 374 | N/A ^f | |
| | | 70 | 307 | 122 | 429 | | |
| | | 80 | 390 | 118 | 508 | | |
| Interior Bents 2 and 3 | | | | | | | |
| Bent 2 ^c | 24 | 60 | 210 | 221 | 431 | 311 | 99 |
| | | 70 | 288 | 314 | 602 | 481 | |
| | | 80 | 378 | 330 | 708 | 587 | |
| | 30 | 60 | 263 | 339 | 602 | 451 | 124 |
| | | 70 | 360 | 490 | 850 | 700 | |
| | | 80 | 472 | 515 | 987 | 837 | |
| Bent 3 ^d | 24 | 60 | 275 | 184 | 459 | 418 | 23 |
| | | 70 | 362 | 207 | 569 | 528 | |
| | | 80 | 464 | 330 | 794 | 753 | |
| | 30 | 60 | 344 | 288 | 632 | 580 | 28 |
| | | 70 | 452 | 324 | 776 | 725 | |
| | | 80 | 580 | 515 | 1095 | 1044 | |

- ^a Embedment length referenced from approximate ground surface elevation of El 220.
^b Embedment length referenced from approximate ground surface elevation of El 222.
^c Embedment length referenced from approximate ground surface elevation of El 193.
^d Embedment length referenced from approximate ground surface elevation of El 204.
^e For the purposes of downdrag analyses, liquefaction-induced downdrag has been limited to the upper 50 feet of embedment. Liquefiable layers below 50 feet not considered for downdrag analyses.
^f Liquefaction not anticipated in upper 50 feet of Bent 4; downdrag not considered.

Resistance Factors. Resistance factors should be applied to the nominal resistances provided. Based solely on the static analysis methods used to calculate nominal pile resistances, the factors presented in Table 10 may be applied.



Table 10. Resistance Factors Based on Static Analysis Methods.

| Deep Foundation and Condition | Clay | | Sand | |
|---|-----------------|-------------|-----------------|-------------|
| | Side Resistance | End-Bearing | Side Resistance | End-Bearing |
| Nominal Compressive Resistance of Single Pile | 0.35 | 0.35 | 0.45 | 0.45 |
| Uplift Resistance of Single Pile | 0.25 | -- | 0.35 | -- |

Based on the AASHTO LRFD (2020) Table 10.5.5.2.3-1, a higher resistance factor can be used in accordance with the method of pile testing performed as indicated in Table 11.

Table 11. Resistance Factors for Driven Piles.

| Condition/Resistance Determination Method | | Resistance Factor |
|---|--|-------------------|
| Nominal Bearing Resistance of Single Pile – Dynamic Analysis and Static Load Test Methods | Driving criteria established by successful static load test of at least one pile per site condition and dynamic testing of at least two piles per site, but no less than 2% of the production piles* | 0.80 |
| | Driving criteria established by successful static load test of at least one pile per site condition without dynamic testing | 0.75 |
| | Driving criteria established by dynamic testing conducted on 100% of production piles* | 0.75 |
| | Driving criteria established by dynamic testing, quality control by dynamic testing of at least two piles per site condition, but no less than 2% of production piles* | 0.65 |
| | Wave equation analysis, without pile dynamic measurements or load test but with field confirmation of hammer performance | 0.50 |
| | FHWA-modified Gates dynamic pile formula (End of Drive condition only) | 0.40 |
| Uplift Resistance of Single Pile | Dynamic test with signal matching | 0.50 |

* Dynamic testing requires signal matching, and estimates of nominal resistance are made from a restrrike. Dynamic tests are calibrated to a static load test, when available.

Pile Group Considerations. The settlement of pile groups should be evaluated as per AASHTO LRFD (2020) section 10.7.2.3. Settlement analysis of the pile groups can be performed when the foundation configurations and service loads are available. AASHTO LRFD (2020) section 10.7.3.9 addresses pile group resistance. Group capacity considerations for different pile groups,



center-to-center spacings, and other conditions (cap contact with ground, softness of surface soil etc.) are given in AASHTO LRFD (2020) sections 10.7.3.9 and 10.7.3.11.

Driven Pile Construction Considerations. Minimum hammer energies required to drive the piles were not evaluated for the proposed foundations. If minimum hammer energy evaluations are required, Geotechnology should be contacted to perform analyses for the required minimum hammer energies for driving piles.

Static Pile Load Testing. At least one static pile compression load test should be performed for each bent or abutment location. The testing should be performed in accordance with ASTM D 1143 using the quick loading procedure and AASHTO LRFD (2020) section 10.7.3.8.2. Please refer to the previous Resistance Factors table for additional guidance regarding the minimum number of tests and alternate resistance factors associated with other field methods for determining resistance.

If the piles are to support net uplift loads, at least one tension load test should be performed for each location. The test should be performed in accordance with ASTM D 3689. Piles should be tested to the required nominal uplift resistances.

Load tests are required to verify recommended nominal pile resistance and will not be used to increase the design pile resistance. The piles used in the load tests should not be used for support of any structures. Geotechnology should be consulted regarding the locations of the test piles.

Dynamic Testing of Driven Piles. As an alternative to static pile load testing, high-strain dynamic pile testing can be performed according to AASHTO LRFD (2020) section 10.7.3.8.3 and the procedures given in ASTM D4945. Different resistance factors correspond to different load testing combinations as illustrated in the previous table. We recommend that the test piles be identified according to AASHTO LRFD (2020) Table 10.5.5.2.3-1 or 2 percent of the production piles, whichever results in a larger number of tests. We recommend that the identified piles be tested at the end of initial drive (EIOD) and a restrike performed at a minimum seven days after EIOD.

Pile driving monitoring should be performed by an engineer with a minimum 3 years dynamic pile testing and analysis experience and who has achieved Basic or better certification under the High-Strain Dynamic Pile Testing Examination and Certification process of the Pile Driving Contractors Association and Foundation QA. Pile driving modeling and analyses should be performed by an engineer with a minimum five years dynamic pile testing and analysis experience and who has achieved Advanced or better certification under the High-Strain Dynamic Pile Testing Examination and Certification process of the Pile Driving Contractors Association and Foundation QA.

Dynamic tests are required to monitor hammer and drive system performance, assess driving stresses and structural integrity and to evaluate pile resistance, and should not be used to increase design pile resistance. Dynamic tests should be performed on production piles with the lowest driving



resistance. Geotechnology will be available to assist with the development of specifications for this program and should be on site to perform or observe the testing and establish the pile driving criteria.

Settlement. Settlement of pile foundations depends on the loads applied and the foundation configuration. In general, settlement of deep foundations designed in accordance with the recommendations provided in this report is expected to be less than 1-inch. However, a calculation of the expected settlement of the pile foundations can be performed when the applied service loads and foundation configuration are available.

Uplift Resistance. Uplift forces can be resisted by the effective weight of the piles and caps, and frictional resistance between the piles and surrounding soil. If the anticipated maximum level of groundwater is higher than the tip of the pile then the buoyant unit weight of the pile must be used in computing uplift resistance for pile lengths extending below the design groundwater level.

Lateral Resistance. The lateral resistance of pile foundations depends on the lengths and dimensions of the foundations and the soil characteristics. The lateral resistance of pile foundations can be computed using the computer program LPILE to model the behavior of a single pile or shaft. Soil parameters are provided in Appendix I for the various strata and soil strengths present at the site. Soil parameters are based on field and laboratory test results and empirical correlations with SPT N-values.

The effects of group interaction must be considered when evaluating pile/shaft group horizontal movement. The lateral resistance for individual piles calculated by LPILE must be reduced by the P-multipliers provided in Section 10.7.2.4 of the AASHTO LRFD (2020) to determine lateral resistance of a pile group. Alternatively, the GROUP software can be used to evaluate the lateral resistance of the pile/shaft groups. The resistance factor for lateral resistance of single pile or pile group is 1.0.

Downdrag

The AASHTO LRFD (2020) suggests that soil settlement relative to a pile of 0.4-inch or greater could produce downdrag on pile foundations. Downdrag occurs as the soil strata move downward relative to foundations due to settlement of the soil layers. The relative movement of the soil layers versus the shaft depends on the final foundation configuration.

Downdrag Due to Fill-Induced Settlement. Based on settlement analyses performed for the 8-foot maximum fill placement at the abutments, up to 3¾ inches of settlement is predicted. The settlement due to fill placement is estimated to occur immediately after fill placement. Pile driving should not begin or at least 4 weeks following completion of fill placement to allow time for settlement to be essentially complete. Piles driven prior to completion of settlement may be subject to drag loads as the soil below the fill consolidates due to the weight of the fill.

Downdrag Due to Dynamic Settlement. Based on the results of the liquefaction analyses, up to 2½ inches of dynamic settlement in the upper 50 feet of soil is anticipated during the design earthquake



event (7% exceedance in 75 years) at the exploration locations. Additional analyses were performed to evaluate post-liquefaction pile resistances. The nominal post-liquefaction resistances and resistances are provided in Table 9. Pre-drilling and/or applying viscous coatings are not recommended to reduce liquefaction-induced downdrag because such methods will reduce the nominal static compressive resistance of the piles.

Liquefaction-induced downdrag can be reduced by performing ground improvement on the potentially liquefiable soil layers. It is our professional opinion that driving the closed-ended pipe piles through the liquefiable layers will densify the susceptible layers in the vicinity of the piles; however, the extent of the densification cannot be determined at this stage. The extent of densification can be estimated by advancing CPT soundings in the vicinity of the piles after they are driven. If more information is desired, please contact Geotechnology.

Corrosion Potential

In addition to laboratory soil classification and strength testing, soil resistivity testing was also conducted. The purpose of soil resistivity testing is to provide soil data for use by a structural engineer for analysis of any necessary protection of the piling, concrete, reinforcing steel, etc. Corrosion and deterioration protection requirements and guidelines for piling are set forth in Section 10.7.5 of the AASHTO LRFD Bridge Design Specifications. The corrosion and deterioration testing results are summarized in Table 12 and are included in Appendix E.

Table 12. Results of pH and Soil Resistivity Testing.

| Boring | Sample No. | Sample Depth (feet) | pH | Soil Resistivity (ohm-cm) |
|--------|------------|---------------------|------|---------------------------|
| B2-3 | SS-4 | 8.5 | 8.08 | 4,577.10 |
| B2-4 | SS-11 | 43.5 | 7.47 | 1,784.10 |
| | SS-14 | 58.5 | 8.06 | 1,734.51 |
| B2-5 | ST-7 | 15 | 8.09 | 649.23 |
| | SS-12 | 38.5 | 7.77 | 3,724.95 |

The following soil conditions should be considered as indicative of a potential for steel pile deterioration or corrosion:

- Resistivity values less than 2,000 ohms-cm; or
- pH less than 5.5.



The following soil conditions should be considered as indicative of a potential for steel reinforcement corrosion or deterioration:

- Resistivity values less than 3,000 ohms-cm; or
- pH less than 5.5.

Interpretation of the data and corrosion protection of the bridge structural components should be performed by the design team.

7.0 RECOMMENDED ADDITIONAL SERVICES

The conclusions and recommendations given in this report are based on: Geotechnology's understanding of the proposed design and construction, as outlined in this report; site observations; interpretation of the exploration data; and our experience. Since the intent of the design recommendations is best understood by Geotechnology, we recommend Geotechnology be included in the final design and construction process, and be retained to review the project plans and specifications to confirm the recommendations given in this report have been correctly implemented. We recommend Geotechnology be retained to participate in pre-bid and preconstruction conferences to reduce the risk of misinterpretation of the conclusions and recommendations in this report relative to the proposed construction of the subject project.

Since actual subsurface conditions between boring locations could vary from those encountered in the borings, our design recommendations are subject to adjustment in the field based on the subsurface conditions encountered during construction. Therefore, we recommend Geotechnology be retained to provide construction observation services as a continuation of the design process to confirm the recommendations in this report and to revise them accordingly to accommodate differing subsurface conditions. Construction observation is intended to enhance compliance with project plans and specifications. It is not insurance, nor does it constitute a warranty or guarantee of any type. Regardless of construction observation, contractors, suppliers, and others are solely responsible for the quality of their work and for adhering to plans and specifications.

8.0 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.*



Telephone: 301/565-2733

e-mail: info@geoprofessional.org www.geoprofessional.org



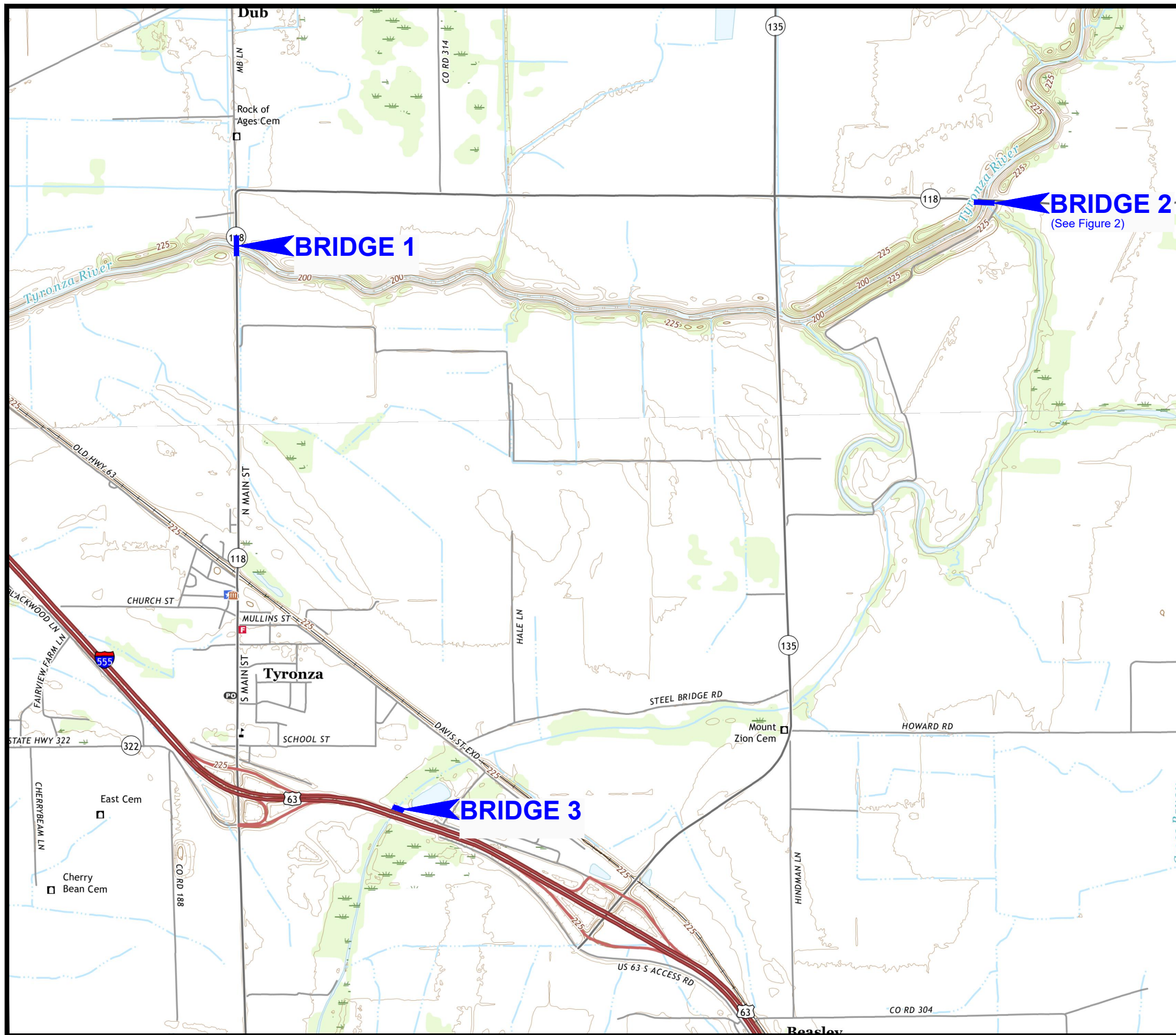
APPENDIX B – FIGURES

Figure 1 – Site Location and Topography

Figure 2 – Aerial Photograph of Site and Boring Locations

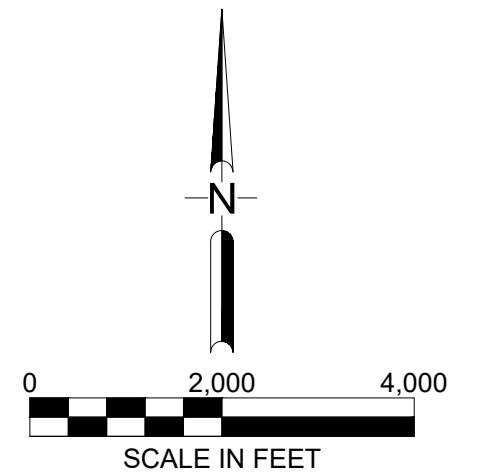
Figure 3 – Shear Wave Velocity Profile




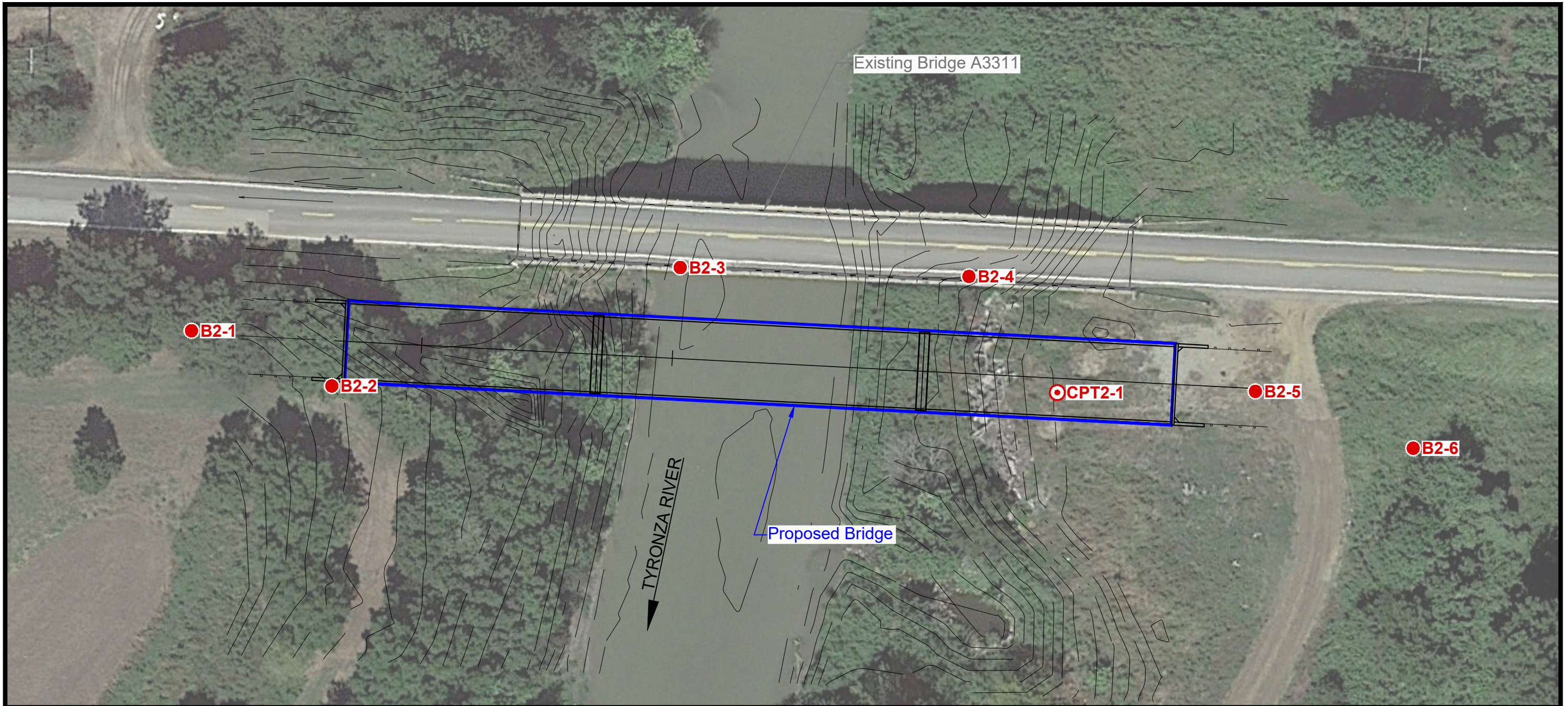


NOTES

1. Plan adapted from 7.5 minute U.S.G.S. maps for Lepanto and Tyronza, Arkansas quadrangles, last revised in 2020.



| | | |
|--|-----------------|----------------|
| Drawn By: WAH | Ck'd By: JDM | App'vd By: ASE |
| Date: 5-5-23 | Date: 11-27-23 | Date: 11-27-23 |
|  GEOTECHNOLOGY <small>A UES Company</small> | | |
| ARDOT 101017 Tyronza River & Ditch No. 6 Strs. & Apprs. (S) Poinsett County, Arkansas | | |
| SITE LOCATION AND TOPOGRAPHY | | |
| Project Number J042992.01 | FIGURE 1 | |

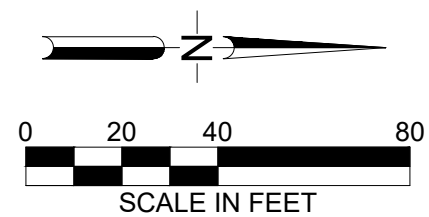


NOTES

1. Plan adapted from a September 5, 2021 aerial photograph courtesy of Google Earth and a drawing dated January 20, 2023, titled "Soil Boring Request Layout of Bridge Over Tyronza River", prepared by Arkansas State Highway Commission"
2. Borings and CPT were located in the field with reference to site features and are shown approximate only.

LEGEND

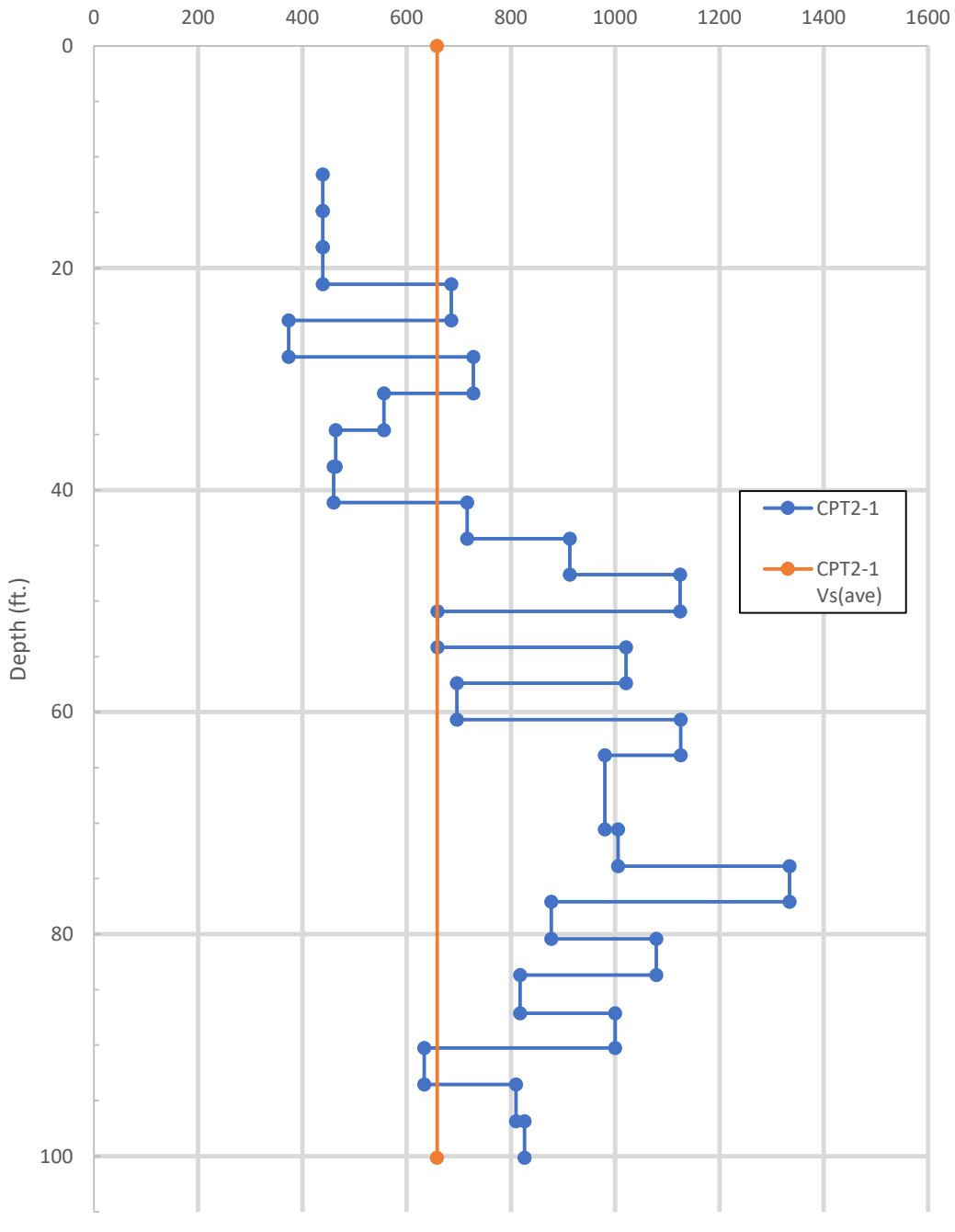
- Boring Location
- ⊙ CPT Sounding Location



| | | |
|---|----------------|-----------------|
| Drawn By: WAH | Ck'd By: JDM | App'vd By: ASE |
| Date: 5-5-23 | Date: 11-27-23 | Date: 11-27-23 |
| | | |
| ARDOT 101017 Tyronza River & Ditch No. 6 Strs. & Apprs. (S) Poinsett County, Arkansas | | |
| AERIAL PHOTOGRAPH OF BRIDGE 2 AND BORING AND CPT LOCATIONS | | |
| Project Number J042992.01 | | FIGURE 2 |

Shear Wave Velocity vs. Depth - CPT2-1

Shear Wave Velocity (ft./sec)





APPENDIX C – BORING INFORMATION

Boring Logs

Boring Log Terms and Symbols

Surface Elevation: 222

Completion Date: 7/13/23

Datum NAVD88

Lat: 433054.5664

Long: 1815220.4502

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 | 10 20 30 40 50 | LL

DEPTH
IN FEET

ELEVATION
IN FEET

DESCRIPTION OF MATERIAL

GRAPHIC LOG

DRY UNIT WEIGHT (pcf)
SPT BLOW COUNTS
CORE RECOVERY/RQD

SAMPLES

Topsoil: 12 inches

Stiff, brown, LEAN CLAY - CL
little organics

Medium stiff to stiff, brown to gray, FAT CLAY - (CH)
trace organics and trace sand

trace sand

3-4-5 SS1
ST2

2-3-4 SS3

3-4-5 SS4
97 ST5

3-3-6 SS6

3-2-3 SS7

3-3-4 SS8

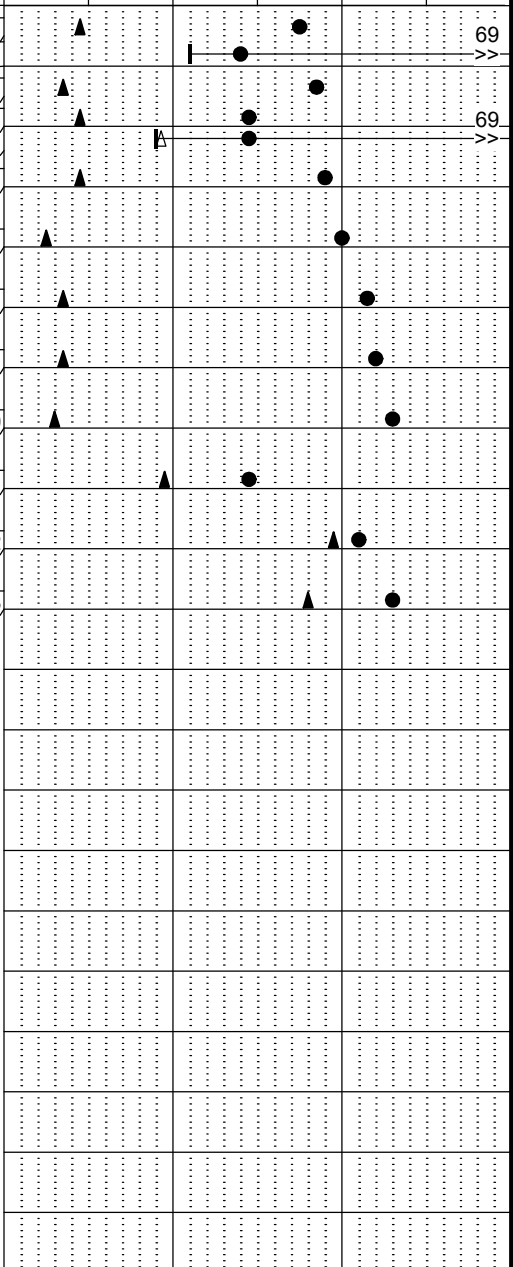
2-3-4 SS9

3-2-4 SS10

6-4-15 SS11

17-18-21 SS12

12-17-19 SS13



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES
LOG OF BORING 2020 JDM - ELEVATIONS .J042992.01.GPJ GTINC 0638301.GPJ 1/4/24 AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

AUGER 3 3/4" HOLLOW STEM
WASHBORING FROM 15 FEET
JCG DRILLER JWD LOGGER
Diedrich D-50 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 93 %

REMARKS:

Drawn by: LLP Checked by: JDM App'vd. by: ASE
Date: 7/14/23 Date: 11/27/23 Date: 11/27/23



**Tyrnza River
& Ditch No.6 Strs. & Apprs. (S)
Poinsett County, Arkansas**

LOG OF BORING: B2-1

Project No. J042992.01

Surface Elevation: 220

Completion Date: 7/19/23

Datum NAVD88

Lat: 433032.5904

Long: 1815276.3746

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 | 10 20 30 40 50 | LL

DEPTH
IN FEET

ELEVATION
IN FEET

DESCRIPTION OF MATERIAL

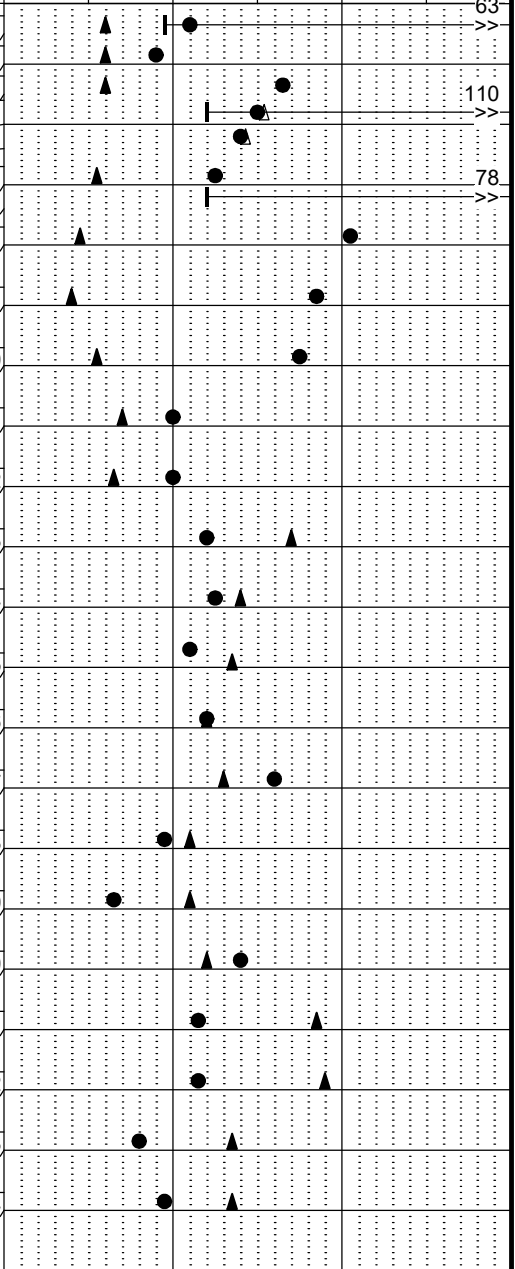
GRAPHIC LOG

DRY UNIT WEIGHT (pcf)
SPT BLOW COUNTS
CORE RECOVERY/RQD

SAMPLES

| | | | |
|-----|-----|---|--|
| | | Topsoil: 12 inches | |
| | | Stiff to medium stiff, brown to gray, FAT CLAY - (CH) | |
| | | trace organics | |
| 5 | 215 | | |
| 10 | 210 | | |
| 15 | 205 | | |
| 20 | 200 | | |
| 25 | 195 | | |
| 30 | 190 | 88.1% passing No. 200 sieve | |
| 35 | 185 | Medium dense, gray, SILTY SAND - SM | |
| 40 | 180 | 21.7% passing No. 200 sieve | |
| 45 | 175 | Medium dense to dense, gray SAND - SP | |
| 50 | 170 | | |
| 55 | 165 | | |
| 60 | 160 | | |
| 65 | 155 | | |
| 70 | 150 | | |
| 75 | 145 | trace gravel | |
| 80 | 140 | | |
| 85 | 135 | | |
| 90 | 130 | | |
| 95 | 125 | | |
| 100 | 120 | Boring terminated at 100 feet. | |

| | | | |
|----------|------|--|--|
| 4-5-7 | SS1 | | |
| 4-5-7 | SS2 | | |
| 3-5-7 | SS3 | | |
| 89 | ST4 | | |
| 89 | ST5 | | |
| 3-5-6 | SS6 | | |
| | ST7 | | |
| 3-3-6 | SS8 | | |
| 3-3-5 | SS9 | | |
| 2-2-9 | SS10 | | |
| 4-5-9 | SS11 | | |
| 8-8-5 | SS12 | | |
| 11-17-17 | SS13 | | |
| 10-12-16 | SS14 | | |
| 11-13-14 | SS15 | | |
| 10-11-13 | SS16 | | |
| 10-12-14 | SS17 | | |
| 11-12-10 | SS18 | | |
| 9-11-11 | SS19 | | |
| 14-12-12 | SS20 | | |
| 11-15-22 | SS21 | | |
| 12-19-19 | SS22 | | |
| 14-13-14 | SS23 | | |
| 13-13-14 | SS24 | | |



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES 1/4/24 AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 35 FEET ∇

DRILLING DATA

___ AUGER 3 3/4" HOLLOW STEM
WASHBORING FROM 35 FEET
JCG DRILLER JMP LOGGER
Diedrich D-50 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 93 %

REMARKS:

| | | |
|---------------|-----------------|-----------------|
| Drawn by: RSP | Checked by: JDM | App'vd. by: ASE |
| Date: 7/20/23 | Date: 11/27/23 | Date: 11/27/23 |



**Tyronza River
& Ditch No.6 Strs. & Apprs. (S)
Poinsett County, Arkansas**

LOG OF BORING: B2-2

Project No. J042992.01

Surface Elevation: 193

Completion Date: 7/18/23

Datum NAVD88

Lat: 433079.7532

Long: 1815415.2427

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 | 10 20 30 40 50 | LL

DEPTH
IN FEET

ELEVATION
IN FEET

DESCRIPTION OF MATERIAL

GRAPHIC LOG

DRY UNIT WEIGHT (pcf)
SPT BLOW COUNTS
CORE RECOVERY/RQD

SAMPLES

Medium stiff, gray, silty, LEAN CLAY - CL

4-3-3 SS1

Medium dense, brown to gray SAND with silt - SP-SM
11.9% passing No. 200 sieve
little clay

10-11-11 SS2

8-10-12 SS3

12-15-17 SS4

Medium dense to dense, brown to gray SAND - SP
3.2% passing No. 200 sieve

8-9-11 SS5

8-11-12 SS6

14-15-15 SS7

10-12-20 SS8

10-10-10 SS9

1.7% passing No. 200 sieve

6-6-7 SS10

14-17-17 SS11

3.1% passing No. 200 sieve

9-9-17 SS12

14-14-15 SS13

14-21-16 SS14

little gravel

17-18-13 SS15

Boring terminated at 70 feet due to loss of drilling fluid into formation and collapse of boring.

NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES
LOG OF BORING 2020 JDM - ELEVATIONS .J042992.01.GPJ GTINC 0638301.GPJ 1/4/24 AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

AUGER 3 3/4" HOLLOW STEM WASHBORING FROM 0 FEET
JCG DRILLER JMP LOGGER
Diedrich D-50 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 93 %

REMARKS: Mudline 29 feet below top of bridge deck.
Boring terminated at 70 feet due to loss of drilling fluid into formation and collapse of boring.

Drawn by: RSP Checked by: JDM App'vd. by: ASE
Date: 7/25/23 Date: 11/27/23 Date: 11/27/23



Tyronza River & Ditch No.6 Strs. & Apprs. (S)
Poinsett County, Arkansas

LOG OF BORING: B2-3

Project No. J042992.01

Surface Elevation: 204

Completion Date: 7/13/23

Datum NAVD88

Lat: 433076.1918

Long: 1815530.2921

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 | 10 20 30 40 50 | LL

NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES 1/4/24 AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

| DEPTH IN FEET | ELEVATION IN FEET | DESCRIPTION OF MATERIAL | GRAPHIC LOG | DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD | SAMPLES | SHEAR STRENGTH, tsf | STANDARD PENETRATION RESISTANCE | WATER CONTENT, % |
|---------------|-------------------|---|-------------|---|---------|---------------------|---------------------------------|------------------|
| | | Loose, gray, CLAYEY SAND - SC | | | | | | |
| 5 | 199 | 49.9% passing No. 200 sieve | | 1-3-6 | SS1 | | | |
| | | Soft, brown to gray, FAT CLAY - CH | | 0-0-2 | SS2 | | | |
| 10 | 194 | | | 0-0-2 | SS3 | | | |
| | | 99.7% passing No. 200 sieve | | 0-2-2 | SS4 | | | |
| 15 | 189 | | | | | | | |
| 20 | 184 | Medium dense, gray, CLAYEY SAND - SC 49.9% passing No. 200 sieve | | 1-5-9 | SS5 | | | 63 |
| 25 | 179 | Medium dense to very dense, gray SAND - SP | | 15-18-16 | SS6 | | | |
| 30 | 174 | | | 6-9-12 | SS7 | | | |
| 35 | 169 | trace gravel | | 11-15-21 | SS8 | | | |
| 40 | 164 | | | 10-13-16 | SS9 | | | |
| 45 | 159 | trace lignite | | 7-10-12 | SS10 | | | |
| 50 | 154 | | | 12-21-33 | SS11 | | | |
| 55 | 149 | | | 12-19-27 | SS12 | | | |
| 60 | 144 | | | 14-20-15 | SS13 | | | |
| 65 | 139 | | | | | | | |
| 70 | 134 | | | 10-13-16 | SS14 | | | |
| 75 | 129 | | | | | | | |
| 80 | 124 | | | 9-17-25 | SS15 | | | |
| 85 | 119 | | | | | | | |
| 90 | 114 | | | 16-25-25 | SS16 | | | |
| 95 | 109 | | | | | | | |
| 100 | 104 | Boring terminated at 100 feet. | | 13-28 -50/3 | SS17 | | | 78 32819" |

GROUNDWATER DATA

FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

AUGER 3 3/4" HOLLOW STEM WASHBORING FROM 0 FEET
JCG DRILLER JMP LOGGER
CME 550X DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 88 %

REMARKS: Mudline 16 feet below top of bridge deck.
Casing seated approximately 3.5 feet into river bed.

| | | |
|----------------------|------------------------|------------------------|
| Drawn by: <u>JMP</u> | Checked by: <u>JDM</u> | App'vd. by: <u>ASE</u> |
| Date: <u>7/13/23</u> | Date: <u>11/27/23</u> | Date: <u>11/27/23</u> |



Tyrnza River & Ditch No.6 Strs. & Apprs. (S)
Poinsett County, Arkansas

LOG OF BORING: B2-4

Project No. J042992.01

Surface Elevation: 222

Completion Date: 6/28/23

Datum NAVD88

Lat: 433030.4097

Long: 1815644.4922

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 | 10 20 30 40 50 | LL

DEPTH
IN FEET

ELEVATION
IN FEET

DESCRIPTION OF MATERIAL

GRAPHIC LOG

DRY UNIT WEIGHT (pcf)
SPT BLOW COUNTS
CORE RECOVERY/RQD

SAMPLES

| | | | | | | | | | |
|-----|-----|--|--|---------------|----|-----|---|--|----|
| | | Topsoil: 6 inches | | | | | | | |
| 5 | 217 | Medium stiff to very stiff, brown to gray, FAT CLAY - (CH) trace organics | | 2-3-3 SS1 | 91 | ST2 | ▲ | | 97 |
| 10 | 212 | | | 2-3-4 SS3 | | | ▲ | | |
| 15 | 207 | Medium stiff, gray, LEAN CLAY - (CL) | | 2-2-4 SS4 | 92 | ST5 | ▲ | | |
| 20 | 202 | Medium stiff, gray, FAT CLAY - CH | | 3-3-4 SS6 | | ST7 | ▲ | | |
| 25 | 197 | Loose, gray SAND with clay - SP-SC 9.1% passing No. 200 sieve | | 2-2-3 SS8 | | | ▲ | | |
| 30 | 192 | trace sand | | 6-4-3 SS9 | | | ▲ | | |
| 35 | 187 | Hard, gray, sandy, FAT CLAY - CH 72.7% passing No. 200 sieve | | 1-2-4 SS10 | | | ▲ | | |
| 40 | 182 | Medium dense to very dense, gray, SAND - SP | | 4-12-20 SS11 | | | ▲ | | |
| 45 | 177 | | | 16-19-24 SS12 | | | | | |
| 50 | 172 | | | 12-16-17 SS13 | | | | | |
| 55 | 167 | trace gravel | | 15-20-19 SS14 | | | | | |
| 60 | 162 | | | 12-13-17 SS15 | | | | | |
| 65 | 157 | | | 12-12-16 SS16 | | | | | |
| 70 | 152 | | | 15-22-29 SS17 | | | | | |
| 75 | 147 | | | | | | | | |
| 80 | 142 | | | 10-14-15 SS18 | | | | | |
| 85 | 137 | | | | | | | | |
| 90 | 132 | | | 14-12-13 SS19 | | | | | |
| 95 | 127 | | | | | | | | |
| 100 | 122 | Boring terminated at 100 feet. | | 12-14-13 SS20 | | | | | |

NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES
LOG OF BORING 2020 JDM - ELEVATIONS J042992.01.GPJ GTINC 0638301.GPJ 1/4/24 AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 33.5 FEET ▽

DRILLING DATA

___ AUGER 3 3/4" HOLLOW STEM
WASHBORING FROM 33.5 FEET
JCG DRILLER JMP LOGGER
Diedrich D-50 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 93 %

REMARKS:

Drawn by: RSP Checked by: JDM App'vd. by: ASE
Date: 7/20/23 Date: 11/27/23 Date: 11/27/23



**Tyronza River
& Ditch No.6 Strs. & Apprs. (S)
Poinsett County, Arkansas**

LOG OF BORING: B2-5

Project No. J042992.01

LOG OF BORING 2020 JDM - ELEVATIONS .J042992.01.GPJ GTINC 0638301.GPJ 1/4/24 AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

| Surface Elevation: <u>222</u> | | Completion Date: <u>6/30/23</u> | | GRAPHIC LOG | DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD | SAMPLES | SHEAR STRENGTH, tsf | | | | |
|-------------------------------|----------------------|--|------------------|--------------------------|---|---------|---------------------------------|-----|----|----|--|
| Datum <u>NAVD88</u> | | Lat: <u>433007.7617</u> | | | | | STANDARD PENETRATION RESISTANCE | | | | |
| | | Long: <u>1815707.3724</u> | | | | | (ASTM D 1586) | | | | |
| DEPTH IN FEET | ELEVATION IN FEET | DESCRIPTION OF MATERIAL | WATER CONTENT, % | | | | | | | | |
| | | | PL | N-VALUE (BLOWS PER FOOT) | | | LL | | | | |
| | | | | 0.5 | 1.0 | 1.5 | 2.0 | 2.5 | | | |
| | | Topsoil: 6 inches | | | | | | | | | |
| 5 | 217 | Medium stiff, gray, FAT CLAY - (CH) trace gravel | / | 3-2-3 SS1 | 84 ST2 | ▲ | ● | ■ | 95 | >> | |
| 10 | 212 | Medium stiff, gray, LEAN CLAY - (CL) | / | 2-3-3 SS3 | 97 ST4 | ▲ | ● | ■ | | | |
| 15 | 207 | trace sand | / | 2-2-3 SS5 | | ▲ | ● | ■ | | | |
| 20 | 202 | | / | 1-2-3 SS6 | | ▲ | ● | ■ | | | |
| 25 | 197 | Loose, gray, CLAYEY SAND - SC 34.7% passing No. 200 sieve | / | 2-2-4 SS7 | | ▲ | ● | ■ | | | |
| 30 | 192 | Medium stiff, gray, LEAN CLAY - CL | / | 2-3-3 SS8 | | ▲ | ● | ■ | | | |
| 35 | 187 | 86.6% passing No. 200 sieve | / | 2-1-4 SS9 | | ▲ | ● | ■ | | | |
| 40 | 182 | Dense, gray SAND - SP | . | 4-21-24 SS10 | | | ● | ■ | ▲ | | |
| 45 | 177 | | . | 11-16-24 SS11 | | | | ■ | ▲ | | |
| 50 | 172 | Boring terminated at 50 feet. | . | 13-14-17 SS12 | | | | ▲ | | | |
| 55 | 167 | | | | | | | | | | |
| 60 | 162 | | | | | | | | | | |
| 65 | 157 | | | | | | | | | | |
| 70 | 152 | | | | | | | | | | |
| 75 | 147 | | | | | | | | | | |
| 80 | 142 | | | | | | | | | | |
| 85 | 137 | | | | | | | | | | |
| 90 | 132 | | | | | | | | | | |
| 95 | 127 | | | | | | | | | | |
| 100 | 122 | | | | | | | | | | |

GROUNDWATER DATA

FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

AUGER 3 3/4" HOLLOW STEM WASHBORING FROM 10 FEET
JCG DRILLER JMP LOGGER
Diedrich D-50 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 93 %

REMARKS:

Drawn by: LLP Checked by: JDM App'vd. by: ASE
Date: 7/6/23 Date: 11/27/23 Date: 11/27/23



**Tyronza River
& Ditch No.6 Strs. & Apprs. (S)
Poinsett County, Arkansas**

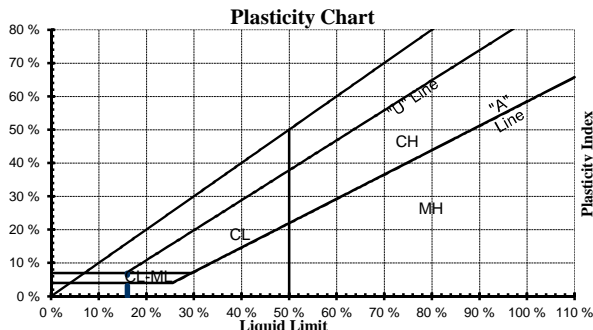
LOG OF BORING: B2-6

Project No. J042992.01

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

STRENGTH OF COHESIVE SOILS

DENSITY OF GRANULAR SOILS

| Consistency | Undrained Shear Strength (tsf) | Unconfined Comp. Strength (tsf) | Descriptive Term | Approximate N_{60} -Value Range |
|--------------|--------------------------------|---------------------------------|------------------|-----------------------------------|
| Very Soft | less than 0.125 | less than 0.25 | Very Loose | 0 to 4 |
| Soft | 0.125 to 0.25 | 0.25 to 0.5 | Loose | 5 to 10 |
| Medium Stiff | 0.25 to 0.5 | 0.5 to 1.0 | Medium Dense | 11 to 30 |
| Stiff | 0.5 to 1.0 | 1.0 to 2.0 | Dense | 31 to 50 |
| Very Stiff | 1.0 to 2.0 | 2.0 to 3.0 | Very Dense | >50 |
| Hard | greater than 2.0 | greater than 4.0 | | |

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

OTHER TERMS

| | | |
|--------|-----------|---|
| Trace | 0 to 10% | Layer - Inclusion greater than 3 inches thick. |
| Little | 10 to 20% | Seam - Inclusion 1/8-inch to 3 inches thick |
| Some | 20 to 35% | Parting - Inclusion less than 1/8-inch thick |
| And | 35 to 50% | Pocket - Inclusion of material that is smaller than sample diameter |



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 – CPT2-1 SOUNDING PLOT



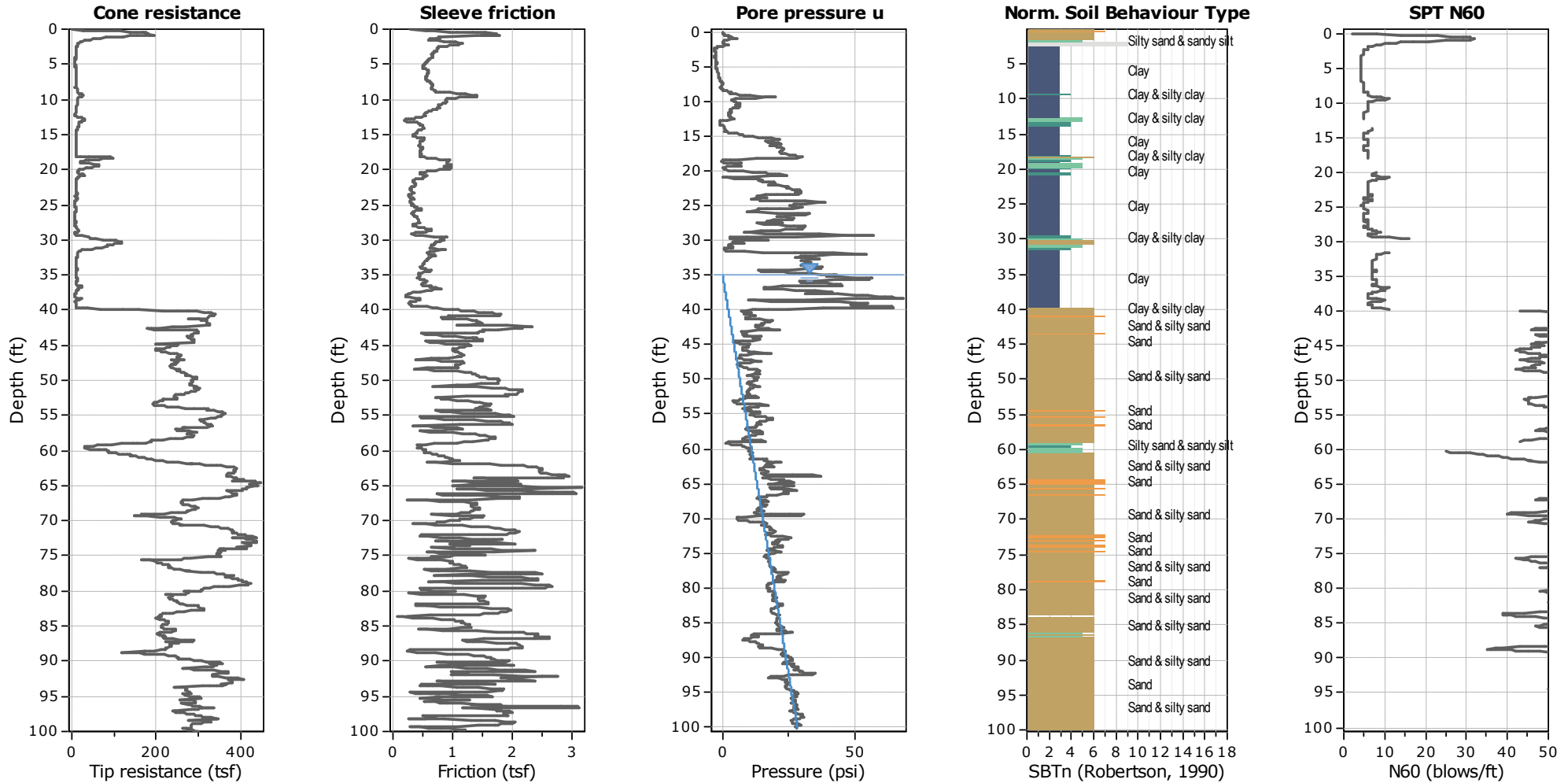


Geotechnology, LLC
 3312 Winbrook Dr
 Memphis, TN 38116
 (901) 353-1981

CPT: CPT2-1

Total depth: 100.20 ft, Date: 6/13/2023
 Coords: lat 35.511141° lon -90.308923°
 Cone Type: 15 Square-Centimeter
 Cone Operator: DWJ

Project: ARDOT G003 101017 Tyronza River & Ditch No. 6 Strs. & Apprs. (S)
Location: Poinsett County, Arkansas



SBTn legend

- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravelly sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |



APPENDIX E – LABORATORY TEST DATA

Atterberg Limits

Grain Size Distributions

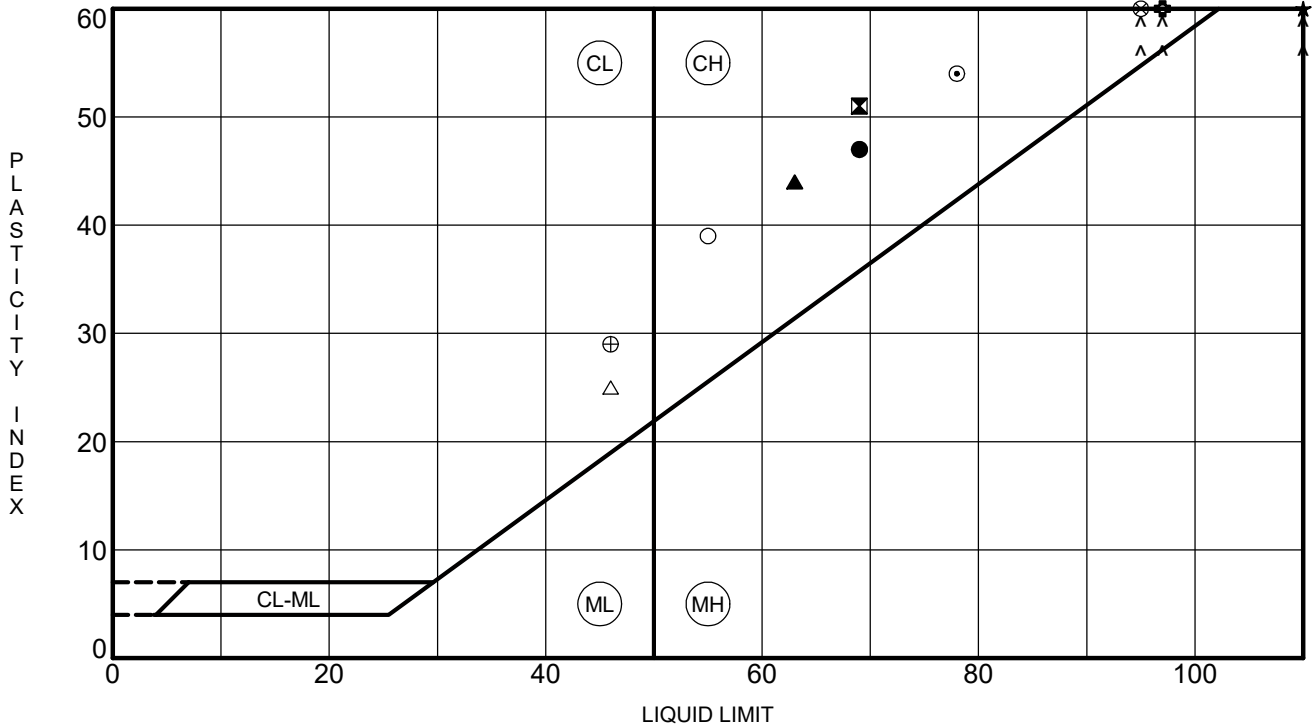
Unconsolidated-Undrained Triaxial Compression

One-Dimensional Consolidation

Direct Shear

Resistivity

pH

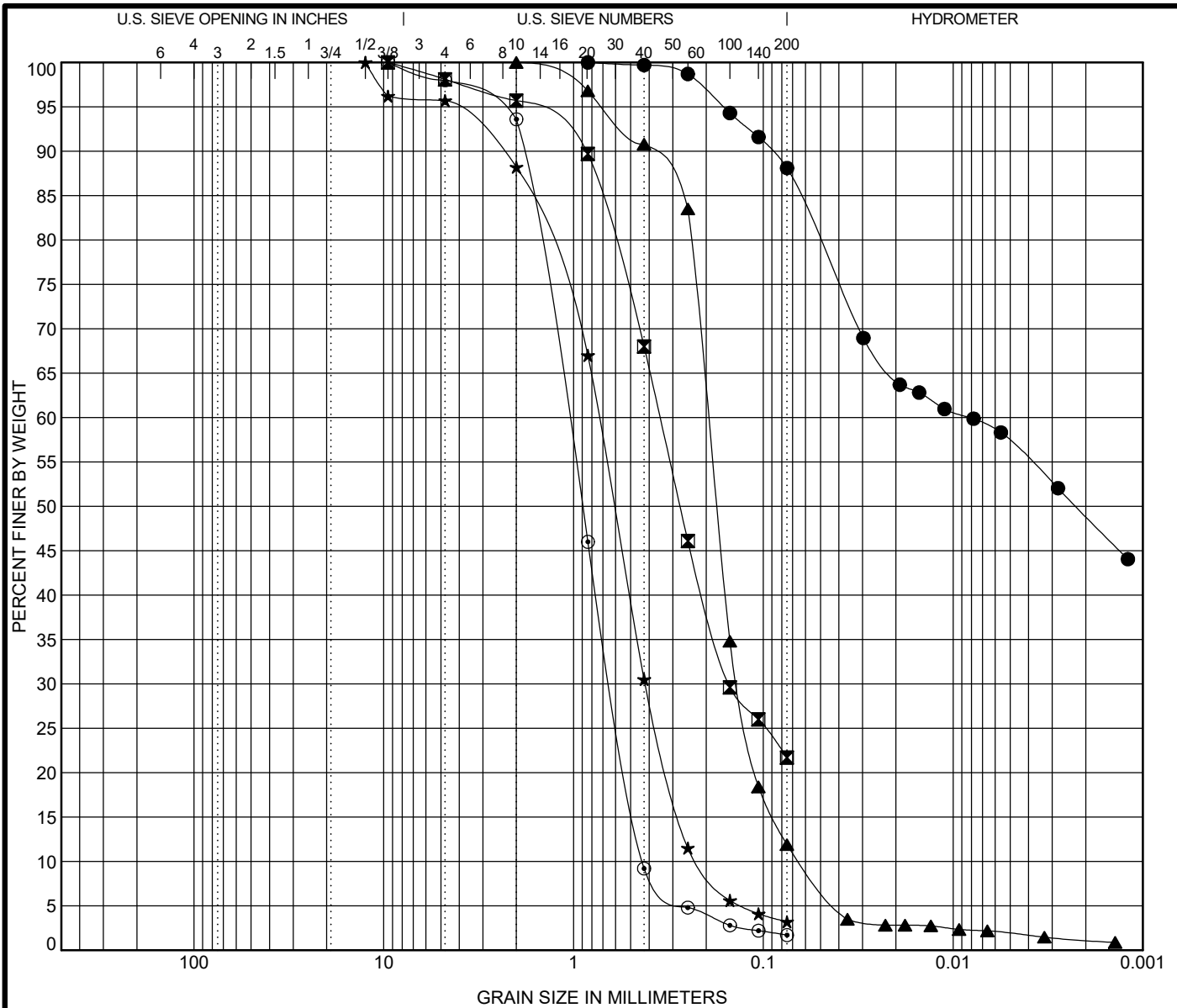


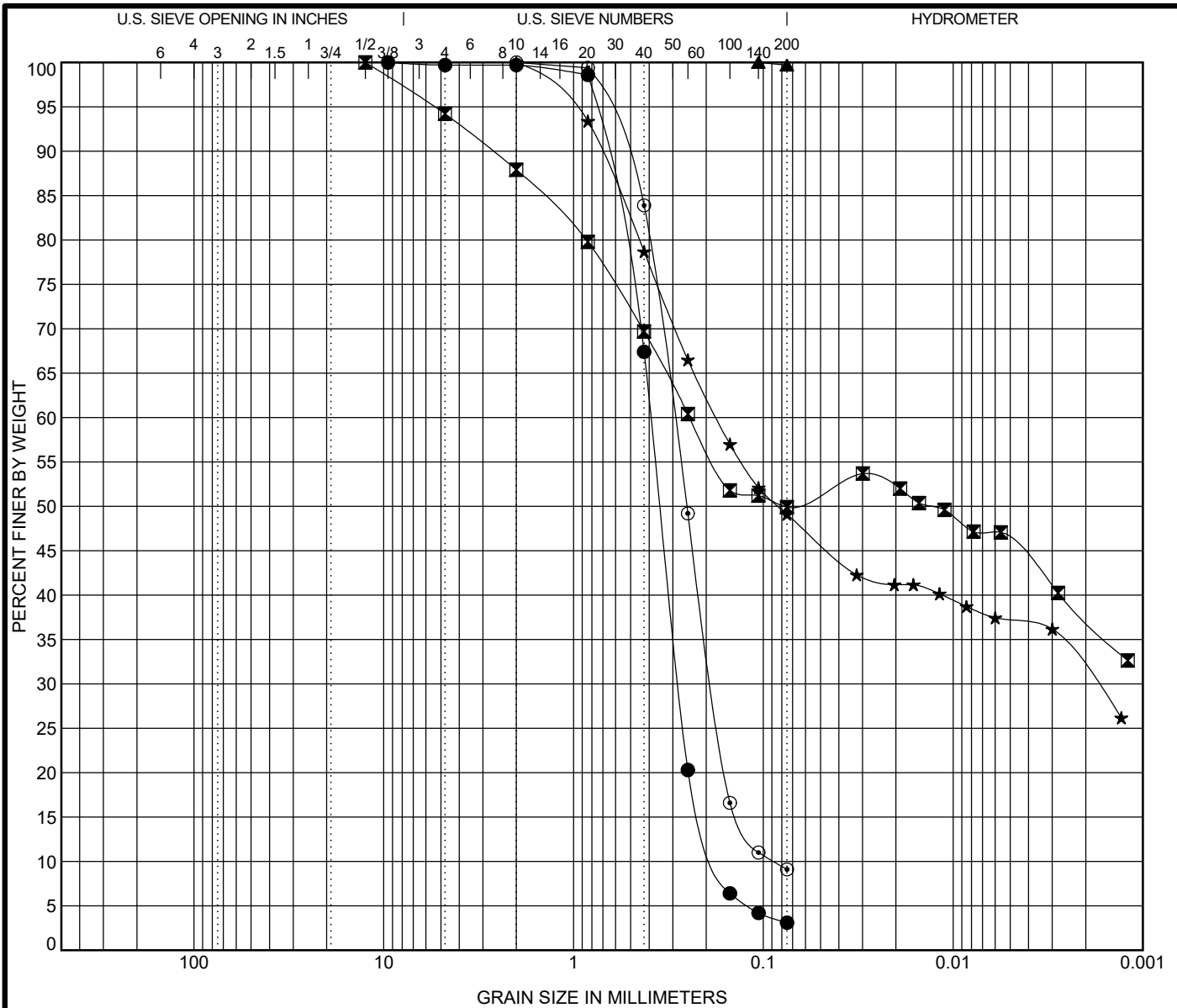
| Specimen Identification | LL | PL | PI | Fines | Classification |
|-------------------------|------|-----|----|-------|----------------|
| ● B2-1 | 3.0 | 69 | 22 | 47 | FAT CLAY(CH) |
| ⊠ B2-1 | 10.0 | 69 | 18 | 51 | FAT CLAY(CH) |
| ▲ B2-2 | 1.0 | 63 | 19 | 44 | FAT CLAY(CH) |
| ★ B2-2 | 8.0 | 110 | 24 | 86 | FAT CLAY(CH) |
| ⊙ B2-2 | 16.0 | 78 | 24 | 54 | FAT CLAY(CH) |
| ⊕ B2-5 | 3.0 | 97 | 22 | 75 | FAT CLAY(CH) |
| ○ B2-5 | 10.0 | 55 | 16 | 39 | FAT CLAY(CH) |
| △ B2-5 | 15.0 | 46 | 21 | 25 | LEAN CLAY(CL) |
| ⊗ B2-6 | 3.0 | 95 | 26 | 69 | FAT CLAY(CH) |
| ⊕ B2-6 | 8.0 | 46 | 17 | 29 | LEAN CLAY(CL) |
| | | | | | |
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US ATTERBERG LIMITS J042992.01.GPJ US LAB.GDT 11/27/23



ATTERBERG LIMITS RESULTS
 Tyronza River
 & Ditch No.6 Strs. & Apprs. (S)
 Poinsett County, Arkansas
 J042992.01





| COBBLES | GRAVEL | | SAND | | | SILT OR CLAY |
|---------|--------|------|--------|--------|------|--------------|
| | coarse | fine | coarse | medium | fine | |

| Specimen Identification | Classification | LL | PL | PI | Cc | Cu | | |
|-------------------------|-------------------------------------|-------|-------|-------|---------|-------|-------|-------|
| ● B2-3 48.5 | POORLY GRADED SAND(SP) | | | | 1.16 | 2.28 | | |
| ☒ B2-4 3.5 | CLAYEY SAND(SC) | | | | | | | |
| ▲ B2-4 13.5 | LEAN CLAY(CL) | | | | | | | |
| ★ B2-4 18.5 | CLAYEY SAND(SC) | | | | | | | |
| ⊙ B2-5 23.5 | POORLY GRADED SAND with CLAY(SP-SC) | | | | 1.31 | 3.34 | | |
| Specimen Identification | D100 | D60 | D30 | D10 | %Gravel | %Sand | %Silt | %Clay |
| ● B2-3 48.5 | 9.5 | 0.391 | 0.279 | 0.171 | 0.3 | 96.6 | 3.1 | |
| ☒ B2-4 3.5 | 12.5 | 0.22 | | | 5.8 | 44.3 | 3.9 | 46.0 |
| ▲ B2-4 13.5 | 0.106 | | | | 0.0 | 0.3 | | 99.7 |
| ★ B2-4 18.5 | 2 | 0.176 | 0.002 | | 0.0 | 50.9 | 12.0 | 37.1 |
| ⊙ B2-5 23.5 | 2 | 0.295 | 0.185 | 0.088 | 0.0 | 90.9 | | 9.1 |

US GRAIN SIZE J042992.01.GPJ US LAB.GDT 11/27/23

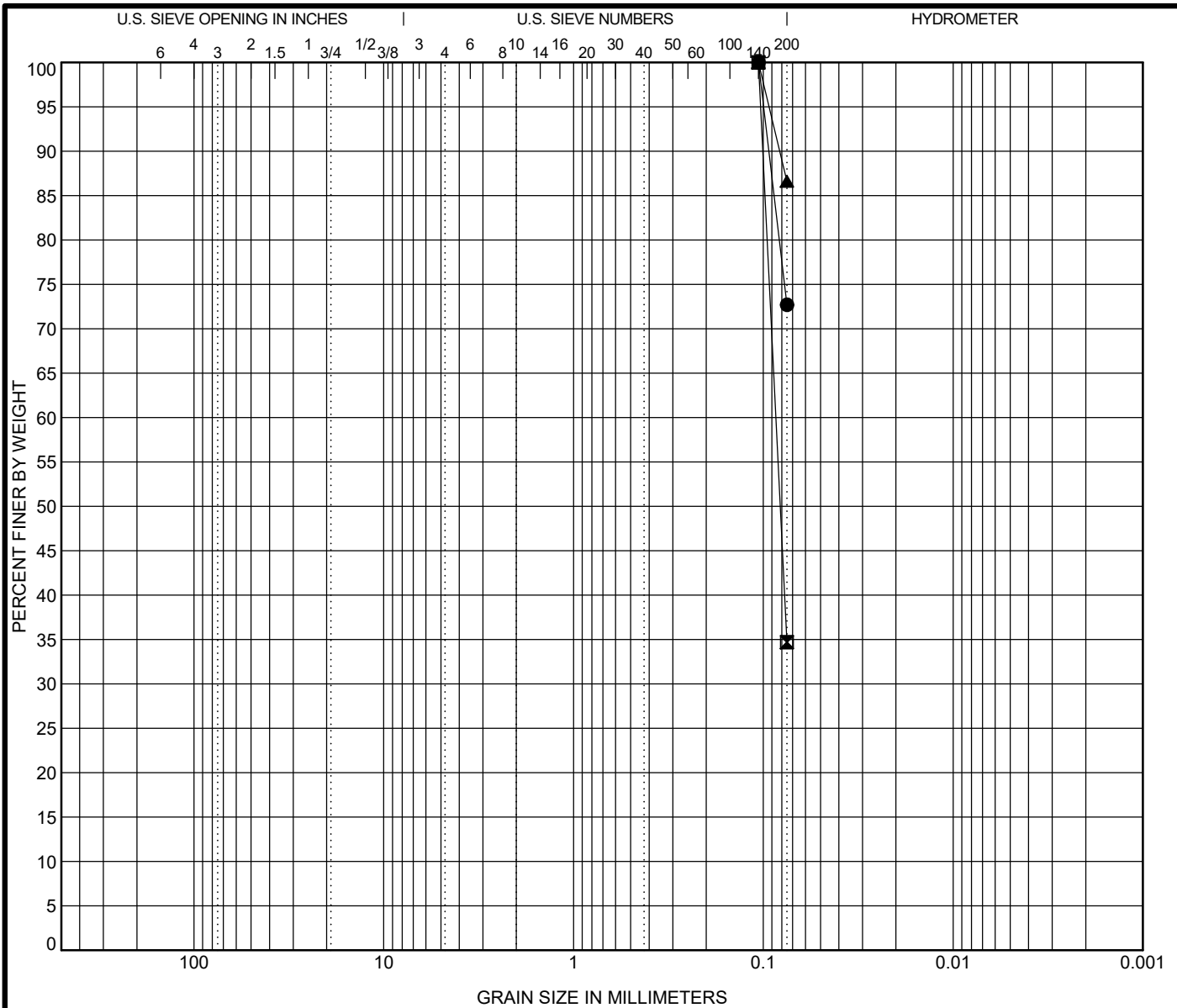


GEOTECHNOLOGY

A UES Company

GRAIN SIZE DISTRIBUTION

Tyrnza River
 & Ditch No.6 Strs. & Apprs. (S)
 Poinsett County, Arkansas
 J042992.01



| COBBLES | GRAVEL | | SAND | | | SILT OR CLAY |
|---------|--------|------|--------|--------|------|--------------|
| | coarse | fine | coarse | medium | fine | |

| Specimen Identification | Classification | LL | PL | PI | Cc | Cu |
|-------------------------|---------------------------|----|----|----|----|----|
| ● B2-5 33.5 | SANDY FAT CLAY(CH) | | | | | |
| ☒ B2-6 23.5 | CLAYEY SAND(SC) | | | | | |
| ▲ B2-6 33.5 | LEAN CLAY(CL) | | | | | |

| Specimen Identification | D100 | D60 | D30 | D10 | %Gravel | %Sand | %Silt | %Clay |
|-------------------------|-------|-------|-----|-----|---------|-------|-------|-------|
| ● B2-5 33.5 | 0.106 | | | | 0.0 | 27.3 | 72.7 | |
| ☒ B2-6 23.5 | 0.106 | 0.086 | | | 0.0 | 65.3 | 34.7 | |
| ▲ B2-6 33.5 | 0.106 | | | | 0.0 | 13.4 | 86.6 | |

US GRAIN SIZE J042992.01.GPJ US LAB.GDT 11/27/23



GRAIN SIZE DISTRIBUTION
 Tyrnza River
 & Ditch No.6 Strs. & Apprs. (S)
 Poinsett County, Arkansas
 J042992.01

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS ASTM D2850

CLIENT : Arkansas State Highway and Transportation Department
 PROJECT NO.: J042992.01
 PROJECT: ARDOT 2023-2027 Contract TO G003 Tyrnza River & Ditch No. 6 Strs & App. Poinsett County
 LOCATION: Poinsett County, Arkansas

DATE: 8/6/2023

BORING NO.: B2-1
 SAMPLE OBTAINED BY: Shelby Tube
 SAMPLE DESCRIPTION: Brown Fat Clay

SAMPLE NO.: ST-5
 CONDITION: Undisturbed

DEPTH (ft.): 10.0-12.0

LIQUID LIMIT (%): 69 PLASTIC LIMIT (%): 18 PLASTICITY INDEX (%): 51 USCS: CH

SPECIFIC GRAVITY OF SOLIDS: 2.70 (Assumed)

LOAD CELL NO.:

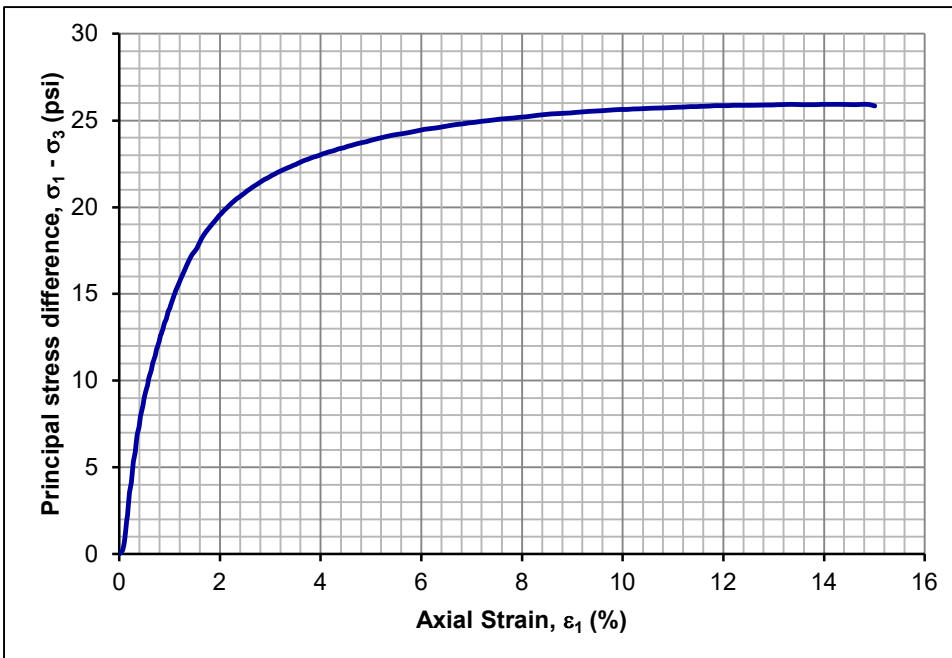
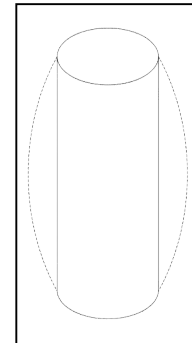
INITIAL SAMPLE DATA

FAILURE DATA***

| | |
|---------------------------|-------|
| AVERAGE DIAMETER (in.): | 2.84 |
| HEIGHT (in.): | 5.88 |
| HEIGHT TO DIAMETER RATIO: | 2.07 |
| WET UNIT WEIGHT (pcf): | 124.9 |
| DRY UNIT WEIGHT (pcf): | 96.8 |
| VOID RATIO: | 0.74 |
| MOISTURE CONTENT (%)*: | 29.0 |
| DEGREE OF SATURATION (%): | 100.0 |

| | |
|--|-------|
| MOISTURE CONTENT AFTER FAILURE (%)**: | 28.8 |
| 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): | 25.9 |
| MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi): | 6.4 |
| MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi): | 32.3 |
| UNDRAINED COMPRESSIVE STRENGTH, U_u (psf): | 3,730 |
| UNDRAINED SHEAR STRENGTH, s_u (psf): | 1,865 |
| LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf): | 3,690 |

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 : Arkansas State Highway and Transportation Department
 PROJECT NO.: J042992.01
 PROJECT: ARDOT 2023-2027 Contract TO G003 Tyrnza River & Ditch No. 6 Strs & App. Poinsett County
 LOCATION: Poinsett County, Arkansas

DATE: 8/4/2023

BORING NO.: B2-2
 SAMPLE OBTAINED BY: Shelby Tube
 SAMPLE DESCRIPTION: Gray Fat Clay

SAMPLE NO.: ST-4
 CONDITION: Undisturbed

DEPTH (ft.): 8.0-10.0

LIQUID LIMIT (%): 110 PLASTIC LIMIT (%): 24 PLASTICITY INDEX (%): 86 USCS: CH

SPECIFIC GRAVITY OF SOLIDS: 2.70 (Assumed)

LOAD CELL NO.:

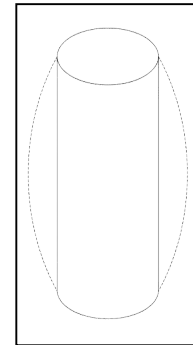
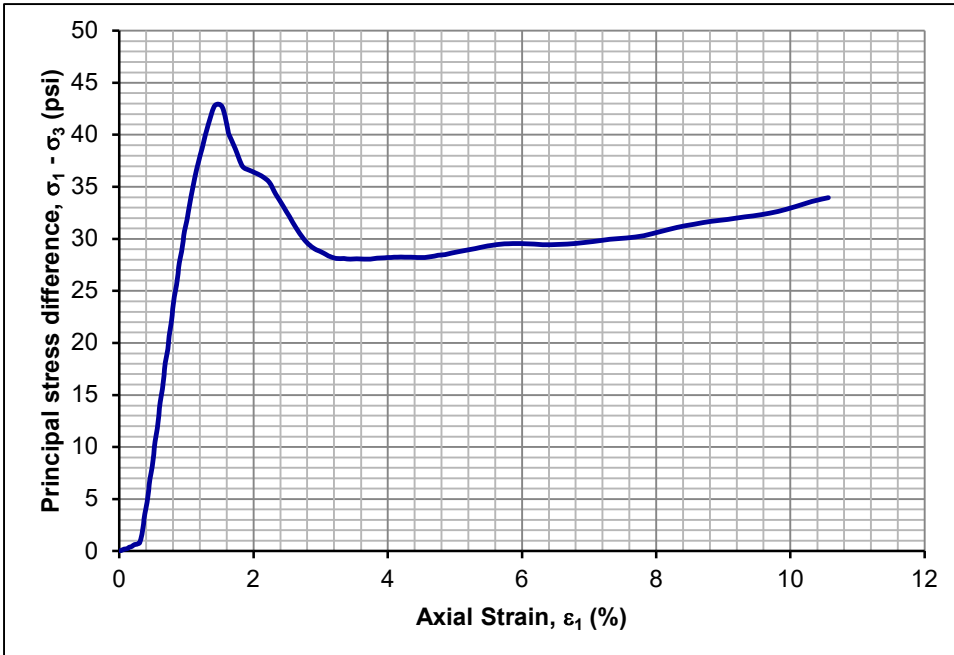
INITIAL SAMPLE DATA

FAILURE DATA***

| | |
|---------------------------|-------|
| AVERAGE DIAMETER (in.): | 2.84 |
| HEIGHT (in.): | 5.95 |
| HEIGHT TO DIAMETER RATIO: | 2.09 |
| WET UNIT WEIGHT (pcf): | 116.0 |
| DRY UNIT WEIGHT (pcf): | 89.1 |
| VOID RATIO: | 0.89 |
| MOISTURE CONTENT (%)*: | 30.2 |
| DEGREE OF SATURATION (%): | 91.5 |

| | |
|--|-------|
| MOISTURE CONTENT AFTER FAILURE (%)**: | 30.3 |
| AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.): | 1.0 |
| AXIAL STRAIN AT FAILURE (%): | 1.4 |
| PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi): | 42.8 |
| MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi): | 5.3 |
| MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi): | 48.1 |
| UNDRAINED COMPRESSIVE STRENGTH, U_u (psf): | 6,170 |
| UNDRAINED SHEAR STRENGTH, s_u (psf): | 3,085 |
| LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf): | N/A |

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 : Arkansas State Highway and Transportation Department
 PROJECT NO.: J042992.01
 PROJECT: ARDOT 2023-2027 Contract TO G003 Tyrnza River & Ditch No. 6 Strs & App. Poinsett County
 LOCATION: Tyrnza, AR

DATE: 8/4/2023

BORING NO.: B2-2
 SAMPLE OBTAINED BY: Shelby Tube
 SAMPLE DESCRIPTION: Gray Fat Clay

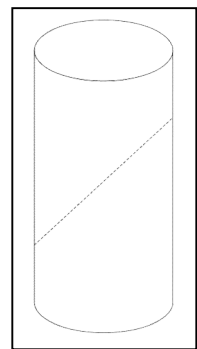
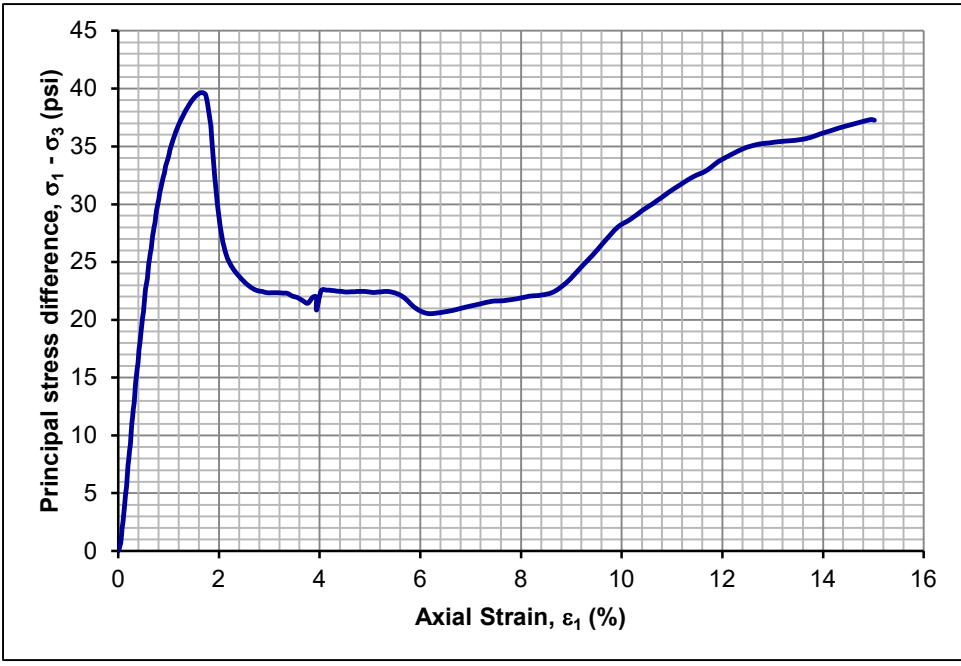
SAMPLE NO.: ST-5
 CONDITION: Undisturbed

DEPTH (ft.): 10.0-12.0

LIQUID LIMIT (%): PLASTIC LIMIT (%): PLASTICITY INDEX (%): USCS: CH

SPECIFIC GRAVITY OF SOLIDS: 2.70 (Assumed) LOAD CELL NO.:

| INITIAL SAMPLE DATA | | FAILURE DATA*** | |
|---------------------------|-------|--|--------------|
| AVERAGE DIAMETER (in.): | 2.84 | MOISTURE CONTENT AFTER FAILURE (%)**: | 28.3 |
| HEIGHT (in.): | 6.08 | AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.): | 0.9 |
| HEIGHT TO DIAMETER RATIO: | 2.14 | AXIAL STRAIN AT FAILURE (%): | 1.6 |
| WET UNIT WEIGHT (pcf): | 115.0 | PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi): | 39.7 |
| DRY UNIT WEIGHT (pcf): | 89.3 | MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi): | 6.4 |
| VOID RATIO: | 0.89 | MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi): | 46.1 |
| MOISTURE CONTENT (%)*: | 28.7 | UNDRAINED COMPRESSIVE STRENGTH, U_u (psf): | 5,710 |
| DEGREE OF SATURATION (%): | 87.4 | UNDRAINED SHEAR STRENGTH, s_u (psf): | 2,855 |
| | | LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf): | N/A |



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 : Arkansas State Highway and Transportation Department
 PROJECT NO.: J042992.01
 PROJECT: ARDOT 2023-2027 Contract TO G003 Tyrnza River & Ditch No. 6 Strs & App. Poinsett County
 LOCATION: Poinsett County, Arkansas

DATE: 8/4/2023

BORING NO.: B2-5
 SAMPLE OBTAINED BY: Shelby Tube
 SAMPLE DESCRIPTION: Gray Fat Clay

SAMPLE NO.: ST-2
 CONDITION: Undisturbed

DEPTH (ft.): 3.0-5.0

LIQUID LIMIT (%): 97 PLASTIC LIMIT (%): 22 PLASTICITY INDEX (%): 75 USCS: CH

SPECIFIC GRAVITY OF SOLIDS: 2.70 (Assumed)

LOAD CELL NO.:

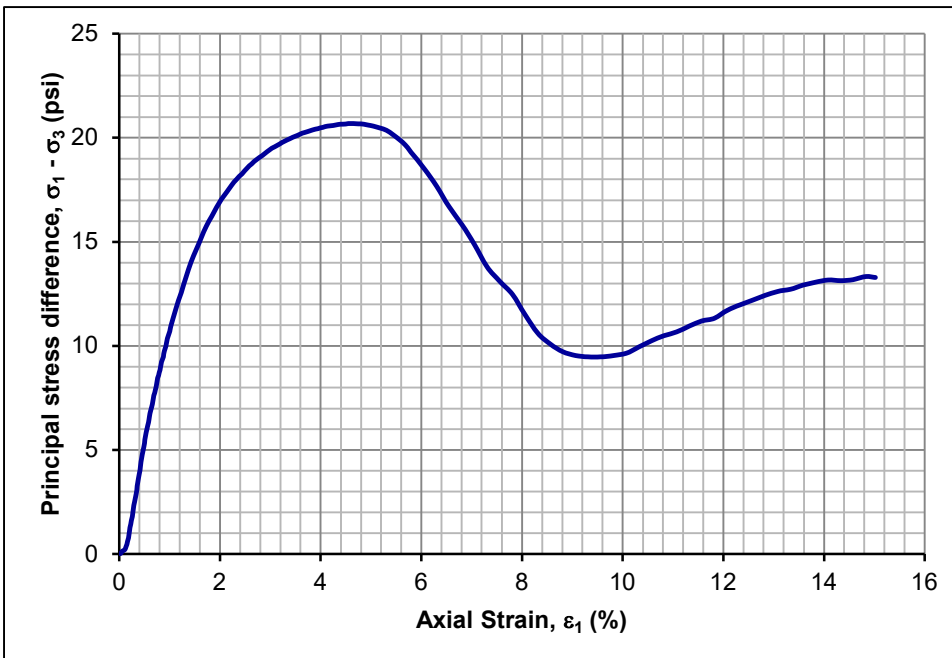
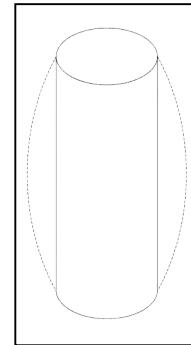
INITIAL SAMPLE DATA

FAILURE DATA***

| | |
|---------------------------|-------|
| AVERAGE DIAMETER (in.): | 2.87 |
| HEIGHT (in.): | 5.58 |
| HEIGHT TO DIAMETER RATIO: | 1.94 |
| WET UNIT WEIGHT (pcf): | 115.0 |
| DRY UNIT WEIGHT (pcf): | 90.6 |
| VOID RATIO: | 0.86 |
| MOISTURE CONTENT (%)*: | 27.0 |
| DEGREE OF SATURATION (%): | 84.7 |

| | |
|--|--------------|
| MOISTURE CONTENT AFTER FAILURE (%)**: | 36.0 |
| AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.): | 1.0 |
| AXIAL STRAIN AT FAILURE (%): | 4.6 |
| PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi): | 20.7 |
| MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi): | 2.3 |
| MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi): | 23.0 |
| UNDRAINED COMPRESSIVE STRENGTH, U_u (psf): | 2,980 |
| UNDRAINED SHEAR STRENGTH, s_u (psf): | 1,490 |
| LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf): | N/A |

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 : Arkansas State Highway and Transportation Department
 PROJECT NO.: J042992.01
 PROJECT: ARDOT 2023-2027 Contract TO G003 Tyrnza River & Ditch No. 6 Strs & App. Poinsett County
 LOCATION: Poinsett County, Arkansas

DATE: 8/15/2023

BORING NO.: B2-5
 SAMPLE OBTAINED BY: Shelby Tube
 SAMPLE DESCRIPTION: Brown Fat Clay

SAMPLE NO.: ST-5
 CONDITION: Undisturbed

DEPTH (ft.): 10.0-12.0

LIQUID LIMIT (%): 55 PLASTIC LIMIT (%): 16 PLASTICITY INDEX (%): 39 USCS: CH

SPECIFIC GRAVITY OF SOLIDS: 2.70 (Assumed)

LOAD CELL NO.:

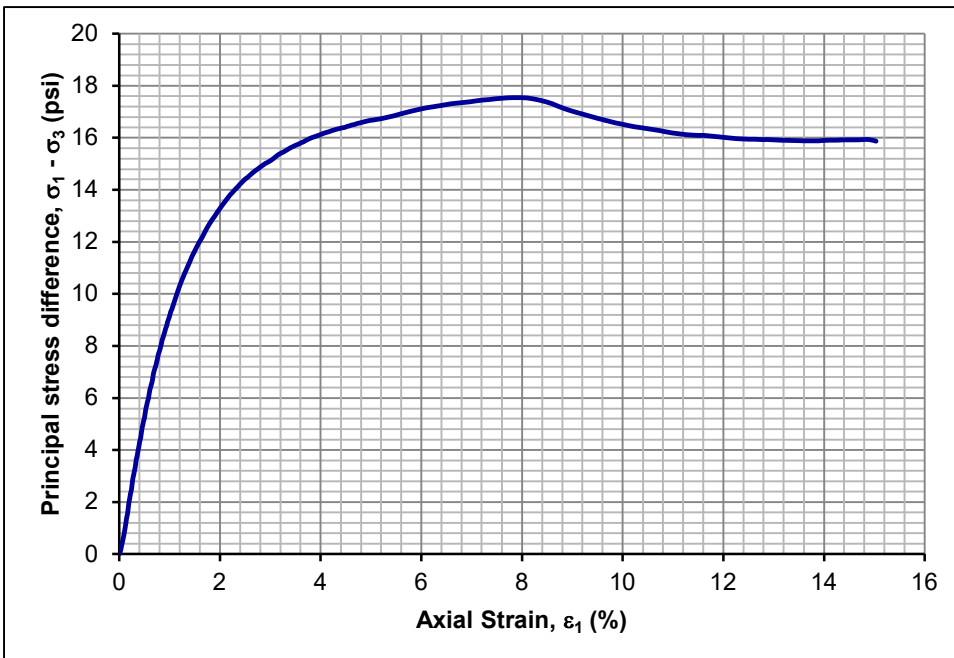
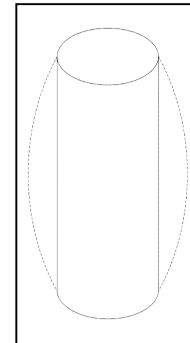
INITIAL SAMPLE DATA

FAILURE DATA***

| | |
|---------------------------|-------|
| AVERAGE DIAMETER (in.): | 2.81 |
| HEIGHT (in.): | 5.57 |
| HEIGHT TO DIAMETER RATIO: | 1.98 |
| WET UNIT WEIGHT (pcf): | 118.6 |
| DRY UNIT WEIGHT (pcf): | 91.9 |
| VOID RATIO: | 0.83 |
| MOISTURE CONTENT (%)*: | 29.0 |
| DEGREE OF SATURATION (%): | 94.1 |

| | |
|--|--------------|
| MOISTURE CONTENT AFTER FAILURE (%)**: | 29.0 |
| AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.): | 1.0 |
| AXIAL STRAIN AT FAILURE (%): | 7.8 |
| PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi): | 17.5 |
| MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi): | 0.0 |
| MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi): | 17.5 |
| UNDRAINED COMPRESSIVE STRENGTH, U_u (psf): | 2,530 |
| UNDRAINED SHEAR STRENGTH, s_u (psf): | 1,265 |
| LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf): | N/A |

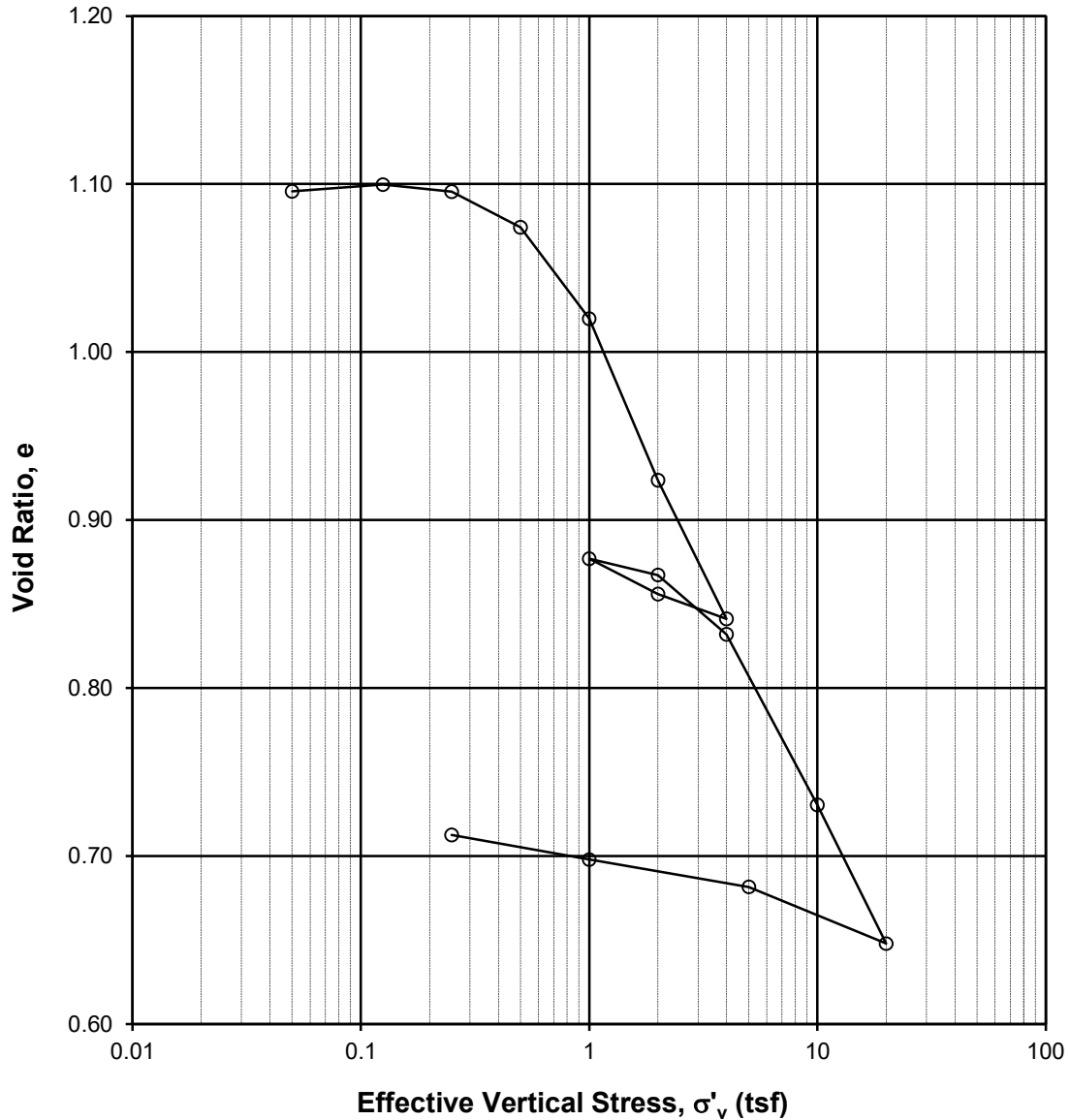
FAILURE SHAPES



REMARKS :

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

| | | | |
|--|---|------------------------------|-----------------|
| Liquid Limit= <u>69</u> | Plastic Limit= <u>22</u> | Plasticity Index = <u>47</u> | USCS: <u>CH</u> |
| Compression Index, $C_c =$ <u>0.26</u> | Void Ratio, $e_o =$ <u>1.096</u> | | |
| Recompression Index, $C_r =$ <u>0.05</u> | Preconsolidation Pressure = <u>0.65</u> tsf | | |



1-D CONSOLIDATION TEST: INCREMENTAL

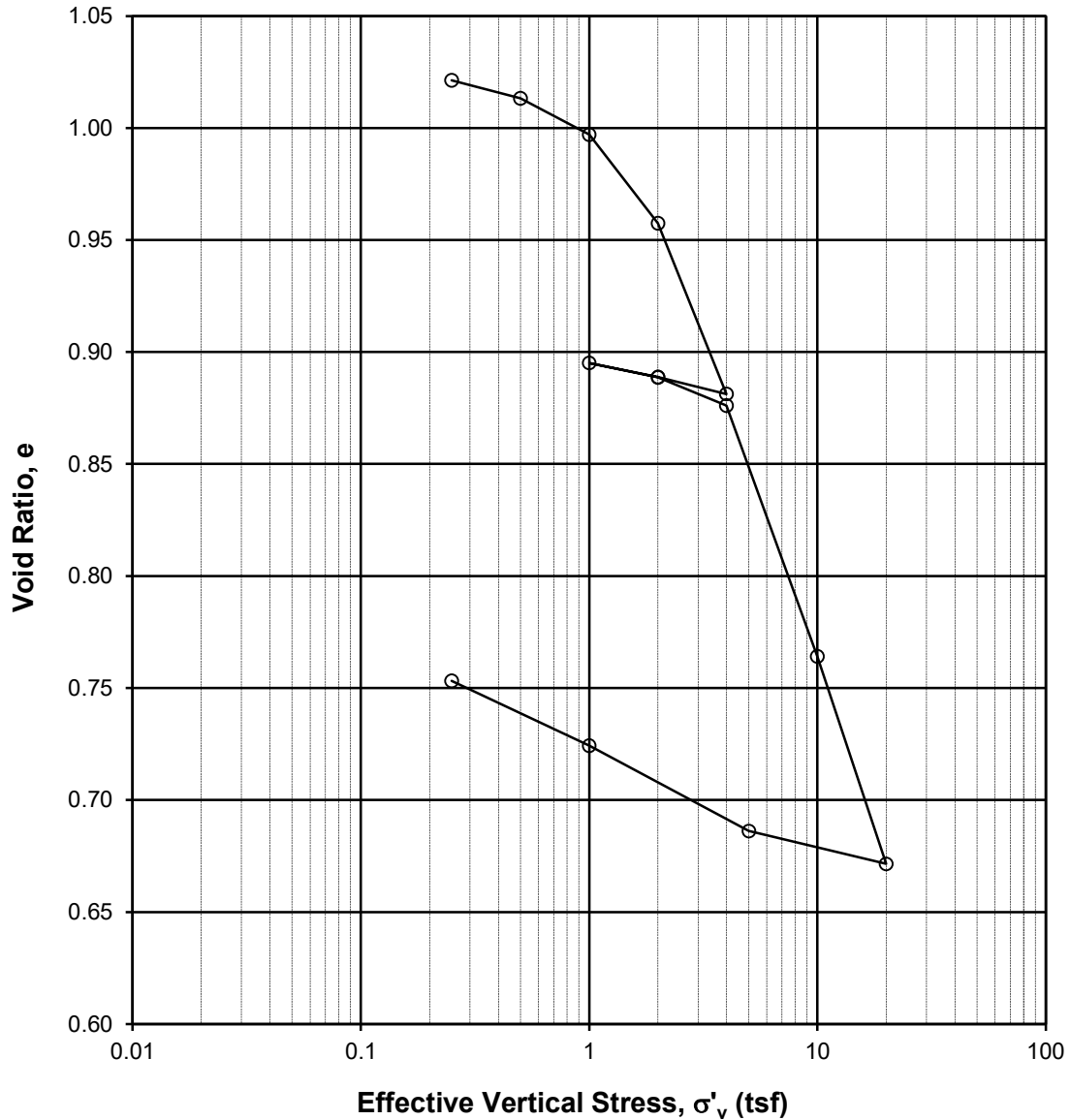
ASTM D 2435

Project No.: J042992.01

Boring: B2-1

Sample: ST-2 - Depth: 3.0

| | | | |
|--|---|------------------------------|-----------------|
| Liquid Limit= <u>46</u> | Plastic Limit= <u>21</u> | Plasticity Index = <u>25</u> | USCS: <u>CL</u> |
| Compression Index, $C_c =$ <u>0.30</u> | Void Ratio, $e_o =$ <u>1.021</u> | | |
| Recompression Index, $C_r =$ <u>0.05</u> | Preconsolidation Pressure = <u>1.70</u> tsf | | |



1-D CONSOLIDATION TEST: INCREMENTAL

ASTM D 2435

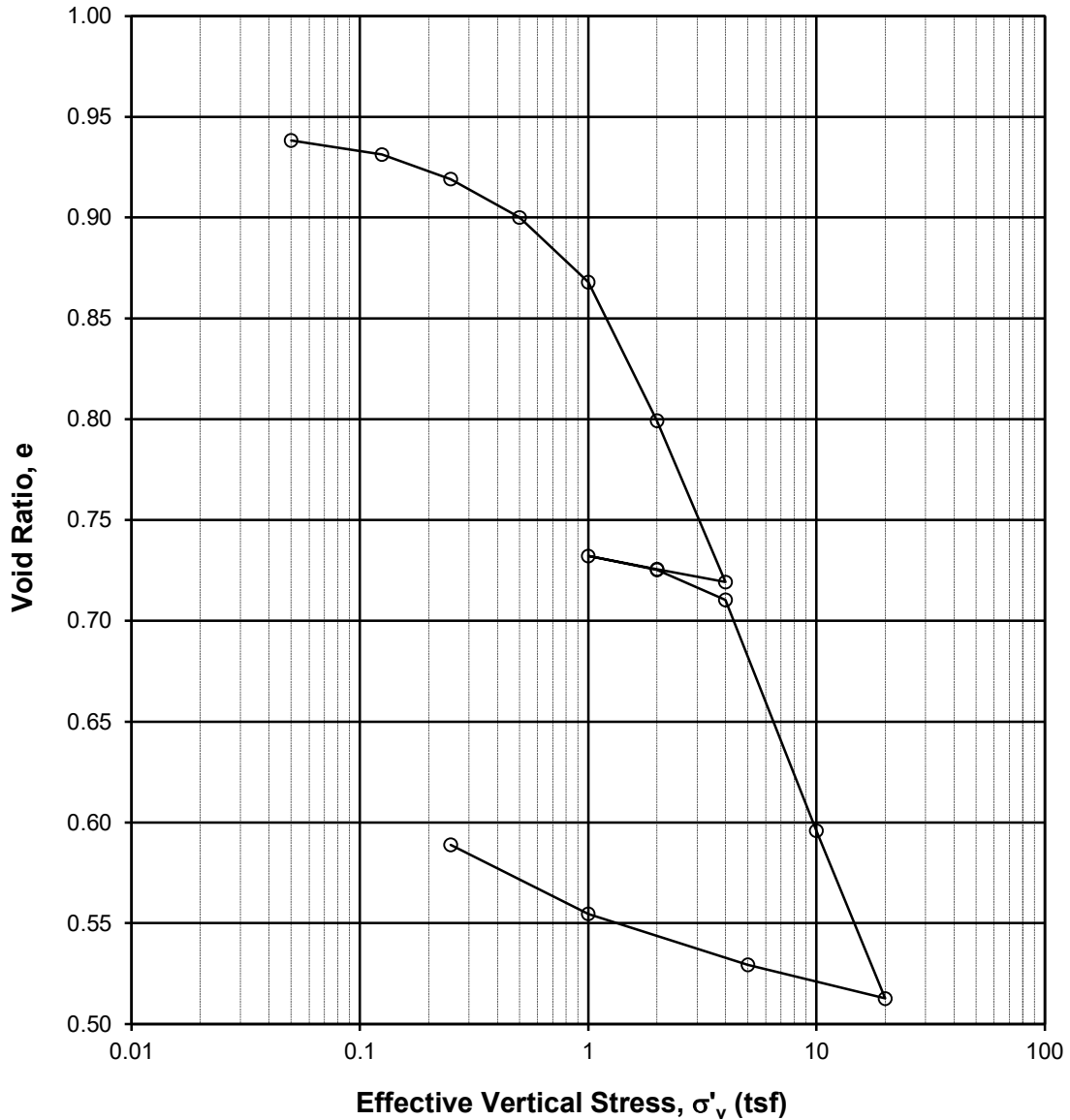
Project No.: J042992.01

Boring: B2-5

Sample: ST-7 - Depth: 15.0

Liquid Limit= 46 Plastic Limit= 17 Plasticity Index = 29 USCS: CL

Compression Index, C_c = 0.28 Void Ratio, e_o = 0.938
 Recompression Index, C_r = 0.05 Preconsolidation Pressure = 0.95 tsf



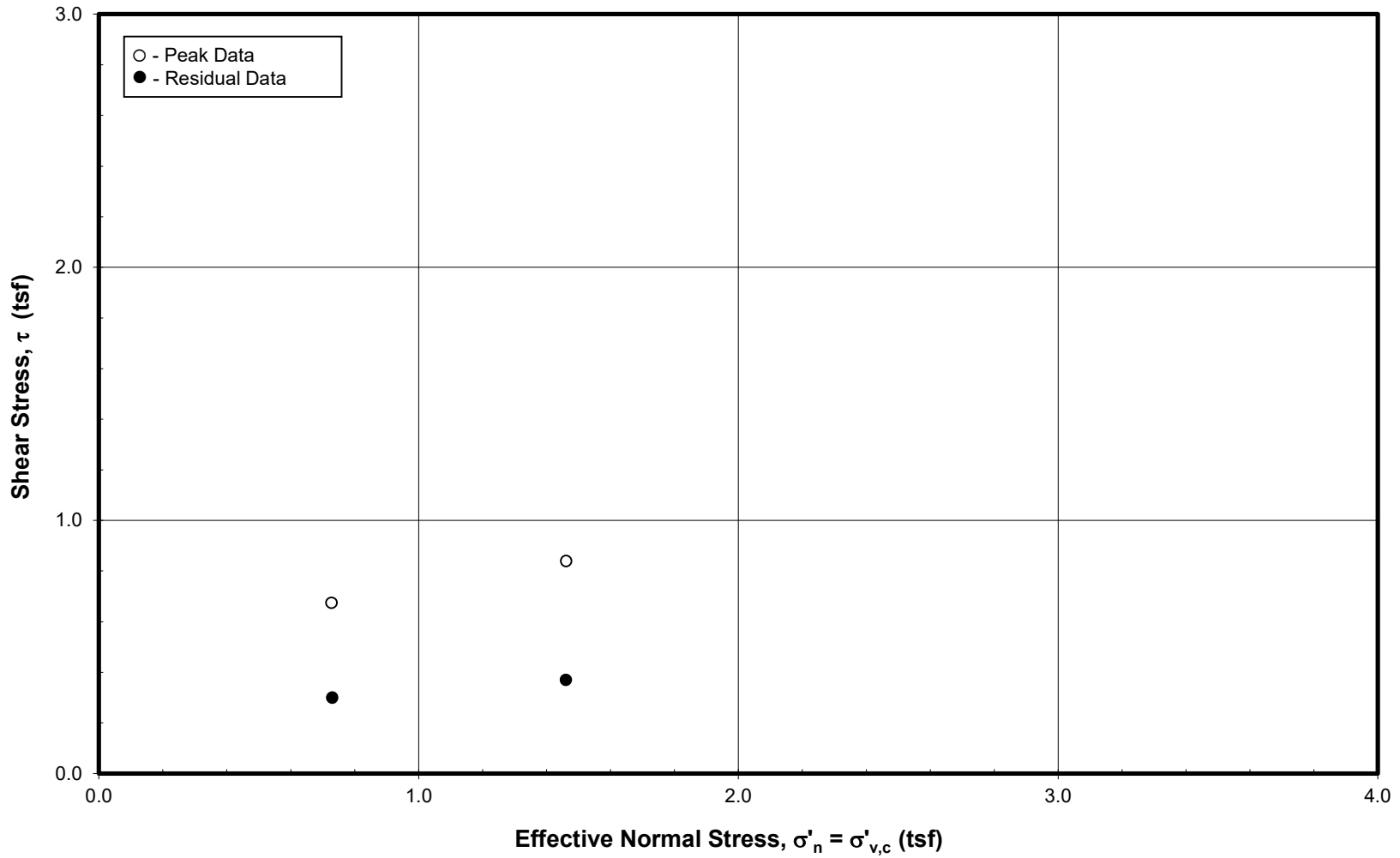
1-D CONSOLIDATION TEST: INCREMENTAL

ASTM D 2435

Project No.: J042992.01

Boring: B2-6

Sample: ST-4 - Depth: 8.0



DRAINED DIRECT SHEAR TEST
ASTM D 3080
Boring: B2-2 Sample: ST-5 -Depth: 10.0ft



SOIL RESISTIVITY TEST REPORT

| | | |
|----------------|---|------------------|
| Project No.: | J042992.01 | August 4th, 2023 |
| Project Name: | ARDOT 101017 Poinsett County – Bridge 2 | Page 1 of 1 |
| Boring Number: | B2-3 | |
| Sample ID: | SS4 | |
| Depth (ft): | 8.5 | |

MINIMUM LABORATORY SOIL RESISTIVITY AASHTO T288

| <u>Reading</u> | <u>Resistance Measurement</u> | <u>Soil Box Factor (cm)</u> | <u>Soil Resistivity (ohms-cm)</u> | <u>Moisture Content (%)</u> |
|----------------|-------------------------------|-----------------------------|-----------------------------------|-----------------------------|
| #1 | 23,830 | 0.57 | 13,583.10 | 10.0 |
| #2 | 11,490 | 0.57 | 6,549.30 | 16.9 |
| #3 | 8,030 | 0.57 | 4,577.10 | 24.0 |
| #4 | 8,140 | 0.57 | 4,639.80 | 36.4 |

Minimum Soil Resistivity **4,577.10**



SOIL RESISTIVITY TEST REPORT

| | | |
|----------------|---|------------------|
| Project No.: | J042992.01 | August 4th, 2023 |
| Project Name: | ARDOT 101017 Poinsett County – Bridge 2 | Page 1 of 1 |
| Boring Number: | B2-4 | |
| Sample ID: | SS11 | |
| Depth (ft): | 43.5 | |

MINIMUM LABORATORY SOIL RESISTIVITY AASHTO T288

| <u>Reading</u> | <u>Resistance Measurement</u> | <u>Soil Box Factor (cm)</u> | <u>Soil Resistivity (ohms-cm)</u> | <u>Moisture Content (%)</u> |
|----------------|-------------------------------|-----------------------------|-----------------------------------|-----------------------------|
| #1 | 3,130 | 0.57 | 1,784.10 | 9.4 |
| #2 | 4,100 | 0.57 | 2,337.00 | 17.4 |

Minimum Soil Resistivity **1,784.10**



SOIL RESISTIVITY TEST REPORT

| | | |
|-----------------------|--|--------------------------|
| Project No.: | J042992.01 | August 15th, 2023 |
| Project Name: | ARDOT 101017 Poinsett County – Bridge 2 | Page 1 of 1 |
| Boring Number: | B2-4 | |
| Sample ID: | SS14 | |
| Depth (ft): | 58.5 | |

MINIMUM LABORATORY SOIL RESISTIVITY AASHTO T288

| <u>Reading</u> | <u>Resistance Measurement</u> | <u>Soil Box Factor (cm)</u> | <u>Soil Resistivity (ohms-cm)</u> | <u>Moisture Content (%)</u> |
|----------------|-------------------------------|-----------------------------|-----------------------------------|-----------------------------|
| #1 | 12,550 | 0.57 | 7,153.50 | 10.0 |
| #2 | 8,203 | 0.57 | 4,675.71 | 16.9 |
| #3 | 7,109 | 0.57 | 4,052.13 | 19.8 |
| #4 | 3,043 | 0.57 | 1,734.51 | 20.0 |
| #5 | 8,913 | 0.57 | 5,080.41 | 20.9 |

Minimum Soil Resistivity **1,734.51**



SOIL RESISTIVITY TEST REPORT

| | | |
|----------------|---|-------------------|
| Project No.: | J042992.01 | August 15th, 2023 |
| Project Name: | ARDOT 101017 Poinsett County – Bridge 2 | Page 1 of 1 |
| Boring Number: | B2-5 | |
| Sample ID: | SS12 | |
| Depth (ft): | 38.5 | |

MINIMUM LABORATORY SOIL RESISTIVITY AASHTO T288

| <u>Reading</u> | <u>Resistance Measurement</u> | <u>Soil Box Factor (cm)</u> | <u>Soil Resistivity (ohms-cm)</u> | <u>Moisture Content (%)</u> |
|----------------|-------------------------------|-----------------------------|-----------------------------------|-----------------------------|
| #1 | 11,190 | 0.57 | 6,378.30 | 10.1 |
| #2 | 6,535 | 0.57 | 3,724.95 | 17.3 |
| #3 | 6,663 | 0.57 | 3,797.91 | 17.4 |

Minimum Soil Resistivity 3,724.95



SOIL RESISTIVITY TEST REPORT

| | | |
|----------------|---|-------------------|
| Project No.: | J042992.01 | August 15th, 2023 |
| Project Name: | ARDOT 101017 Poinsett County – Bridge 2 | Page 1 of 1 |
| Boring Number: | B2-5 | |
| Sample ID: | ST-7 | |
| Depth (ft): | 15.0 | |

MINIMUM LABORATORY SOIL RESISTIVITY AASHTO T288

| <u>Reading</u> | <u>Resistance Measurement</u> | <u>Soil Box Factor (cm)</u> | <u>Soil Resistivity (ohms-cm)</u> | <u>Moisture Content (%)</u> |
|----------------|-------------------------------|-----------------------------|-----------------------------------|-----------------------------|
| #1 | 1,365 | 0.57 | 778.05 | 16.7 |
| #2 | 1,139 | 0.57 | 649.23 | 32.2 |
| #3 | 1,265 | 0.57 | 721.05 | 41.2 |

Minimum Soil Resistivity **649.23**



APPENDIX F – SITE-SPECIFIC SEISMIC STUDY

Site-Specific Seismic Study
ARDOT G003 101017 Tyronza River &
Ditch No. 6 Strs. & Apprs.
Bridge 2
Pointsett County, Arkansas

By

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

October 7, 2023

TABLE OF CONTENTS

| | <u>Page</u> |
|---|-------------|
| 1.0. EXECUTIVE SUMMARY | 1 |
| 2.0. SCOPE OF WORK..... | 2 |
| 3.0. SUBSURFACE CONDITIONS | 2 |
| 4.0. SHEAR-WAVE VELOCITY PROFILE..... | 3 |
| 5.0. GENERAL INFORMATION..... | 5 |
| 6.0. REGIONAL SEISMICITY..... | 5 |
| 7.0. SEISMIC HAZARD ANALYSIS | 6 |
| 7.1. SEISMIC SOURCE MODELS | 6 |
| 7.2. GROUND MOTION MODELS..... | 6 |
| 7.3. TREATMENT OF UNCERTAINTIES..... | 8 |
| 8.0. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2 nd Edition, 2022 Interim Revisions | 8 |
| 8.1. Dynamic Soil Properties..... | 8 |
| 9.0. CODE-BASED DESIGN APPROACH | 9 |
| 9.1. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2 nd Edition, 2022 Interim Revisions | 9 |
| 10.0. SITE-SPECIFIC PROCEDURE..... | 10 |
| 10.1. Seismic Hazard Analysis | 12 |
| 10.2. Variability in Soil's Shear-Wave and Thickness Profile..... | 12 |
| 10.3. Site-Specific Results..... | 13 |
| 11.0. DESIGN RESPONSE SPECTRAL PARAMETERS | 13 |
| 12.0. LIMITATIONS OF THE REPORT..... | 17 |
| 13.0. REFERENCES | 17 |
| APPENDIX A. Site Location | 20 |

Site-Specific Seismic Study ARDOT G003 101017 Tyronza River & Ditch No. 6 Strs. & Apprs Bridge 2 Pointsett County, Arkansas

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 $35.511411^{\circ}\text{N}$ and $90.3089231^{\circ}\text{W}$ (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_{DI} (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). The shear-wave velocity profiles obtained from SCPT are provide Table 1 and are shown in Figure 1.

Table 1. Shear-Wave Velocities Measured.

| CPT1 | | |
|------------|-------------|----------------|
| Depth1(ft) | Depth2(ft.) | velocity(ft/s) |
| 14.86 | 439.06 | 14.86 |
| 18.14 | 439.06 | 18.14 |
| 21.45 | 439.06 | 21.45 |
| 24.73 | 685.78 | 24.73 |
| 28.01 | 373.56 | 28.01 |
| 31.29 | 727.80 | 31.29 |
| 34.60 | 556.32 | 34.60 |
| 37.88 | 463.73 | 37.88 |
| 41.13 | 460.25 | 41.13 |
| 44.38 | 716.42 | 44.38 |
| 47.63 | 913.25 | 47.63 |
| 50.94 | 1124.58 | 50.94 |
| 54.15 | 659.02 | 54.15 |
| 57.40 | 1020.57 | 57.40 |
| 60.68 | 696.57 | 60.68 |
| 63.89 | 1125.60 | 63.89 |
| 70.55 | 980.29 | 70.55 |
| 73.87 | 1005.84 | 73.87 |
| 77.08 | 1334.30 | 77.08 |
| 80.43 | 877.37 | 80.43 |
| 83.67 | 1078.96 | 83.67 |
| 87.12 | 817.61 | 87.12 |
| 90.27 | 1869.17 | 90.27 |
| 93.55 | 634.06 | 93.55 |
| 96.86 | 810.03 | 96.86 |
| 100.14 | 826.46 | 100.14 |

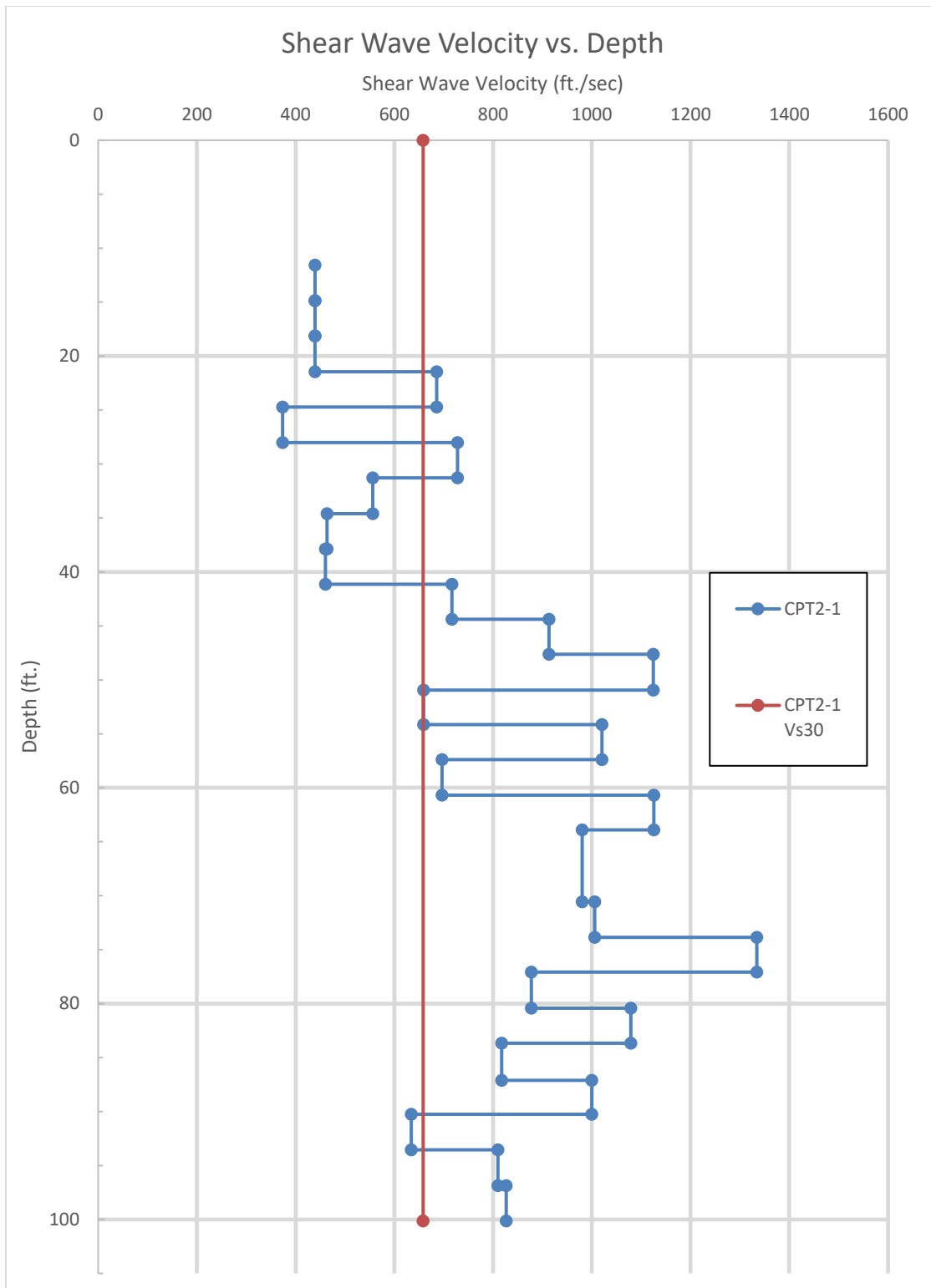


Figure 1. Shear-Wave Velocities Measured by Geotechnology.

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 2 shows the USGS 2018 declustered catalog for CEUS.

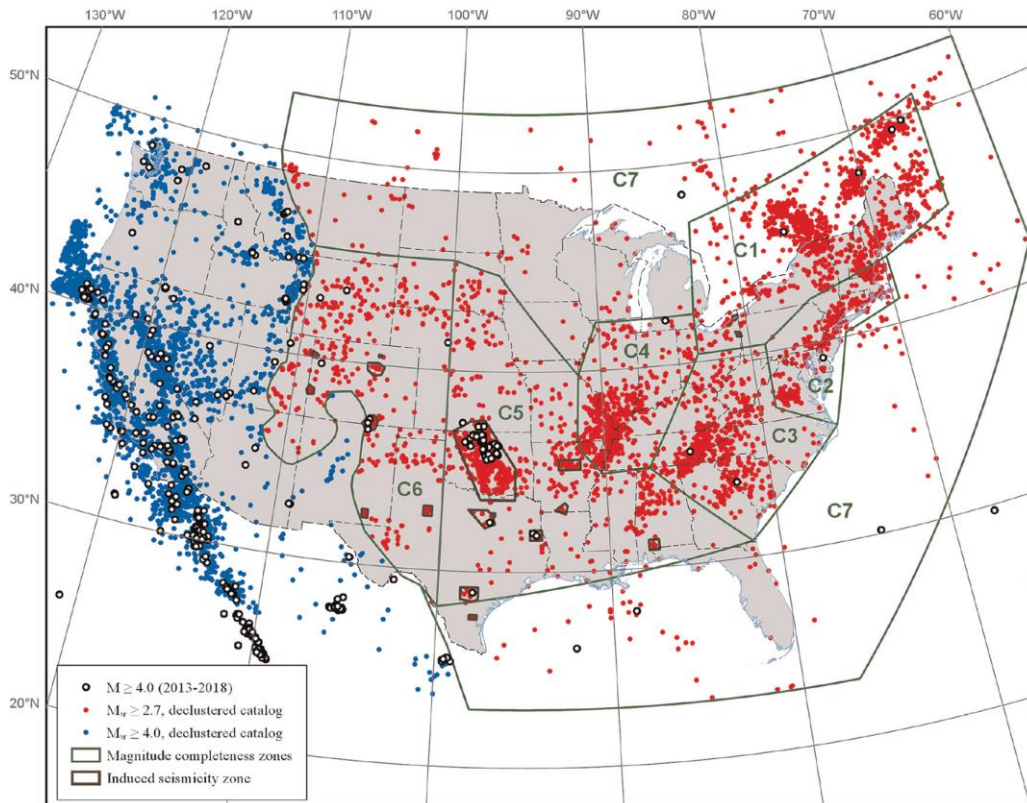


Figure 2. The 2018 NSHM Declustered Catalog for Central and Eastern United States (red) and Western United States (blue).

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).[Source Rezaeian et al. (2021)].

| 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 | Shajouei-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) | Shajouei-Pezeshk | 0 |
| Graizer (replaced with Graizer16 & Graizer17 by USGS) | Graizer | 0 |
| PEER-EX | Hollenback-Kuehn-Goulet-Abrahamson | 0 |
| 17 NGA-East GMMs (used by USGS in 2018 NSHM) | | 0.667 |
| Models 1 to 17 | NGA-East Project | Period-dependen ^a |

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 658 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 | 1.000 |
| F_v | 1.545 |
| F_{PGA} | 1.000 |
| S_s | 1.720 |
| S_1 | 0.455 |
| S_{DS} | 1.720 |
| S_{D1} | 0.703 |
| PGA | 0.973 |
| A_s | 0.973 |

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 3. The same process was repeated for all eleven earthquakes for both components.

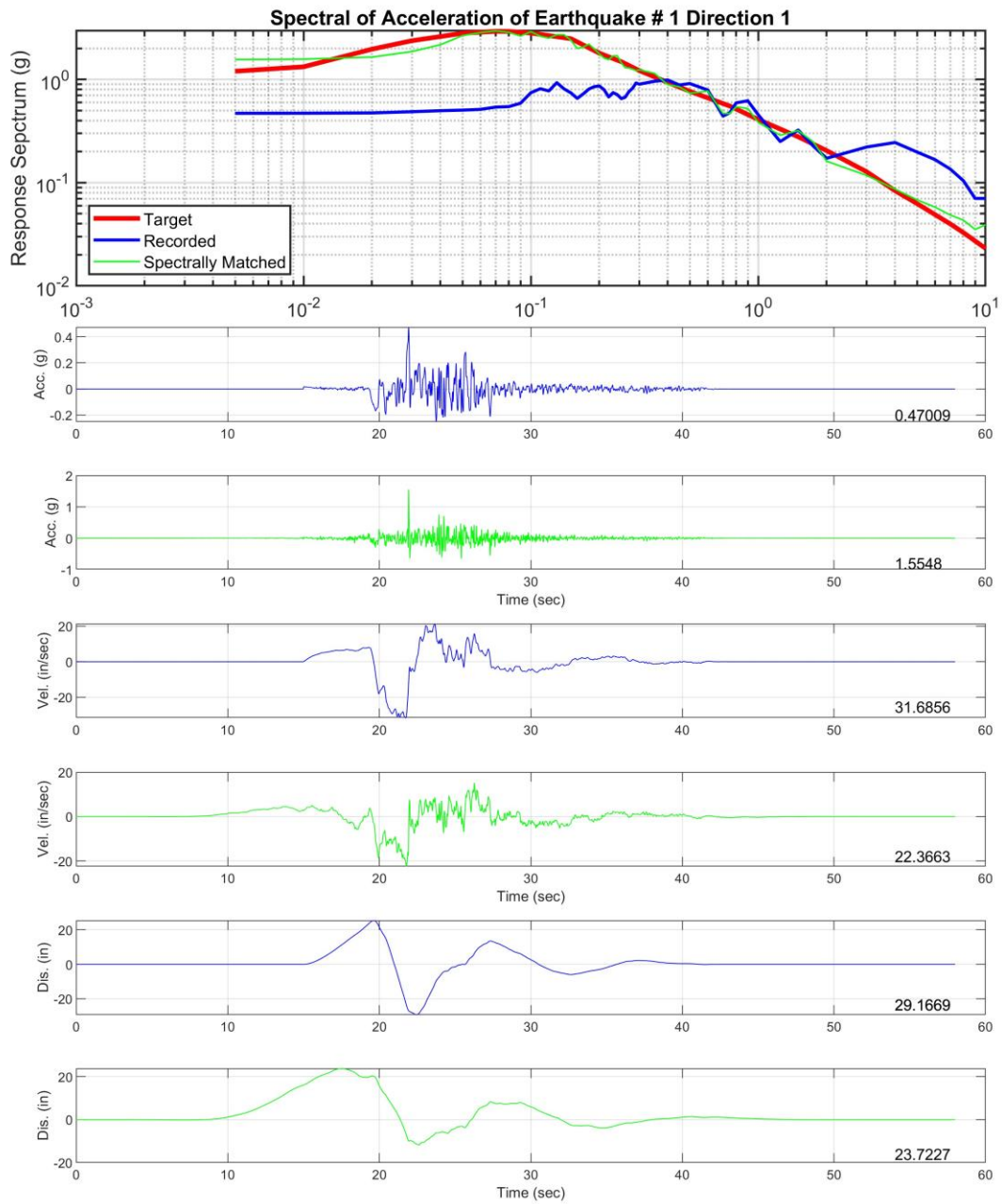


Figure 3. Time Histories Before and After the Spectral Matching Process for Earthquake #1. The numbers Shown in the Bottom right of Each Figure Represent the Absolute Maximum Value of the Graph.

Table 4. Deaggregation.

| Mean and Mode Deaggregation Parameter at 1,033 Years | | | | | |
|--|------|--------|--------|------|--------|
| Mean | | | Mode | | |
| Period | M | R (km) | Period | M | R (km) |
| PGA | 7.36 | 13.96 | PGA | 7.52 | 14.51 |
| 0.01 | 7.36 | 14.03 | 0.01 | 7.55 | 13.35 |
| 0.02 | 7.34 | 14.32 | 0.02 | 7.52 | 14.54 |
| 0.03 | 7.34 | 14.44 | 0.03 | 7.52 | 14.56 |
| 0.05 | 7.35 | 14.93 | 0.05 | 7.52 | 14.54 |
| 0.08 | 7.37 | 15.33 | 0.08 | 7.52 | 14.53 |
| 0.10 | 7.38 | 15.85 | 0.10 | 7.52 | 14.54 |
| 0.15 | 7.42 | 16.29 | 0.15 | 7.52 | 14.55 |
| 0.20 | 7.44 | 16.69 | 0.20 | 7.52 | 14.54 |
| 0.25 | 7.46 | 17.07 | 0.25 | 7.54 | 13.36 |
| 0.30 | 7.46 | 17.50 | 0.30 | 7.54 | 13.36 |
| 0.40 | 7.48 | 17.96 | 0.40 | 7.54 | 13.37 |
| 0.50 | 7.48 | 18.45 | 0.50 | 7.54 | 13.37 |
| 0.75 | 7.51 | 19.73 | 0.75 | 7.54 | 13.40 |
| 1.00 | 7.52 | 20.87 | 1.00 | 7.54 | 13.64 |
| 1.50 | 7.54 | 22.19 | 1.50 | 7.55 | 13.93 |
| 2.00 | 7.56 | 23.58 | 2.00 | 7.55 | 13.93 |
| 3.00 | 7.59 | 24.33 | 3.00 | 7.76 | 13.31 |
| 4.00 | 7.61 | 25.10 | 4.00 | 7.76 | 13.30 |
| 5.00 | 7.63 | 25.81 | 5.00 | 7.76 | 13.32 |
| 7.50 | 7.65 | 26.98 | 7.50 | 7.76 | 13.43 |
| 10.00 | 7.66 | 27.50 | 10.00 | 7.59 | 12.24 |

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 4 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 S_a values provided in Table 5. Figure 5 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 5, 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 | 0.649 |
| 0.030 | 0.831 |
| 0.040 | 0.892 |
| 0.050 | 0.953 |
| 0.070 | 1.075 |
| 0.100 | 1.147 |
| 0.150 | 1.147 |
| 0.200 | 1.284 |
| 0.250 | 1.147 |
| 0.300 | 1.150 |
| 0.400 | 1.375 |
| 0.500 | 1.427 |
| 0.750 | 1.125 |
| 1.000 | 1.017 |
| 1.500 | 0.678 |
| 2.000 | 0.469 |
| 3.000 | 0.199 |
| 4.000 | 0.117 |
| 5.000 | 0.094 |
| 7.500 | 0.062 |
| 10.000 | 0.070 |

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

| PARAMETER | DESIGN ACCELERATION PARAMETERS (g) |
|------------------|---|
| S_{DS} | 1.284 |
| S_{DI} | 1.017 |
| S_{MS} | 1.284 |
| S_{MI} | 1.017 |
| MCE_G | 0.649 |

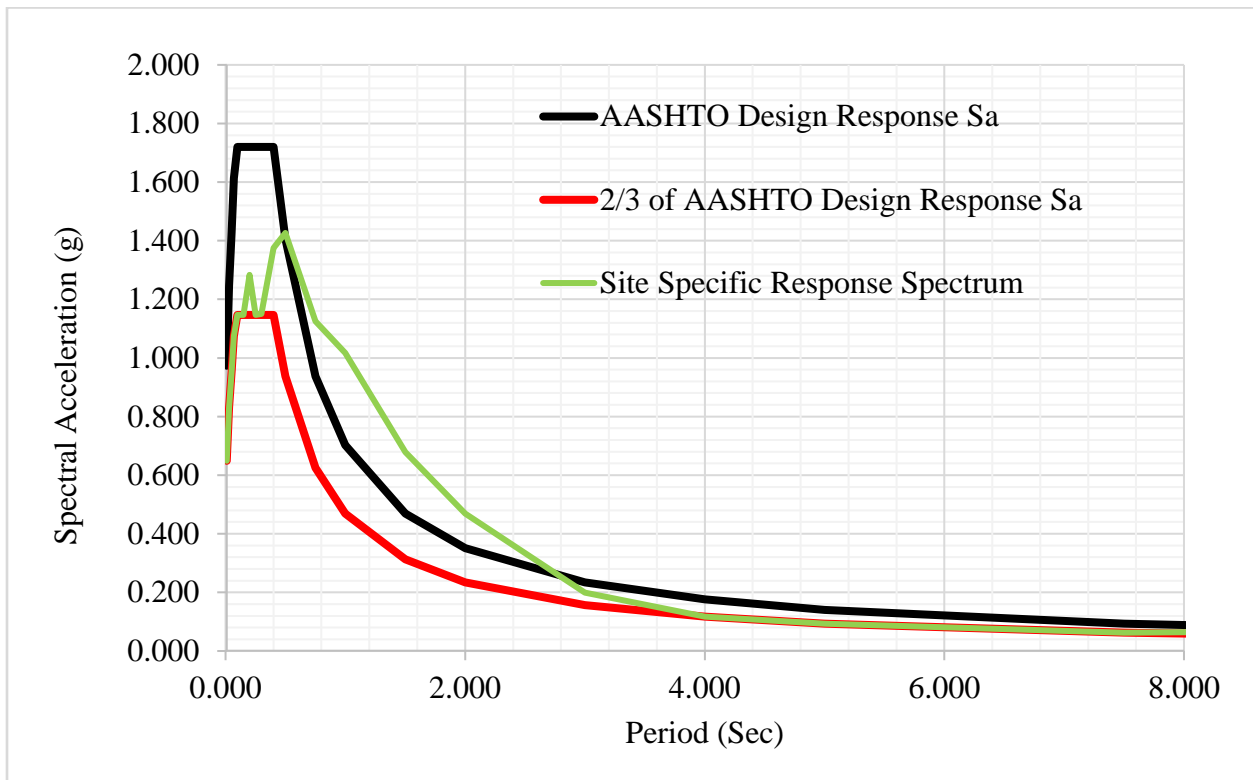


Figure 4. Site-Specific Design Response Spectrum, AASHTO Guide Specifications Design Response Spectrum, and 2/3 of the AASHTO Guide Specifications Design Response Spectrum.

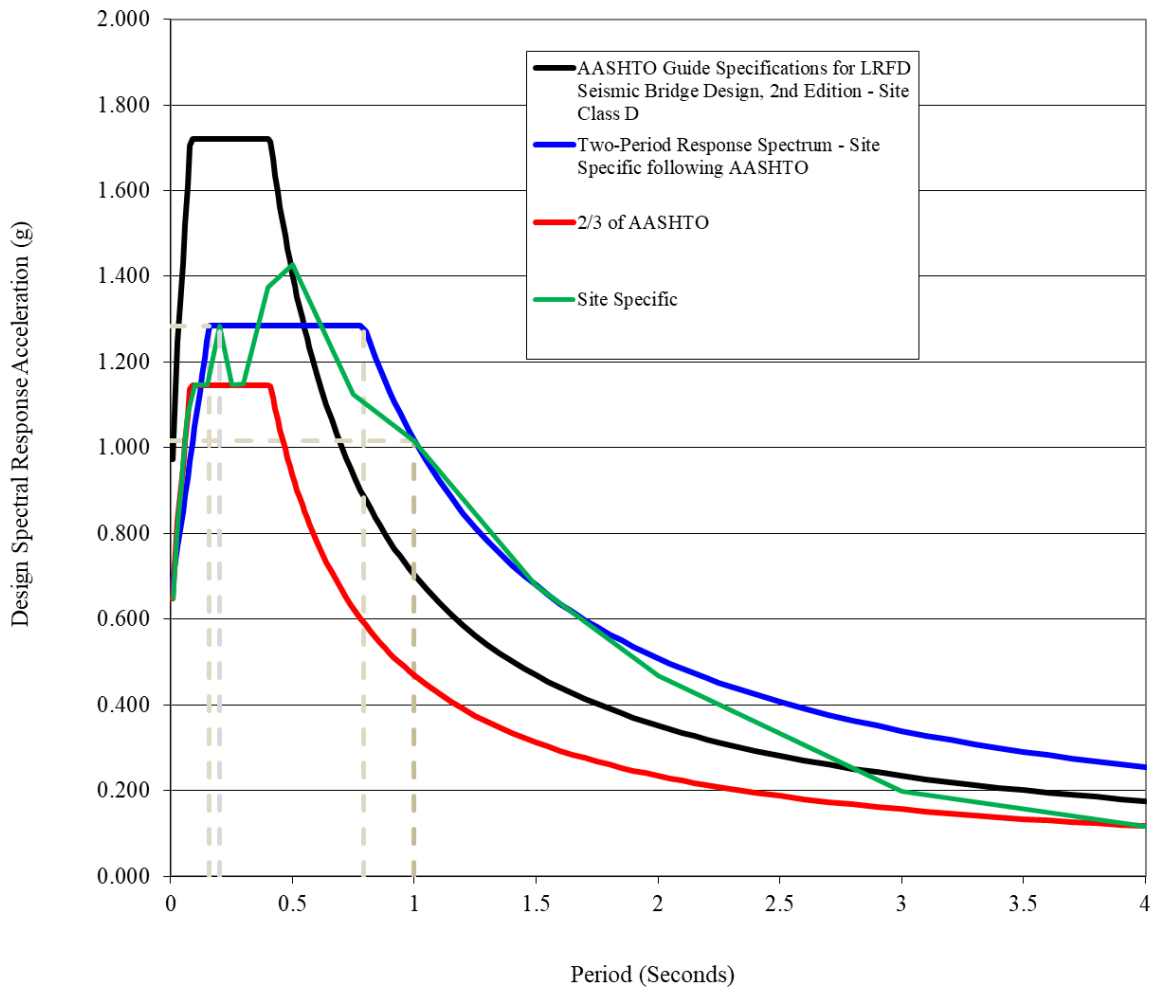


Figure 5. Design Response Spectrum based on AASHTO Guide Specifications, 2/3 of the AASHTO Guide Specifications Site-Specific, and Design Response Spectrum Based on PGA, 0.2, and 1 Second.

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

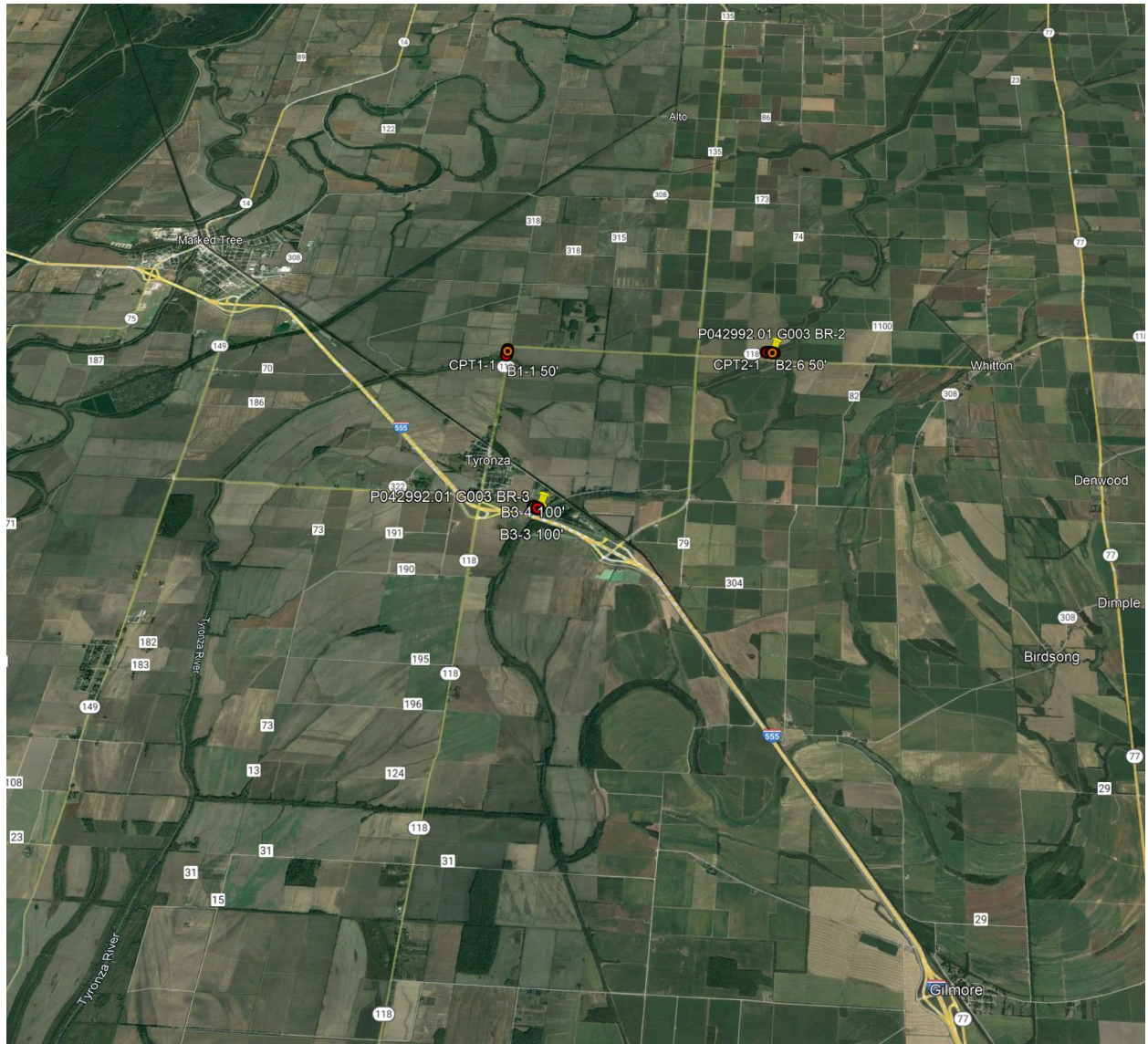


Figure A.1. The Location of the Study Site.



APPENDIX G – AASHTO AND USCS CLASSIFICATIONS



Project: ARDOT 101017
 Tyronza River & Ditch No. 6 Strs. & Apprs. (S)
 Bridge 2 Over Tyronza River
 Number: J042992.01

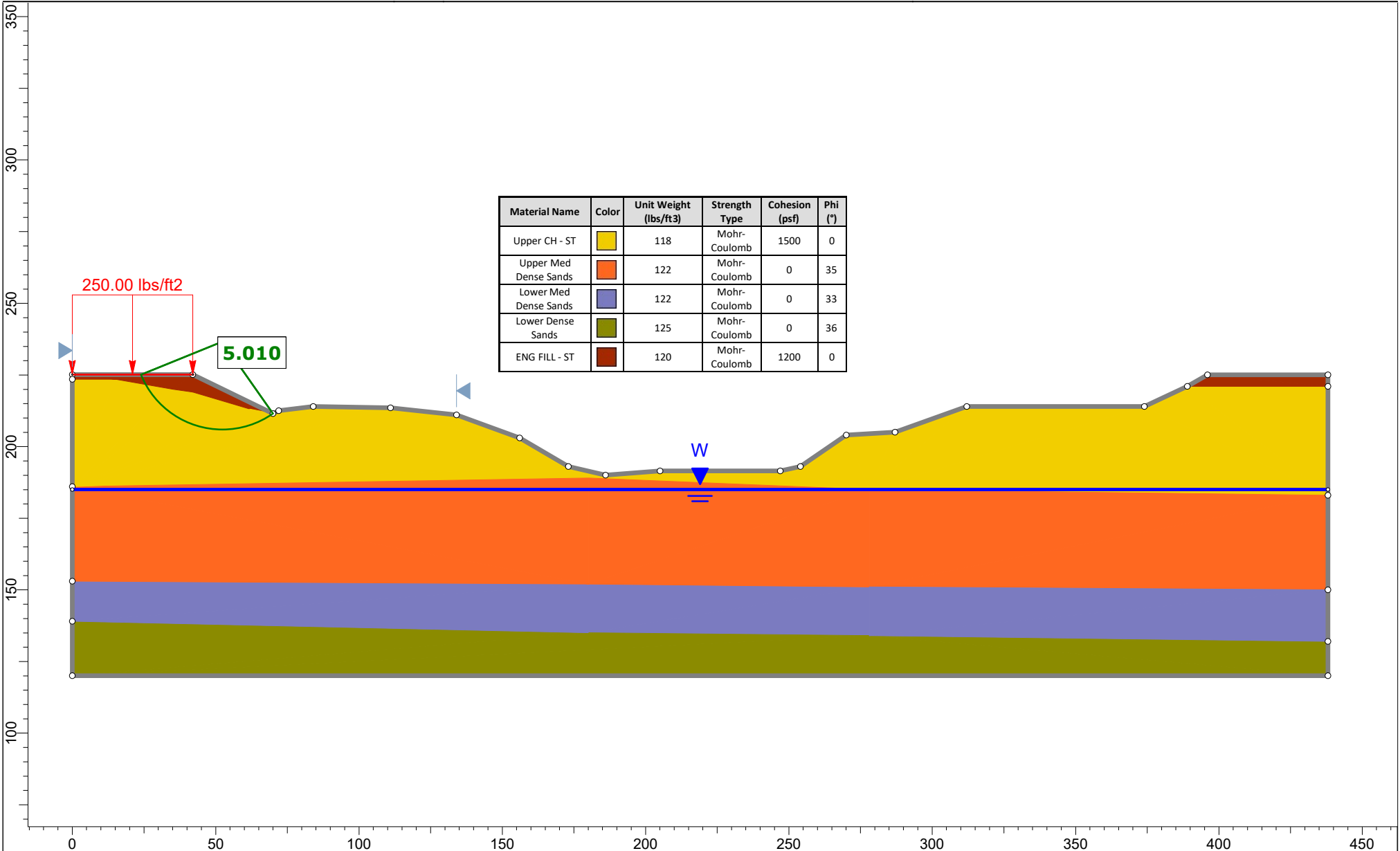
| Borehole | Depth | Liquid Limit (LL) | Plastic Limit (PL) | Plasticity Index (PI) | %<#10 Sieve | %<#40 Sieve | %<#200 Sieve | GI | AASHTO CLASS. | USCS CLASS. |
|----------|-------|-------------------|--------------------|-----------------------|-------------|-------------|--------------|----|---------------|-------------|
| B2-1 | 3 | 69 | 22 | 47 | -- | -- | -- | -- | A-7-6 | CH |
| | 10 | 69 | 18 | 51 | -- | -- | -- | -- | A-7-6 | CH |
| B2-2 | 1 | 63 | 19 | 44 | -- | -- | -- | -- | A-7-6 | CH |
| | 8 | 110 | 24 | 86 | -- | -- | -- | -- | A-7-6 | CH |
| | 16 | 78 | 24 | 54 | -- | -- | -- | -- | A-7-6 | CH |
| B2-3 | 28.5 | -- | -- | -- | 100.0 | 99.7 | 88.1 | -- | A-7-6 | CH |
| | 38.5 | -- | -- | -- | 100.0 | 68.0 | 21.7 | -- | A-2-4 | SM |
| | 3.5 | -- | -- | -- | 100.0 | 90.8 | 11.9 | -- | A-2-4 | SP-SM |
| | 13.5 | -- | -- | -- | 100.0 | 30.5 | 3.2 | -- | A-1-b | SP |
| | 38.5 | -- | -- | -- | 93.6 | 9.2 | 1.7 | -- | A-1-b | SP |
| | 48.5 | -- | -- | -- | 99.7 | 67.4 | 3.1 | -- | A-3 | SP |
| B2-4 | 3.5 | -- | -- | -- | 87.9 | 69.7 | 49.9 | -- | A-7-6 | SC |
| | 13.5 | -- | -- | -- | 100.0 | 100 | 99.7 | -- | A-7-6 | CH |
| | 18.5 | -- | -- | -- | 100.0 | 78.7 | 49.1 | -- | A-7-6 | SC |
| B2-5 | 3 | 97 | 22 | 75 | -- | -- | -- | -- | A-7-6 | CH |
| | 10 | 55 | 16 | 39 | -- | -- | -- | -- | A-7-6 | CH |
| | 15 | 46 | 21 | 25 | -- | -- | -- | -- | A-7-6 | CL |
| | 23.5 | -- | -- | -- | 100.0 | 83.9 | 9.1 | -- | A-2-7 | SP-SC |
| B2-6 | 33.5 | -- | -- | -- | 100.0 | 100.0 | 72.7 | -- | A-7-6 | CH |
| | 3 | 95 | 26 | 69 | -- | -- | -- | -- | A-7-6 | CH |
| | 8 | 46 | 17 | 29 | -- | -- | -- | -- | A-7-6 | CL |
| | 23.5 | -- | -- | -- | 100.0 | 100.0 | 34.7 | -- | A-2-6 | SC |
| | 33.5 | -- | -- | -- | 100.0 | 100.0 | 86.6 | -- | A-4 | CL |



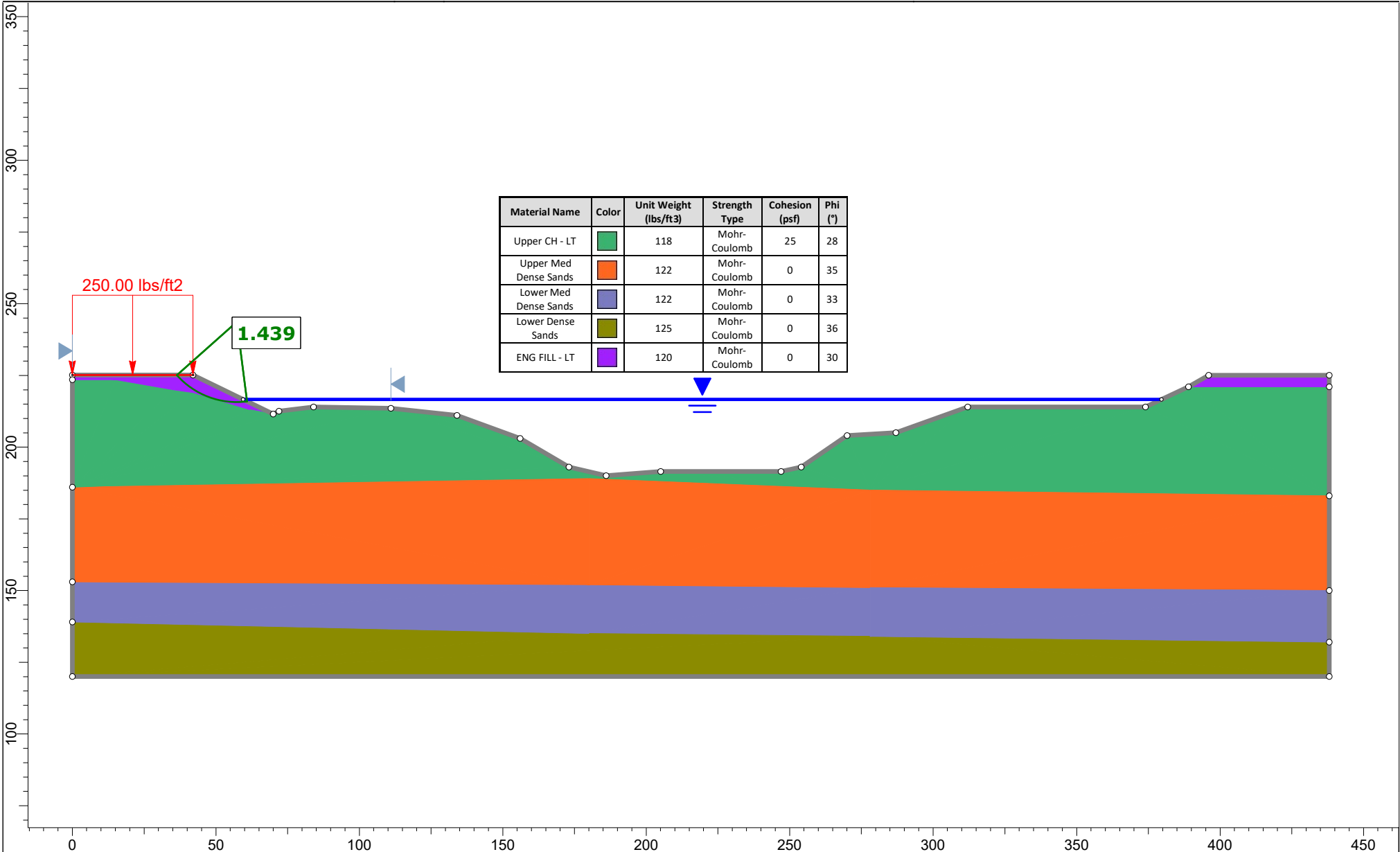
APPENDIX H – GLOBAL STABILITY ANALYSES



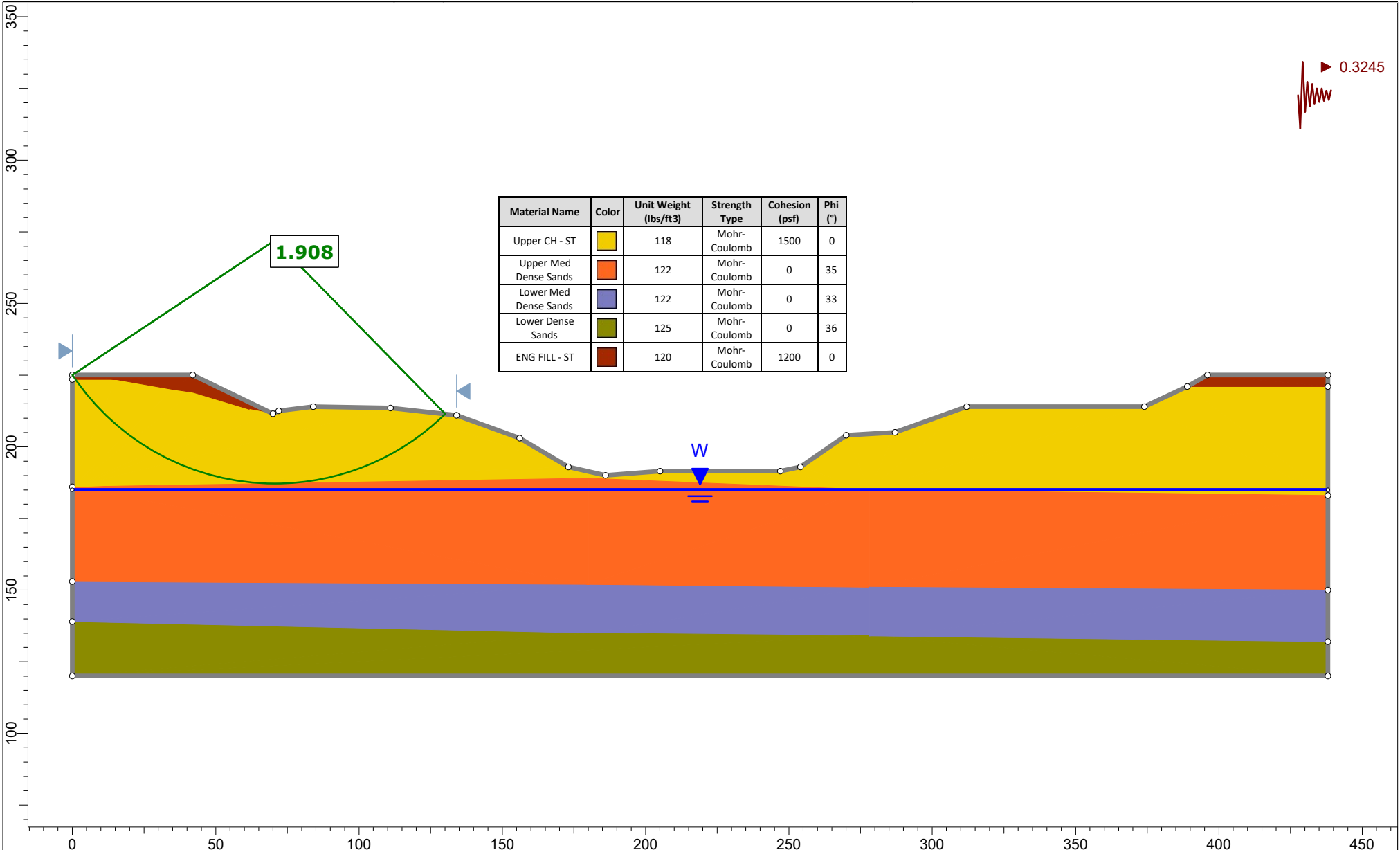
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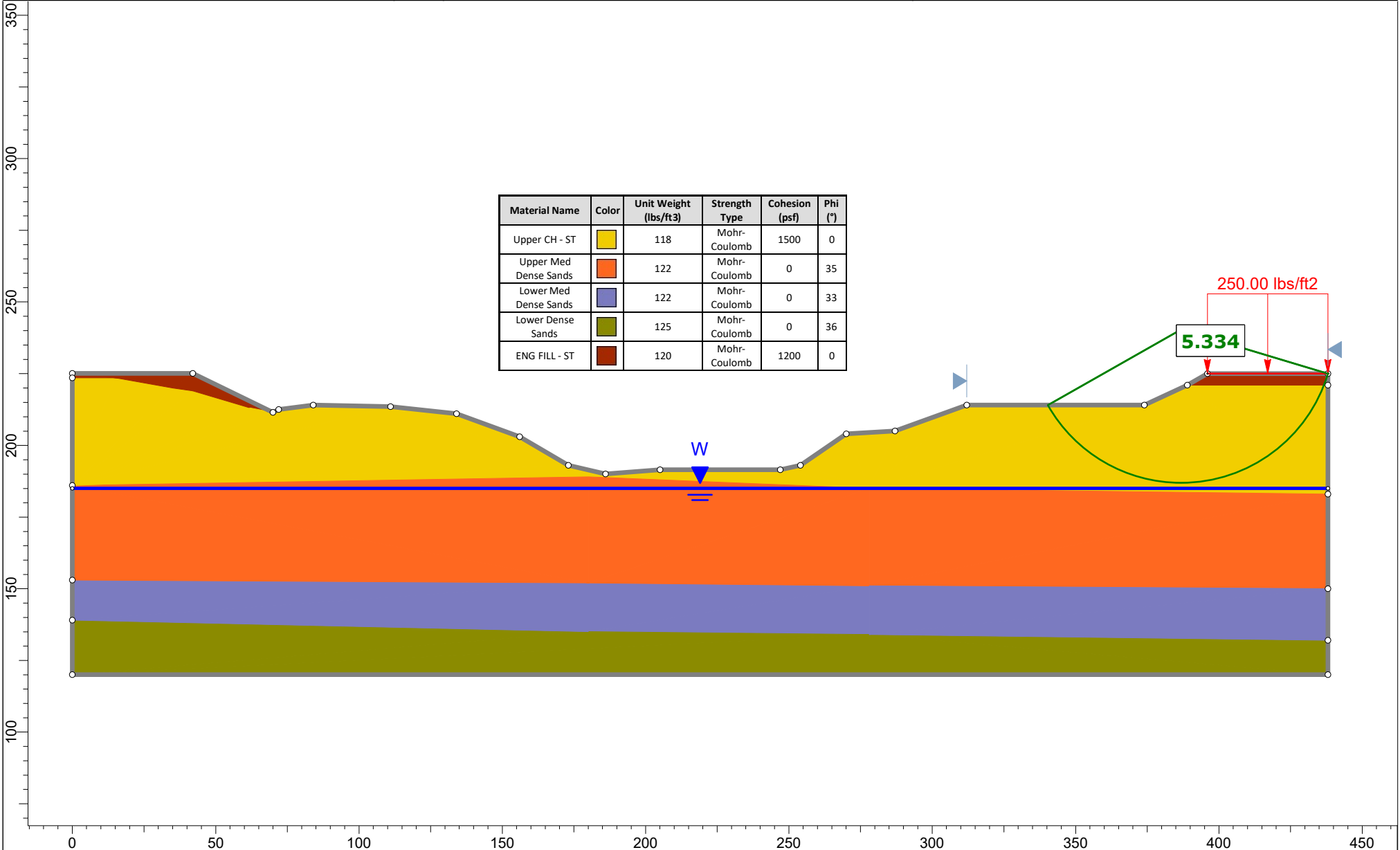
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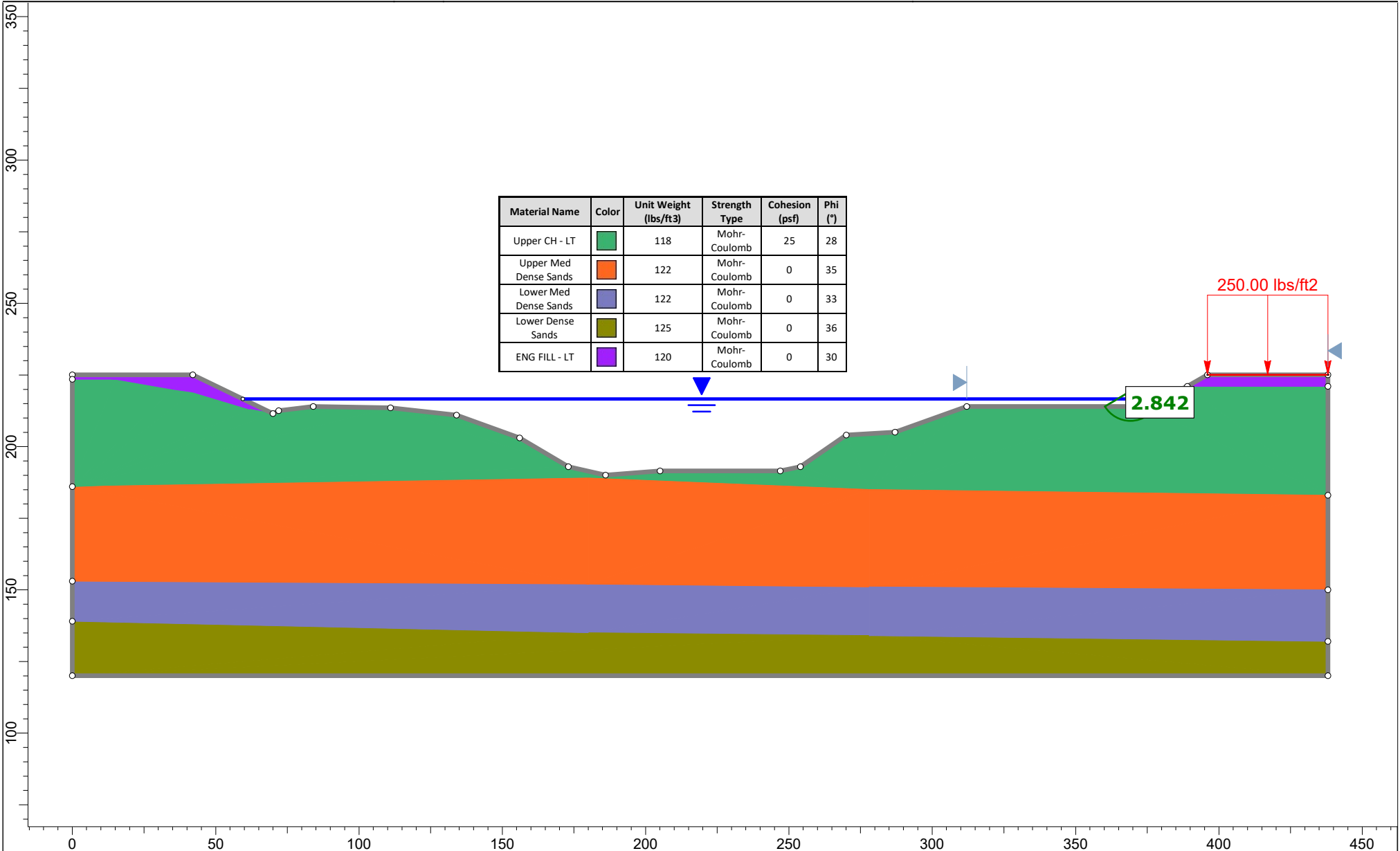
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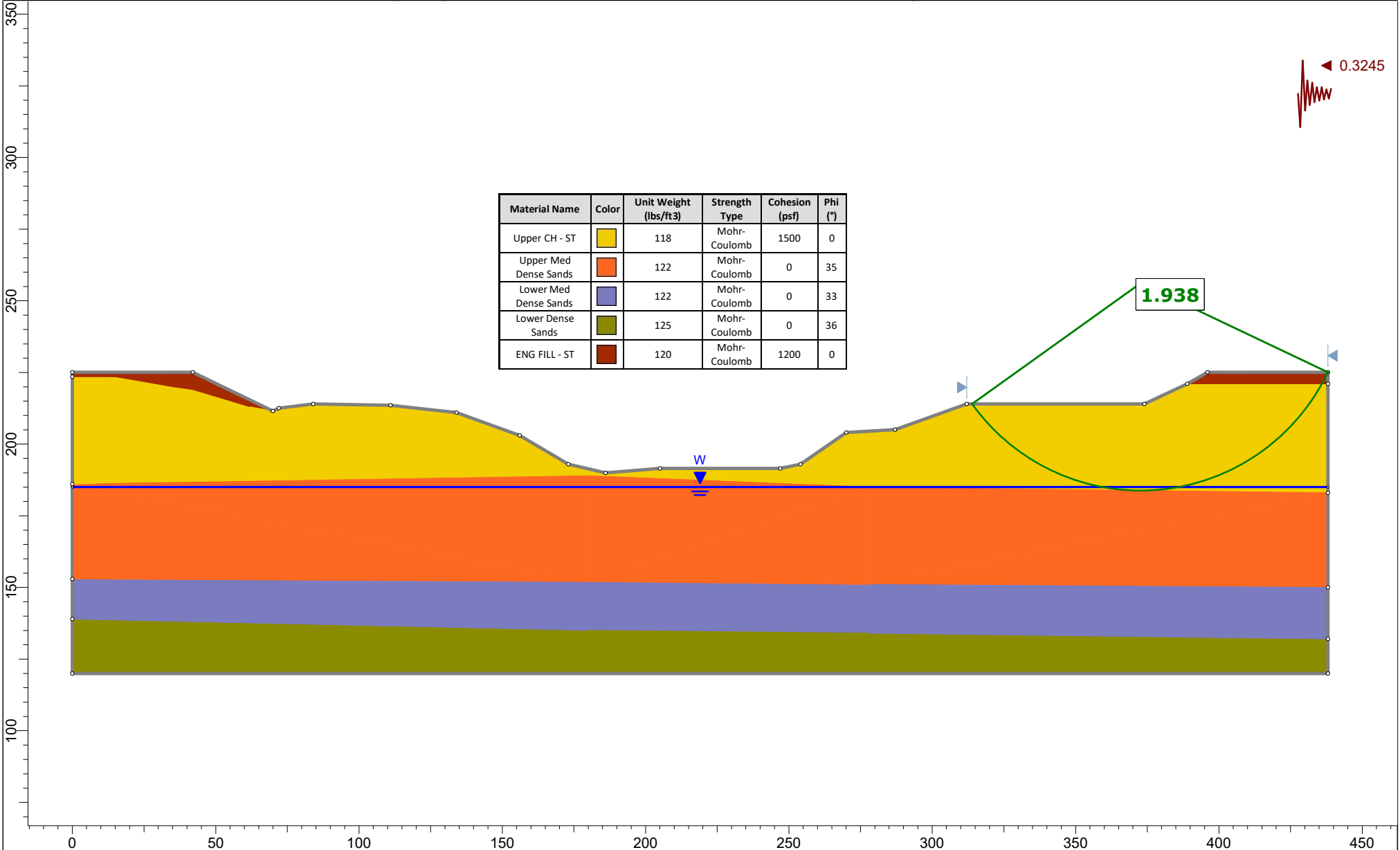
SLIDEINTERPRET 9.031



SLIDEINTERPRET 9.031



SLIDEINTERPRET 9.031





APPENDIX I – SOIL PARAMETERS FOR SYNTHETIC PROFILES



| BRIDGE 2 OVER TYRONZA RIVER – BENT 1 (WEST ABUTMENT; BORING B2-2) | | | | | | | | | | | | | |
|---|--------------------------|--|------|-------------------------|---------------------------|-----------------|--------------------------|------------------|--------------------------------------|--|------------------|--|--------------------------|
| APPROXIMATE GROUND SURFACE ELEVATION = EL 220 | | | | | | | | | | | | | |
| ZONE | SOIL TYPES | DEPTH ^a (feet from ground surface) | | TOTAL UNIT WEIGHT (PCF) | SHEAR STRENGTH PARAMETERS | | | | LATERAL LOAD PARAMETERS ^d | | | LIQUEFACTION SHEAR STRENGTH PARAMETERS | |
| | | FROM | TO | | UNDRAINED (SHORT TERM) | | DRAINED (LONG TERM) | | SOIL STRAIN, E ₅₀ | STATIC SOIL MODULUS (PCI) ^c | LPILE SOIL MODEL | RESIDUAL COHESION (PSF) | RESIDUAL ϕ (DEGREE) |
| | | | | | COHESION (PSF) | ϕ (DEGREE) | EFFECTIVE COHESION (PSF) | ϕ' (DEGREE) | | | | | |
| 1 | Stiff Fat Clay | 0 ^b | 33.5 | 118 | 1,500 | -- | -- | 26 | 0.007 | 500 | Soft Clay | 800 | -- |
| 2 | Medium Dense Sandy Silts | 33.5 | 43.5 | 122 | -- | 32 | -- | 32 | -- | 20 | Sand | -- | 7 |
| 3 | Medium Dense Sands | 43.5 | 63.5 | 122 | -- | 35 | -- | 35 | -- | 90 | | N/A ^e | N/A ^e |
| 4 | Medium Dense Sands | 63.5 | 83.5 | 122 | -- | 33 | -- | 33 | -- | 60 | | N/A ^e | N/A ^e |
| 5 | Dense Sands | 83.5 | 100 | 125 | -- | 35 | -- | 35 | -- | 125 | | N/A ^e | N/A ^e |

Note: Groundwater elevation assumed at El. 188 based on the water levels encountered in the borings. The effective unit weight should be used below the groundwater level. Subtract the density of water (62.4 pounds per cubic foot) from the total unit weight to calculate the effective unit weight.

^a Depth in reference to ground surface at boring locations.

^b Zero depth as measured at top of boring.

^c Pounds per cubic inch.

^d For lateral load analysis only.

^e Liquefaction not anticipated in these layers.

| BRIDGE 2 OVER TYRONZA RIVER – BENT 2 (BORING B2-3) | | | | | | | | | | | | | |
|---|------------------------|--|------------------|-------------------------|---------------------------|-----------------|--------------------------|------------------|--------------------------------------|--|------------------|--|--------------------------|
| APPROXIMATE AVERAGE GROUND SURFACE ELEVATION = EL 193 | | | | | | | | | | | | | |
| ZONE | SOIL TYPES | DEPTH ^a (feet from ground surface) | | TOTAL UNIT WEIGHT (PCF) | SHEAR STRENGTH PARAMETERS | | | | LATERAL LOAD PARAMETERS ^d | | | LIQUEFACTION SHEAR STRENGTH PARAMETERS | |
| | | FROM | TO | | UNDRAINED (SHORT TERM) | | DRAINED (LONG TERM) | | SOIL STRAIN, E ₅₀ | STATIC SOIL MODULUS (PCI) ^c | LPILE SOIL MODEL | RESIDUAL COHESION (PSF) | RESIDUAL ϕ (DEGREE) |
| | | | | | COHESION (PSF) | ϕ (DEGREE) | EFFECTIVE COHESION (PSF) | ϕ' (DEGREE) | | | | | |
| 1 | Medium Stiff Lean Clay | 0 ^b | 3.5 | 118 | 600 | -- | -- | 26 | 0.01 | 100 | Soft Clay | 400 | -- |
| 2 | Dense Silty Sands | 3.5 | 38.5 | 122 | -- | 35 | -- | 35 | -- | 90 | Sand | N/A ^e | N/A ^e |
| 3 | Loose Sands | 38.5 | 43.5 | 122 | -- | 32 | -- | 32 | -- | 20 | | -- | 7 |
| 4 | Dense Sands | 43.5 | 58.5 | 122 | -- | 34 | -- | 34 | -- | 60 | | N/A ^e | N/A ^e |
| 5 | Very Dense Sands | 58.5 | 100 ^f | 125 | -- | 36 | -- | 36 | -- | 125 | | N/A ^e | N/A ^e |

Note: Groundwater elevation assumed at El. 188 based on the water levels encountered in the borings. The effective unit weight should be used below the groundwater level.

Subtract the density of water (62.4 pounds per cubic foot) from the total unit weight to calculate the effective unit weight.

^a Depth in reference to ground surface at boring locations.

^b Zero depth as measured at top of boring.

^c Pounds per cubic inch.

^d For lateral load analysis only.

^e Liquefaction not anticipated in these layers.

^f Boring B2-3 terminated at 70 feet. Soil layer assumed to extend to 100 feet.

| BRIDGE 2 OVER TYRONZA RIVER – BENT 3 (BORING B2-4) | | | | | | | | | | | | | |
|--|--------------------|--|------|-------------------------|---------------------------|-----------------|--------------------------|------------------|--------------------------------------|--|------------------|--|--------------------------|
| APPROXIMATE GROUND SURFACE ELEVATION = EL 204 | | | | | | | | | | | | | |
| ZONE | SOIL TYPES | DEPTH ^a (feet from ground surface) | | TOTAL UNIT WEIGHT (PCF) | SHEAR STRENGTH PARAMETERS | | | | LATERAL LOAD PARAMETERS ^d | | | LIQUEFACTION SHEAR STRENGTH PARAMETERS | |
| | | FROM | TO | | UNDRAINED (SHORT TERM) | | DRAINED (LONG TERM) | | SOIL STRAIN, E ₅₀ | STATIC SOIL MODULUS (PCI) ^c | LPILE SOIL MODEL | RESIDUAL COHESION (PSF) | RESIDUAL ϕ (DEGREE) |
| | | | | | COHESION (PSF) | ϕ (DEGREE) | EFFECTIVE COHESION (PSF) | ϕ' (DEGREE) | | | | | |
| 1 | Soft Lean Clay | 0 ^b | 18.5 | 118 | 600 | -- | -- | 26 | 0.01 | 100 | Soft Clay | 400 | -- |
| 2 | Loose Clayey Sands | 18.5 | 23.5 | 122 | -- | 32 | -- | 32 | -- | 20 | Sand | -- | 7 |
| 3 | Medium Dense Sands | 23.5 | 53.5 | 122 | -- | 35 | -- | 35 | -- | 90 | | N/A ^e | N/A ^e |
| 4 | Dense Sands | 53.5 | 78.5 | 122 | -- | 34 | -- | 34 | -- | 60 | | N/A ^e | N/A ^e |
| 5 | Very Dense Sands | 78.5 | 100 | 125 | -- | 36 | -- | 36 | -- | 125 | | N/A ^e | N/A ^e |

Note: Groundwater elevation assumed at El. 188 based on the water levels encountered in the borings. The effective unit weight should be used below the groundwater level. Subtract the density of water (62.4 pounds per cubic foot) from the total unit weight to calculate the effective unit weight.

^a Depth in reference to ground surface at boring locations.

^b Zero depth as measured at top of boring.

^c Pounds per cubic inch.

^d For lateral load analysis only.

^e Liquefaction not anticipated in these layers.

| BRIDGE 2 OVER TYRONZA RIVER – BENT 4 (EAST ABUTMENT; BORING B2-5) | | | | | | | | | | | | | |
|---|-----------------------------|--|------|-------------------------|---------------------------|-----------------|--------------------------|------------------|--------------------------------------|--|------------------|--|--------------------------|
| APPROXIMATE GROUND SURFACE ELEVATION = EL 222 | | | | | | | | | | | | | |
| ZONE | SOIL TYPES | DEPTH ^a (feet from ground surface) | | TOTAL UNIT WEIGHT (PCF) | SHEAR STRENGTH PARAMETERS | | | | LATERAL LOAD PARAMETERS ^d | | | LIQUEFACTION SHEAR STRENGTH PARAMETERS | |
| | | FROM | TO | | UNDRAINED (SHORT TERM) | | DRAINED (LONG TERM) | | SOIL STRAIN, E ₅₀ | STATIC SOIL MODULUS (PCI) ^c | LPILE SOIL MODEL | RESIDUAL COHESION (PSF) | RESIDUAL ϕ (DEGREE) |
| | | | | | COHESION (PSF) | ϕ (DEGREE) | EFFECTIVE COHESION (PSF) | ϕ' (DEGREE) | | | | | |
| 1 | Stiff/Medium Stiff Fat Clay | 0 ^b | 38.5 | 118 | 1,500 | -- | -- | 26 | 0.007 | 500 | Soft Clay | N/A ^e | N/A ^e |
| 2 | Loose Clayey Sand | 38.5 | 68.5 | 122 | -- | 35 | -- | 35 | -- | 90 | Sand | N/A ^e | N/A ^e |
| 3 | Medium Dense Sand | 68.5 | 100 | 122 | -- | 34 | -- | 34 | -- | 60 | | N/A ^e | N/A ^e |

Note: Groundwater elevation assumed at El. 188 based on the water levels encountered in the borings. The effective unit weight should be used below the groundwater level.

Subtract the density of water (62.4 pounds per cubic foot) from the total unit weight to calculate the effective unit weight.

^a Depth in reference to ground surface at boring locations.

^b Zero depth as measured at top of boring.

^c Pounds per cubic inch.

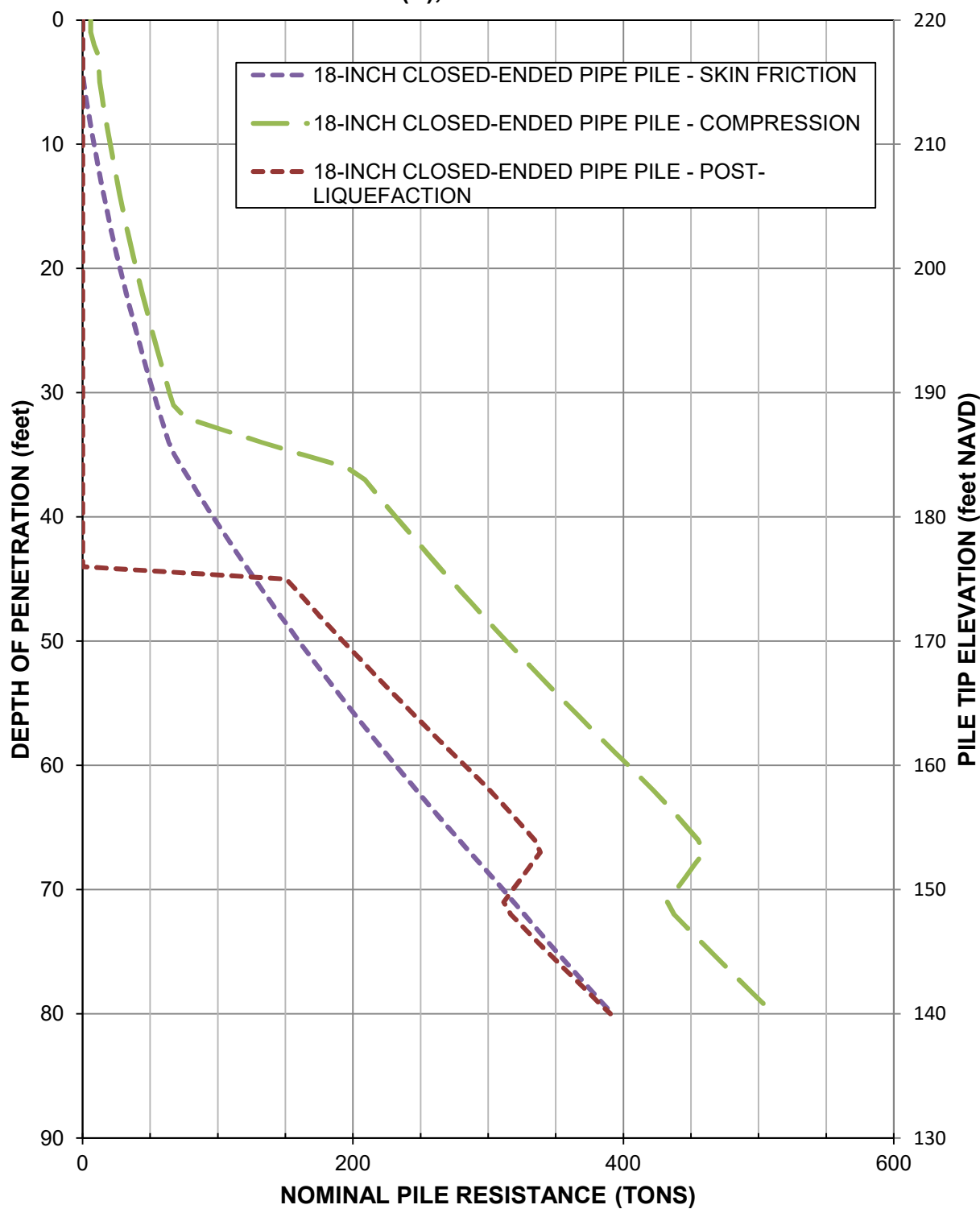
^d For lateral load analysis only.

^e Liquefaction not anticipated in upper 50 feet of Boring B2-5.

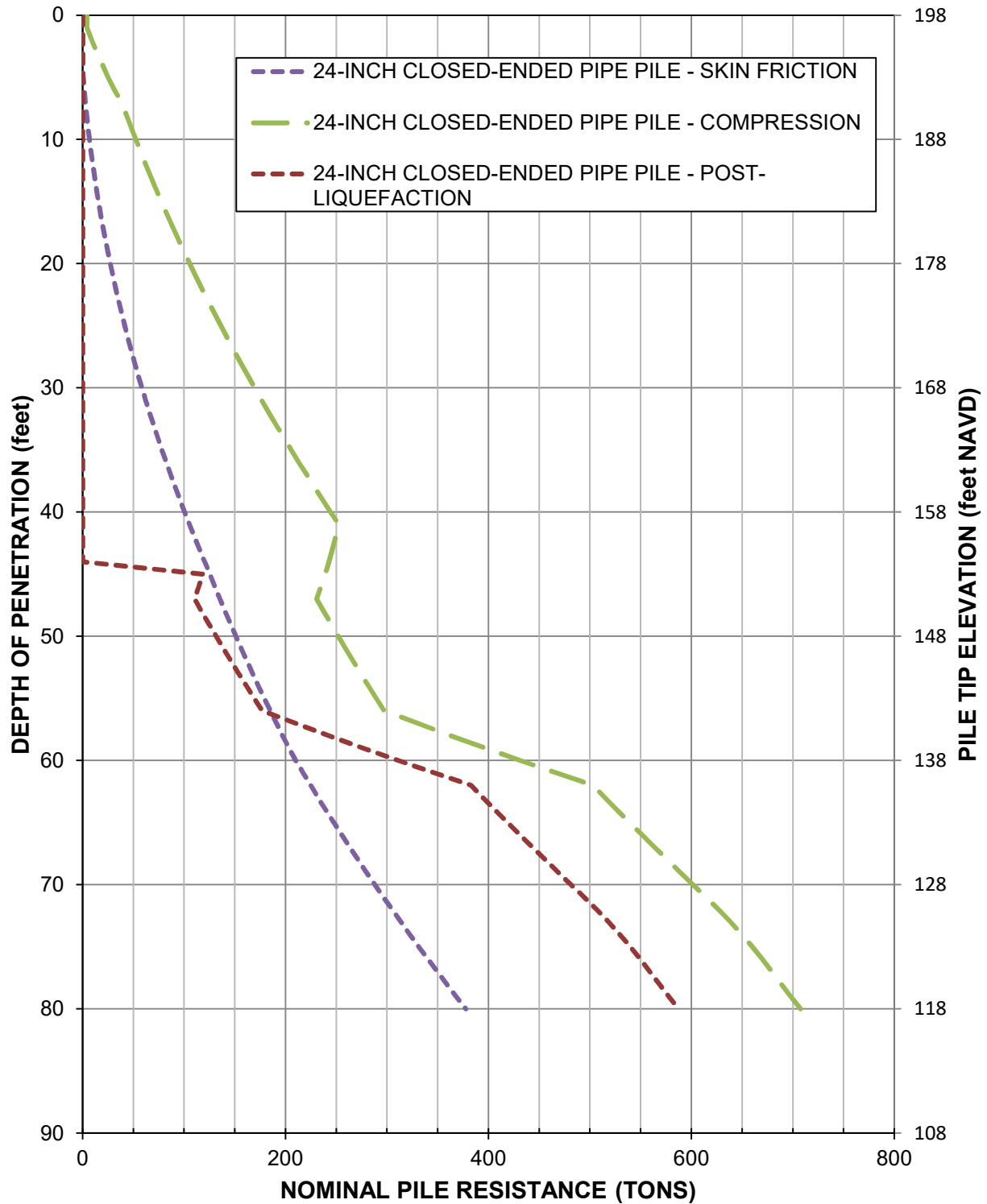


APPENDIX J – NOMINAL RESISTANCE CURVES

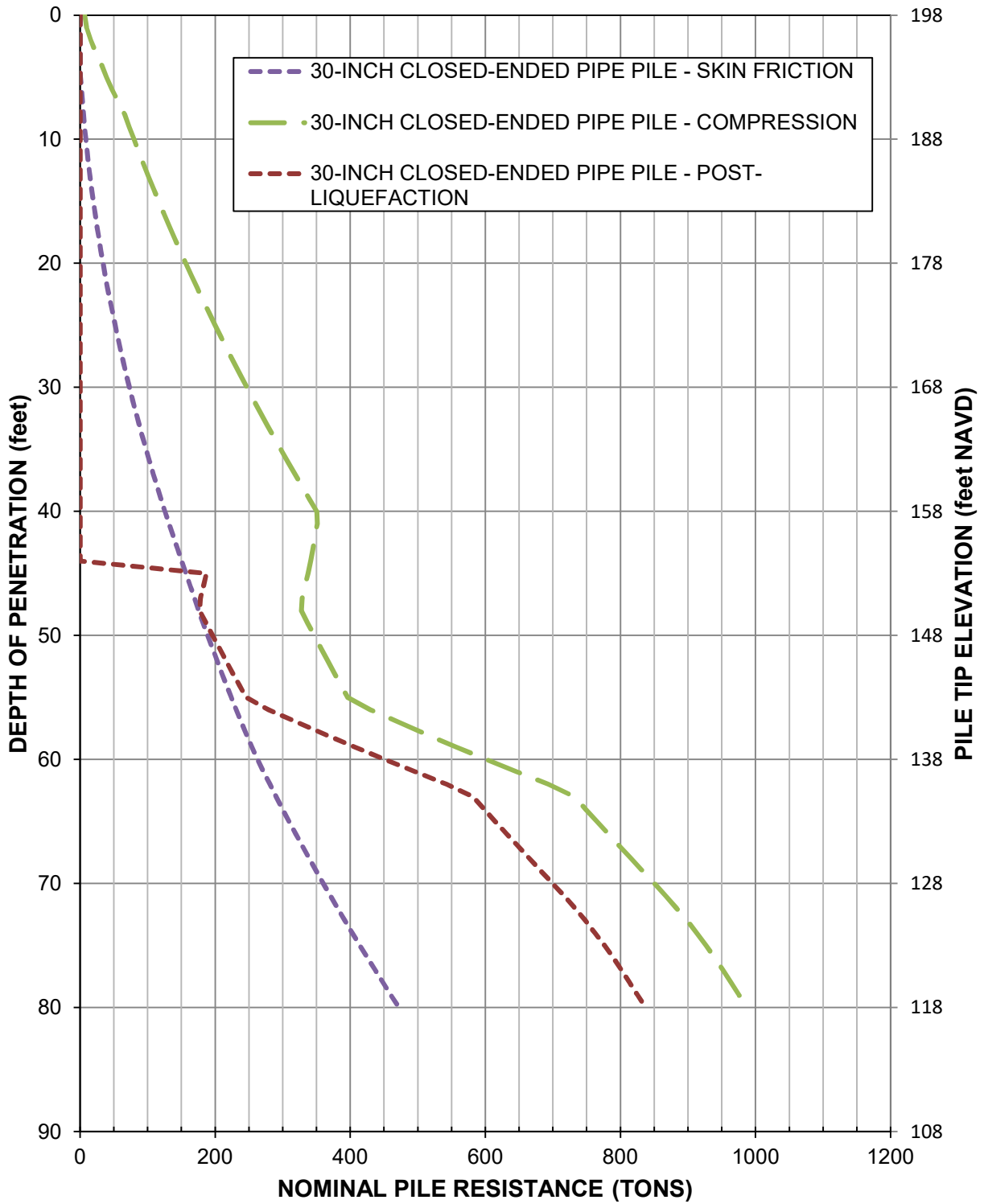
**NOMINAL RESISTANCE CURVES
 18-INCH CLOSED-ENDED PIPE PILE
 ARDOT TYRONZA RIVER & DITCH NO. 6 - BRIDGE 2
 STR. & APPRS. (S), POINSETT COUNTY - BENT 1**



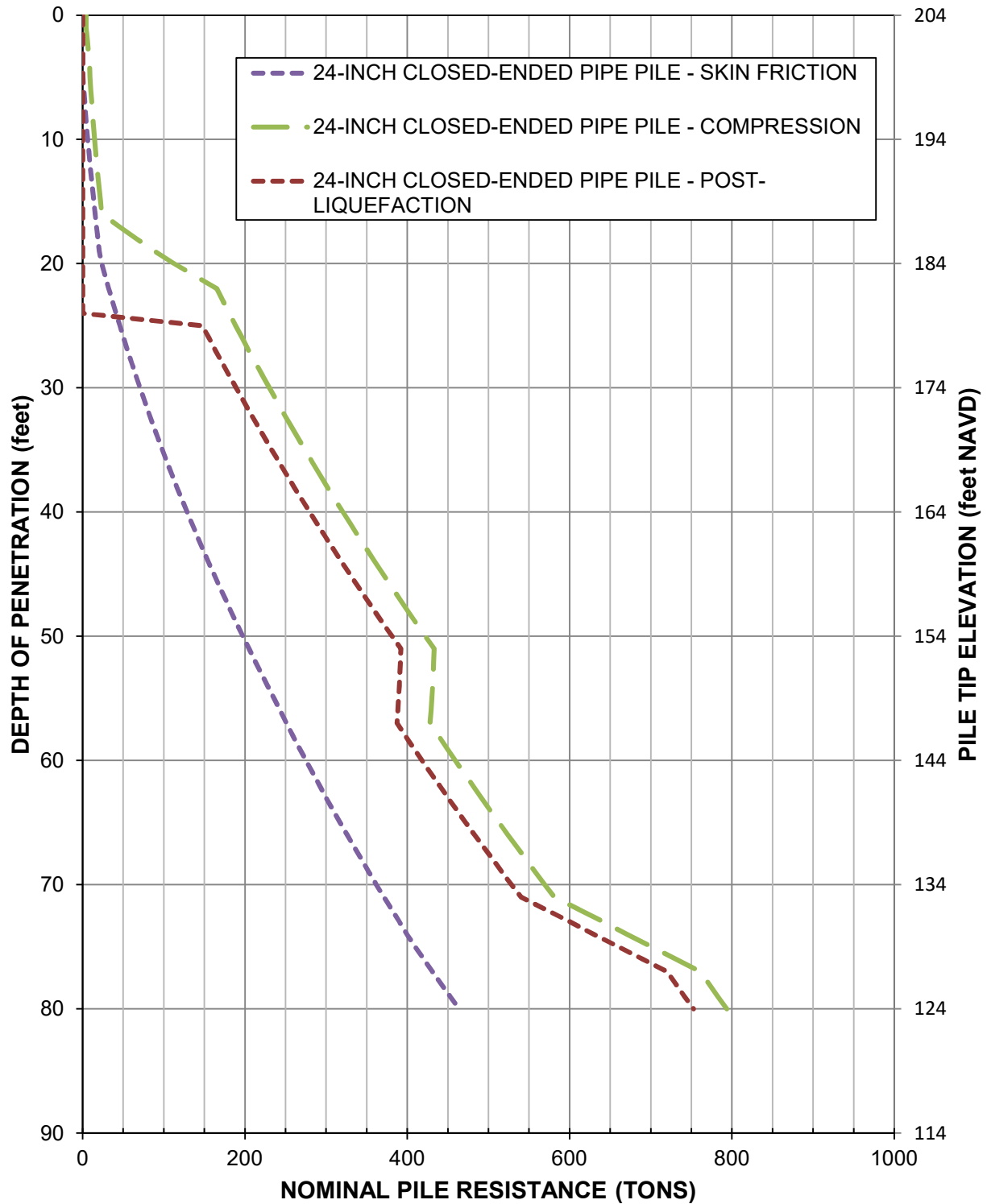
**NOMINAL RESISTANCE CURVES
 24-INCH CLOSED-ENDED PIPE PILE
 ARDOT TYRONZA RIVER & DITCH NO. 6 - BRIDGE 2
 STR. & APPRS. (S), POINSETT COUNTY - BENT 2**



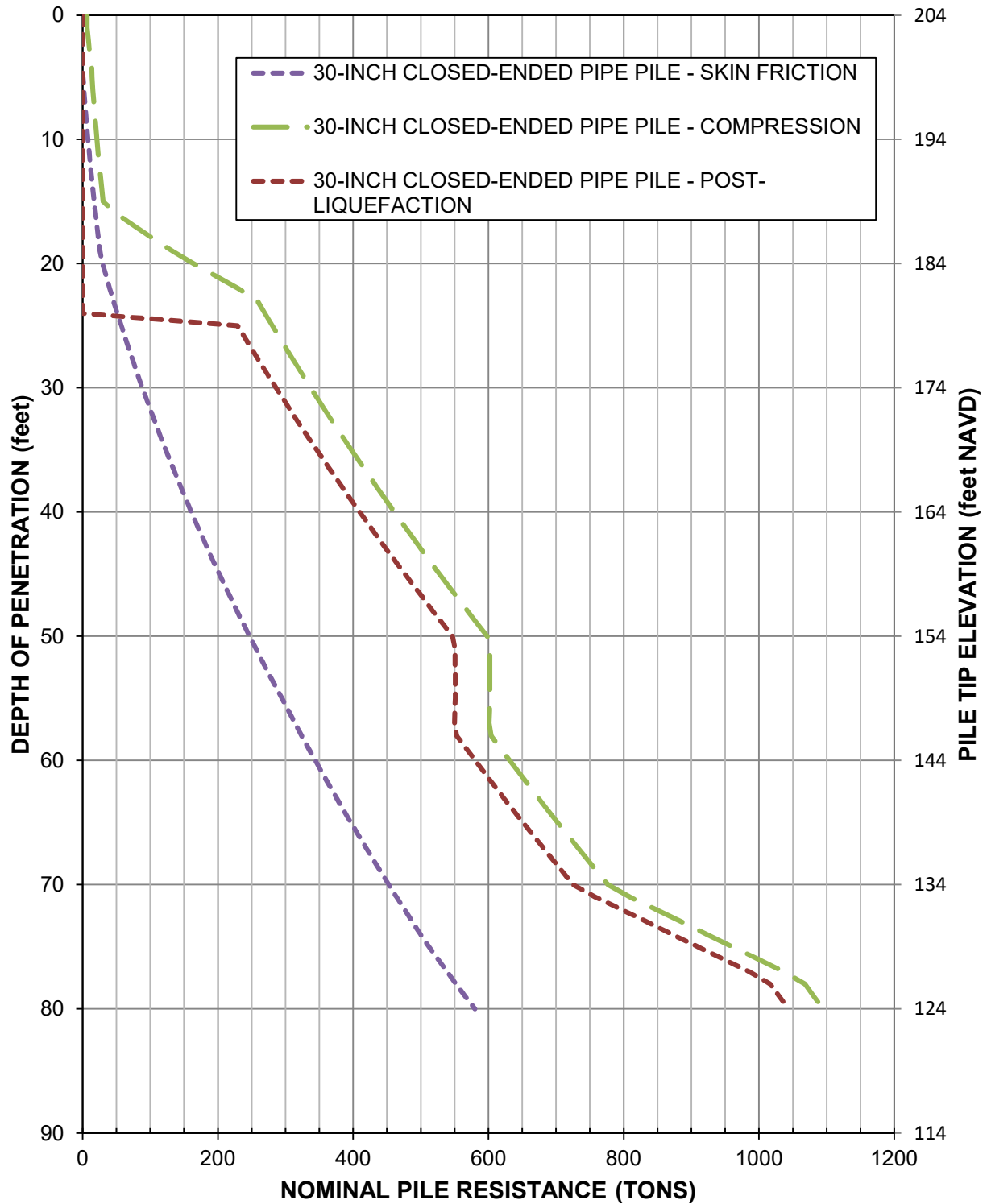
**NOMINAL RESISTANCE CURVES
 30-INCH CLOSED-ENDED PIPE PILE
 ARDOT TYRONZA RIVER & DITCH NO. 6 - BRIDGE 2
 STR. & APPRS. (S), POINSETT COUNTY - BENT 2**



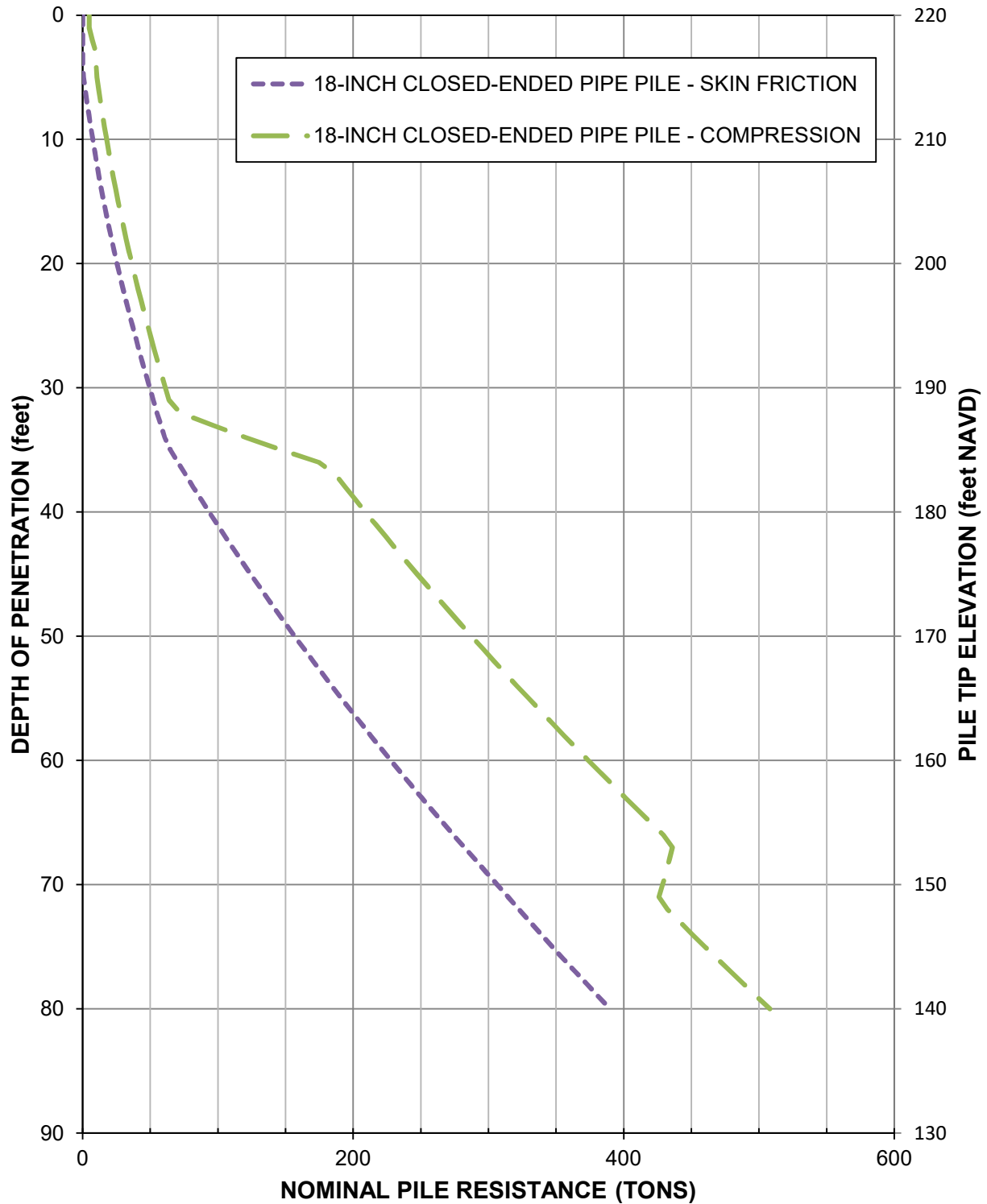
**NOMINAL RESISTANCE CURVES
 24-INCH CLOSED-ENDED PIPE PILE
 ARDOT TYRONZA RIVER & DITCH NO. 6 - BRIDGE 2
 STR. & APPRS. (S), POINSETT COUNTY - BENT 3**



**NOMINAL RESISTANCE CURVES
 30-INCH CLOSED-ENDED PIPE PILE
 ARDOT TYRONZA RIVER & DITCH NO. 6 - BRIDGE 2
 STR. & APPRS. (S), POINSETT COUNTY - BENT 3**



**NOMINAL RESISTANCE CURVES
 18-INCH CLOSED-ENDED PIPE PILE
 ARDOT TYRONZA RIVER & DITCH NO. 6 - BRIDGE 2
 STR. & APPRS. (S), POINSETT COUNTY - BENT 4**



GEOTECHNOLOGY

A UES Company



**GEOTECHNICAL REPORT
TYRONZA RIVER & DITCH No. 6
STRS. & APPRS. (S)
BRIDGE 3 – I-555 SERVICE ROAD
BRIDGE OVER DITCH No. 6
POINSETT COUNTY, ARKANSAS**

**ARKANSAS DEPARTMENT OF TRANSPORTATION
STATE PROJECT No. 101017**

Prepared for:
**ARKANSAS DEPARTMENT OF TRANSPORTATION (ARDOT)
LITTLE ROCK, ARKANSAS**

Prepared by:
**GEOTECHNOLOGY, LLC
MEMPHIS, TENNESSEE**

Date:
FEBRUARY 9, 2024

Geotechnology Project No.:
J042992.01

**SAFETY
QUALITY
INTEGRITY
PARTNERSHIP
OPPORTUNITY
RESPONSIVENESS**



February 9, 2024

Mr. Paul Tierney
Geotechnical Engineer
Arkansas Department of Transportation (ARDOT)
PO Box 2261
Little Rock, Arkansas 72203

Re: Geotechnical Report
Tyronza River & Ditch No. 6 Strs. & Apprs. (S)
Bridge 3 - I-555 Service Road Bridge Over Ditch No. 6
Poinsett County, Arkansas
ARDOT Project No. 101017
Geotechnology Project No. J042992.01

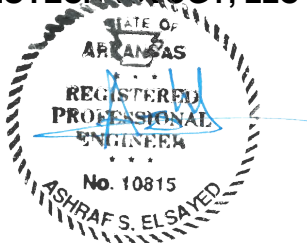
Dear Mr. Tierney:

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, conclusions and/or recommendations, and support 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



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

Jacob Monroe, P.E.
Project Engineer

DFB/ASE/JDM:jdm/dfb

Copies submitted: Client (email)



TABLE OF CONTENTS

1.0 Scope of Services..... 1

2.0 General Information 1

 Planned Construction 1

 Existing Topography and Proposed Grades 2

 Drainage 2

 Geology 2

3.0 Geotechnical Exploration 2

 Cone Penetration Testing 2

 Drilling and Sampling 2

4.0 Laboratory Review and Testing 3

5.0 Subsurface Conditions 4

 Subgrade Materials 4

 Groundwater 4

6.0 Engineering Evaluation, Analysis, and Recommendations 5

 Site Preparation and Earthwork 5

 Seismic Considerations 6

 AASHTO LRFD 2020 Seismic Site Classification and Seismic Design
 Parameters 6

 Site-Specific Ground Motion Assessment 7

 Liquefaction and Dynamic Settlement 7

 Approach Embankment Settlement 9

 Global Stability 10

 Deep Foundations 11

 Corrosion Potential 17

7.0 Recommended Additional Services 18

8.0 Limitations 18

Appendices

- Appendix A – Important Information About This Geotechnical-Engineering Report
 - Appendix B – Figures
 - Appendix C – Boring Information
 - Appendix D – CPT3-1 Sounding Results
 - Appendix E – Laboratory Test Data
 - Appendix F – Site-Specific Seismic Study
 - Appendix G – AASHTO and USCS Classifications
 - Appendix H – Global Stability Analysis Results
 - Appendix I – Soil Parameters for Synthetic Profiles
 - Appendix J –Nominal Axial Resistance Curves
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LIST OF TABLES

| | |
|---|----|
| Table 1. Field Tests and Measurements..... | 3 |
| Table 2. Summary of Laboratory Tests and Methods. | 4 |
| Table 3. Seismic Design Parameters (7% Probability of Exceedance in 75 years)..... | 7 |
| Table 4. Design Acceleration Parameters (7% Probability of Exceedance in 75 Years)..... | 7 |
| Table 5. Results of Liquefaction Analyses. | 8 |
| Table 6. Summary of Approach Embankment Settlement. | 9 |
| Table 7. Results of Slope Stability Analyses. | 11 |
| Table 8. Static Nominal Axial Resistance of Driven Closed-Ended Pipe Piles..... | 12 |
| Table 9. Post-Liquefaction Nominal Axial Resistance of Driven Closed-Ended Pipe Piles. | 13 |
| Table 10. Resistance Factors Based on Static Analysis Methods. | 14 |
| Table 11. Resistance Factors for Driven Piles. | 14 |
| Table 12. Results of pH and Soil Resistivity Testing..... | 17 |

**GEOTECHNICAL REPORT
TYRONZA RIVER & DITCH NO. 6 STRS. & APPRS. (S)
BRIDGE 3 – I-555 SERVICE ROAD BRIDGE OVER DITCH NO. 6
POINSETT COUNTY, ARKANSAS
February 9, 2024 | Geotechnology Project No. J042992.01**

1.0 SCOPE OF SERVICES

Presented in this report are the results of the geotechnical exploration and recommendations for design and construction of the proposed I-555 Service Road Bridge Over Ditch No. 6 in Poinsett County, Arkansas. The project location is shown on Figure 1 included in Appendix B.

Recommendations presented in this report are based on the geology, provided plans and project information, and the results of the geotechnical exploration. Results of the borings, Cone Penetration Test (CPT) sounding, in-situ testing, sampling, and laboratory testing are included in the report. A total of four borings and one seismic CPT sounding were performed in the vicinity of the site as shown on Figure 2 included in Appendix B. The boring logs, CPT sounding plot, and field and laboratory test results are enclosed. The collected data have been analyzed and the physical properties of the in-situ soils summarized. General site conditions are discussed, along with recommendations for subgrade preparation. 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

Planned Construction

The preliminary bridge layout¹ depicts Bridge 3 as a three-span structure, approximately 136 feet in length and 32.5 feet in width out-to-out. Bridge 3 will extend from Sta. 626+49.41 at the west abutment to Sta. 627+85.47 at the east abutment. We understand from ARDOT that bridge foundations will be closed-ended, pipe piles: 14 or 16 inches in diameter at the abutments, 18 or 24 inches in diameter at the interior bents.

Abutment slopes are anticipated to be two horizontal units for every vertical unit (2H:1V) and side slopes are anticipated to be 3H:1V. Approximately 15 feet of fill will be required to reach design grades along the centerline of the proposed road. However, cut will be required into a round hill at the north end of the west abutment.

¹ *Arkansas Department of Transportation Layout of Bridge, I-555 Service Road over Ditch No. 6, Tyronza River & Ditch No. 6 Strs. & Apprs. (S), Poinsett County, dated September 13, 2022.*



Existing Topography and Proposed Grades

The I-555 Service Road will be new construction running along the north side of I-555 in Poinsett County, Arkansas. Based on the provided bridge layout, existing topography along the centerline of the proposed roadway and bridge abutments range from approximately El 210 to El 214. There is a round hill up to El 222 north of the west abutment. The proposed roadway and bridge will be constructed at approximately El 225. Ditch No. 6 is at approximately El 207.

Drainage

Ditch No. 6 drains to the south as part of the Lower St. Francis River Watershed. The St. Francis River flows south and joins the Mississippi River north of Helena, Arkansas.

Geology

Poinsett County is in northeast Arkansas in the Mississippi Embayment. The Mississippi Embayment is a trough-like depression dipping southward along an axis approximately following the Mississippi River. Site geology generally consists of Holocene-aged alluvial deposits of clay and silt underlain by fine- and medium-grained sand.

3.0 GEOTECHNICAL EXPLORATION

Cone Penetration Testing

One seismic cone penetration test (CPT) sounding was performed in the proposed bridge western approach for continuous soil data collection and to measure the average shear wave velocity. The CPT sounding was performed to a depth of approximately 100 feet.

CPT3-1 was advanced using a 20-ton, track-mounted Vertek direct-push rig on June 13, 2023. The data were collected using a Vertek 15 square-centimeter end area, seismic piezometric cone with a u_2 pore pressure location (behind the cone) following the procedures outlined in ASTM D3441 and D5778. A plot of the CPT measurements is presented in Appendix D along with interpreted soil behavior types. Seismic CPT tests were performed in CPT3-1 at approximately 1-meter intervals to collect shear wave velocity data. A plot of the shear wave velocity profile is also presented in Appendix D.

Drilling and Sampling

A total of four borings were drilled to an approximate depth of 100 feet at select locations in the proposed bridge approaches. The borings were drilled from July 20 through 27, 2023 using a rotary drill rig (Diedrich D-50), hollow-stem augers, and wet rotary methods. Sampling procedures included Standard Penetration Test (SPT) and thin-wall (Shelby) tube methods. SPT's were conducted at 2.5, 5, and 10-foot depth intervals using an automatic hammer. Thin-walled Shelby tube samples were collected in cohesive soils at selected depths. Groundwater observations were made during drilling operations.

The collected samples were visually examined by field staff and transported to our laboratory for further evaluation and testing. The samples were examined in the laboratory by a geotechnical



professional who prepared descriptive logs of the materials encountered. The boring logs are presented in Appendix C along with an explanation of the terms and symbols used on the boring logs. Included on each boring log are elevation data estimated from the provided plans. Included 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 |

The boring logs and CPT sounding plot represent conditions observed at the time of exploration and have been edited to incorporate results of the laboratory tests. Unless noted on the boring logs, the lines designating the changes between various strata represent approximate boundaries. The transition between materials could be gradual or 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.

The boring logs and CPT sounding plot 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. A list of the laboratory tests and corresponding test method standards are presented in Table 2. Most of the laboratory test results are presented on the boring logs in Appendix C. Atterberg limits, grain size analyses, unconsolidated-undrained triaxial compression (UU), one-dimension consolidation, pH, and soil resistivity test results are also provided in Appendix E.



Table 2. Summary of Laboratory Tests and Methods.

| Laboratory Test | ASTM | AASHTO |
|--|--------|--------|
| Moisture Content | D 2216 | T 265 |
| Atterberg Limits | D 4318 | T 98 |
| Grain Size Analysis | D 422 | T 88 |
| Percent Finer Than No. 200 Sieve | D 1140 | T 11 |
| Unconsolidated-Undrained Triaxial Compression | D 2850 | T 296 |
| One-Dimensional Consolidation | D 2435 | T 216 |
| Direct Shearing of Soils under Consolidated-Drained Conditions | D 3080 | T 236 |
| pH of Soil | D 4972 | T 289 |
| Soil Electrical Resistivity | G 57 | T 288 |

The boring logs and CPT plot were prepared by a project geotechnical engineer from the field logs, visual classification of the soil samples in the laboratory, 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 strata could be abrupt or gradual.

5.0 SUBSURFACE CONDITIONS

The stratigraphy in the borings generally consisted of fat clay soils underlain by silty sand and sand. A generalized subsurface profile from west to east along the proposed bridge is shown on Figure 3.

Subgrade Materials

The native soils were classified as fat clay (CH), silt (ML), silty sand (SM), and poorly graded sand (SP-SC, SP-SM, SP) by the Unified Soil Classification System (USCS) and A-7-5, A-7-6, A-4, A-2-7, A-2-4, and A-1-b by the AASHTO classification method. Based on N-values, the fine-grained soils were soft to medium stiff, and the coarse-grained soils were loose to very dense.

Shear strengths from UU test in the fine-grained soils range from 560 to 1,300 pounds per square foot (psf). Moisture contents in the fine-grained soil samples ranged from 18 to 54 percent. Liquid limits (LL) and plasticity indices (PI) of tested samples ranged from 25 to 116 percent and 3 to 86 percent, respectively.

The boring logs and CPT sounding plot, with more detailed descriptions, are included in Appendix C and Appendix D, respectively. Laboratory testing used to determine AASHTO classifications are presented in Appendix F.

Groundwater

Groundwater was observed during drilling in Boring B3-3 at an approximate depth of 28 feet, which correlates to approximately El 189. Groundwater was not observed in the other borings; however, groundwater levels might have been masked by the mud rotary drilling method used in the borings.



Groundwater was interpreted in CPT3-1 at approximately 22 feet below the ground surface, which correlates to approximately El 190. Groundwater levels vary over time because of seasonal variations in precipitation, water levels in Ditch No. 6, or other factors not evident at the time of exploration.

6.0 ENGINEERING EVALUATION, ANALYSIS, AND RECOMMENDATIONS

Site Preparation and Earthwork

The following procedures are recommended for site preparation in cut and fill areas. These recommendations do not supersede ARDOT standards and specifications. Site preparation and compaction requirements must conform to the latest ARDOT standards.

Site Preparation. In general, cut areas and areas to receive new fill should be stripped of existing pavements, topsoil, vegetation, and other deleterious materials. Topsoil should be placed in landscape areas or disposed of off-site. Vegetation and tree roots should be over-excavated.

The exposed subgrade should be proof-rolled using a tandem axle dump truck loaded to approximately 20,000 pounds per axle (or equivalent proof-rolling equipment). Soft areas that develop should be over-excavated and backfilled with select fill, which is defined as soil conforming to A-4 or better material and compacted to the unit weights specified in subsequent paragraphs.

Side Slopes. Existing slopes steeper than 4H:1V should be benched prior to placing new fill. Slope ratios of 2H:1V are proposed for abutment slopes. Slope ratios of 3H:1V or flatter are recommended for all cut and fill side slopes along the proposed alignment.

Cut Areas. Based on the preliminary plans, minor cuts will be made north of the west abutment. After excavation, the top 6 inches of the resulting subgrade should be compacted to a minimum of 95% of the maximum dry unit weight as determined by a standard Proctor test (ASTM D698/AASHTO T 99). Subgrades supporting pavement should be compacted to 98% of the maximum unit weight as determined by the standard Proctor test.

Fill Materials. Fill material should consist of natural soils classified as AASHTO A-6 or better², and should meet the minimum requirements set forth in ARDOT's Special Provision³ (SP) dated March 1, 2022. Soils classified as AASHTO A-4 or better are considered to be select fill. Fine-grained "silt-clay" soils (A-4 through A-6) should have a maximum LL of 45 and a PI between 8 and 20 percent. Coarse-grained "sandy" soils used for embankment fills should have a minimum PI of 5 to lower potential for erosion. Fill materials should be free from organic matter, debris, or other deleterious materials, and have a maximum particle size of 2 inches.

² A-6 soils or better as determined by ARDOT.

³ Special Provision "Compacted Embankment", developed by ARDOT, dated March 1, 2022.



Fill and Backfill Placement. Fill and backfill should be placed in level lifts, up to 8 inches in loose thickness. For fill and backfill exhibiting a well-defined moisture-density relationship, each lift should be moisture-conditioned to within $\pm 2\%$ of the optimum moisture content and compacted with a sheepsfoot roller or self-propelled compactor to a minimum of 98% of the maximum dry unit weight as determined by the standard Proctor test. Moisture-conditioning can include: aeration and drying of wetter soils; wetting drier soils; and/or mixing wetter and drier soils into a uniform blend. The upper 3 feet of soil beneath the base of pavement should be compacted to 98% of the maximum unit weight as determined by the standard Proctor test.

For fill and backfill that do not exhibit a well-defined moisture-density relationship, each lift should be compacted to a 70% of the minimum relative density as evaluated from the maximum and minimum index densities measured by ASTM D4253 and D4254, respectively. The upper 3 feet of soil beneath the base of pavement should be compacted to 75% of the minimum relative density.

Fill Placement on Slopes. In areas that require fill to be placed on slopes, benching of existing slopes should be performed during placement of new fill. Fill on the sloped areas should begin from the toe of the slope and proceed upward, placing new fill on horizontal benches. Bench shelves should be 8 to 10 feet wide, and bench faces should be 1 to 2 feet in height. Fill lifts should be keyed into the slope to reduce the potential of a slip plane between the new fill and existing soils. Fill slopes should be constructed by extending the compacted fill beyond the planned profile of the slope and then trimming the slope to the desired configuration.

Moisture Considerations. Maintaining the moisture content of bearing and subgrade soils within the acceptable range is important during and after construction. Silty and clayey subgrade soils should not be allowed to become wet or dry during or after construction, and measures should be taken to hinder water from ponding on these soils. Positive drainage should be established to promote drainage of surface water away from the roadway.

Seismic Considerations

Earthquake Risk. The project area is in the vicinity of the New Madrid Seismic Zone (NMSZ). The NMSZ is 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, centered near New Madrid, Missouri. Three strong earthquakes occurred over the next three months 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 bridge and approaches will be designed in accordance with the AASHTO publication “LRFD Bridge Design Specifications”, eighth edition (2020).

AASHTO LRFD 2020 Seismic Site Classification and Seismic Design Parameters

Seismic Design Parameters. Seismic design parameters based on a seismic hazard with 7% probability of exceedance in 75 years and field and laboratory testing is presented in Table 3.



Table 3. Seismic Design Parameters (7% Probability of Exceedance in 75 years).

| Latitude 35.4793688°N/Longitude 90.348355°W | | |
|---|-----------------------|---|
| Category/ Parameter | Designation/ Value | Reference |
| Seismic Zone | 4 | AASHTO LRFD 2020 Table 3.10.6-1 |
| Seismic Site Class | D | AASHTO LRFD 2020 Table 3.10.3.1-1 |
| S_s | 1.714g | Ground motion parameters obtained from a computer program supplied with the AASHTO Guideline for the Seismic Design of Highway Bridges (2020) using the indicated latitude and coordinates of the project site and the seismic site class based on boring data. |
| S_1 | 0.452g | |
| F_a | 1.000 | |
| F_v | 1.548 | |
| F_{PGA} | 1.000 | |
| t_s | 0.408 | |
| t_0 | 0.082 | |
| S_{DS} | 1.714g | |
| S_{D1} | 0.700g | |
| PGA | 0.968g | |
| As | 0.968g | |

Site-Specific Ground Motion Assessment

A site-specific study was performed for the project site to develop a site-specific seismic design response spectrum. The process included 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 CPT1-1 of 785 feet per second as shown on the Shear Wave Velocity Profile in Appendix D.

The results of the shear wave velocity measurements indicate that the site is a Site Class D, “stiff soil”, profile based on a $V_{S,100}$ of 785 feet per second. According to the results of the site-specific seismic study, the recommended site-specific design accelerations are presented in Table 4.

Table 4. Design Acceleration Parameters (7% Probability of Exceedance in 75 Years).

| Parameter | Value |
|-----------|--------|
| S_{DS} | 1.219g |
| S_{D1} | 0.999g |
| MCE_G | 0.645g |

Liquefaction and Dynamic Settlement

An analysis was performed to evaluate the liquefaction and dynamic settlement potential at the site. Both field and laboratory data were used to perform a spreadsheet analysis following the



deterministic method by Idriss and Boulanger (2014). The field measurements included the depth of the water table, SPT N-values, and information collected in CPT3-1. The laboratory data included USCS classification and soil unit weight. An earthquake magnitude (M_w) of 7.7 with a probability of exceedance of 7% in 75 years was considered. A site peak ground acceleration of 0.645g was utilized as obtained from the site-specific seismic study. Groundwater was set at an elevation of approximately EI 190 as indicated in the CPT and borings.

Subsurface conditions (as characterized by the field and laboratory data) and earthquake characteristics were used to estimate the safety factors against liquefaction in each soil layer, as well as the associated dynamic settlement during the design seismic event. Based on the analysis, there is liquefaction potential at the site. The analysis results are presented in Table 5.

Table 5. Results of Liquefaction Analyses.

| Location | Total Depth of Boring (feet) | Liquefiable Zones ^a (feet below ground surface) | Estimated Dynamic Settlement (inches) | |
|----------|------------------------------|---|---------------------------------------|------------------------------------|
| | | | Upper 50 Feet | Total Depth of Boring/CPT Sounding |
| B3-1 | 100 | 38.5-38.5 48.5-53.5 | 2.5 | 3.5 |
| B3-2 | 100 | 23.5-38.5 | 3 | 3 |
| B3-3 | 100 | 28.5-33.5 43.5-48.5 | 2.5 | 3 |
| B3-4 | 100 | 28.5-33.5 38.5-48.5 53.5-58.5 | 3 | 4.5 |
| CPT3-1 | 100 | 22-23 25-26 28 29-38 41-49 52-55 57.5 | 6 | 6.5 |

^a Defined as zones with a factor of safety against liquefaction in the design earthquake of less than 1.0, as computed using the deterministic method by Idriss and Boulanger, 2014.

In general, based on the liquefaction analysis results, the silty sand layer from approximately EI 195 to EI 175 along the west approach and EI 194 to EI 168 along the east approach is susceptible to liquefaction when saturated. The CPT-based analysis of data from CPT3-1 indicates the liquefiable zone extends from approximate depths of 22 to 49 feet (approximately EI 192 to EI 163).

Please note that the current state of practice for liquefaction hazard assessment is based on what is known as “The Simplified Method” as introduced by Seed (1971) and subsequent modifications/revisions by others (Seed 1982, Idriss 1999, Youd 2001, and Idriss and Boulanger 2014, among others). The simplified method is based on observations and assessments of soil zones that either liquefied or did not liquefy in the upper 50 feet. Because of reported uncertainties in the literature, the occurrence of substantial liquefaction in relatively deep sand deposits below 50 feet is considered unlikely.

Driven piles supporting the bridge abutments would be subject to downdrag from settlement of the surrounding soil due to liquefaction. Calculated drag loads at each abutment are provided in Table 9.



Lateral Spreading. Lateral spreading is triggered and sustained by earthquake ground motions. Based on our seismic slope stability analyses, it is our professional opinion the potential for lateral spreading is low at the site. However, in the event of an earthquake, the soils that liquefy will have residual strengths which can have potentially destabilizing effects on overlying slopes. Post-liquefaction residual shear strength parameters are presented in Appendix I.

Approach Embankment Settlement

Settlement analyses of natural soils were performed to assess fill-induced settlement for the approaches. Based on the provided preliminary plans, up to approximately 15 feet of fill will be required at the bridge approaches to bring the site to design grade. For settlement analyses, we assumed cohesive, engineered fill will be used for the fill material, with a granular drainage layer placed at the base of the fill embankment. The results of the settlement due to fill placement are shown in Table 6. If grade changes require the placement of additional fill, Geotechnology should be contacted to perform additional settlement analyses for fill-induced settlement at the approaches.

Table 6. Summary of Approach Embankment Settlement.

| Location, Sta. | Maximum Fill Height (feet) | Estimated Settlement (inches) | | | Time (months) for Settlement <0.4 inch |
|-----------------------|----------------------------|-------------------------------|---------------------------|-------|--|
| | | Immediate | Long-Term (Consolidation) | Total | |
| West Abutment, 626+40 | 15 | 2.5 | 3.5 | 6 | 4 |
| East Abutment, 628+00 | 15 | 2.5 | 3.5 | 6 | 3 |

The results of the settlement analyses indicate immediate and long-term (primary consolidation) settlement at the bridge approaches. We anticipate the immediate settlement to occur shortly after fill placement, and practical completion of consolidation (less than 0.4 inches remaining) approximately 3 to 4 months after fill placement.

If the bridge foundations are installed while soil consolidation is in progress, downdrag on pile foundations could occur, as well as differential settlement of facing panels, coping, curbing, piping, and pavement distress. These effects can be mitigated by implementing a waiting period and monitoring subgrade settlement during and after fill placement. Alternatively, driven piles supporting the bridge abutments can be designed to withstand downdrag due to consolidation settlement. Calculated drag loads at each abutment are provided in Table 8.

Settlement Mitigation. Settlement of embankment fill placed for the approach embankments and abutments of the I-555 Service Road can be mitigated by undercutting compressible, cohesive soils and replacing it with compacted granular material. Settlement mitigation must include a monitoring program to determine when embankment settlement is effectively complete.



Analysis results indicate that over-excavating 5 and 6 feet of cohesive subgrade soils at the west and east abutments, respectively, and replacing it with compacted, granular material would reduce settlement to 0.4-inch or less, approximately one month after embankment fill placement. ARDOT Select Material (SM-1) (Section 302) or ARDOT Class 7 Aggregate Base Course (Section 303, Table 303-1) can be specified for backfilling the undercut. These materials should be compacted to at least 95 percent of the maximum dry unit weight from the modified Proctor compaction test (AASHTO T-180).

Replacement of the in-situ clayey soils with these materials will reduce the amount of predicted consolidation settlement and facilitate drainage of excess pore water pressure from the top of the clayey soils, generated by embankment fill placement. The undercut and granular backfill should extend a minimum of 5 feet beyond the toe of the spill slope and side slopes of the abutment, and at least 40 feet back from the abutments. The granular backfill must be drained to a deeper swale or storm drain.

Settlement Monitoring. The duration of subgrade settlement at the bridge approaches is difficult to predict due to variations in permeability and drainage of the different soil layers. Settlement monitoring can be utilized at the bridge abutments to determine when pile foundations and roadway components can be constructed. Settlement monitoring devices include settlement plates and riser pipes, hydrostatic level cells referenced to a fixed fluid reservoir, or horizontal inclinometers installed at the base of the fill prior to fill placement. Settlement plates are the simplest method but are prone to disturbance or destruction by grading equipment. Therefore, redundant settlement plate locations should be installed.

Settlement plates can be installed approximately 1-foot below the existing ground surface and extend in 5-foot calibrated increments as the height of embankment fill increases. To protect the riser pipes, fill should be hand-compacted within a 4-foot radius of each plate. A typical settlement plate detail is presented in Figure 4 in Appendix B. We recommend settlement plates be placed no further than 50 feet apart, with at least one in the deepest area of fill at the abutments. The project surveyor should be retained to monitor the settlement plate riser pipe. Settlement at the site should be measured twice weekly during fill placement and weekly after filling is completed.

Global Stability

Geotechnology performed stability analyses for deep-seated, global failure of bridge abutment slopes using the computer program SLIDE2. Short-term, long-term, and seismic conditions were considered using the Spencer method to compute factors of safety for the proposed slopes.

Calculated minimum factors of safety are summarized in the following table. Minimum required factors of safety for the proposed bridge were based on the ARDOT Minimum Acceptable Factors of Safety as provided by ARDOT using a seismic operational class of "Other". A pseudo-static seismic acceleration of 0.323g, corresponding to one-half the peak ground acceleration (per FHWA Publication HI-99-012) was utilized.



A groundwater elevation of approximately El 190, as measured in the borings and interpreted in CPT3-1, was utilized in the analyses. Section profiles with critical slip surfaces and utilized soil parameters are presented in Appendix H for the selected analyses. The analysis models did not consider the effect of foundation piles driven at the abutments or riprap placed on the abutment slopes that would provide additional restraining force to stabilize the slopes.

Table 7. Results of Slope Stability Analyses.

| Location, Applicable Borings | Description | Slope Height (ft.) | Calculated Factor of Safety | | |
|------------------------------|-------------|--------------------|----------------------------------|---------------------------------|------------------------|
| | | | Short-Term Static ^{a,c} | Long-Term Static ^{a,d} | Seismic ^{b,c} |
| West Abutment, B3-1, B3-2 | 2H:1V | 15 | 2.58 | 1.43 | 1.24 |
| | Fill Slope | | | | |
| East Abutment B3-3, B3-4 | 2H:1V | 15 | 2.42 | 1.43 | 1.14 |
| | Fill Slope | | | | |

^a Target factor of safety = 1.3, approximately equivalent to a global stability resistance factor = 0.75, as provided by ARDOT.

^b Target factor of safety = 1.1, approximately equivalent to a global stability resistance factor = 0.9, as provided by ARDOT.

Sufficient factors of safety (FOS) against global stability failure were computed for abutment spill slopes and side slopes constructed at 1V:2H or shallower. To reduce the potential for slip planes developing between new fill and existing subgrade soils, existing slopes should be benched prior to placing new fill.

Deep Foundations

Foundation design recommendations are provided herein based on the AASHTO LRFD Bridge Design Specifications (2020).

It is our understanding that driven, closed-ended, pipe piles will be installed to support the bridge. Pile diameters being considered by ARDOT are 14 and 16 inches at the abutments, and 18 and 24 inches at the intermediate bents. Geotechnology should be notified if other foundation types or sizes are to be considered. Based on the provided information, we modeled the bottom of pile cap at El 219 at the exterior bents, and the ground surface at El 210 at the interior bents. Soil parameters, including LPILE lateral load analysis parameters, are included in Appendix I for each bent.

Nominal resistance curves showing axial resistance from skin friction and total axial capacity (skin friction + end bearing) for Bents 1 through 4 are presented in Appendix J. Nominal static resistances for driven, closed-ended, pipe piles at each bent are presented in Table 8. Uplift (tension) resistance can be calculated using the resistance provided by skin friction.



Table 8. Static Nominal Axial Resistance of Driven Closed-Ended Pipe Piles.

| Location, Top of Pile Elevation (ft, NAVD) | Pile Diameter (inches) | Embedment Length (feet) | Nominal Static Resistance ^c (tons) | | | Nominal Drag Load from Consolidation Settlement (tons) |
|--|------------------------------|-------------------------------|---|----------------|----------------------|--|
| | | | Skin Friction | End Bearing | Compression Total | |
| Bent 1 ^a (West Abutment), El 219 | 14 | 60 | 178 | 107 | 285 | 12 |
| | | 70 | 236 | 107 | 343 | |
| | | 80 | 301 | 107 | 408 | |
| | 16 | 60 | 203 | 140 | 343 | 14 |
| | | 70 | 270 | 140 | 410 | |
| | | 80 | 344 | 140 | 484 | |
| Bent 4 ^a (East Abutment), El 219 | 14 | 60 | 161 | 107 | 268 | 12 |
| | | 70 | 218 | 107 | 325 | |
| | | 80 | 281 | 109 | 390 | |
| | 16 | 60 | 184 | 140 | 324 | 13 |
| | | 70 | 249 | 140 | 389 | |
| | | 80 | 321 | 143 | 464 | |
| Bent 2 ^b , El 210 | 18 | 60 | 212 | 127 | 339 | 0 |
| | | 70 | 281 | 143 | 424 | |
| | | 80 | 363 | 194 | 557 | |
| | 24 | 60 | 283 | 225 | 508 | 0 |
| | | 70 | 375 | 255 | 630 | |
| | | 80 | 484 | 346 | 830 | |
| Bent 3 ^b , El 210 | 18 | 60 | 203 | 168 | 371 | 0 |
| | | 70 | 275 | 186 | 461 | |
| | | 80 | 357 | 186 | 543 | |
| | 24 | 60 | 270 | 299 | 569 | 0 |
| | | 70 | 367 | 330 | 697 | |
| | | 80 | 476 | 330 | 806 | |

^a Embedment length referenced from an estimated bottom of pile cap at El 219.

^b Embedment length referenced from approximate ground surface elevation at El 210.

^c Total static compressive resistance has not been reduced by the static drag load.



Presented in Table 9 are post-liquefaction nominal resistances and nominal drag loads on the piles due to liquefaction settlement. For analysis of post-liquefaction pile resistance, liquefaction was limited to the upper 50 feet of the natural soil profile in the borings. Post-liquefaction nominal resistance curves are presented in Appendix J.

Table 9. Post-Liquefaction Nominal Axial Resistance of Driven Closed-Ended Pipe Piles.

| Location, Top of Pile Elevation (ft, NAVD) | Pile Diameter (inches) | Embedment Length (feet) | Post-Liquefaction Nominal Resistance ^c (tons) | | | Nominal Drag Load from Liquefaction Settlement (tons) |
|--|------------------------------|-------------------------------|---|----------------|----------------------|---|
| | | | Skin Friction | End Bearing | Compression Total | |
| Bent 1 ^a (West Abutment), El 219 | 14 | 60 | 90 | 107 | 197 | 32 |
| | | 70 | 148 | 107 | 255 | |
| | | 80 | 213 | 107 | 320 | |
| | 16 | 60 | 102 | 140 | 242 | 36 |
| | | 70 | 169 | 140 | 309 | |
| | | 80 | 243 | 140 | 383 | |
| Bent 4 ^a (East Abutment), El 219 | 14 | 60 | 45 | 107 | 152 | 39 |
| | | 70 | 102 | 107 | 209 | |
| | | 80 | 166 | 109 | 275 | |
| | 16 | 60 | 52 | 140 | 192 | 45 |
| | | 70 | 117 | 140 | 257 | |
| | | 80 | 189 | 143 | 332 | |
| Bent 2 ^b , El 210 | 18 | 50 | 140 | 127 | 267 | 32 |
| | | 60 | 209 | 143 | 352 | |
| | | 70 | 291 | 194 | 485 | |
| | 24 | 50 | 186 | 225 | 411 | 43 |
| | | 60 | 279 | 255 | 534 | |
| | | 70 | 387 | 355 | 732 | |
| Bent 3 ^b , El 210 | 18 | 50 | 109 | 168 | 277 | 40 |
| | | 60 | 181 | 186 | 367 | |
| | | 70 | 263 | 186 | 449 | |
| | 24 | 50 | 145 | 299 | 444 | 53 |
| | | 60 | 241 | 330 | 571 | |
| | | 70 | 350 | 330 | 680 | |

^a Embedment length referenced from an estimated bottom of pile cap at El 219.

^b Embedment length referenced from approximate ground surface elevation at El 210.

^c Post-liquefaction nominal axial resistance was not reduced by the drag load from liquefaction.



Resistance Factors. Resistance factors should be applied to the nominal resistances provided. Based solely on the static analysis methods used to calculate nominal pile resistances, the factors presented in Table 10 may be applied.

Table 10. Resistance Factors Based on Static Analysis Methods.

| Deep Foundation and Condition | Clay | | Sand | |
|---|-----------------|-------------|-----------------|-------------|
| | Side Resistance | End-Bearing | Side Resistance | End-Bearing |
| Nominal Compressive Resistance of Single Pile | 0.35 | 0.35 | 0.45 | 0.45 |
| Uplift Resistance of Single Pile | 0.25 | -- | 0.35 | -- |

Based on the AASHTO LRFD (2020) Table 10.5.5.2.3-1, a higher resistance factor can be used in accordance with the method of pile testing performed as indicated in Table 11.

Table 11. Resistance Factors for Driven Piles.

| Condition/Resistance Determination Method | | Resistance Factor |
|---|--|-------------------|
| Nominal Bearing Resistance of Single Pile – Dynamic Analysis and Static Load Test Methods | Driving criteria established by successful static load test of at least one pile per site condition and dynamic testing of at least two piles per site, but no less than 2% of the production piles* | 0.80 |
| | Driving criteria established by successful static load test of at least one pile per site condition without dynamic testing | 0.75 |
| | Driving criteria established by dynamic testing conducted on 100% of production piles* | 0.75 |
| | Driving criteria established by dynamic testing, quality control by dynamic testing of at least two piles per site condition, but no less than 2% of production piles* | 0.65 |
| | Wave equation analysis, without pile dynamic measurements or load test but with field confirmation of hammer performance | 0.50 |
| | FHWA-modified Gates dynamic pile formula (End of Drive condition only) | 0.40 |
| Uplift Resistance of Single Pile | Dynamic test with signal matching | 0.50 |

* Dynamic testing requires signal matching and estimates of nominal resistance made from a restrike. Dynamic tests are calibrated to a static load test, when available.

Pile Group Considerations. The settlement of pile groups should be evaluated as per AASHTO LRFD (2020) section 10.7.2.3. Settlement analysis of the pile groups can be performed when the



foundation configurations and service loads are available. AASHTO LRFD (2020) section 10.7.3.9 addresses pile group resistance. Group capacity considerations for different pile groups, center-to-center spacings, and other conditions (cap contact with ground, softness of surface soil) are given in AASHTO LRFD (2020) sections 10.7.3.9 and 10.7.3.11.

Driven Pile Construction Considerations. Minimum hammer energies required to drive the piles were not evaluated for the proposed foundations. If minimum hammer energy evaluations are required, Geotechnology should be contacted to perform analyses for the required minimum hammer energies for driving piles.

Static Pile Load Testing. At least one static pile compression load test should be performed for each bent or abutment location. The testing should be performed in accordance with ASTM D 1143 using the quick loading procedure and AASHTO LRFD (2020) section 10.7.3.8.2. Please refer to the previous Resistance Factors table for additional guidance regarding the minimum number of tests and alternate resistance factors associated with other field methods for determining resistance.

If the piles are to support net uplift loads, at least one tension load test should be performed for each location. The test should be performed in accordance with ASTM D 3689. Piles should be tested to the required nominal uplift resistance.

Load tests are required to verify recommended nominal pile resistance and will not be used to increase the design pile resistance. The piles used in the load tests should not be used for support of any structures. Geotechnology should be consulted regarding the locations of the test piles.

Dynamic Testing of Driven Piles. As an alternative to static pile load testing, high-strain dynamic pile testing can be performed according to AASHTO LRFD (2020) section 10.7.3.8.3 and the procedures given in ASTM D4945. Different resistance factors correspond to different load testing combinations as illustrated in the previous table. We recommend that the test piles be identified according to AASHTO LRFD (2020) Table 10.5.5.2.3-1 or 2 percent of the production piles, whichever results in a larger number of tests. We recommend that the identified piles be tested at the end of initial drive (EOD) and a restrike performed at a minimum seven days after EOD.

Pile driving monitoring should be performed by an engineer with a minimum 3 years of dynamic pile testing and analysis experience and who has achieved Basic or better certification under the High-Strain Dynamic Pile Testing Examination and Certification process of the Pile Driving Contractors Association and Foundation QA. Pile driving modeling and analyses should be performed by an engineer with a minimum five years of dynamic pile testing and analysis experience and who has achieved Advanced or better certification under the High-Strain Dynamic Pile Testing Examination and Certification process of the Pile Driving Contractors Association and Foundation QA.

Dynamic tests are required to monitor hammer and drive system performance, assess driving stresses and structural integrity and to evaluate pile resistance, and should not be used to increase



design pile resistance. Dynamic tests should be performed on production piles with the lowest driving resistance. Geotechnology will be available to assist with development of specifications for this program and should be on site to perform or observe the testing and establish the pile driving criteria.

Settlement. Settlement of pile foundations depends on the loads applied and the foundation configuration. In general, settlement of deep foundations designed in accordance with the recommendations provided in this report is expected to be less than 1-inch. However, a calculation of the expected settlement of the pile foundations can be performed when the applied service loads and foundation configuration are available.

Uplift Resistance. Uplift forces can be resisted by the effective weight of the piles and caps, and frictional resistance between the piles and surrounding soil. If the anticipated maximum level of groundwater is higher than the tip of the pile, then the buoyant unit weight of the pile must be used in computing uplift resistance for pile lengths extending below the design groundwater level.

Lateral Resistance. The lateral resistance of pile foundations depends on the lengths and dimensions of the foundations and the soil characteristics. The lateral resistance of pile foundations can be computed using the computer program LPILE to model the behavior of a single pile or shaft. LPILE soil parameters for effective unit weight in pounds per cubic foot (pcf), strain, and static modulus in pounds per cubic inch (pci) are provided in Appendix I for the various strata and soil strengths present at the site. LPILE soil parameters are based on field and laboratory test results and empirical correlations with SPT N-values.

The effects of group interaction must be considered when evaluating pile/shaft group horizontal movement. The lateral resistance for individual piles calculated by LPILE must be reduced by the P-multipliers provided in Section 10.7.2.4 of the AASHTO LRFD (2020) to determine lateral resistance of a pile group. Alternatively, the GROUP software can be used to evaluate the lateral resistance of the pile/shaft groups. The resistance factor for lateral resistance of single pile or pile group is 1.0.

Downdrag Due to Fill-Induced Settlement. Downdrag occurs when the soil moves downward relative to pile foundations because of consolidation settlement of fine-grained soils. The AASHTO LRFD Bridge Design Specifications (2020) suggest that settlement of soil strata relative to the pile of greater than 0.4-inch could produce downdrag on pile foundations at the bridge abutments. The relative movement of the soil layers versus the pile depends on the final foundation configuration.

As discussed in the section titled, "Approach Embankment Settlement", consolidation settlement of approximately 3.5 inches is calculated to occur under the weight of the approach embankments. Consolidation settlement is calculated to take approximately 3 months to be effectively complete, with 0.4-inch or less of additional settlement anticipated. Therefore, piles driven through the fill embankments at the abutments could be subject to downdrag as the fine-grained soils consolidate under the fill load. Nominal (unfactored) drag loads from consolidation settlement are provided in



Table 8 based on the calculated cumulative side resistance of piles at the depth where less than 0.4-inch of consolidation settlement is predicted to occur.

Load factors for downdrag are provided in AASHTO LRFD (2020) Table 3.4.1-2. The alpha method was used to calculate side resistance and downdrag on the piles. Additional guidance for design of piles with downdrag is provided in AASHTO LRFD (2020) Sections 10.7.1.6.2 and 10.7.3.7.

Downdrag Due to Post-Liquefaction Settlement. Based on the results of the liquefaction analyses, post-liquefaction settlement in the range of 3 to 6 inches is anticipated within the upper 50 feet of soil during the design earthquake event (7% exceedance in 75 years) at the exploration locations. Liquefaction-induced downdrag can be reduced by performing ground improvement on the potentially liquefiable soil layers. It is our professional opinion that driving the closed-ended pipe piles through the liquefiable layers will densify the susceptible layers in the vicinity of the piles; however, the amount and extent of densification cannot be ascertained during the design phase of the project. However, the extent of densification can be determined by advancing CPT soundings in the vicinity of the piles after they are driven. Pre-drilling or applying viscous coatings to piles are not recommended to reduce liquefaction-induced downdrag, because such methods will also reduce the nominal static compressive resistance of the piles. Please contact Geotechnology if more information is desired.

Corrosion Potential

In addition to laboratory soil classification and strength testing, soil resistivity testing was also conducted. The purpose of soil resistivity testing is to provide soil data for use by a structural engineer for analysis of any necessary protection of the piling, concrete, reinforcing steel, etc. Corrosion and deterioration protection requirements and guidelines for piling are set forth in Section 10.7.5 of the AASHTO LRFD Bridge Design Specifications. The corrosion and deterioration testing results are summarized in Table 12 and are included in Appendix E.

Table 12. Results of pH and Soil Resistivity Testing.

| Boring | Sample No. | Sample Depth (feet) | pH | Soil Resistivity (ohm-cm) |
|--------|------------|---------------------|------|---------------------------|
| B3-1 | ST7 | 15.0 | 7.59 | 929 |
| | SS12 | 38.5 | 8.14 | 1,784 |
| B3-2 | SS13 | 48.5 | 8.24 | 929 |
| B3-3 | ST8 | 20.0 | 7.21 | 929 |
| | SS15 | 53.5 | 7.69 | 2,069 |
| B3-4 | ST8 | 20.0 | 7.84 | 1,844 |
| | SS10 | 28.5 | 5.91 | 1,024 |



The following soil conditions should be considered as indicative of a potential for steel pile deterioration or corrosion:

- Resistivity values less than 2,000 ohms-cm; or
- pH less than 5.5.

The following soil conditions should be considered as indicative of a potential for steel reinforcement corrosion or deterioration:

- Resistivity values less than 3,000 ohms-cm; or
- pH less than 5.5.

Interpretation of the data and corrosion protection of the bridge structural components should be performed by the design team.

7.0 RECOMMENDED ADDITIONAL SERVICES

The conclusions and recommendations given in this report are based on: Geotechnology's understanding of the proposed design and construction, as outlined in this report; site observations; interpretation of the exploration data; and our experience. Since the intent of the design recommendations is best understood by Geotechnology, we recommend Geotechnology be included in the final design and construction process and be retained to review the project plans and specifications to confirm the recommendations given in this report have been correctly implemented. We recommend Geotechnology be retained to participate in pre-bid and preconstruction conferences to reduce the risk of misinterpretation of the conclusions and recommendations in this report relative to the proposed construction of the subject project.

Since actual subsurface conditions between boring locations could vary from those encountered in the borings, our design recommendations are subject to adjustment in the field based on the subsurface conditions encountered during construction. Therefore, we recommend Geotechnology be retained to provide construction observation services as a continuation of the design process to confirm the recommendations in this report and to revise them accordingly to accommodate differing subsurface conditions. Construction observation is intended to enhance compliance with project plans and specifications. It is not insurance, nor does it constitute a warranty or guarantee of any type. Regardless of construction observation, contractors, suppliers, and others are solely responsible for the quality of their work and for adhering to plans and specifications.

8.0 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.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a civil engineer may not fulfill the needs of a constructor — a construction contractor — or even another civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. No one except you should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you — should apply this report for any purpose or project except the one originally contemplated.*

Read the Full Report

Serious problems have occurred because those relying on a geotechnical-engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

Geotechnical Engineers Base Each Report on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk-management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical-engineering report that was:

- not prepared for you;
- not prepared for your project;
- not prepared for the specific site explored; or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical-engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an

assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical-engineering report is based on conditions that existed at the time the geotechnical engineer performed the study. *Do not rely on a geotechnical-engineering report whose adequacy may have been affected by:* the passage of time; man-made events, such as construction on or adjacent to the site; or natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Contact the geotechnical engineer before applying this report to determine if it is still reliable.* A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ — sometimes significantly — from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide geotechnical-construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are Not Final

Do not overrely on the confirmation-dependent recommendations included in your report. *Confirmation-dependent recommendations are not final*, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations *only* by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's confirmation-dependent recommendations if that engineer does not perform the geotechnical-construction observation required to confirm the recommendations' applicability.*

A Geotechnical-Engineering Report Is Subject to Misinterpretation

Other design-team members' misinterpretation of geotechnical-engineering reports has resulted in costly

problems. Confront that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Constructors can also misinterpret a geotechnical-engineering report. Confront that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing geotechnical construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical-engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make constructors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give constructors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise constructors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure constructors have sufficient time* to perform additional study. Only then might you be in a position to give constructors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and constructors fail to recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of 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.

Environmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform an *environmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold-prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold-prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical-engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; *none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.*

Rely, on Your GBC-Member Geotechnical Engineer for Additional Assistance

Membership in the Geotechnical Business Council of the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your GBC-Member geotechnical engineer for more information.



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e-mail: info@geoprofessional.org www.geoprofessional.org

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APPENDIX B – FIGURES

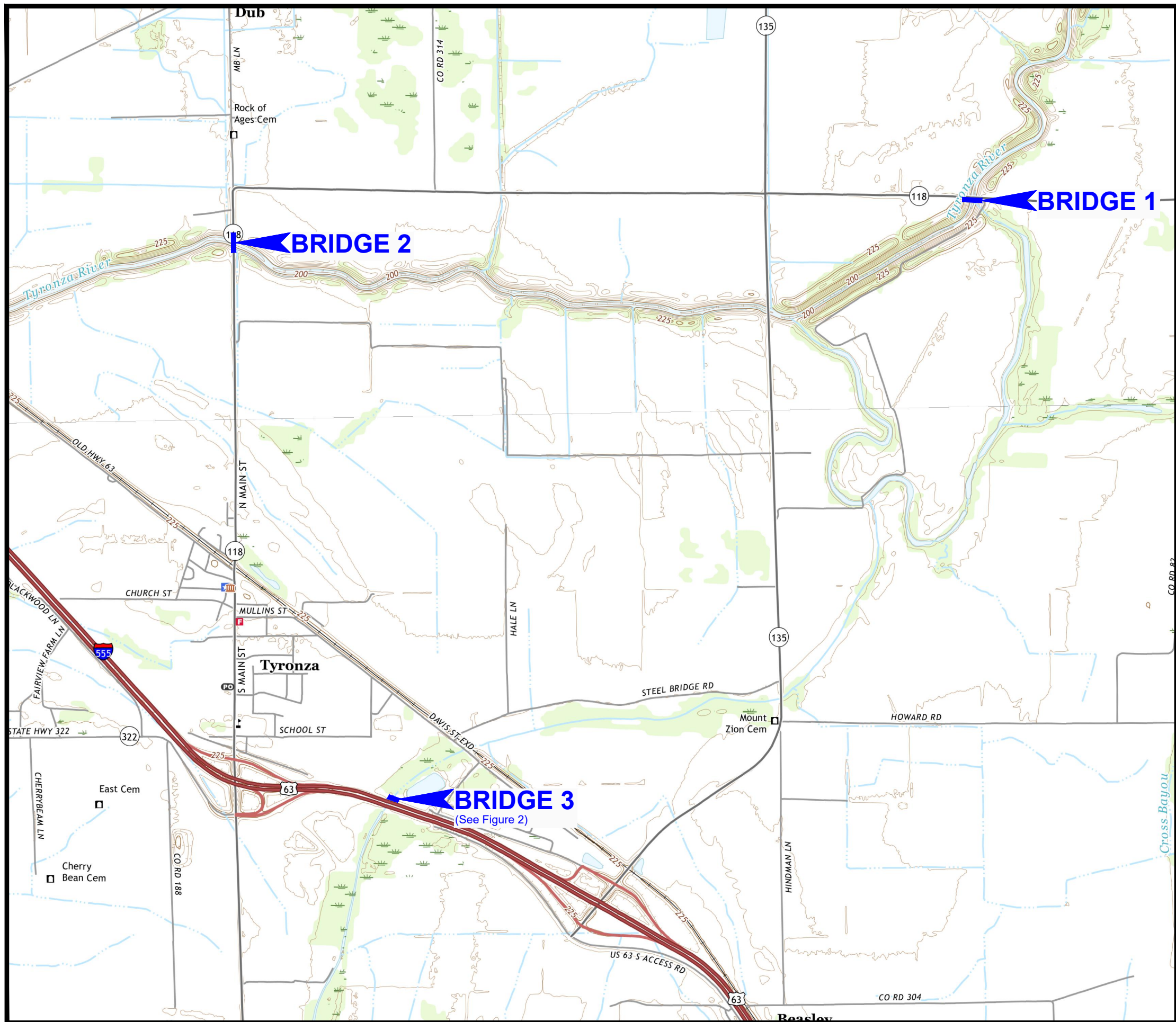
Figure 1 – Site Location and Topography

Figure 2 – Aerial Photograph of Site and Boring Locations

Figure 3 – Generalized Subsurface Profile

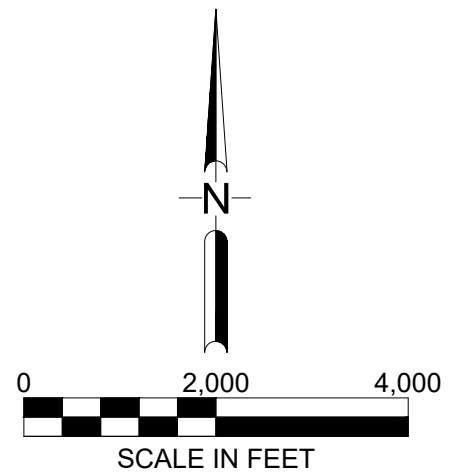
Figure 4 -Settlement Plate Detail





NOTES

1. Plan adapted from 7.5 minute U.S.G.S. maps for Lepanto and Tyronza, Arkansas quadrangles, last revised in 2020.



| | | |
|---------------|---------------|----------------|
| Drawn By: WAH | Ck'd By: DFB | App'vd By: ASE |
| Date: 5-5-23 | Date: 11-1-23 | Date: 11-27-23 |

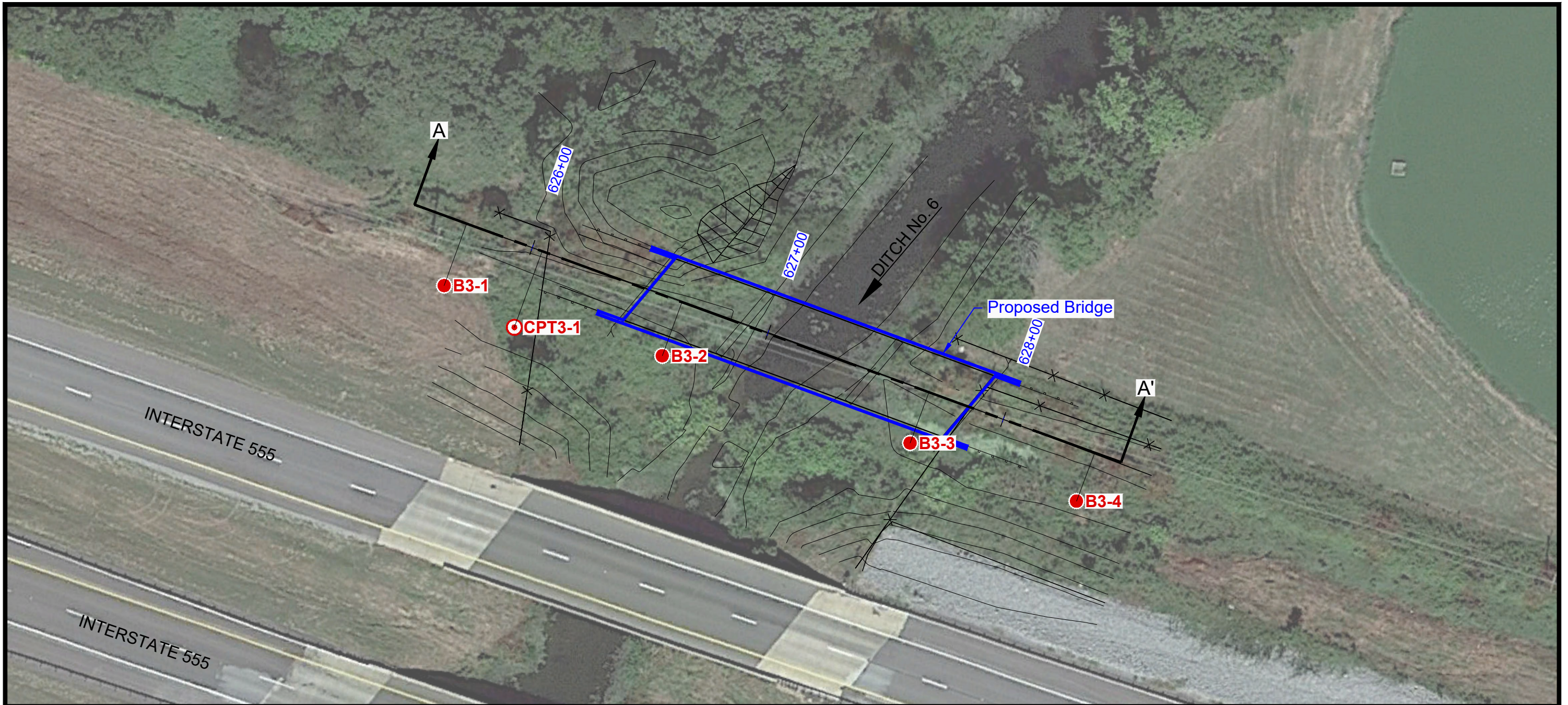


ARDOT 101017 Tyronza River &
Ditch No. 6 Strs. & Apprs. (S)
Poinsett County, Arkansas

**SITE LOCATION
AND TOPOGRAPHY**

Project Number
J042992.01

FIGURE 1




NOTES

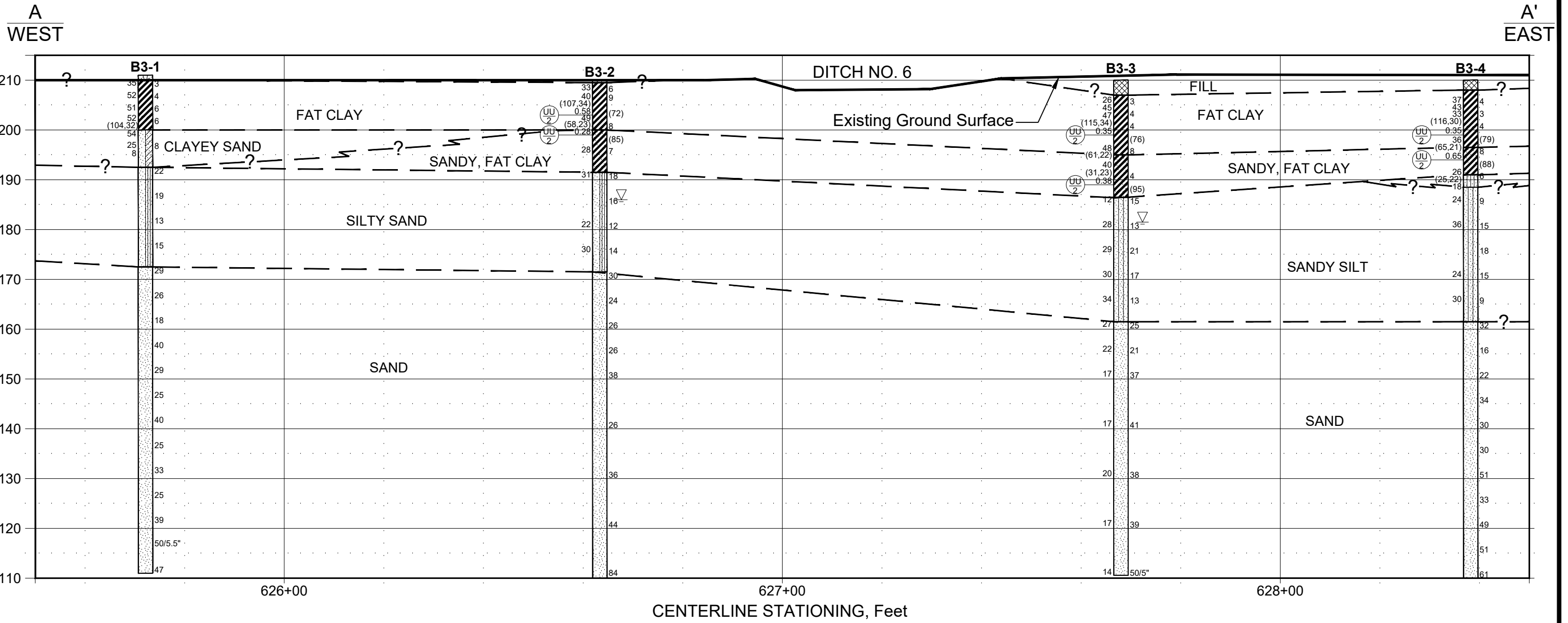
1. Plan adapted from a September 5, 2021 aerial photograph courtesy of Google Earth and a drawing dated January 20, 2023, titled "Soil Boring Request I-555 Service Road Over Ditch No. 6", prepared by Arkansas State Highway Commission"
2. Borings and CPT were located in the field with reference to site features and are shown approximate only.

LEGEND

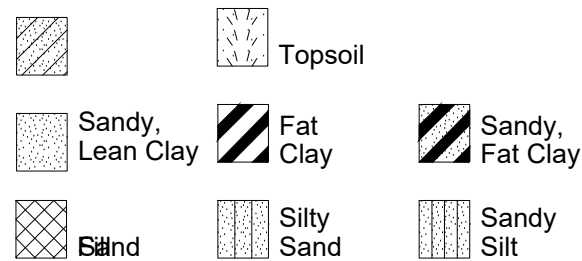
- Boring Location
- ⊙ CPT Sounding Location
- Subsurface Profile (See Figure 3)



| | | |
|---|-----------------|----------------|
| Drawn By: WAH | Ck'd By: DFB | App'vd By: ASE |
| Date: 11-1-23 | Date: 11-1-23 | Date: 11-27-23 |
|  GEOTECHNOLOGY <small>A UES Company</small> | | |
| ARDOT 101017 Tyronza River & Ditch No. 6 Strs. & Apprs. (S) Poinsett County, Arkansas | | |
| AERIAL PHOTOGRAPH OF BRIDGE 3, BORING AND CPT LOCATIONS | | |
| Project Number J042992.01 | FIGURE 2 | |

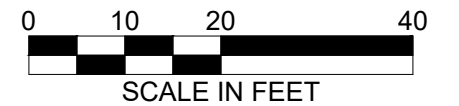


KEY TO BOREHOLE SYMBOLS



KEY TO TEST DATA

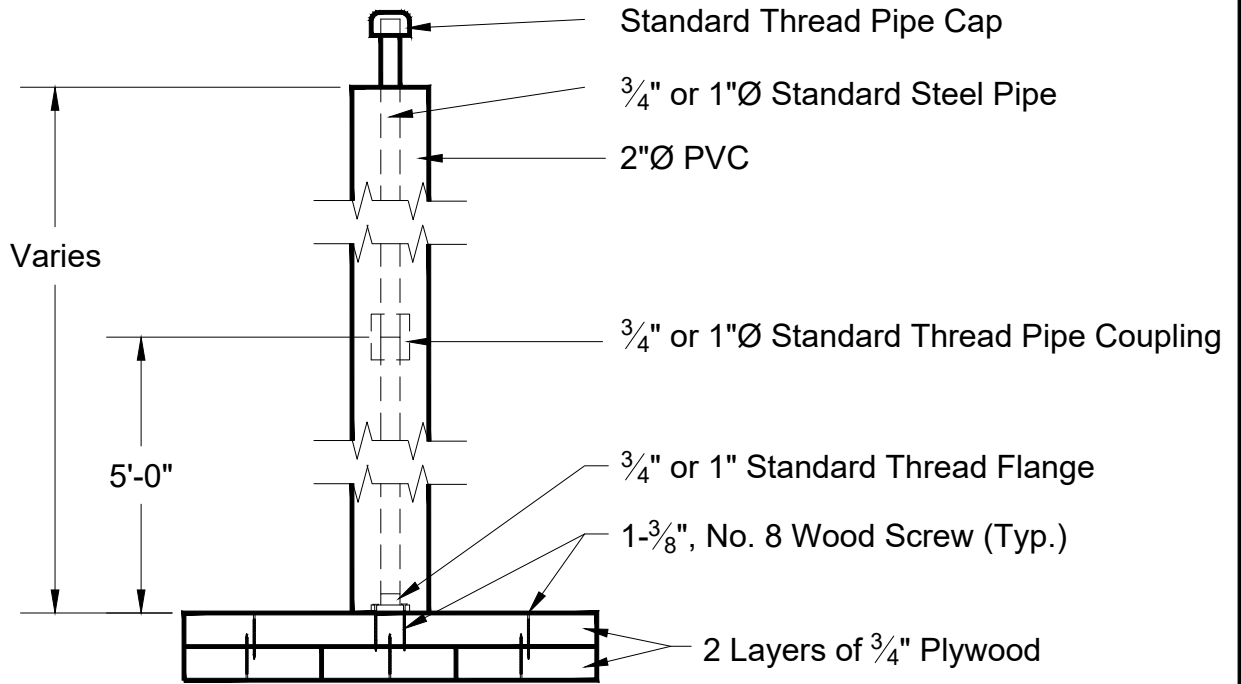
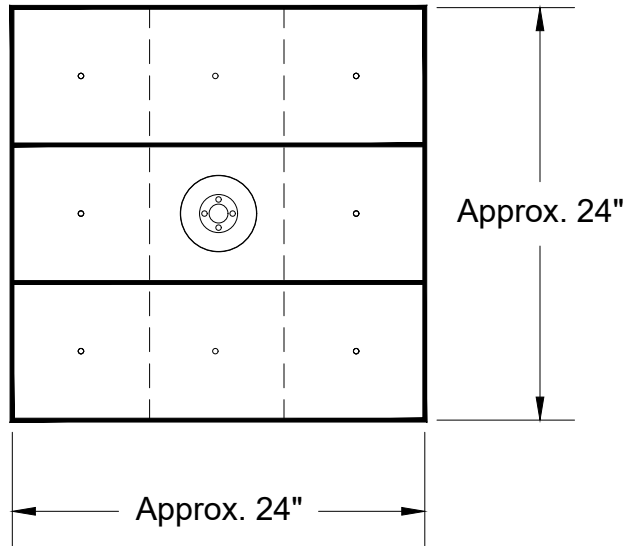
| | | | |
|---|---------|--------|--|
| Natural Water Content in Percent | 21 | 9 | Standard Penetration Test Resistance (No. of Blows of a 140-lb. Hammer Dropping 30-in. Required to Drive a 2-in. O.D. Split Spoon One Foot or Indicated Depth - "S" Denotes Seating) |
| Liquid and Plastic Limits | (58,31) | | Dry Unit Weight in Pounds per Cubic Foot |
| Shear Strength in TSF from Unconfined Compression | UU 0.73 | (97) | Groundwater Elevation Observed at Time of Drilling |
| | | ▽ | Percent Core Recovery |
| | | 100/83 | R.Q.D.* |
| | | | *R.Q.D. Denotes Modified Core Recovery Percentage in Which Only Pieces of Sound Core Over 4 Inches Long are Counted as Recovery |



NOTES

- See Figure 2 for location of Subsurface Profile.
- Data concerning subsurface conditions were obtained at boring locations only. Actual conditions at locations between borings could differ from the generalized subsurface profile shown here.

| | | |
|--|---------------|-----------------|
| Drawn By: WAH | Ck'd By: DFB | App'vd By: ASE |
| Date: 11-1-23 | Date: 11-1-23 | Date: 11-27-23 |
| | | |
| ARDOT 101017 Tyronza River & Ditch No. 6 Strs. & Apprs. (S) Poinsett County, Arkansas GENERALIZED SUBSURFACE PROFILE | | |
| Project Number J042992.01 | | FIGURE 3 |



NOTES

1. Place plate on level surface, a minimum of 2 feet below ground level and hand compact backfill adjacent to PVC.

| | | |
|--|-----------------|----------------|
| Drawn By: WAH | Ck'd By: DFB | App'vd By: ASE |
| Date: 11-1-23 | Date: 11-1-23 | Date: 11-27-23 |
| | | |
| ARDOT 101017 Tyrnza River & Ditch No. 6 Strs. & Apprs. (S) Poinsett County, Arkansas | | |
| SETTLEMENT PLATE DETAIL | | |
| Project Number J042992.01 | FIGURE 4 | |



APPENDIX C – BORING INFORMATION

Boring Logs

Boring Log Terms and Symbols

Surface Elevation: 216

Completion Date: 7/27/23

Datum NAVD88

Lat: 421293.8052

Long: 1803922.6205

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 | 10 20 30 40 50 | LL

NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES
LOG OF BORING 2020 JDM - ELEVATIONS .J042992.01.GPJ GTINC 0638301.GPJ 1/4/24 AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

| DEPTH IN FEET | ELEVATION IN FEET | DESCRIPTION OF MATERIAL | GRAPHIC LOG | DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD | SAMPLES | SHEAR STRENGTH, tsf | STANDARD PENETRATION RESISTANCE | WATER CONTENT, % |
|---------------|-------------------|---|-------------|---|-----------------|---------------------|---------------------------------|------------------|
| | | Topsoil: 12 inches | | | | | | |
| 5 | 211 | Soft to medium stiff, gray to brown, FAT CLAY, some silt partings - (CH) | | | 2-1-2 SS1 | ▲ | | |
| | | | | | 2-1-3 SS2 | ▲ | | |
| | | | | | 2-3-3 SS3 | ▲ | | |
| 10 | 206 | | | | 2-3-3 SS4 | ▲ | | |
| | | | | | ST5 | | | |
| 15 | 201 | Loose, brown to gray, fine SAND, trace clay - (SP-SC) 9.9% passing No. 200 sieve 6.5% passing No. 200 sieve | | | 3-3-5 SS6 | ▲ | | |
| | | | | | ST7 | | | |
| 20 | 196 | Medium dense, gray, fine to medium SAND, trace silt - (SP-SM) trace organics | | | 6-9-13 SS8 | ▲ | | |
| 25 | 191 | | | | 8-7-12 SS9 | ▲ | | |
| 30 | 186 | 5.2% passing No. 200 sieve | | | 5-5-8 SS10 | ▲ | | |
| 35 | 181 | | | | 8-8-7 SS11 | ▲ | | |
| 40 | 176 | Medium dense to very dense, gray, medium SAND - SP trace organics | | | 12-14-15 SS12 | ▲ | | |
| 45 | 171 | | | | 12-12-14 SS13 | ▲ | | |
| 50 | 166 | | | | 6-7-11 SS14 | ▲ | | |
| 55 | 161 | | | | 18-19-21 SS15 | ▲ | | |
| 60 | 156 | | | | 12-13-16 SS16 | ▲ | | |
| 65 | 151 | | | | 14-12-13 SS17 | ▲ | | |
| 70 | 146 | | | | 14-19-21 SS18 | ▲ | | |
| 75 | 141 | | | | 14-13-12 SS19 | ▲ | | |
| 80 | 136 | | | | 13-14-19 SS20 | ▲ | | |
| 85 | 131 | some organics | | | 9-11-14 SS21 | ▲ | | |
| 90 | 126 | | | | 15-17-22 SS22 | ▲ | | |
| 95 | 121 | | | | 42-50/5.5' SS23 | | | 5.5" |
| 100 | 116 | Boring terminated at 100 feet. | | | 14-19-28 SS24 | ▲ | | |

GROUNDWATER DATA

FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

AUGER 3 3/4" HOLLOW STEM WASHBORING FROM 20 FEET
JCG DRILLER SAS LOGGER
Diedrich D-50 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 93 %

REMARKS:

Drawn by: SAS Checked by: DFB App'vd. by: ASE
Date: 7/28/23 Date: 11/13/23 Date: 11/27/23



Tyrnza River & Ditch No.6 Strs. & Apprs. (S)
Poinsett County, Arkansas

LOG OF BORING: B3-1

Project No. J042992.01

Surface Elevation: 214

Completion Date: 7/26/23

Datum NAVD88

Lat: 421265.8088

Long: 1804009.636

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 | 10 20 30 40 50 | LL

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

| DEPTH IN FEET | ELEVATION IN FEET | DESCRIPTION OF MATERIAL | GRAPHIC LOG | DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD | SAMPLES | SHEAR STRENGTH, tsf | STANDARD PENETRATION RESISTANCE | WATER CONTENT, % |
|---------------|-------------------|---|-------------|---|---------------|---------------------|---------------------------------|------------------|
| | | Topsoil: 6 inches | | | | | | |
| 5 | 209 | Medium stiff to stiff, brown to gray, FAT CLAY - (CH) trace organics | | | 1-2-4 SS1 | ▲ | | |
| | | | | | 2-3-6 SS2 | ▲ | | |
| | | | | | 72 ST3 | | | |
| 10 | 204 | Medium stiff, gray, FAT CLAY, some sand - (CH) | | | 2-3-5 SS4 | ▲ | | |
| | | | | | 85 ST5 | | | |
| 15 | 199 | 73.4% passing No. 200 sieve | | | 2-3-4 SS6 | ▲ | | |
| 20 | 194 | Medium dense, gray, fine SAND, little silt - (SP-SM) | | | 4-10-8 SS7 | | | |
| 25 | 189 | | | | 4-5-11 SS8 | | | |
| 30 | 184 | | | | 8-8-4 SS9 | | | |
| 35 | 179 | 11.4% passing No. 200 sieve | | | 7-8-6 SS10 | | | |
| 40 | 174 | Medium dense to very dense dense, gray, medium SAND - SP | | | 12-16-14 SS11 | | | |
| 45 | 169 | | | | 10-11-13 SS12 | | | |
| 50 | 164 | | | | 12-12-14 SS13 | | | |
| 55 | 159 | | | | 11-12-14 SS14 | | | |
| 60 | 154 | | | | 16-18-20 SS15 | | | |
| 65 | 149 | | | | | | | |
| 70 | 144 | | | | 13-14-12 SS16 | | | |
| 75 | 139 | | | | | | | |
| 80 | 134 | | | | 11-16-20 SS17 | | | |
| 85 | 129 | | | | | | | |
| 90 | 124 | with gravel | | | 17-22-22 SS18 | | | |
| 95 | 119 | | | | | | | |
| 100 | 114 | with gravel Boring terminated at 100 feet. | | | 26-38-46 SS19 | | | 84 |

GROUNDWATER DATA

ENCOUNTERED AT 24 FEET ∇

DRILLING DATA

___ AUGER 3 3/4" HOLLOW STEM
WASHBORING FROM 25 FEET
JCG DRILLER EJJ LOGGER
Diedrich D-50 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 93 %

REMARKS:

Drawn by: RSP Checked by: DFB App'vd. by: ASE
Date: 7/26/23 Date: 11/13/23 Date: 11/27/23



Tyrnza River
& Ditch No.6 Strs. & Apprs. (S)
Poinsett County, Arkansas

LOG OF BORING: B3-2

Project No. J042992.01

Surface Elevation: 217

Completion Date: 7/20/23

Datum NAVD88

Lat: 421231.0163

Long: 1804108.5044

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 | 10 20 30 40 50 | LL

DEPTH IN FEET
ELEVATION IN FEET

DESCRIPTION OF MATERIAL

GRAPHIC LOG

DRY UNIT WEIGHT (pcf)
SPT BLOW COUNTS
CORE RECOVERY/RQD

SAMPLES

Fill: Rip Rap

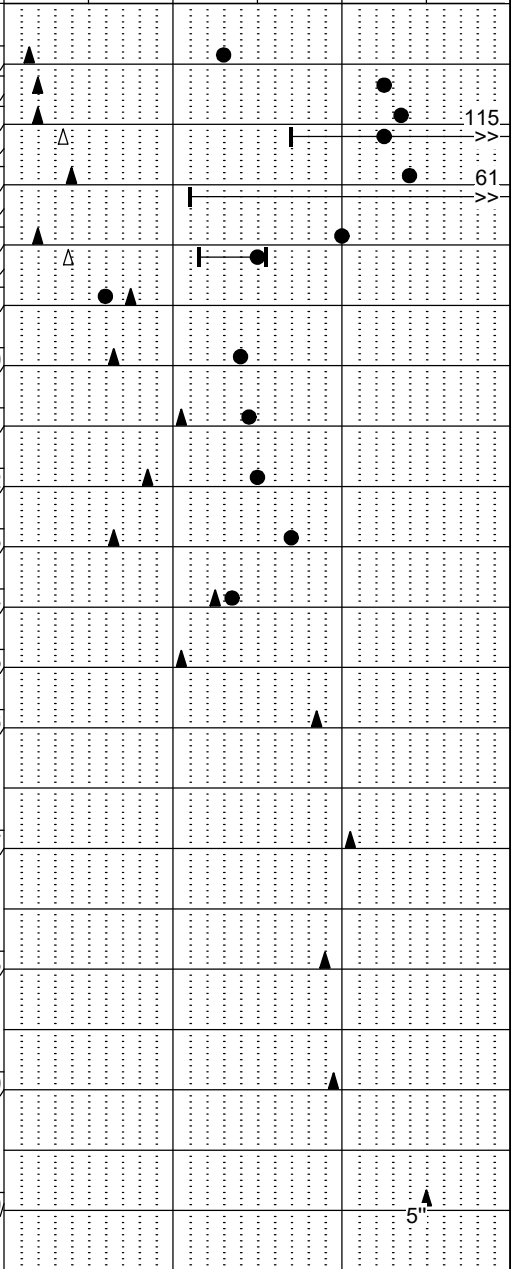
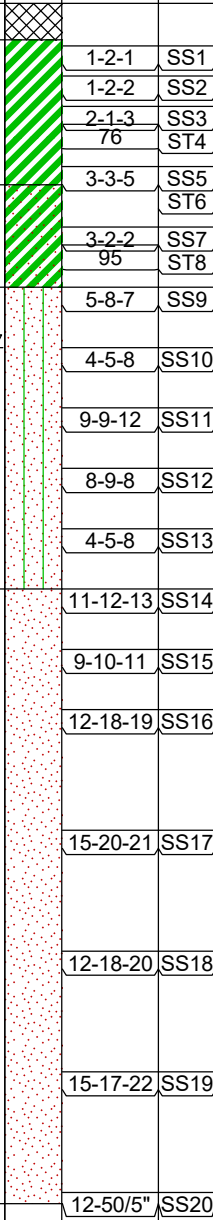
Soft to medium stiff, gray, FAT CLAY - (CH)

Soft to medium stiff, gray, sandy, FAT CLAY - (CH)

Medium dense, brown to gray, SILTY SAND - (SM)

Medium dense to very dense, gray, medium SAND - SP

little gravel
Spoon refusal at 99.4 feet.



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES
LOG OF BORING 2020 JDM - ELEVATIONS J042992.01.GPJ GTINC 0638301.GPJ 1/4/24 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 30 FEET
JCG DRILLER JMP LOGGER
Diedrich D-50 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 93 %

REMARKS:

Drawn by: RSP Checked by: DFB App'vd. by: ASE
Date: 7/21/23 Date: 11/13/23 Date: 11/27/23



**Tyrnza River
& Ditch No.6 Strs. & Apprs. (S)
Poinsett County, Arkansas**

LOG OF BORING: B3-3

Project No. J042992.01

LOG OF BORING 2020 JDM - ELEVATIONS .J042992.01.GPJ GTINC 0638301.GPJ 1/4/24 AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

| Surface Elevation: <u>216</u> | | Completion Date: <u>7/25/23</u> | | GRAPHIC LOG | DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD | SAMPLES | SHEAR STRENGTH, tsf | | | | |
|-------------------------------|----------------------|--|--|--|---|---------|--|---|--|-----|--|
| Datum <u>NAVD88</u> | | Lat: <u>421207.8259</u> | | | | | Δ - UU/2 \circ - QU/2 \square - SV 0.5 1.0 1.5 2.0 2.5 | | | | |
| DEPTH IN FEET | ELEVATION IN FEET | DESCRIPTION OF MATERIAL | | | | | STANDARD PENETRATION RESISTANCE (ASTM D 1586) ▲ N-VALUE (BLOWS PER FOOT) | | | | |
| | | | | WATER CONTENT, % PL 10 20 30 40 50 LL | | | | | | | |
| | | Fill: Rip Rap | | | | | | | | | |
| 5 | 211 | Soft to medium stiff, gray, FAT CLAY - (CH) | | [Green Diagonal Hatching] | 2-2-2 | SS1 | ▲ | | | | |
| | | | | | 2-1-2 | SS2 | ▲ | | | | |
| 10 | 206 | | | | 2-2-2 | SS3 | ▲ | | | | |
| | | | | | 79 | ST4 | ▲ | △ | | 116 | |
| 15 | 201 | Medium stiff to stiff, gray to brownish-gray, FAT CLAY, with sand - CH | | [Green Diagonal Hatching] | 2-3-5 | SS5 | ▲ | | | | |
| | | | | | 88 | ST6 | ▲ | △ | | 65 | |
| 20 | 196 | Medium stiff, brown and gray, sandy SILT - (ML) | | | 4-3-3 | SS7 | ▲ | | | | |
| | | | | | | ST8 | | | | | |
| 25 | 191 | Loose to medium dense, gray, SILTY SAND, trace gravel - (SM) | | | 3-4-5 | SS9 | ▲ | | | | |
| 30 | 186 | | | | 5-6-9 | SS10 | ▲ | | | | |
| 35 | 181 | | | | 9-8-10 | SS11 | ▲ | | | | |
| 40 | 176 | 27.3% passing No. 200 sieve | | | 3-6-9 | SS12 | ▲ | | | | |
| 45 | 171 | | | | 7-5-4 | SS13 | ▲ | | | | |
| 50 | 166 | Medium dense to very dense, gray, medium SAND - (SP) | | | 10-12-20 | SS14 | ▲ | | | | |
| | | | | | 8-7-9 | SS15 | ▲ | | | | |
| 55 | 161 | 4.1% passing No. 200 sieve | | | 10-10-12 | SS16 | ▲ | | | | |
| 60 | 156 | | | | 12-15-19 | SS17 | ▲ | | | | |
| 65 | 151 | | | | 13-15-15 | SS18 | ▲ | | | | |
| 70 | 146 | | | | 13-15-15 | SS19 | ▲ | | | | |
| 75 | 141 | | | | 13-20-31 | SS20 | ▲ | | | | |
| 80 | 136 | | | | 13-16-17 | SS21 | ▲ | | | | |
| 85 | 131 | little gravel | | | 21-23-26 | SS22 | ▲ | | | | |
| 90 | 126 | | | | 24-21-30 | SS23 | ▲ | | | | |
| 95 | 121 | some gravel | | | 21-30-31 | SS24 | ▲ | | | | |
| 100 | 116 | some gravel | | | | | | | | 61 | |
| | | Boring terminated at 100 feet. | | | | | | | | | |

GROUNDWATER DATA

FREE WATER NOT ENCOUNTERED DURING DRILLING

REMARKS:

DRILLING DATA

___ AUGER 3 3/4" HOLLOW STEM WASHBORING FROM 25 FEET
 WEC DRILLER SAS LOGGER
Diedrich D-50 DRILL RIG
 HAMMER TYPE Auto
 HAMMER EFFICIENCY 93 %

| | | |
|---------------|-----------------|-----------------|
| Drawn by: RSP | Checked by: DFB | App'vd. by: ASE |
| Date: 7/26/23 | Date: 11/13/23 | Date: 11/27/23 |



Tyrnza River & Ditch No.6 Strs. & Apprs. (S)
Poinsett County, Arkansas

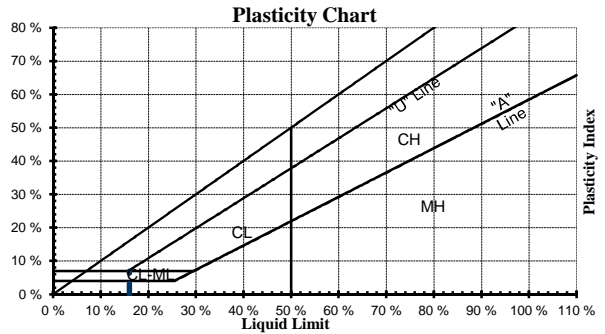
LOG OF BORING: B3-4

Project No. J042992.01

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

| <i>Major Divisions</i> | | <i>Symbol</i> | <i>Description</i> | |
|--|--|---|---|---|
| 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 | |
| | | Gravels with Appreciable Fines | GP Poorly-Graded Gravel, Gravel-Sand Mixture | |
| | | Sand and Sandy Soils | Clean Sands Little or no Fines | GM Silty Gravel, Gravel-Sand-Silt Mixture |
| | | | Sands with Appreciable Fines | GC Clayey-Gravel, Gravel-Sand-Clay Mixture |
| | Fine-Grained Soils (More than 50% Smaller than No. 200 Sieve Size) | Silts and Clays | Liquid Limit Less Than 50 | SW Well-Graded Sand, Gravelly Sand |
| | | | | SP Poorly-Graded Sand, Gravelly Sand |
| | | | | SM Silty Sand, Sand-Silt Mixture |
| | | Silts and Clays | Liquid Limit Greater Than 50 | SC Clayey-Sand, Sand-Clay Mixture |
| | ML Silt, Sandy Silt, Clayey Silt, Slight Plasticity | | | |
| | CL Lean Clay, Sandy Clay, Silty Clay, Low to Medium Plasticity | | | |
| | | 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

DENSITY OF GRANULAR SOILS

| <i>Consistency</i> | <i>Undrained Shear Strength (tsf)</i> | <i>Unconfined Comp. Strength (tsf)</i> | <i>Descriptive Term</i> | <i>Approximate N₆₀-Value Range</i> |
|--------------------|---------------------------------------|--|-------------------------|---|
| Very Soft | less than 0.125 | less than 0.25 | Very Loose | 0 to 4 |
| Soft | 0.125 to 0.25 | 0.25 to 0.5 | Loose | 5 to 10 |
| Medium Stiff | 0.25 to 0.5 | 0.5 to 1.0 | Medium Dense | 11 to 30 |
| Stiff | 0.5 to 1.0 | 1.0 to 2.0 | Dense | 31 to 50 |
| Very Stiff | 1.0 to 2.0 | 2.0 to 3.0 | Very Dense | >50 |
| Hard | greater than 2.0 | greater than 4.0 | | |

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

OTHER TERMS

| | | |
|--------|-----------|---|
| Trace | 0 to 10% | Layer - Inclusion greater than 3 inches thick. |
| Little | 10 to 20% | Seam - Inclusion 1/8-inch to 3 inches thick |
| Some | 20 to 35% | Parting - Inclusion less than 1/8-inch thick |
| And | 35 to 50% | Pocket - Inclusion of material that is smaller than sample diameter |



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.



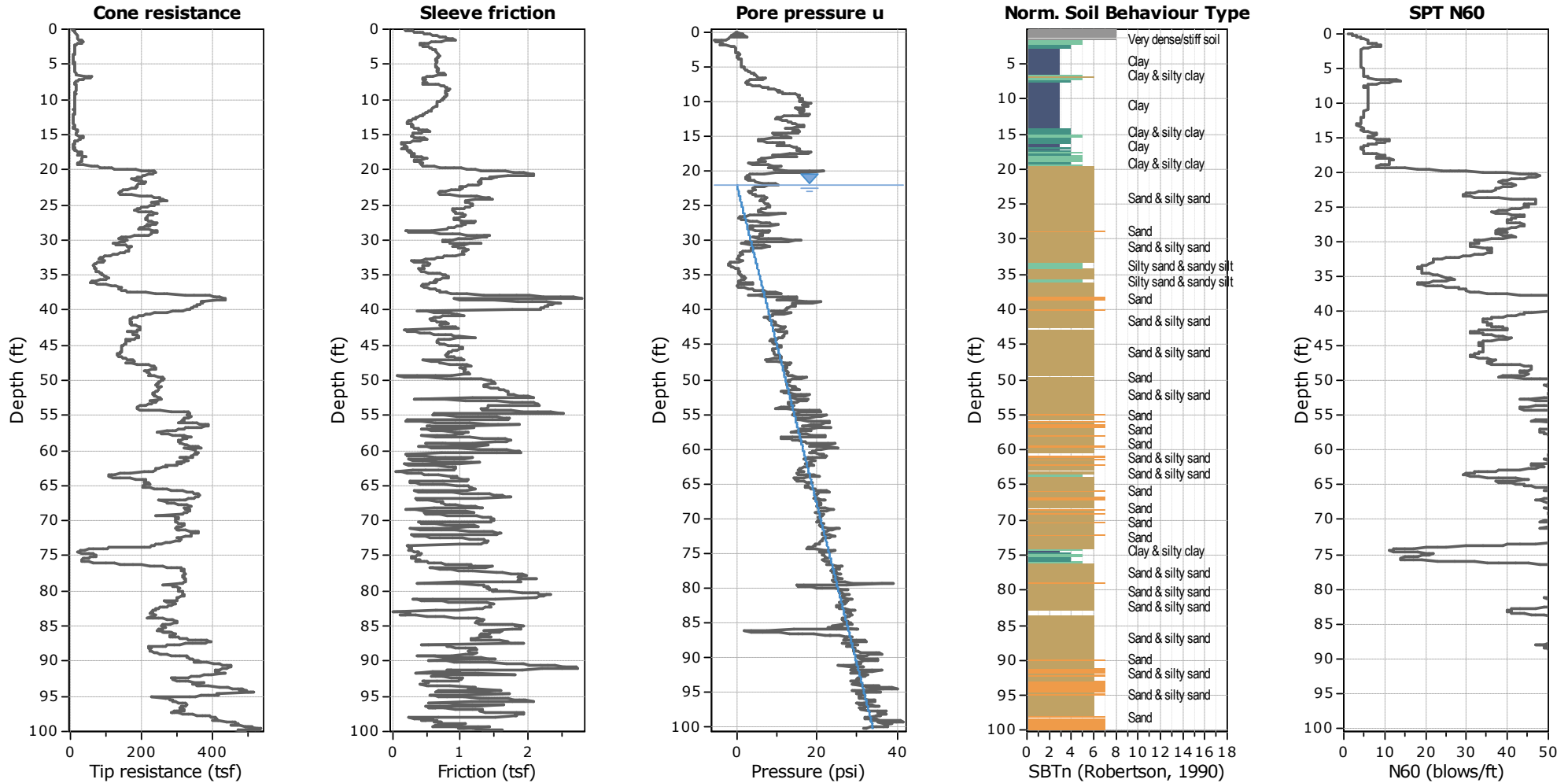
APPENDIX D – CPT3-1 SOUNDING RESULTS

CPT Plot

Shear Wave Velocity Profile



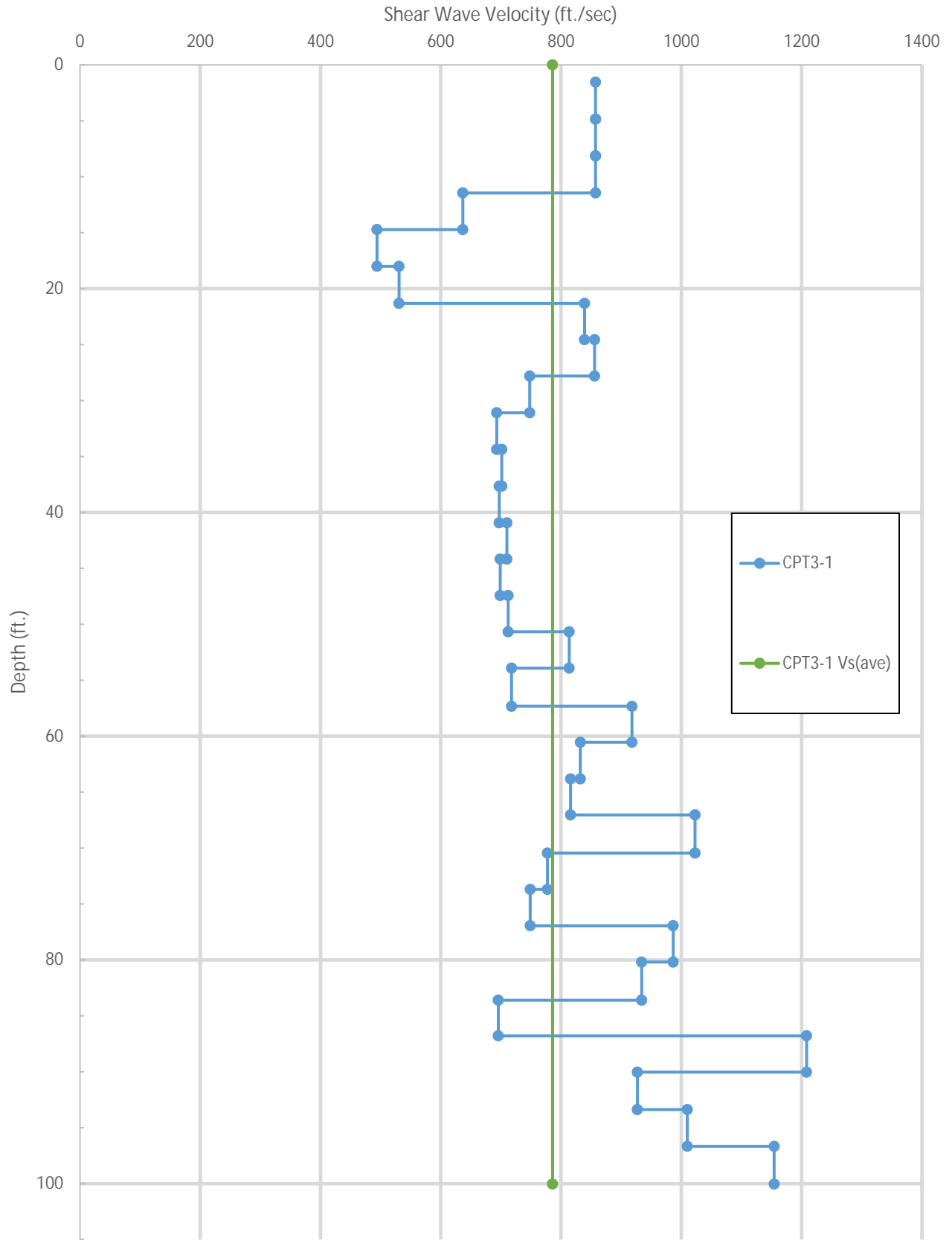
Project: ARDOT G003 101017 Tyrnza River & Ditch No. 6 Strs. & Apprs. (S)
Location: Poinsett County, Arkansas



SBTn legend

- | | | |
|--|---|---|
| ■ 1. Sensitive fine grained | ■ 4. Clayey silt to silty clay | ■ 7. Gravelly sand to sand |
| ■ 2. Organic material | ■ 5. Silty sand to sandy silt | ■ 8. Very stiff sand to clayey sand |
| ■ 3. Clay to silty clay | ■ 6. Clean sand to silty sand | ■ 9. Very stiff fine grained |

Shear Wave Velocity vs. Depth





APPENDIX E – LABORATORY TEST DATA

Atterberg Limits

Grain Size Distributions

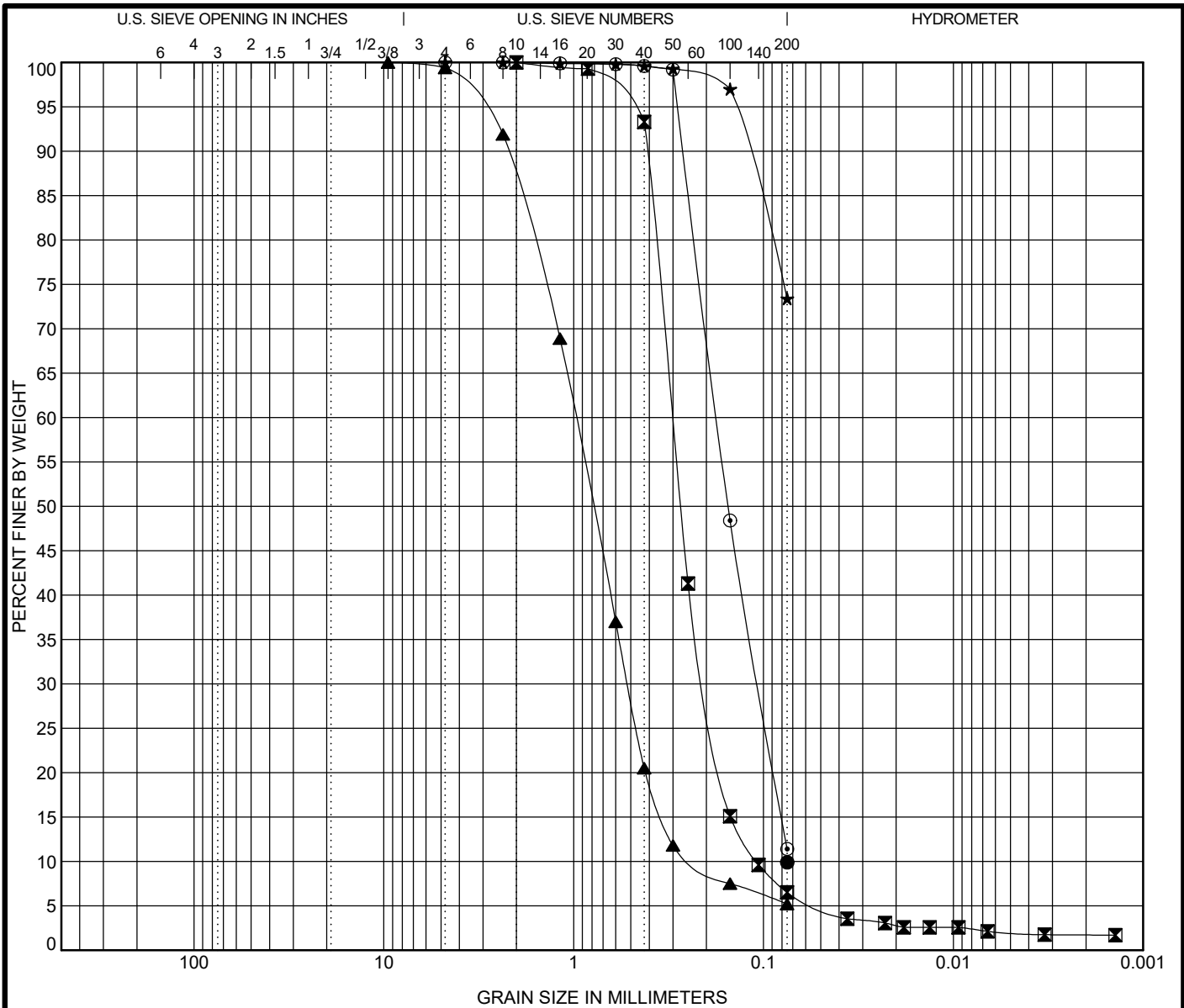
Unconsolidated-Undrained Triaxial Compression

One-Dimensional Consolidation

Direct Shear

Soil Resistivity

pH of Soil



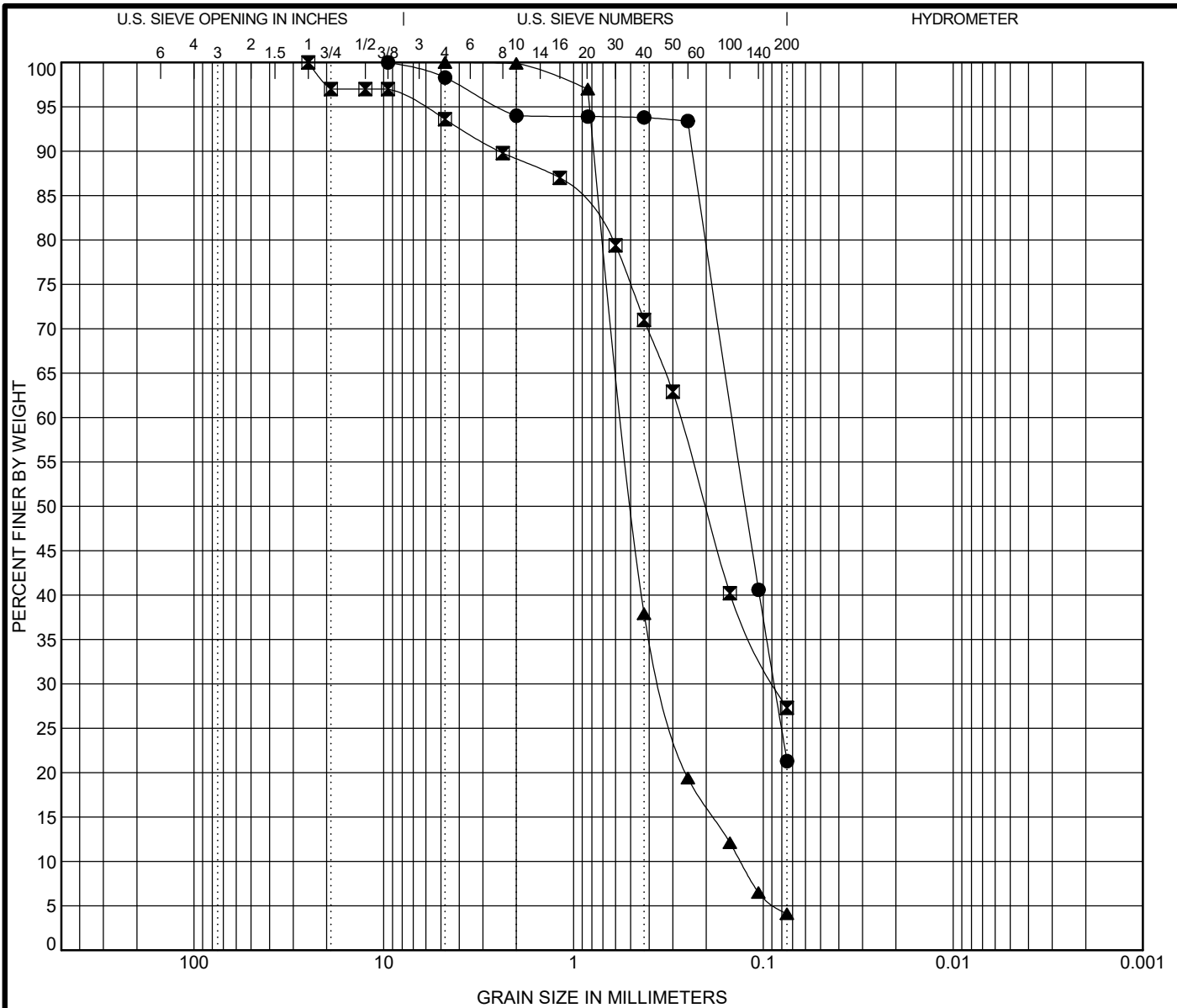
| COBBLES | GRAVEL | | SAND | | | SILT OR CLAY |
|---------|--------|------|--------|--------|------|--------------|
| | coarse | fine | coarse | medium | fine | |

| Specimen Identification | Classification | LL | PL | PI | Cc | Cu | | |
|-------------------------|----------------------------------|-------|-------|-------|---------|-------|-------|-------|
| ● B3-1 11.5 | POORLY GRADED SAND(SP-SC), A-2-7 | | | | | | | |
| ☒ B3-1 15.0 | POORLY GRADED SAND(SP-SC), A-2-7 | | | | 1.22 | 2.78 | | |
| ▲ B3-1 28.5 | POORLY GRADED SAND(SP-SM), A-1-b | | | | 1.23 | 4.35 | | |
| ★ B3-2 13.5 | FAT CLAY with SAND(CH), A-7-6 | | | | | | | |
| ◎ B3-2 33.5 | POORLY GRADED SAND(SP-SM), A-2-4 | | | | 0.88 | 2.41 | | |
| Specimen Identification | D100 | D60 | D30 | D10 | %Gravel | %Sand | %Silt | %Clay |
| ● B3-1 11.5 | 0.075 | | | | 0.0 | 0.0 | 9.9 | |
| ☒ B3-1 15.0 | 2 | 0.303 | 0.201 | 0.109 | 0.0 | 93.5 | 4.5 | 2.0 |
| ▲ B3-1 28.5 | 9.5 | 0.977 | 0.518 | 0.224 | 0.6 | 94.2 | | 5.2 |
| ★ B3-2 13.5 | 4.75 | | | | 0.0 | 26.6 | | 73.4 |
| ◎ B3-2 33.5 | 4.75 | 0.176 | 0.106 | | 0.0 | 88.6 | | 11.4 |

US GRAIN SIZE J042992.01.GPJ US LAB.GDT 11/24/23



GRAIN SIZE DISTRIBUTION
 I-555 Service Road Bridge
 Tyronza River & Ditch No. 6 Strs. & Apprs.
 Poinsett County, Arkansas
 J042992.01



| COBBLES | GRAVEL | | SAND | | | SILT OR CLAY |
|---------|--------|------|--------|--------|------|--------------|
| | coarse | fine | coarse | medium | fine | |

| Specimen Identification | Classification | LL | PL | PI | Cc | Cu |
|-------------------------|-------------------------------|----|----|----|------|------|
| ● B3-3 43.5 | SILTY SAND(SM), A-2-4 | | | | | |
| ☒ B3-4 38.5 | SILTY SAND(SM), A-2-4 | | | | | |
| ▲ B3-4 53.5 | POORLY GRADED SAND(SP), A-1-b | | | | 1.59 | 4.16 |

| Specimen Identification | D100 | D60 | D30 | D10 | %Gravel | %Sand | %Silt | %Clay |
|-------------------------|------|-------|-------|-------|---------|-------|-------|-------|
| ● B3-3 43.5 | 9.5 | 0.145 | 0.088 | | 1.7 | 77.0 | 21.3 | |
| ☒ B3-4 38.5 | 25 | 0.275 | 0.087 | | 6.4 | 66.3 | 27.3 | |
| ▲ B3-4 53.5 | 4.75 | 0.548 | 0.339 | 0.132 | 0.0 | 95.9 | 4.1 | |

US GRAIN SIZE J042992.01.GPJ US LAB.GDT 11/24/23



GRAIN SIZE DISTRIBUTION
 I-555 Service Road Bridge
 Tyronza River & Ditch No. 6 Strs. & Apprs.
 Poinsett County, Arkansas
 J042992.01

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS ASTM D2850

CLIENT : Arkansas State Highway and Transportation Department

DATE: 9/26/2023

PROJECT NO.: J042992.01

PROJECT: ARDOT 2023-2027 Contract TO G003 Tyrnza River & Ditch No. 6 Strs & App. Poinsett County

LOCATION: Tyrnza, AR

BORING NO.: B3-2

SAMPLE NO.: ST-3

DEPTH (ft.): 6.0-8.0

SAMPLE OBTAINED BY: Shelby Tube

CONDITION: Undisturbed

SAMPLE DESCRIPTION: Dark Brown Fat Clay, Trace Sand

LIQUID LIMIT (%): 107

PLASTIC LIMIT (%): 34

PLASTICITY INDEX (%): 73

USCS: CH

SPECIFIC GRAVITY OF SOLIDS: 2.70 (Assumed)

LOAD CELL NO.:

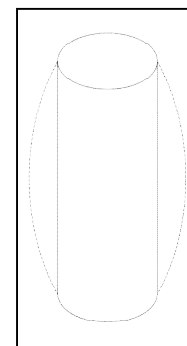
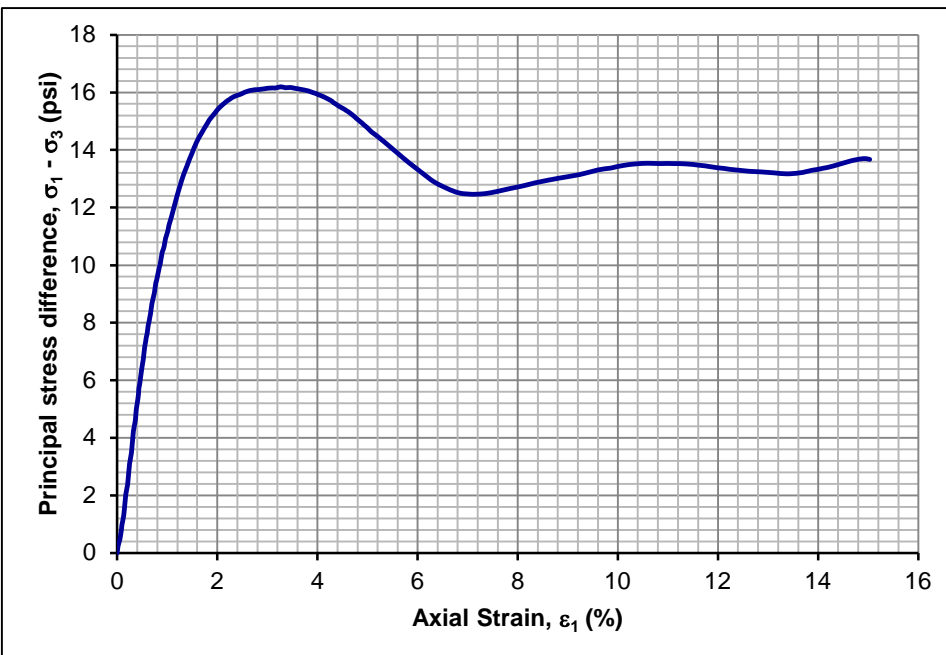
INITIAL SAMPLE DATA

| | |
|---------------------------|-------|
| AVERAGE DIAMETER (in.): | 2.84 |
| HEIGHT (in.): | 5.97 |
| HEIGHT TO DIAMETER RATIO: | 2.11 |
| WET UNIT WEIGHT (pcf): | 103.1 |
| DRY UNIT WEIGHT (pcf): | 72.4 |
| VOID RATIO: | 1.33 |
| MOISTURE CONTENT (%)*: | 42.5 |
| DEGREE OF SATURATION (%): | 86.4 |

FAILURE DATA***

| | |
|--|-------|
| MOISTURE CONTENT AFTER FAILURE (%)**: | 41.4 |
| AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.): | 1.0 |
| AXIAL STRAIN AT FAILURE (%): | 3.3 |
| PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi): | 16.2 |
| MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi): | 4.1 |
| MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi): | 20.3 |
| UNDRAINED COMPRESSIVE STRENGTH, U_u (psf): | 2,330 |
| UNDRAINED SHEAR STRENGTH, s_u (psf): | 1,165 |
| LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf): | N/A |

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 : Arkansas State Highway and Transportation Department

DATE: 9/18/2023

PROJECT NO.: J042992.01

PROJECT: ARDOT 2023-2027 Contract TO G003 Tyronza River & Ditch No. 6 Strs & App. Poinsett County

LOCATION: Tyronza, AR

BORING NO.: B3-2

SAMPLE NO.: ST-5

DEPTH (ft.): 10.0-12.0

SAMPLE OBTAINED BY: Shelby Tube

CONDITION: Undisturbed

SAMPLE DESCRIPTION: Gray Fat Clay with Sand

LIQUID LIMIT (%): 58

PLASTIC LIMIT (%): 23

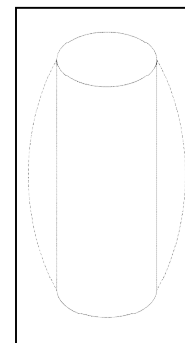
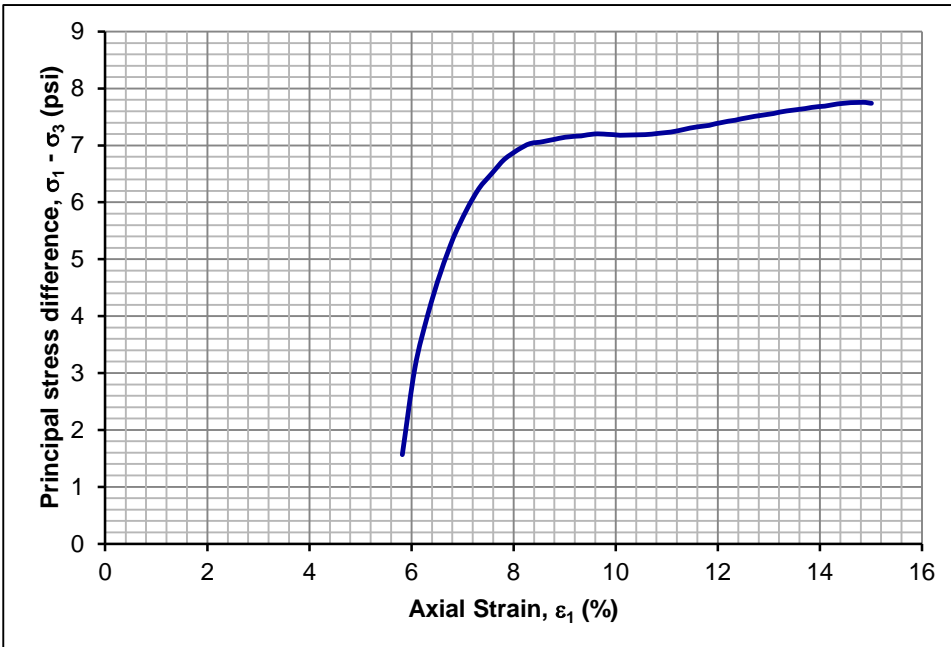
PLASTICITY INDEX (%): 35

USCS: CH

SPECIFIC GRAVITY OF SOLIDS: 2.70 (Assumed)

LOAD CELL NO.:

| INITIAL SAMPLE DATA | | FAILURE DATA*** | |
|---------------------------|-------|--|--------------|
| AVERAGE DIAMETER (in.): | 2.83 | MOISTURE CONTENT AFTER FAILURE (%)**: | 38.8 |
| HEIGHT (in.): | 5.83 | AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.): | 1.0 |
| HEIGHT TO DIAMETER RATIO: | 2.06 | AXIAL STRAIN AT FAILURE (%): | 14.8 |
| WET UNIT WEIGHT (pcf): | 113.6 | PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi): | 7.8 |
| DRY UNIT WEIGHT (pcf): | 84.5 | MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi): | 6.4 |
| VOID RATIO: | 0.99 | MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi): | 14.2 |
| MOISTURE CONTENT (%)*: | 34.4 | UNDRAINED COMPRESSIVE STRENGTH, U_u (psf): | 1,120 |
| DEGREE OF SATURATION (%): | 93.6 | UNDRAINED SHEAR STRENGTH, s_u (psf): | 560 |
| | | LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf): | 1,035 |



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 : Arkansas State Highway and Transportation Department

DATE: 9/18/2023

PROJECT NO.: J042992.01

PROJECT: ARDOT 2023-2027 Contract TO G003 Tyrnza River & Ditch No. 6 Strs & App. Poinsett County

LOCATION: Tyrnza, AR

BORING NO.: B3-3

SAMPLE NO.: ST-4

DEPTH (ft.): 10.0-12.0

SAMPLE OBTAINED BY: Shelby Tube

CONDITION: Undisturbed

SAMPLE DESCRIPTION: Dark Brown/Gray Fat Clay, Trace Organics

LIQUID LIMIT (%): 115

PLASTIC LIMIT (%): 34

PLASTICITY INDEX (%): 81

USCS: CH

SPECIFIC GRAVITY OF SOLIDS: 2.70 (Assumed)

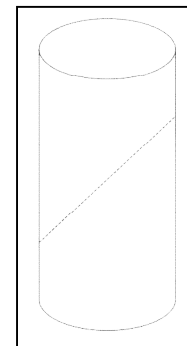
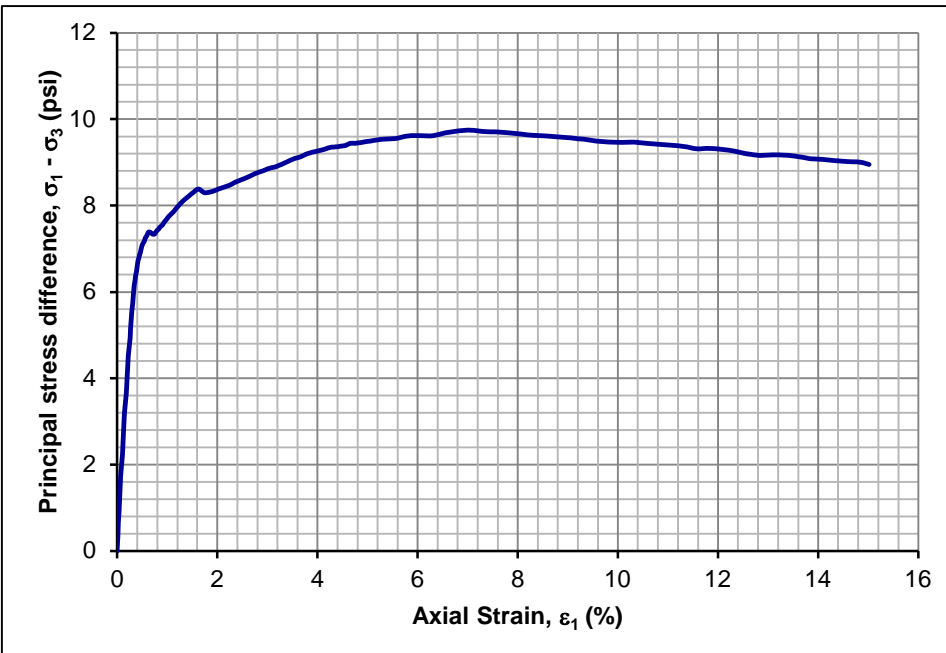
LOAD CELL NO.:

INITIAL SAMPLE DATA

| | |
|---------------------------|-------|
| AVERAGE DIAMETER (in.): | 2.87 |
| HEIGHT (in.): | 5.80 |
| HEIGHT TO DIAMETER RATIO: | 2.03 |
| WET UNIT WEIGHT (pcf): | 109.8 |
| DRY UNIT WEIGHT (pcf): | 75.5 |
| VOID RATIO: | 1.23 |
| MOISTURE CONTENT (%)*: | 45.4 |
| DEGREE OF SATURATION (%): | 99.5 |

FAILURE DATA***

| | |
|--|-------|
| MOISTURE CONTENT AFTER FAILURE (%)**: | 46.4 |
| AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.): | 1.0 |
| AXIAL STRAIN AT FAILURE (%): | 7.1 |
| PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi): | 9.7 |
| MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi): | 6.4 |
| MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi): | 16.2 |
| UNDRAINED COMPRESSIVE STRENGTH, U_u (psf): | 1,400 |
| UNDRAINED SHEAR STRENGTH, s_u (psf): | 700 |
| LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf): | N/A |



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 : Arkansas State Highway and Transportation Department

DATE: 9/25/2023

PROJECT NO.: J042992.01

PROJECT: ARDOT 2023-2027 Contract TO G003 Tyrnza River & Ditch No. 6 Strs & App. Poinsett County

LOCATION: Tyrnza, AR

BORING NO.: B3-3

SAMPLE NO.: ST-8

DEPTH (ft.): 20.0-22.0

SAMPLE OBTAINED BY: Shelby Tube

CONDITION: Undisturbed

SAMPLE DESCRIPTION: Gray/Brown Sandy Fat Clay

LIQUID LIMIT (%): 61

PLASTIC LIMIT (%): 22

PLASTICITY INDEX (%): 39

USCS: CH

SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)

LOAD CELL NO.:

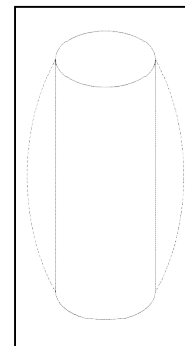
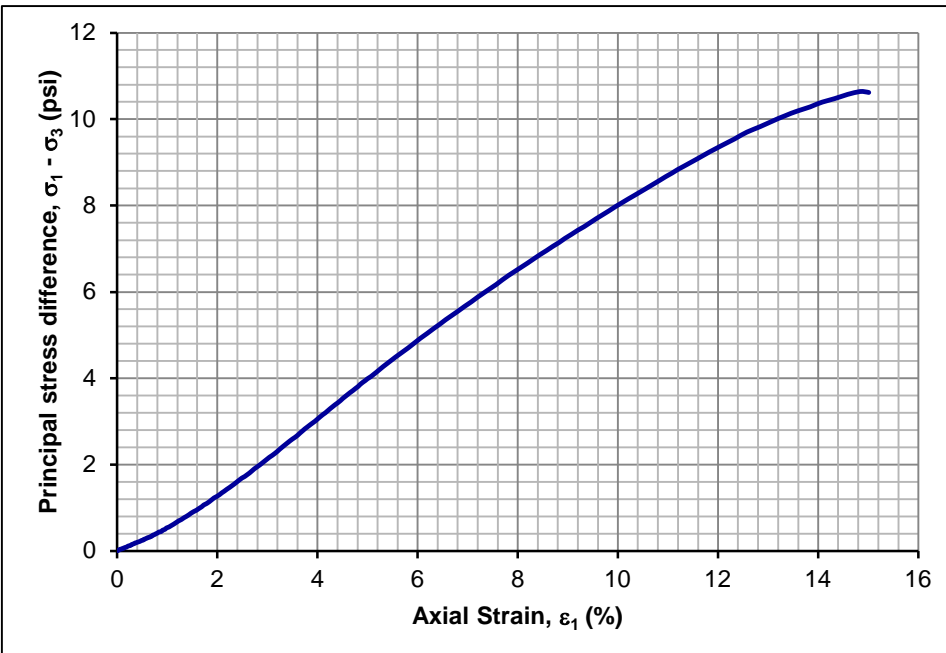
INITIAL SAMPLE DATA

| | |
|---------------------------|-------|
| AVERAGE DIAMETER (in.): | 2.83 |
| HEIGHT (in.): | 6.08 |
| HEIGHT TO DIAMETER RATIO: | 2.15 |
| WET UNIT WEIGHT (pcf): | 123.3 |
| DRY UNIT WEIGHT (pcf): | 94.8 |
| VOID RATIO: | 0.81 |
| MOISTURE CONTENT (%)*: | 30.2 |
| DEGREE OF SATURATION (%): | 100.0 |

FAILURE DATA***

| | |
|--|-------|
| MOISTURE CONTENT AFTER FAILURE (%)**: | 29.5 |
| 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): | 10.6 |
| MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi): | 12.3 |
| MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi): | 22.9 |
| UNDRAINED COMPRESSIVE STRENGTH, U_u (psf): | 1,530 |
| UNDRAINED SHEAR STRENGTH, s_u (psf): | 765 |
| LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf): | 1,155 |

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 : Arkansas State Highway and Transportation Department

DATE: 9/18/2023

PROJECT NO.: J042992.01

PROJECT: ARDOT 2023-2027 Contract TO G003 Tyrnza River & Ditch No. 6 Strs & App. Poinsett County

LOCATION: Tyrnza, AR

BORING NO.: B3-4

SAMPLE NO.: ST-4

DEPTH (ft.): 10.0-12.0

SAMPLE OBTAINED BY: Shelby Tube

CONDITION: Undisturbed

SAMPLE DESCRIPTION: Brown/Gray Fat Clay

LIQUID LIMIT (%): 116

PLASTIC LIMIT (%): 30

PLASTICITY INDEX (%): 86

USCS: CH

SPECIFIC GRAVITY OF SOLIDS: 2.70 (Assumed)

LOAD CELL NO.:

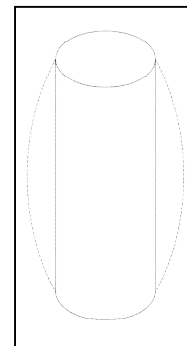
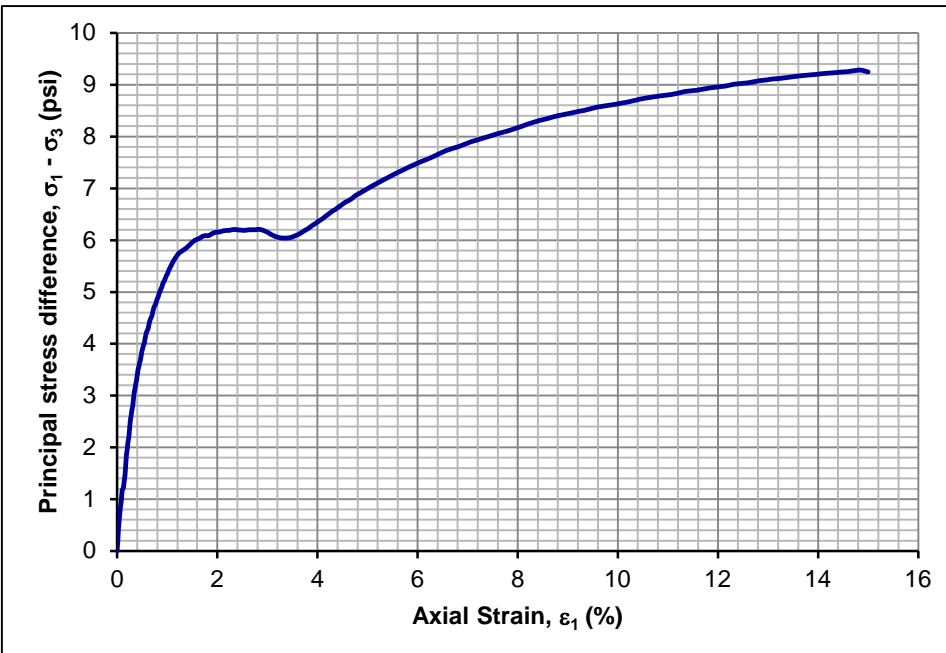
INITIAL SAMPLE DATA

| | |
|---------------------------|-------|
| AVERAGE DIAMETER (in.): | 2.83 |
| HEIGHT (in.): | 5.65 |
| HEIGHT TO DIAMETER RATIO: | 2.00 |
| WET UNIT WEIGHT (pcf): | 113.8 |
| DRY UNIT WEIGHT (pcf): | 78.9 |
| VOID RATIO: | 1.13 |
| MOISTURE CONTENT (%)*: | 44.2 |
| DEGREE OF SATURATION (%): | 100.0 |

FAILURE DATA***

| | |
|--|-------|
| MOISTURE CONTENT AFTER FAILURE (%)**: | 44.3 |
| 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): | 9.3 |
| MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi): | 6.4 |
| MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi): | 15.7 |
| UNDRAINED COMPRESSIVE STRENGTH, U_u (psf): | 1,340 |
| UNDRAINED SHEAR STRENGTH, s_u (psf): | 670 |
| LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf): | 1,245 |

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 : Arkansas State Highway and Transportation Department

DATE: 9/18/2023

PROJECT NO.: J042992.01

PROJECT: ARDOT 2023-2027 Contract TO G003 Tyrnza River & Ditch No. 6 Strs & App. Poinsett County

LOCATION: Tyrnza, AR

BORING NO.: B3-4

SAMPLE NO.: ST-6

DEPTH (ft.): 15.0-17.0

SAMPLE OBTAINED BY: Shelby Tube

CONDITION: Undisturbed

SAMPLE DESCRIPTION: Brown/Gray Fat Clay, Trace Sand

LIQUID LIMIT (%): 65

PLASTIC LIMIT (%): 21

PLASTICITY INDEX (%): 44

USCS: CH

SPECIFIC GRAVITY OF SOLIDS: 2.70 (Assumed)

LOAD CELL NO.:

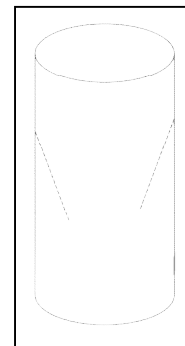
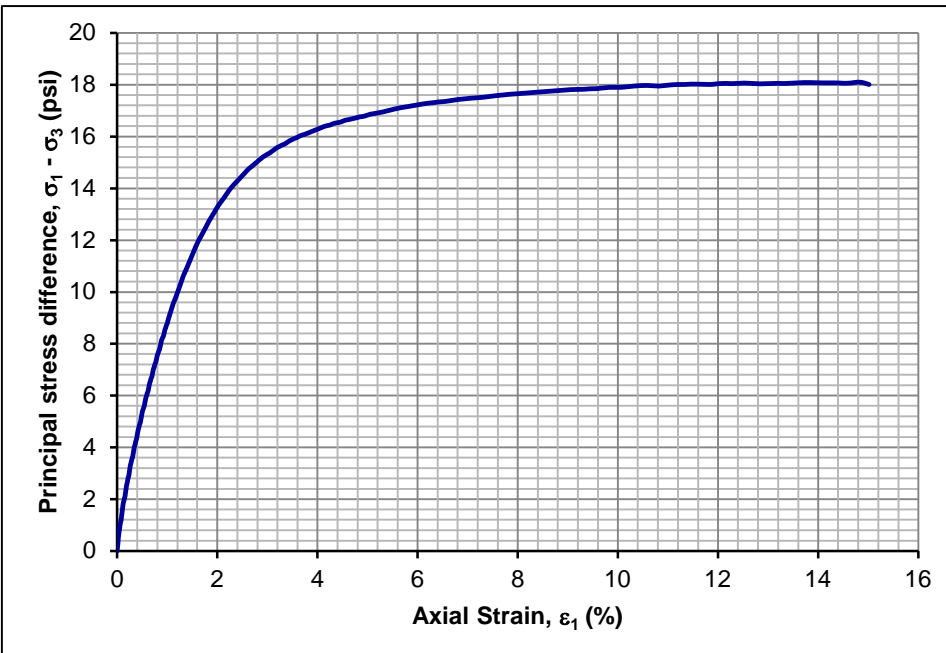
INITIAL SAMPLE DATA

| | |
|---------------------------|-------|
| AVERAGE DIAMETER (in.): | 2.85 |
| HEIGHT (in.): | 5.80 |
| HEIGHT TO DIAMETER RATIO: | 2.04 |
| WET UNIT WEIGHT (pcf): | 118.6 |
| DRY UNIT WEIGHT (pcf): | 88.2 |
| VOID RATIO: | 0.91 |
| MOISTURE CONTENT (%)*: | 34.4 |
| DEGREE OF SATURATION (%): | 100.0 |

FAILURE DATA***

| | |
|--|-------|
| MOISTURE CONTENT AFTER FAILURE (%)**: | 30.4 |
| 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): | 18.1 |
| MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi): | 9.3 |
| MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi): | 27.4 |
| UNDRAINED COMPRESSIVE STRENGTH, U_u (psf): | 2,610 |
| UNDRAINED SHEAR STRENGTH, s_u (psf): | 1,305 |
| LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf): | 2,575 |

FAILURE SHAPES



REMARKS :

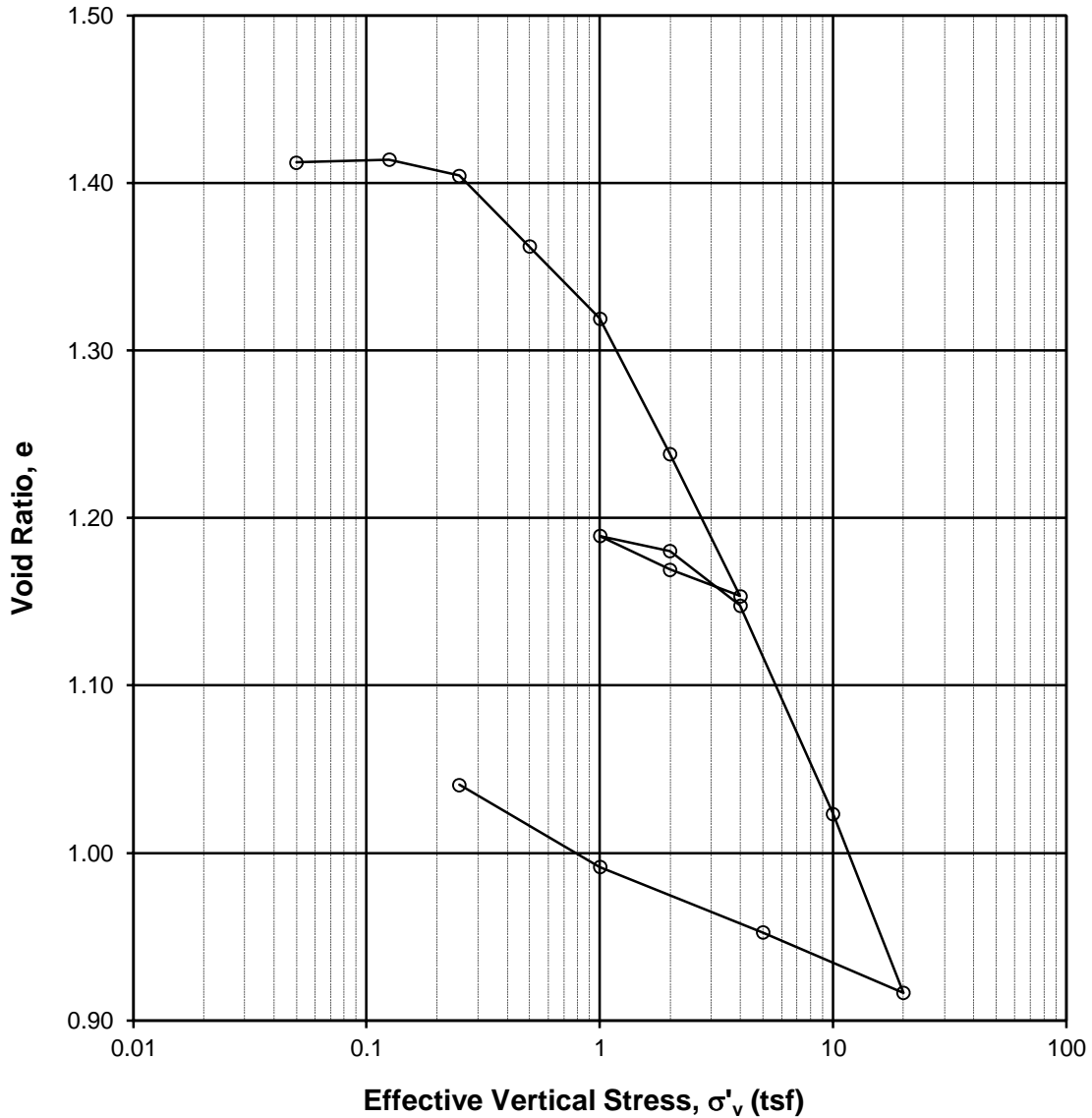
* Initial moisture content determined from sample cuttings.

** Final moisture content determined from entire sample.

*** Failure stress values have been corrected for membrane effects.

Liquid Limit= 115 Plastic Limit= 34 Plasticity Index = 81 USCS: CH

Compression Index, C_c = 0.32 Void Ratio, e_o = 1.41
 Recompression Index, C_r = 0.12 Preconsolidation Pressure = 1.4 tsf



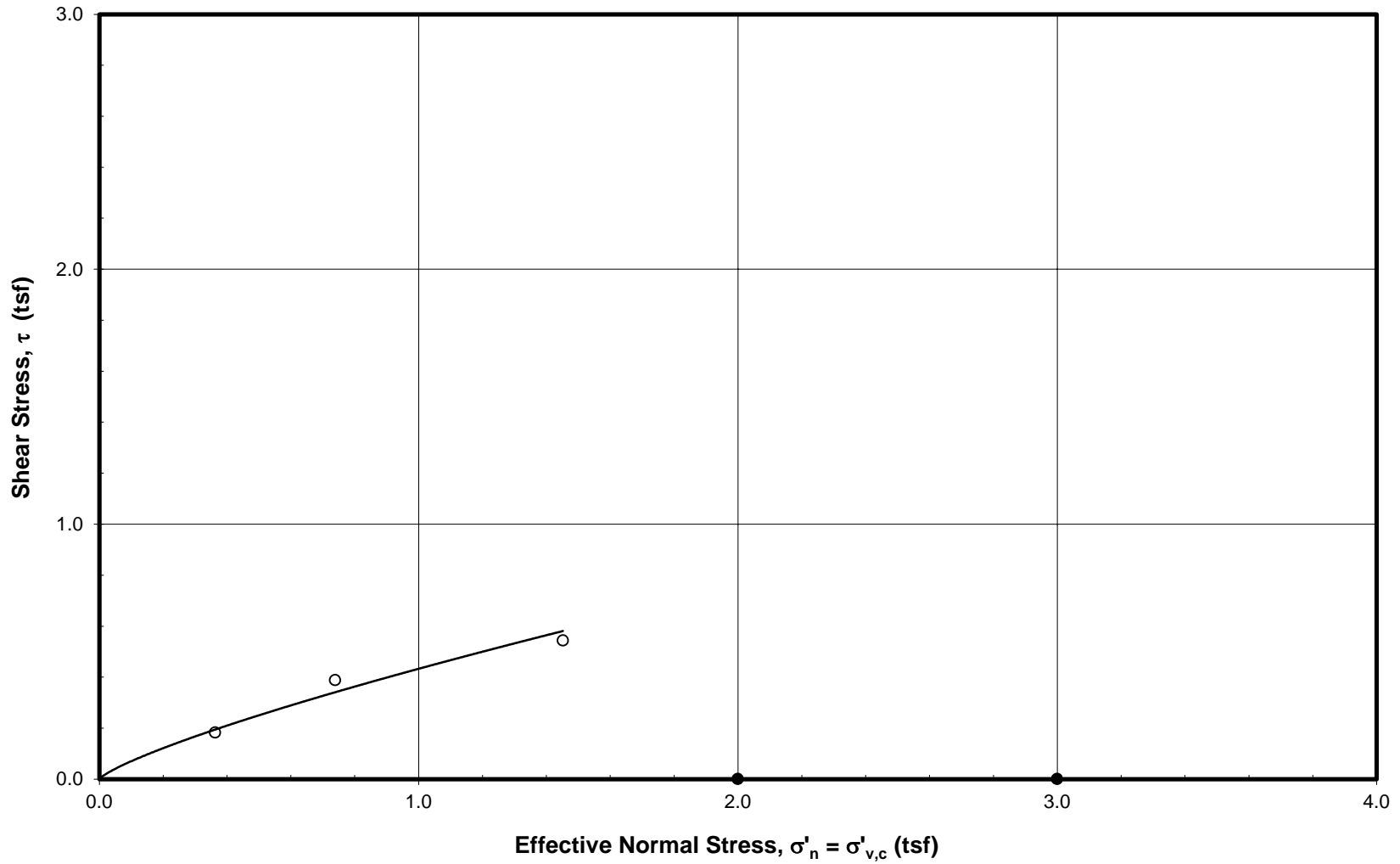
1-D CONSOLIDATION TEST: INCREMENTAL

ASTM D 2435

Project No.: J042992.01

Boring: B3-3

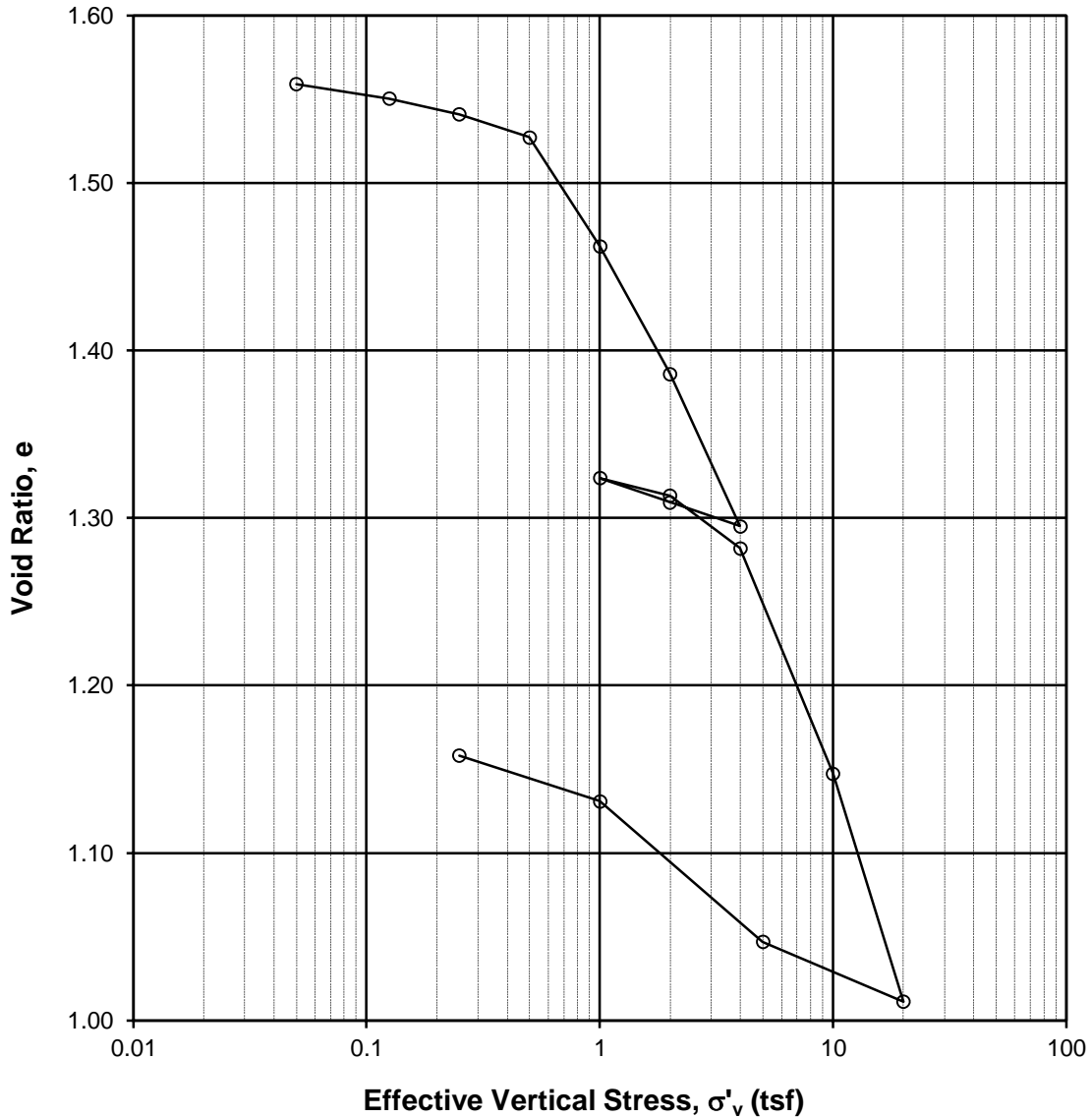
Sample: ST4 - Depth: 10.0



DRAINED DIRECT SHEAR TEST
ASTM D 3080
Boring: B3-4 Sample: ST4 -Depth: 10.0ft

Liquid Limit= 107 Plastic Limit= 34 Plasticity Index = 73 USCS: CH

Compression Index, $C_c = \underline{0.39}$ Void Ratio, $e_o = \underline{1.56}$
 Recompression Index, $C_r = \underline{0.10}$ Preconsolidation Pressure = 1.5 tsf



1-D CONSOLIDATION TEST: INCREMENTAL

ASTM D 2435

Project No.: J042992.01

Boring: B3-2

Sample: ST3 - Depth: 6.0



SOIL RESISTIVITY TEST REPORT

| | | |
|----------------|---|----------------------|
| Project No.: | J042992.01 | September 15th, 2023 |
| Project Name: | ARDOT 101017 Poinsett County – Bridge 3 | Page 1 of 1 |
| Boring Number: | B3-1 | |
| Sample ID: | ST-7 | |
| Depth (ft): | 15.0 | |

MINIMUM LABORATORY SOIL RESISTIVITY AASHTO T288

| <u>Reading</u> | <u>Resistance Measurement</u> | <u>Soil Box Factor (cm)</u> | <u>Soil Resistivity (ohms-cm)</u> | <u>Moisture Content (%)</u> |
|----------------|-------------------------------|-----------------------------|-----------------------------------|-----------------------------|
| #1 | 6,430 | 0.57 | 3,665.10 | 11.1 |
| #2 | 1,630 | 0.57 | 929.10 | 17.3 |
| #3 | 13,600 | 0.57 | 7,752.00 | 26.5 |

Minimum Soil Resistivity 929.10



SOIL RESISTIVITY TEST REPORT

| | | |
|----------------|---|----------------------|
| Project No.: | J042992.01 | September 15th, 2023 |
| Project Name: | ARDOT 101017 Poinsett County – Bridge 3 | Page 1 of 1 |
| Boring Number: | B3-1 | |
| Sample ID: | SS-12 | |
| Depth (ft): | 38.5 | |

MINIMUM LABORATORY SOIL RESISTIVITY AASHTO T288

| <u>Reading</u> | <u>Resistance Measurement</u> | <u>Soil Box Factor (cm)</u> | <u>Soil Resistivity (ohms-cm)</u> | <u>Moisture Content (%)</u> |
|----------------|-------------------------------|-----------------------------|-----------------------------------|-----------------------------|
| #1 | 10,430 | 0.57 | 5,945.10 | 10.2 |
| #2 | 4,030 | 0.57 | 2,297.10 | 17.1 |
| #3 | 3,130 | 0.57 | 1,784.10 | 23.5 |
| #4 | 4,630 | 0.57 | 2,639.10 | 23.2 |

Minimum Soil Resistivity **1,784.10**



SOIL RESISTIVITY TEST REPORT

| | | |
|----------------|---|----------------------|
| Project No.: | J042992.01 | September 15th, 2023 |
| Project Name: | ARDOT 101017 Poinsett County – Bridge 3 | Page 1 of 1 |
| Boring Number: | B3-2 | |
| Sample ID: | SS-13 | |
| Depth (ft): | 48.5 | |

MINIMUM LABORATORY SOIL RESISTIVITY AASHTO T288

| <u>Reading</u> | <u>Resistance Measurement</u> | <u>Soil Box Factor (cm)</u> | <u>Soil Resistivity (ohms-cm)</u> | <u>Moisture Content (%)</u> |
|----------------|-------------------------------|-----------------------------|-----------------------------------|-----------------------------|
| #1 | 11,130 | 0.57 | 6,344.10 | 10.3 |
| #2 | 3,630 | 0.57 | 2,069.10 | 16.9 |
| #3 | 1,630 | 0.57 | 929.10 | 23.9 |
| #4 | 3,430 | 0.57 | 1,955.10 | 24.6 |

Minimum Soil Resistivity 929.10



SOIL RESISTIVITY TEST REPORT

| | | |
|----------------|---|----------------------|
| Project No.: | J042992.01 | September 15th, 2023 |
| Project Name: | ARDOT 101017 Poinsett County – Bridge 3 | Page 1 of 1 |
| Boring Number: | B3-3 | |
| Sample ID: | ST-8 | |
| Depth (ft): | 20.0 | |

MINIMUM LABORATORY SOIL RESISTIVITY AASHTO T288

| <u>Reading</u> | <u>Resistance Measurement</u> | <u>Soil Box Factor (cm)</u> | <u>Soil Resistivity (ohms-cm)</u> | <u>Moisture Content (%)</u> |
|----------------|-------------------------------|-----------------------------|-----------------------------------|-----------------------------|
| #1 | 10,430 | 0.57 | 5,945.10 | 11.6 |
| #2 | 6,730 | 0.57 | 3,836.10 | 18.2 |
| #3 | 6,330 | 0.57 | 3,608.10 | 25.4 |
| #4 | 1,630 | 0.57 | 929.10 | 32.4 |
| #5 | 8,530 | 0.57 | 4,862.10 | 28.3 |

Minimum Soil Resistivity **929.10**



SOIL RESISTIVITY TEST REPORT

| | | |
|----------------|---|----------------------|
| Project No.: | J042992.01 | September 15th, 2023 |
| Project Name: | ARDOT 101017 Poinsett County – Bridge 3 | Page 1 of 1 |
| Boring Number: | B3-3 | |
| Sample ID: | SS-15 | |
| Depth (ft): | 53.5 | |

MINIMUM LABORATORY SOIL RESISTIVITY AASHTO T288

| <u>Reading</u> | <u>Resistance Measurement</u> | <u>Soil Box Factor (cm)</u> | <u>Soil Resistivity (ohms-cm)</u> | <u>Moisture Content (%)</u> |
|----------------|-------------------------------|-----------------------------|-----------------------------------|-----------------------------|
| #1 | 20,330 | 0.57 | 11,588.10 | 10.5 |
| #2 | 11,230 | 0.57 | 6,401.10 | 16.4 |
| #3 | 3,630 | 0.57 | 2,069.10 | 23.6 |
| #4 | 12,330 | 0.57 | 7,028.10 | 23.6 |

Minimum Soil Resistivity **2,069.10**



SOIL RESISTIVITY TEST REPORT

| | | |
|----------------|---|----------------------|
| Project No.: | J042992.01 | September 15th, 2023 |
| Project Name: | ARDOT 101017 Poinsett County – Bridge 3 | Page 1 of 1 |
| Boring Number: | B3-4 | |
| Sample ID: | ST-8 | |
| Depth (ft): | 20.0 | |

MINIMUM LABORATORY SOIL RESISTIVITY AASHTO T288

| <u>Reading</u> | <u>Resistance Measurement</u> | <u>Soil Box Factor (cm)</u> | <u>Soil Resistivity (ohms-cm)</u> | <u>Moisture Content (%)</u> |
|----------------|-------------------------------|-----------------------------|-----------------------------------|-----------------------------|
| #1 | 4,930 | 0.57 | 2,810.10 | 12.3 |
| #2 | 3,235 | 0.57 | 1,843.95 | 19.0 |
| #3 | 3,413 | 0.57 | 1,945.41 | 27.2 |

Minimum Soil Resistivity **1,843.95**



SOIL RESISTIVITY TEST REPORT

| | | |
|----------------|---|----------------------|
| Project No.: | J042992.01 | September 15th, 2023 |
| Project Name: | ARDOT 101017 Poinsett County – Bridge 3 | Page 1 of 1 |
| Boring Number: | B3-4 | |
| Sample ID: | ST-10 | |
| Depth (ft): | 28.5 | |

MINIMUM LABORATORY SOIL RESISTIVITY AASHTO T288

| <u>Reading</u> | <u>Resistance Measurement</u> | <u>Soil Box Factor (cm)</u> | <u>Soil Resistivity (ohms-cm)</u> | <u>Moisture Content (%)</u> |
|----------------|-------------------------------|-----------------------------|-----------------------------------|-----------------------------|
| #1 | 6,225 | 0.57 | 3,548.25 | 11.8 |
| #2 | 2,883 | 0.57 | 1,643.31 | 18.7 |
| #3 | 2,233 | 0.57 | 1,272.81 | 26.5 |
| #4 | 2,113 | 0.57 | 1,204.41 | 33.9 |
| #5 | 2,133 | 0.57 | 1,215.81 | 42.1 |

Minimum Soil Resistivity **1,204.41**

pH TESTS (ASTM D 4972 or AASHTO T-289)



| | | |
|-------------------|---|---------------------------|
| DATE 9/12/2023 | PROJECT NAME ARDOT Poinsett Bridge 3 | PROJECT NO. J042992.01 |
|-------------------|---|---------------------------|

General Test Information: pH Meter: Humboldt Ph Testr H-4371 or _____
 Distilled Water: required pH=5.5 to 7.5 Measured value: _____
 Soil/Water Ratio: Typically 1/1 or 1/2, but 1/5 for lime stabilized soils

| Boring No. | Sample No. | Depth (ft) | Visual Identification (Color, Group Name & Symbol) | Soil : Water Ratio (g/g) or (g/mL) | pH of Solution (Meter/Temp.) | Tare No. Air Drying | Jar Number | Remarks |
|------------|------------|------------|--|------------------------------------|------------------------------|---------------------|------------|---------|
| B3-1 | ST7 | 15.00 | | 1:1 | 7.59 ----- 21.7 | | | |
| B3-1 | SS12 | 38.50 | | 1:1 | 8.14 ----- 21.9 | | | |
| B3-2 | SS13 | 48.50 | | 1:1 | 8.24 ----- 21.8 | | | |
| B3-3 | ST8 | 20.00 | | 1:1 | 7.21 ----- 21.6 | | | |
| B3-3 | SS15 | 53.50 | | 1:1 | 7.69 ----- 21.9 | | | |
| B3-4 | ST8 | 20.00 | | 1:1 | 7.84 ----- 21.6 | | | |
| B3-4 | SS10 | 28.50 | | 1:1 | 5.91 ----- 21.2 | | | |
| | | | | | ----- | | | |
| | | | | | ----- | | | |
| | | | | | ----- | | | |
| | | | | | ----- | | | |
| | | | | | ----- | | | |
| | | | | | ----- | | | |

pH by Meter is Method A; pH by Paper is Method B

Tested By: CG
Date: 09/14/23

Calculated By: HP
Date: 09/15/23

Checked By: DFB
Date: 11/14/23



APPENDIX F – SITE-SPECIFIC SEISMIC STUDY

**Site-Specific Seismic Study
ARDOT G003 101017 Tyronza River &
Ditch No. 6 Strs. & Apprs.
Bridge 3
Pointsett County, Arkansas**

By

Shahram Pezeshk, Ph.D., P.E.

Email: s.pezeshk@aol.com

901-606-6934

October 7, 2023

TABLE OF CONTENTS

| | <u>Page</u> |
|---|-------------|
| 1.0. EXECUTIVE SUMMARY | 1 |
| 2.0. SCOPE OF WORK..... | 2 |
| 3.0. SUBSURFACE CONDITIONS | 2 |
| 4.0. SHEAR-WAVE VELOCITY PROFILE..... | 3 |
| 5.0. GENERAL INFORMATION..... | 5 |
| 6.0. REGIONAL SEISMICITY..... | 5 |
| 7.0. SEISMIC HAZARD ANALYSIS | 6 |
| 7.1. SEISMIC SOURCE MODELS | 6 |
| 7.2. GROUND MOTION MODELS..... | 6 |
| 7.3. TREATMENT OF UNCERTAINTIES..... | 8 |
| 8.0. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2 nd Edition, 2022 Interim Revisions | 8 |
| 8.1. Dynamic Soil Properties..... | 8 |
| 9.0. CODE-BASED DESIGN APPROACH | 9 |
| 9.1. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2 nd Edition, 2022 Interim Revisions | 9 |
| 10.0. SITE-SPECIFIC PROCEDURE..... | 10 |
| 10.1. Seismic Hazard Analysis | 12 |
| 10.2. Variability in Soil's Shear-Wave and Thickness Profile..... | 12 |
| 10.3. Site-Specific Results..... | 13 |
| 11.0. DESIGN RESPONSE SPECTRAL PARAMETERS | 13 |
| 12.0. LIMITATIONS OF THE REPORT..... | 17 |
| 13.0. REFERENCES | 17 |
| APPENDIX A. Site Location | 20 |

Site-Specific Seismic Study ARDOT G003 101017 Tyronza River & Ditch No. 6 Strs. & Apprs. Bridge 3 Pointsett County, Arkansas

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 35.4793688°N and 90.348355°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_{DI} (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). The shear-wave velocity profiles obtained from SCPT are provide Table 1 and are shown in Figure 1.

Table 1. Shear-Wave Velocities Measured.

| CPT1 | | |
|------------|-------------|----------------|
| Depth1(ft) | Depth2(ft.) | velocity(ft/s) |
| 1.54 | 4.85 | 857.39 |
| 4.85 | 8.13 | 857.39 |
| 8.13 | 11.45 | 857.39 |
| 11.45 | 14.73 | 636.65 |
| 14.73 | 18.01 | 493.94 |
| 18.01 | 21.32 | 530.67 |
| 21.32 | 24.57 | 839.12 |
| 24.57 | 27.81 | 855.85 |
| 27.81 | 31.09 | 747.97 |
| 31.09 | 34.37 | 693.10 |
| 34.37 | 37.65 | 701.49 |
| 37.65 | 40.93 | 697.23 |
| 40.93 | 44.18 | 710.02 |
| 44.18 | 47.43 | 698.77 |
| 47.43 | 50.68 | 712.06 |
| 50.68 | 53.92 | 813.60 |
| 53.92 | 57.33 | 717.73 |
| 57.33 | 60.55 | 918.01 |
| 60.55 | 63.83 | 832.10 |
| 63.83 | 67.04 | 815.90 |
| 67.04 | 70.45 | 1022.84 |
| 70.45 | 73.70 | 777.36 |
| 73.70 | 76.95 | 748.82 |
| 76.95 | 80.20 | 986.56 |
| 80.20 | 83.61 | 934.14 |
| 83.61 | 86.79 | 695.66 |
| 86.79 | 90.04 | 1208.32 |
| 90.04 | 93.38 | 926.96 |
| 93.38 | 96.66 | 1009.94 |
| 96.66 | 100.04 | 1154.43 |

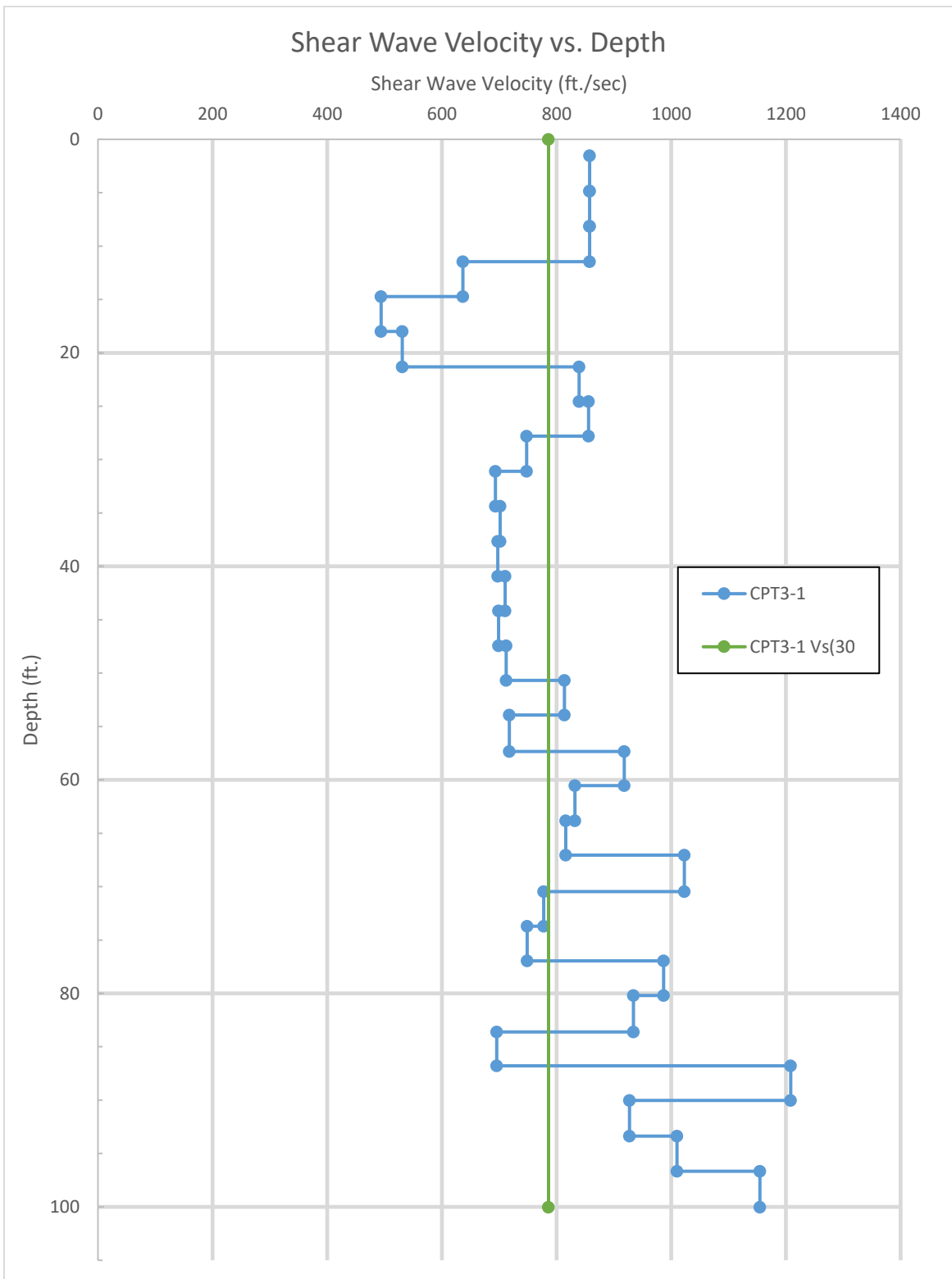


Figure 1. Shear-Wave Velocities Measured by Geotechnology.

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 2 shows the USGS 2018 declustered catalog for CEUS.

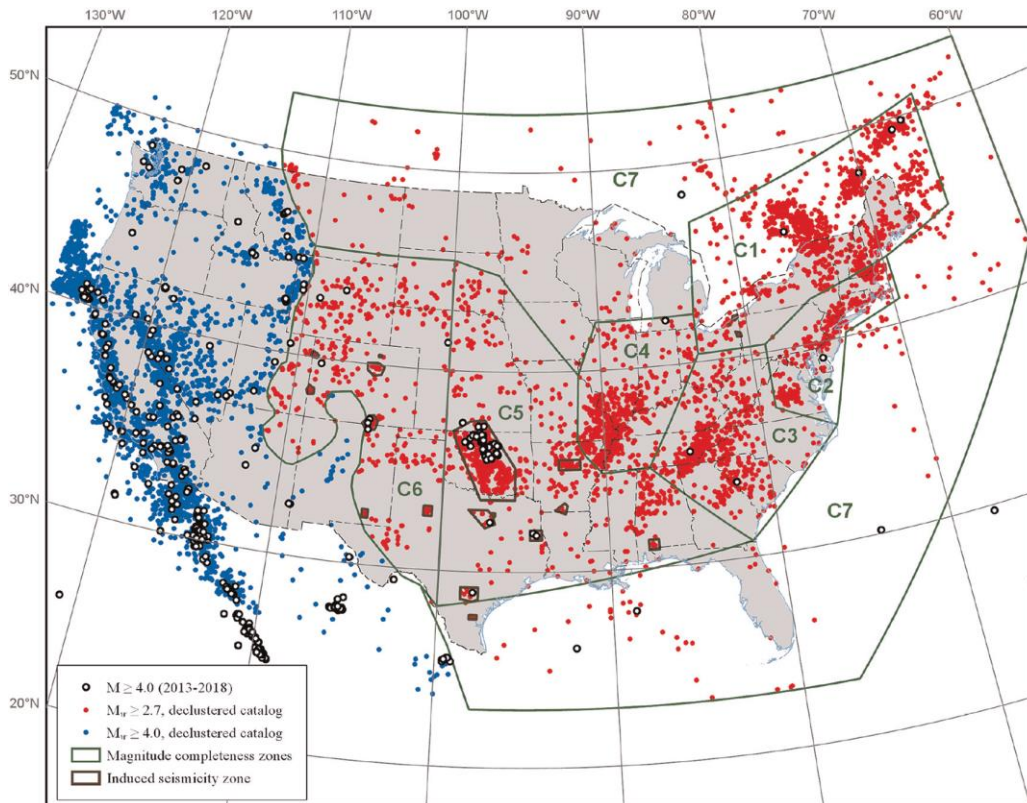


Figure 2. The 2018 NSHM Declustered Catalog for Central and Eastern United States (red) and Western United States (blue).

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).[Source Rezaeian et al. (2021)].

| 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 | Shajouei-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) | Shajouei-Pezeshk | 0 |
| Graizer (replaced with Graizer16 & Graizer17 by USGS) | Graizer | 0 |
| PEER-EX | Hollenback-Kuehn-Goulet-Abrahamson | 0 |
| 17 NGA-East GMMs (used by USGS in 2018 NSHM) | | 0.667 |
| Models 1 to 17 | NGA-East Project | Period-dependen ^a |

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 785 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 | 1.000 |
| F_v | 1.548 |
| F_{PGA} | 1.000 |
| S_s | 1.714 |
| S_1 | 0.452 |
| S_{DS} | 1.714 |
| S_{D1} | 0.700 |
| PGA | 0.968 |
| A_s | 0.968 |

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 3. The same process was repeated for all eleven earthquakes for both components.

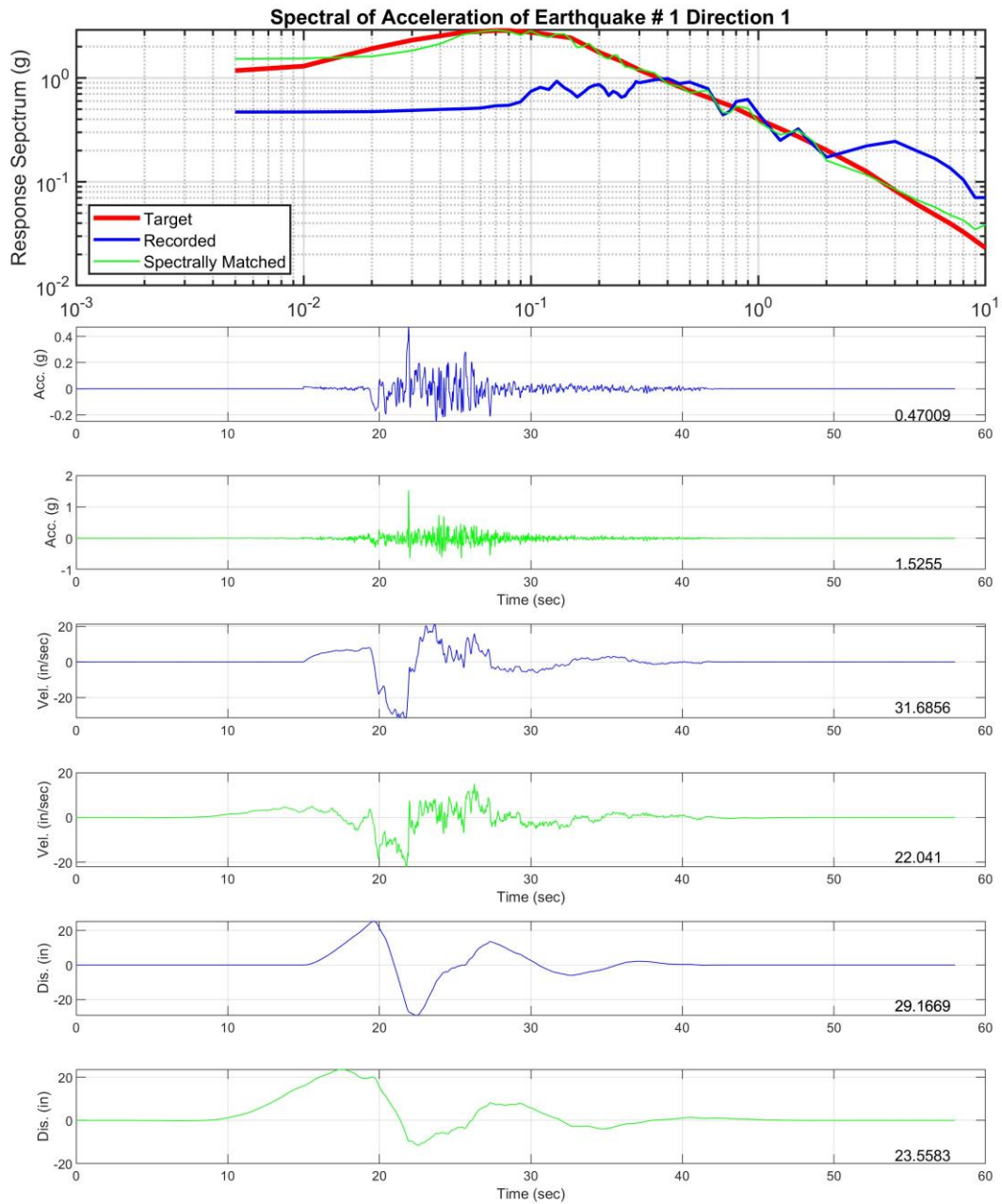


Figure 3. Time Histories Before and After the Spectral Matching Process for Earthquake #1. The numbers Shown in the Bottom right of Each Figure Represent the Absolute Maximum Value of the Graph.

Table 4. Deaggregation.

| Mean and Mode Deaggregation Parameter at 1,033 Years | | | | | |
|--|------|--------|--------|------|--------|
| Mean | | | Mode | | |
| Period | M | R (km) | Period | M | R (km) |
| PGA | 7.35 | 14.34 | PGA | 7.52 | 14.78 |
| 0.01 | 7.36 | 14.41 | 0.01 | 7.54 | 13.72 |
| 0.02 | 7.33 | 14.69 | 0.02 | 7.52 | 14.80 |
| 0.03 | 7.34 | 14.80 | 0.03 | 7.52 | 14.81 |
| 0.05 | 7.35 | 15.27 | 0.05 | 7.52 | 14.80 |
| 0.08 | 7.36 | 15.70 | 0.08 | 7.52 | 14.79 |
| 0.10 | 7.38 | 16.23 | 0.10 | 7.52 | 14.80 |
| 0.15 | 7.42 | 16.69 | 0.15 | 7.52 | 14.80 |
| 0.20 | 7.43 | 17.13 | 0.20 | 7.52 | 14.83 |
| 0.25 | 7.45 | 17.56 | 0.25 | 7.54 | 13.72 |
| 0.30 | 7.46 | 17.99 | 0.30 | 7.54 | 13.72 |
| 0.40 | 7.48 | 18.51 | 0.40 | 7.54 | 13.73 |
| 0.50 | 7.49 | 18.99 | 0.50 | 7.54 | 13.73 |
| 0.75 | 7.51 | 20.33 | 0.75 | 7.54 | 13.76 |
| 1.00 | 7.52 | 21.53 | 1.00 | 7.54 | 13.88 |
| 1.50 | 7.55 | 22.88 | 1.50 | 7.55 | 14.33 |
| 2.00 | 7.57 | 24.37 | 2.00 | 7.55 | 14.33 |
| 3.00 | 7.59 | 25.14 | 3.00 | 7.76 | 13.76 |
| 4.00 | 7.61 | 25.98 | 4.00 | 7.76 | 13.76 |
| 5.00 | 7.63 | 26.74 | 5.00 | 7.76 | 13.77 |
| 7.50 | 7.65 | 27.90 | 7.50 | 7.76 | 13.86 |
| 10.00 | 7.66 | 28.44 | 10.00 | 7.59 | 12.81 |

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 4 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 S_a values provided in Table 5. Figure 5 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 5, 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 | 0.645 |
| 0.030 | 0.828 |
| 0.040 | 0.889 |
| 0.050 | 0.950 |
| 0.070 | 1.072 |
| 0.100 | 1.143 |
| 0.150 | 1.194 |
| 0.200 | 1.219 |
| 0.250 | 1.184 |
| 0.300 | 1.197 |
| 0.400 | 1.244 |
| 0.500 | 1.201 |
| 0.750 | 1.023 |
| 1.000 | 0.999 |
| 1.500 | 0.666 |
| 2.000 | 0.480 |
| 3.000 | 0.230 |
| 4.000 | 0.122 |
| 5.000 | 0.093 |
| 7.500 | 0.062 |
| 10.000 | 0.070 |

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

| PARAMETER | DESIGN ACCELERATION PARAMETERS (g) |
|-----------|------------------------------------|
| S_{DS} | 1.219 |
| S_{DI} | 0.999 |
| S_{MS} | 1.219 |
| S_{MI} | 0.999 |
| MCE_G | 0.645 |

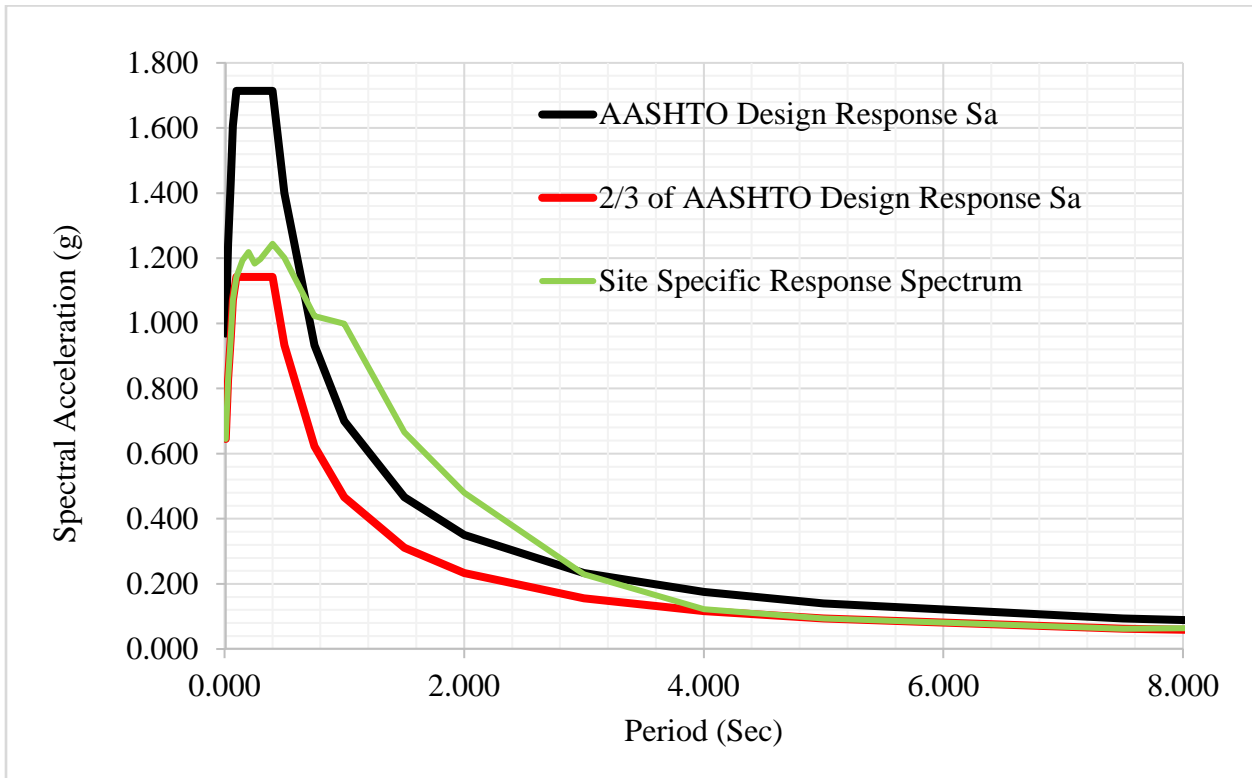


Figure 4. Site-Specific Design Response Spectrum, AASHTO Guide Specifications Design Response Spectrum, and 2/3 of the AASHTO Guide Specifications Design Response Spectrum.

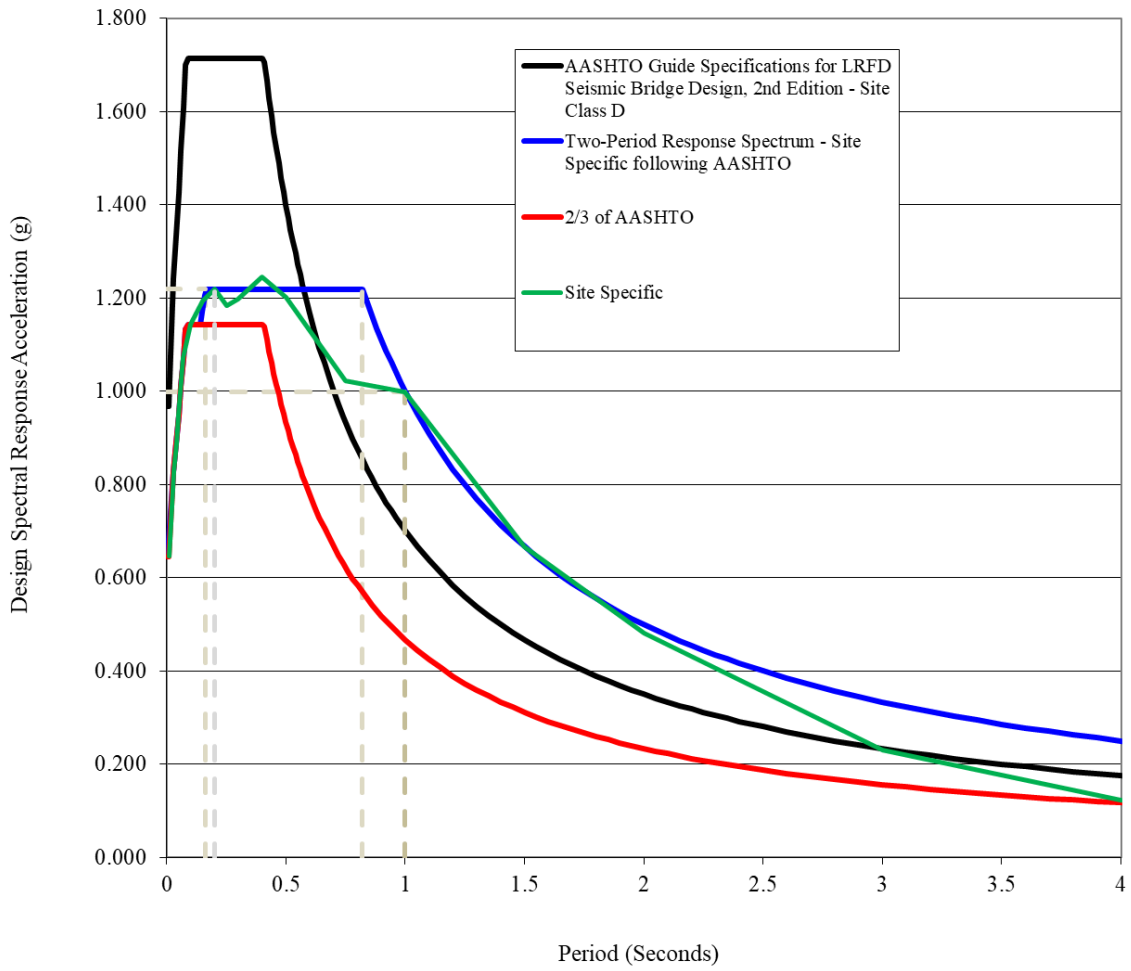


Figure 5. Design Response Spectrum based on AASHTO Guide Specifications, 2/3 of the AASHTO Guide Specifications Site-Specific, and Design Response Spectrum Based on PGA, 0.2, and 1 Second.

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.

13.0 REFERENCES

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APPENDIX A. Site Location



APPENDIX G – AASHTO AND USCS CLASSIFICATIONS

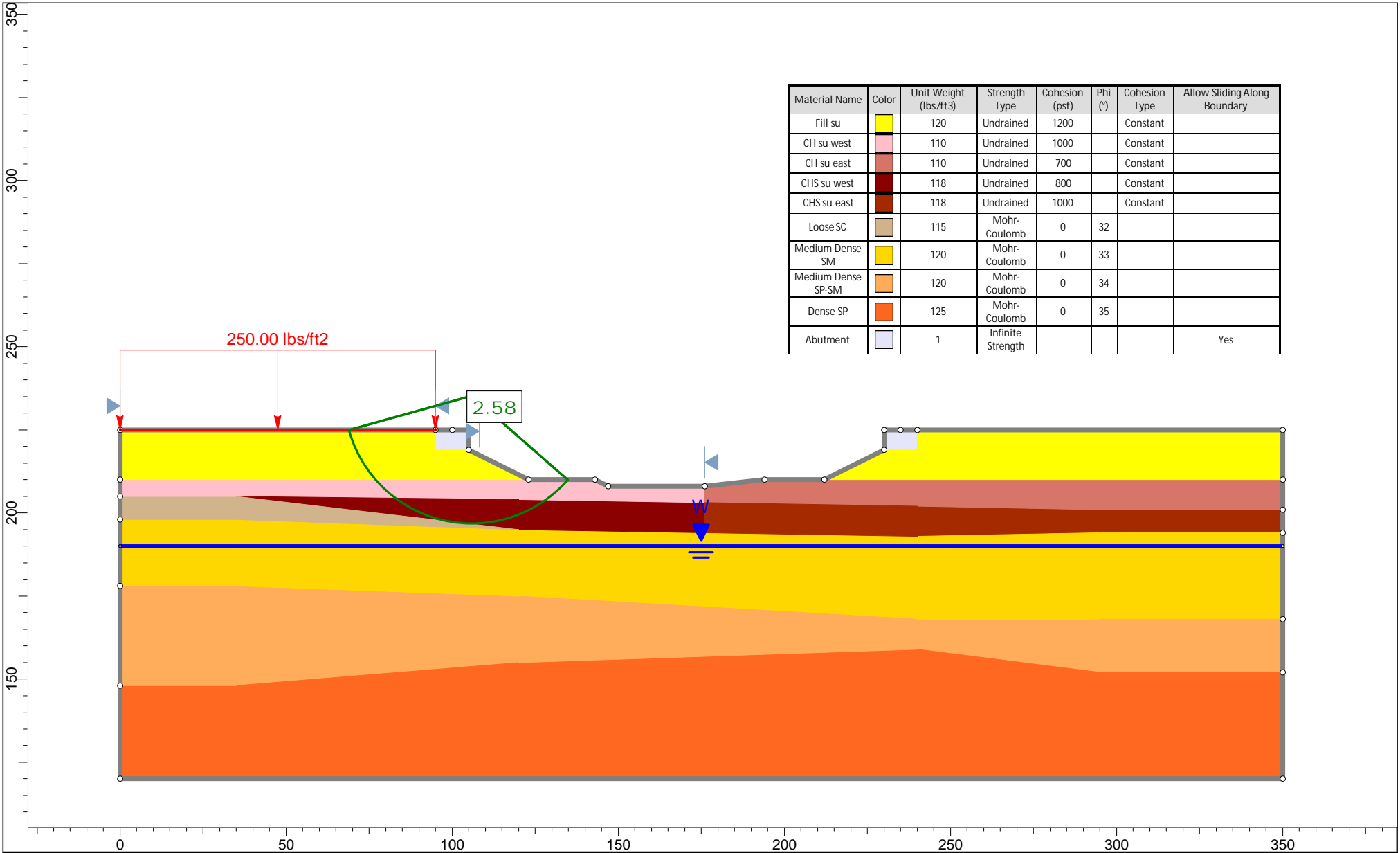


| ARDOT I-555 Service Road J042992.01 | | | | | | | | | | | | | | | | | |
|---|-------|-------------------|--------------------|-----------------------|-----------------------------------|-------|---------|---------|-------|-------|------|------|-------|---------------|-------------|--|--|
| | | Atterberg Only | | | | | | | | | | | | | | | |
| | | Sieve Only | | | | | | | | | | | | | | | |
| | | Att and Sieve | | | | | | | | | | | | | | | |
| Borehole | Depth | Liquid Limit (LL) | Plastic Limit (PL) | Plasticity Index (PI) | Sieve Analysis Percent Passing | | | | | | | | GI | AASHTO CLASS. | USCS CLASS. | | |
| | | | | | 2 in. | 1 in. | 3/4 in. | 3/8 in. | #4 | #10 | #40 | #200 | | | | | |
| B3-1 | 10 | 104 | 32 | 72 | | | | | | 100 | | 73 | 56 | A-7-5 | CH | | |
| B3-1 | 11.5 | 104 | 32 | 72 | | | | | | 100 | | 9.9 | 0 | A-2-7 | SP-SC | | |
| B3-1 | 15 | 58 | 23 | 35 | | | | | | 100.0 | 93.3 | 6.5 | 0 | A-2-7 | SP-SC | | |
| B3-1 | 28.5 | | | | | | | 100.0 | 99.4 | 88 | 20.5 | 5.2 | 0 | A-1-b | SP-SM | | |
| B3-2 | 6 | 107 | 34 | 73 | | | | | | | | 90 | 77 | A-7-5 | CH | | |
| B3-2 | 10 | 58 | 23 | 35 | | | | | | | | 73 | 26 | A-7-6 | CH | | |
| B3-2 | 13.5 | 58 | 23 | 35 | | | | | 100.0 | 100 | 99.6 | 73.4 | 26 | A-7-6 | CH | | |
| B3-2 | 33.5 | | | | | | | 100.0 | 100.0 | 99.6 | 11.4 | 0 | A-2-4 | SP-SM | | | |
| B3-3 | 10 | 115 | 34 | 81 | | | | | | | | 90 | 85 | A-7-5 | CH | | |
| B3-3 | 15 | 61 | 22 | 39 | | | | | | | | 73 | 28 | A-7-6 | CH | | |
| B3-3 | 20 | 31 | 23 | 8 | | | | | | | | 73 | 5 | A-4 | ML | | |
| B3-3 | 43.5 | 31 | 23 | 8 | | | | 100.0 | 98.3 | 94.0 | 93.8 | 21.3 | 0 | A-2-4 | SM | | |
| B3-4 | 10 | 116 | 30 | 86 | | | | | | | | 90 | 89 | A-7-5 | CH | | |
| B3-4 | 15 | 65 | 21 | 44 | | | | | | | | 73 | 32 | A-7-6 | CH | | |
| B3-4 | 20 | 25 | 22 | 3 | | | | | | | | 73 | 1 | A-4 | ML | | |
| B3-4 | 38.5 | 25 | 22 | 3 | | 100.0 | 97.0 | 97.0 | 93.6 | 89 | 71.0 | 27.3 | 0 | A-2-4 | SM | | |
| B3-4 | 53.5 | | | | | | | | 100.0 | 99.9 | 37.9 | 4.1 | 0 | A-1-b | SP | | |




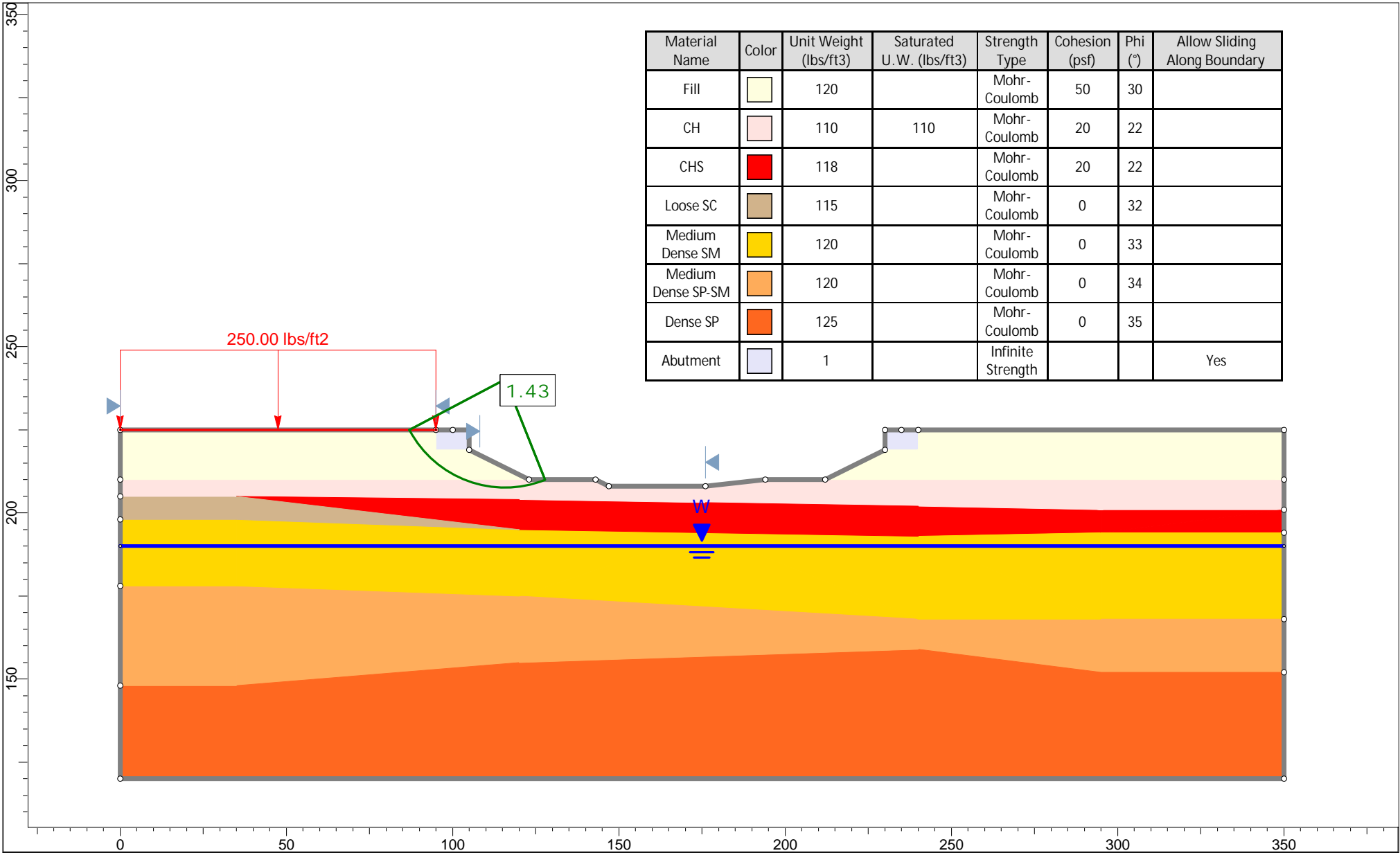
APPENDIX H – GLOBAL STABILITY ANALYSIS RESULTS




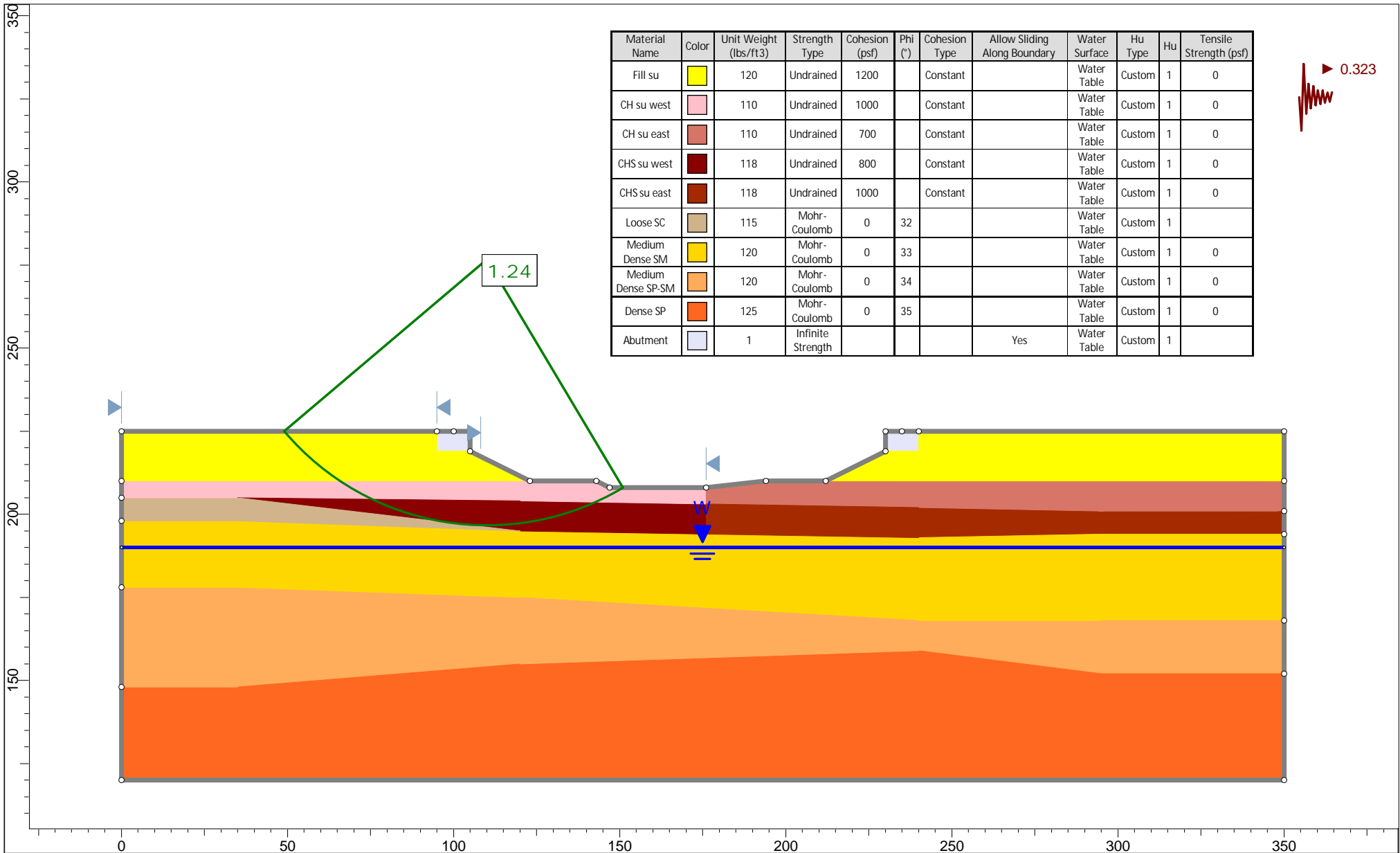


| Material Name | Color | Unit Weight (lbs/ft ³) | Strength Type | Cohesion (psf) | Phi (°) | Cohesion Type | Allow Sliding Along Boundary |
|--------------------|---------------|------------------------------------|-------------------|----------------|---------|---------------|------------------------------|
| Fill su | Yellow | 120 | Undrained | 1200 | | Constant | |
| CH su west | Pink | 110 | Undrained | 1000 | | Constant | |
| CH su east | Red | 110 | Undrained | 700 | | Constant | |
| CHS su west | Dark Red | 118 | Undrained | 800 | | Constant | |
| CHS su east | Brown | 118 | Undrained | 1000 | | Constant | |
| Loose SC | Light Brown | 115 | Mohr-Coulomb | 0 | 32 | | |
| Medium Dense SM | Yellow-Orange | 120 | Mohr-Coulomb | 0 | 33 | | |
| Medium Dense SP-SM | Orange | 120 | Mohr-Coulomb | 0 | 34 | | |
| Dense SP | Dark Orange | 125 | Mohr-Coulomb | 0 | 35 | | |
| Abutment | Light Blue | 1 | Infinite Strength | | | | Yes |

| | | | | |
|--|----------------------|---|-------------------------------------|--------------------------|
|  | Project | | I-555 Service Road over Ditch No. 6 | |
| | Group | I-555 Service Road over Ditch No. 6 - West Abutment | Scenario | Short-Term Stability |
| | Drawn By | DFB | Company | Geotechnology, LLC |
| | Date | 10/6/23 | File Name | J042992.01 Bridge 3.slmd |
| | SLIDEINTERPRET 9.029 | | | |



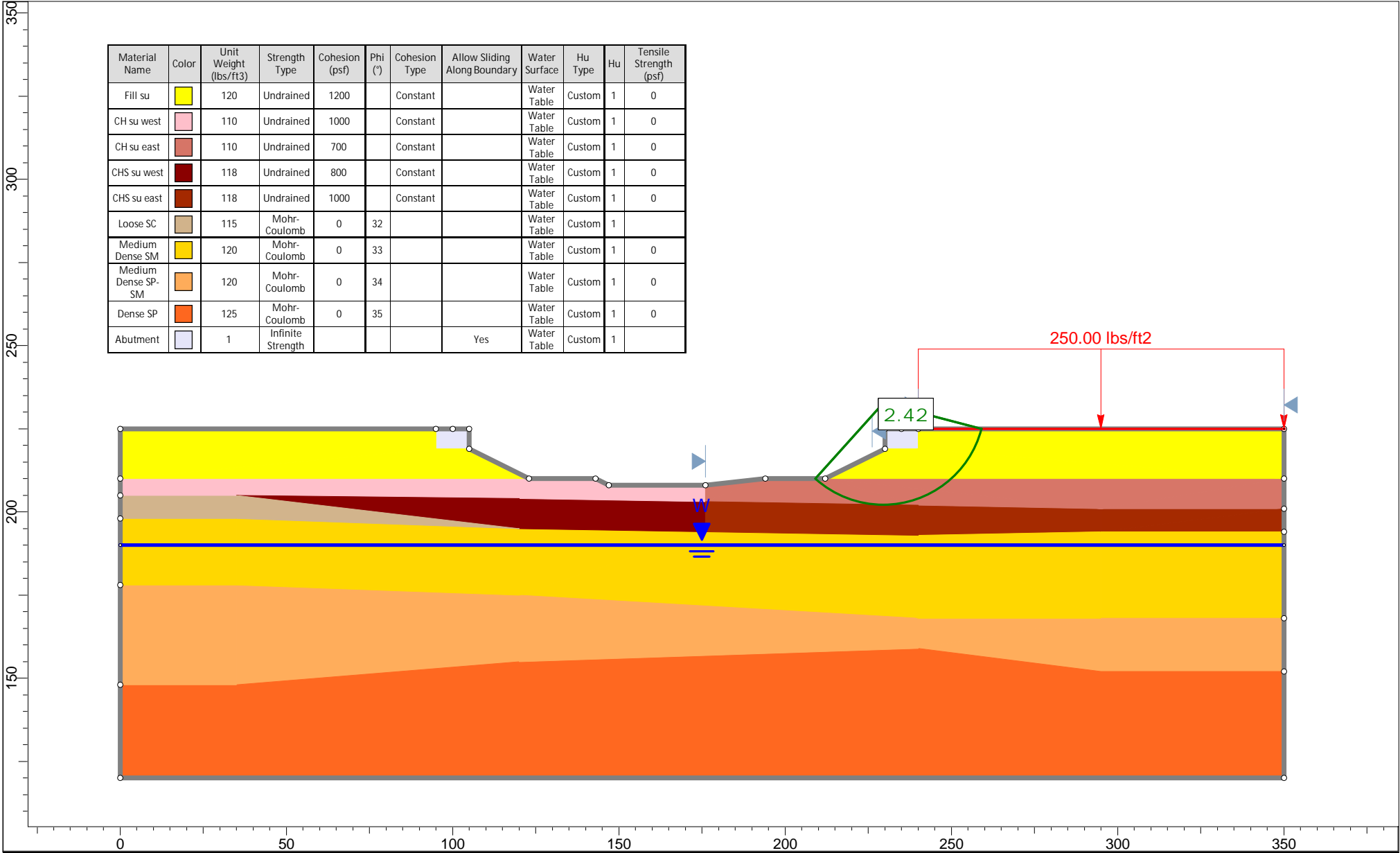
| | | | | |
|--|----------------------|---|-------------------------------------|--------------------------|
|  | Project | | I-555 Service Road over Ditch No. 6 | |
| | Group | I-555 Service Road over Ditch No. 6 - West Abutment | Scenario | Long-Term Stability |
| | Drawn By | DFB | Company | Geotechnology, LLC |
| | Date | 10/6/23 | File Name | J042992.01 Bridge 3.slmd |
| | SLIDEINTERPRET 9.029 | | | |



| Material Name | Color | Unit Weight (lbs/ft ³) | Strength Type | Cohesion (psf) | Phi (°) | Cohesion Type | Allow Sliding Along Boundary | Water Surface | Hu Type | Hu | Tensile Strength (psf) |
|--------------------|--------------|------------------------------------|-------------------|----------------|---------|---------------|------------------------------|---------------|---------|----|------------------------|
| Fill su | Yellow | 120 | Undrained | 1200 | | Constant | | Water Table | Custom | 1 | 0 |
| CH su west | Pink | 110 | Undrained | 1000 | | Constant | | Water Table | Custom | 1 | 0 |
| CH su east | Red | 110 | Undrained | 700 | | Constant | | Water Table | Custom | 1 | 0 |
| CHS su west | Dark Red | 118 | Undrained | 800 | | Constant | | Water Table | Custom | 1 | 0 |
| CHS su east | Brown | 118 | Undrained | 1000 | | Constant | | Water Table | Custom | 1 | 0 |
| Loose SC | Tan | 115 | Mohr-Coulomb | 0 | 32 | | | Water Table | Custom | 1 | |
| Medium Dense SM | Yellow | 120 | Mohr-Coulomb | 0 | 33 | | | Water Table | Custom | 1 | 0 |
| Medium Dense SP-SM | Light Orange | 120 | Mohr-Coulomb | 0 | 34 | | | Water Table | Custom | 1 | 0 |
| Dense SP | Dark Orange | 125 | Mohr-Coulomb | 0 | 35 | | | Water Table | Custom | 1 | 0 |
| Abutment | Light Blue | 1 | Infinite Strength | | | | Yes | Water Table | Custom | 1 | |



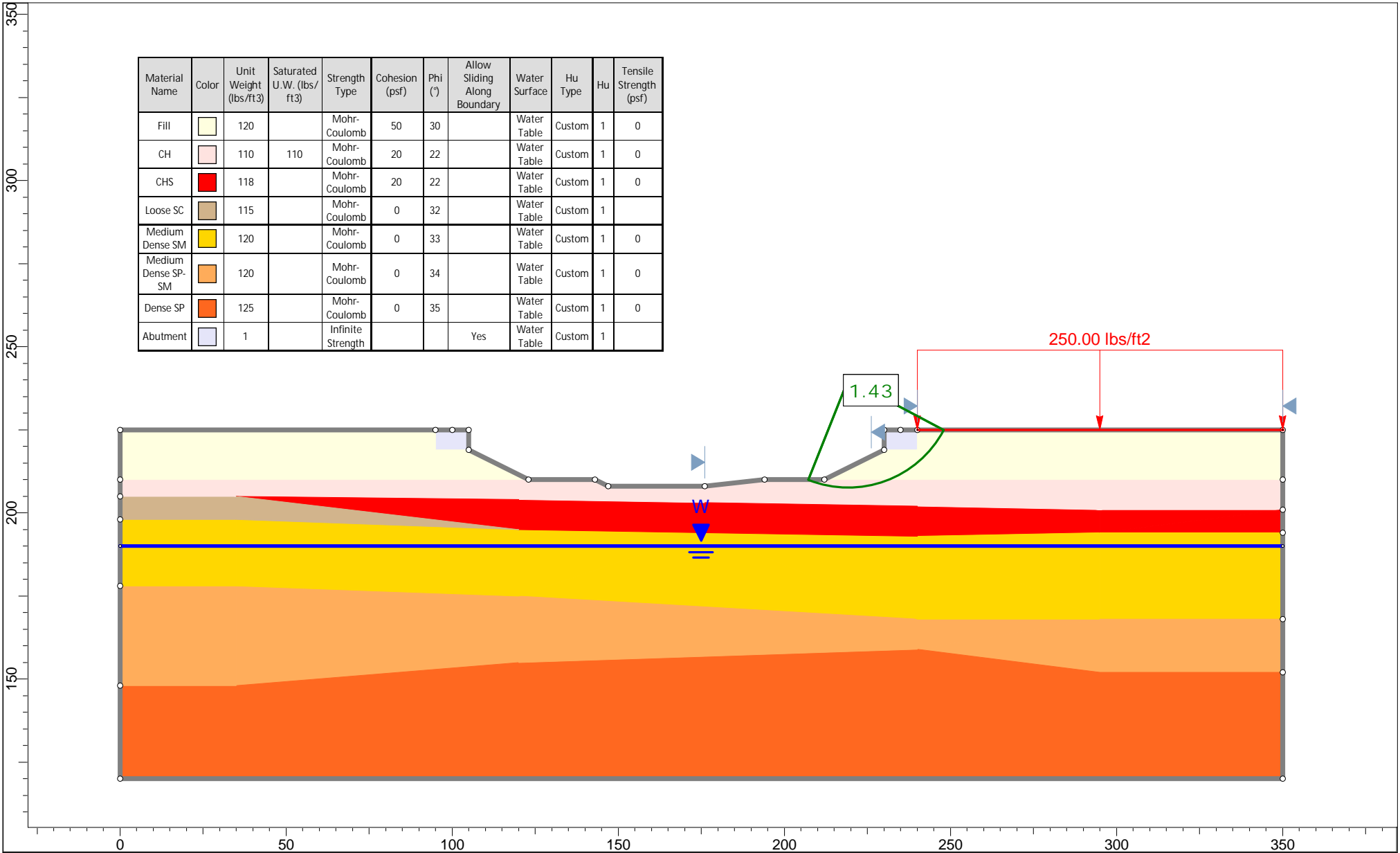
| | | | |
|----------|---|-------------------------------------|--------------------------|
| Project | | I-555 Service Road over Ditch No. 6 | |
| Group | I-555 Service Road over Ditch No. 6 - West Abutment | Scenario | Seismic Stability |
| Drawn By | DFB | Company | Geotechnology, LLC |
| Date | 10/6/23 | File Name | J042992.01 Bridge 3.slmd |



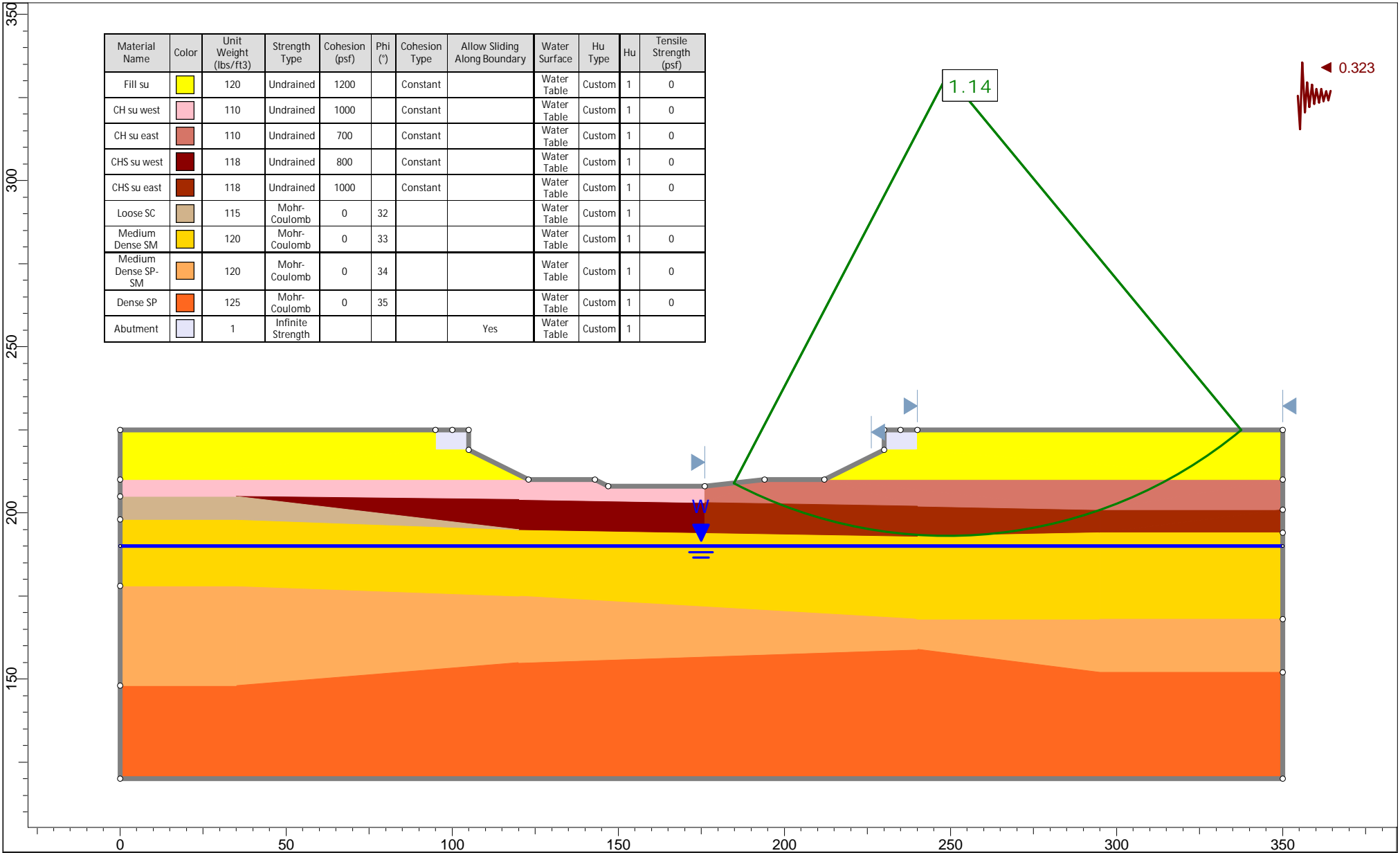
| Material Name | Color | Unit Weight (lbs/ft ³) | Strength Type | Cohesion (psf) | Phi (°) | Cohesion Type | Allow Sliding Along Boundary | Water Surface | Hu Type | Hu | Tensile Strength (psf) |
|--------------------|-------------|------------------------------------|-------------------|----------------|---------|---------------|------------------------------|---------------|---------|----|------------------------|
| Fill su | Yellow | 120 | Undrained | 1200 | | Constant | | Water Table | Custom | 1 | 0 |
| CH su west | Pink | 110 | Undrained | 1000 | | Constant | | Water Table | Custom | 1 | 0 |
| CH su east | Red | 110 | Undrained | 700 | | Constant | | Water Table | Custom | 1 | 0 |
| CHS su west | Dark Red | 118 | Undrained | 800 | | Constant | | Water Table | Custom | 1 | 0 |
| CHS su east | Brown | 118 | Undrained | 1000 | | Constant | | Water Table | Custom | 1 | 0 |
| Loose SC | Light Brown | 115 | Mohr-Coulomb | 0 | 32 | | | Water Table | Custom | 1 | |
| Medium Dense SM | Yellow | 120 | Mohr-Coulomb | 0 | 33 | | | Water Table | Custom | 1 | 0 |
| Medium Dense SP-SM | Orange | 120 | Mohr-Coulomb | 0 | 34 | | | Water Table | Custom | 1 | 0 |
| Dense SP | Dark Orange | 125 | Mohr-Coulomb | 0 | 35 | | | Water Table | Custom | 1 | 0 |
| Abutment | Light Blue | 1 | Infinite Strength | | | | Yes | Water Table | Custom | 1 | |

| | | | | |
|--|----------------------|---|-------------------------------------|--------------------------|
| | Project | | I-555 Service Road over Ditch No. 6 | |
| | Group | I-555 Service Road over Ditch No. 6 - East Abutment | Scenario | Short-Term Stability |
| | Drawn By | DFB | Company | Geotechnology, LLC |
| | Date | 10/6/23 | File Name | J042992.01 Bridge 3.slmd |
| | SLIDEINTERPRET 9.029 | | | |

| Material Name | Color | Unit Weight (lbs/ft3) | Saturated U.W. (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (°) | Allow Sliding Along Boundary | Water Surface | Hu Type | Hu | Tensile Strength (psf) |
|--------------------|--------------|-----------------------|--------------------------|-------------------|----------------|---------|------------------------------|---------------|---------|----|------------------------|
| Fill | Light Yellow | 120 | | Mohr-Coulomb | 50 | 30 | | Water Table | Custom | 1 | 0 |
| CH | Light Pink | 110 | 110 | Mohr-Coulomb | 20 | 22 | | Water Table | Custom | 1 | 0 |
| CHS | Red | 118 | | Mohr-Coulomb | 20 | 22 | | Water Table | Custom | 1 | 0 |
| Loose SC | Light Brown | 115 | | Mohr-Coulomb | 0 | 32 | | Water Table | Custom | 1 | |
| Medium Dense SM | Yellow | 120 | | Mohr-Coulomb | 0 | 33 | | Water Table | Custom | 1 | 0 |
| Medium Dense SP-SM | Light Orange | 120 | | Mohr-Coulomb | 0 | 34 | | Water Table | Custom | 1 | 0 |
| Dense SP | Orange | 125 | | Mohr-Coulomb | 0 | 35 | | Water Table | Custom | 1 | 0 |
| Abutment | Light Blue | 1 | | Infinite Strength | | | Yes | Water Table | Custom | 1 | |



| | | | |
|----------|---|-------------------------------------|--------------------------|
| Project | | I-555 Service Road over Ditch No. 6 | |
| Group | I-555 Service Road over Ditch No. 6 - East Abutment | Scenario | Long-Term Stability |
| Drawn By | DFB | Company | Geotechnology, LLC |
| Date | 10/6/23 | File Name | J042992.01 Bridge 3.slmd |



| | | |
|--|---|--|
| | <i>Project</i> I-555 Service Road over Ditch No. 6 | |
| | <i>Group</i> I-555 Service Road over Ditch No. 6 - East Abutment | <i>Scenario</i> Seismic Stability |
| | <i>Drawn By</i> DFB | <i>Company</i> Geotechnology, LLC |
| | <i>Date</i> 10/6/23 | <i>File Name</i> J042992.01 Bridge 3.slmd |



APPENDIX I – SOIL PARAMETERS FOR SYNTHETIC PROFILES



| BRIDGE 3 OVER DITCH NO. 6 – BENT 1 (WEST ABUTMENT; BORING B3-1) | | | | | | | | | | | | | |
|---|----------------------------|------------------------------|-----|----------------------------|---------------------------|-----------------|--------------------------|------------------|--------------------------------------|--|-------------------------------|--|--------------------------|
| APPROXIMATE TOP OF PILE = EL 219 | | | | | | | | | | | | | |
| ZONE | SOIL TYPES | DEPTH ^a (feet) | | TOTAL UNIT WEIGHT (PCF) | SHEAR STRENGTH PARAMETERS | | | | LATERAL LOAD PARAMETERS ^c | | | LIQUEFACTION SHEAR STRENGTH PARAMETERS | |
| | | FROM | TO | | UNDRAINED (SHORT TERM) | | DRAINED (LONG TERM) | | SOIL STRAIN, E ₅₀ | STATIC SOIL MODULUS (PCI) ^b | LPILE SOIL MODEL | RESIDUAL COHESION (PSF) | RESIDUAL ϕ (DEGREE) |
| | | | | | COHESION (PSF) | ϕ (DEGREE) | EFFECTIVE COHESION (PSF) | ϕ' (DEGREE) | | | | | |
| 1 | Compacted, Lean Clay Fill | 0 | 3 | 120 | 1,200 | -- | -- | 30 | 0.007 | 200 | Stiff Clay without Free Water | 1,200 | -- |
| 2 | Soft, Fat Clay | 3 | 9 | 110 | 500 | -- | -- | 24 | 0.020 | 30 | Soft Clay | 425 | -- |
| 3 | Medium Stiff, Fat Clay | 9 | 14 | 118 | 700 | -- | -- | 26 | 0.010 | 100 | Soft Clay | 600 | -- |
| 4 | Loose Sand, trace clay | 14 | 21 | 110 | -- | 31 | -- | 31 | -- | 25 | Sand | 200 | -- |
| 5 | Medium Dense Sand | 21 | 41 | 120 | -- | 34 | -- | 34 | -- | 40 | Sand | 450 | -- |
| 6 | Medium Dense to Dense Sand | 41 | 91 | 125 | | 34 | | 34 | -- | 60 | Sand | -- | 34 |
| 7 | Dense to Very Dense Sand | 91 | 103 | 130 | | 36 | | 36 | -- | 120 | Sand | -- | 36 |

Note: Groundwater elevation assumed at El. 190 based on the water levels encountered in the borings. The effective unit weight should be used below the groundwater level.

Subtract the density of water (62.4 pounds per cubic foot) from the total unit weight to calculate the effective unit weight.

^a Depth in reference to the approximate top of pile at Bent 1.

^b Pounds per cubic inch.

^c For lateral load analysis only.

| BRIDGE 3 OVER DITCH NO. 6 – BENT 2 (BORING B3-2) | | | | | | | | | | | | | |
|--|-------------------------------------|------------------------------|----|----------------------------------|---------------------------|---------------|--------------------------------|----------------|--------------------------------------|---|--|--|---------------------------|
| APPROXIMATE TOP OF PILE = EL 210 | | | | | | | | | | | | | |
| ZONE | SOIL TYPES | DEPTH ^a (feet) | | TOTAL UNIT WEIGHT (PCF) | SHEAR STRENGTH PARAMETERS | | | | LATERAL LOAD PARAMETERS ^c | | | LIQUEFACTION SHEAR STRENGTH PARAMETERS | |
| | | FROM | TO | | UNDRAINED (SHORT TERM) | | DRAINED (LONG TERM) | | SOIL STRAIN, E ₅₀ | STATIC SOIL MODULUS (PCI) ^b | LPILE SOIL MODEL | RESIDUAL COHESION (PSF) | RESIDUAL Φ (DEGREE) |
| | | | | | COHESION (PSF) | Φ (DEGREE) | EFFECTIVE COHESION (PSF) | Φ' (DEGREE) | | | | | |
| 1 | Stiff, Fat Clay | 0 | 6 | 110 | 1,100 | -- | -- | 24 | 0.007 | 180 | Stiff Clay without Free Water | 900 | -- |
| 2 | Medium Stiff, Sandy, Fat Clay | 6 | 14 | 118 | 700 | -- | -- | 24 | 0.010 | 120 | Soft Clay | 600 | -- |
| 3 | Medium Dense Sand | 14 | 34 | 120 | -- | 33 | -- | 33 | -- | 60 | Sand | 400 | -- |
| 4 | Medium Dense to Dense Sand | 34 | 74 | 125 | -- | 35 | -- | 35 | -- | 90 | Sand | -- | 35 |
| 5 | Dense to Very Dense Sand | 74 | 96 | 130 | -- | 37 | -- | 37 | -- | 120 | Sand | -- | 37 |

Note: Groundwater elevation assumed at El. 190 based on the water levels encountered in the borings. The effective unit weight should be used below the groundwater level.

Subtract the density of water (62.4 pounds per cubic foot) from the total unit weight to calculate the effective unit weight.

^a Depth in reference to the approximate top of pile at Bent 2.

^b Pounds per cubic inch.

^c For lateral load analysis only.

| BRIDGE 3 OVER DITCH NO. 6 – BENT 3 (BORING B2-3) | | | | | | | | | | | | | |
|--|-------------------------------------|------------------------------|----|----------------------------------|---------------------------|---------------|--------------------------------|----------------|--------------------------------------|---|------------------------|--|---------------------------|
| APPROXIMATE TOP OF PILE ELEVATION = EL 210 | | | | | | | | | | | | | |
| ZONE | SOIL TYPES | DEPTH ^a (feet) | | TOTAL UNIT WEIGHT (PCF) | SHEAR STRENGTH PARAMETERS | | | | LATERAL LOAD PARAMETERS ^c | | | LIQUEFACTION SHEAR STRENGTH PARAMETERS | |
| | | FROM | TO | | UNDRAINED (SHORT TERM) | | DRAINED (LONG TERM) | | SOIL STRAIN, E ₅₀ | STATIC SOIL MODULUS (PCI) ^b | LPILE SOIL MODEL | RESIDUAL COHESION (PSF) | RESIDUAL Φ (DEGREE) |
| | | | | | COHESION (PSF) | Φ (DEGREE) | EFFECTIVE COHESION (PSF) | Φ' (DEGREE) | | | | | |
| 1 | Soft, Fat Clay | 0 | 3 | 110 | 500 | -- | -- | 24 | 0.020 | 30 | Soft Clay | 425 | -- |
| 2 | Medium Stiff, Fat Clay | 3 | 8 | 110 | 700 | -- | -- | 24 | 0.010 | 100 | Soft Clay | 600 | -- |
| 3 | Soft, Sandy, Fat Clay | 8 | 13 | 118 | 500 | -- | -- | 24 | 0.020 | 30 | Soft Clay | 425 | -- |
| 4 | Medium Stiff, Sandy, Fat Clay | 13 | 16 | 118 | 700 | -- | -- | 24 | 0.010 | 100 | Soft Clay | 600 | -- |
| 5 | Medium Dense, Silty Sand | 16 | 41 | 120 | -- | 32 | -- | 32 | -- | 60 | Sand | 500 | -- |
| 6 | Medium Dense Sand | 41 | 51 | 125 | -- | 35 | -- | 35 | -- | 90 | Sand | -- | 35 |
| 7 | Dense to Very Dense Sand | 51 | 92 | 130 | -- | 36 | -- | 36 | -- | 120 | Sand | -- | 36 |

Note: Groundwater elevation assumed at El. 195 based on the water levels encountered in the borings. The effective unit weight should be used below the groundwater level.

Subtract the density of water (62.4 pounds per cubic foot) from the total unit weight to calculate the effective unit weight.

^a Depth in reference to the approximate top of pile elevation at Bent 3.

^b Pounds per cubic inch.

^c For lateral load analysis only.

| BRIDGE 3 OVER DITCH NO. 6 – BENT 4 (EAST ABUTMENT; BORING B3-4) | | | | | | | | | | | | | |
|---|--|------------------------------|-----|----------------------------------|---------------------------|---------------|--------------------------------|----------------|--------------------------------------|---|--|--|---------------------------|
| APPROXIMATE TOP OF PILE ELEVATION = EL 219 | | | | | | | | | | | | | |
| ZONE | SOIL TYPES | DEPTH ^a (feet) | | TOTAL UNIT WEIGHT (PCF) | SHEAR STRENGTH PARAMETERS | | | | LATERAL LOAD PARAMETERS ^c | | | LIQUEFACTION SHEAR STRENGTH PARAMETERS | |
| | | FROM | TO | | UNDRAINED (SHORT TERM) | | DRAINED (LONG TERM) | | SOIL STRAIN, E ₅₀ | STATIC SOIL MODULUS (PCI) ^b | LPILE SOIL MODEL | RESIDUAL COHESION (PSF) | RESIDUAL φ (DEGREE) |
| | | | | | COHESION (PSF) | φ (DEGREE) | EFFECTIVE COHESION (PSF) | φ' (DEGREE) | | | | | |
| 1 | Compacted, Lean Clay Fill | 0 | 5 | 120 | 1,200 | -- | -- | 30 | 0.007 | 200 | Stiff Clay without Free Water | 1,200 | -- |
| 2 | Soft, Fat Clay | 5 | 14 | 110 | 500 | -- | -- | 24 | 0.020 | 30 | Soft Clay | 400 | -- |
| 3 | Medium Stiff to Stiff, Sandy, Fat Clay | 14 | 22 | 118 | 900 | -- | -- | 24 | 0.010 | 150 | Stiff Clay without Free Water | 900 | -- |
| 4 | Medium Stiff, Sandy Silt | 22 | 29 | 115 | 700 | -- | -- | 28 | 0.010 | 100 | Soft Clay | 150 | -- |
| | Loose to Medium Dense, Silty Sand | 29 | 51 | 110 | -- | 32 | -- | 32 | -- | 40 | Sand | 500 | -- |
| | Medium Dense to Dense Sand | 51 | 81 | 120 | -- | 35 | -- | 35 | -- | 60 | Sand | -- | 35 |
| | Dense to Very Dense Sand | 81 | 103 | 125 | -- | 37 | -- | 37 | -- | 120 | Sand | -- | 37 |

Note: Groundwater elevation assumed at El. 195 based on the water levels encountered in the borings. The effective unit weight should be used below the groundwater level. Subtract the density of water (62.4 pounds per cubic foot) from the total unit weight to calculate the effective unit weight.

^a Depth in reference to the approximate top of pile at Bent 4.

^b Pounds per cubic inch.

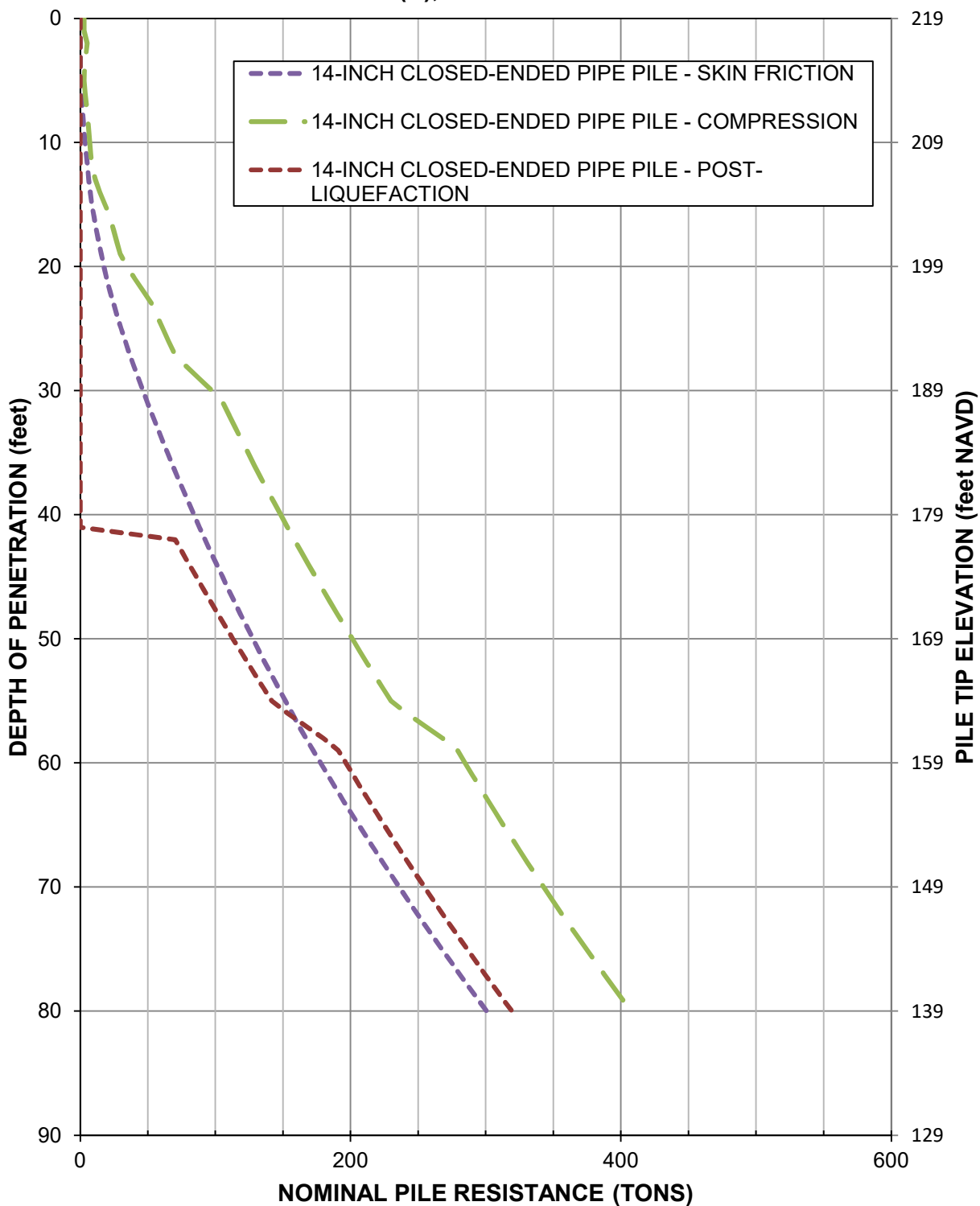
^c For lateral load analysis only.



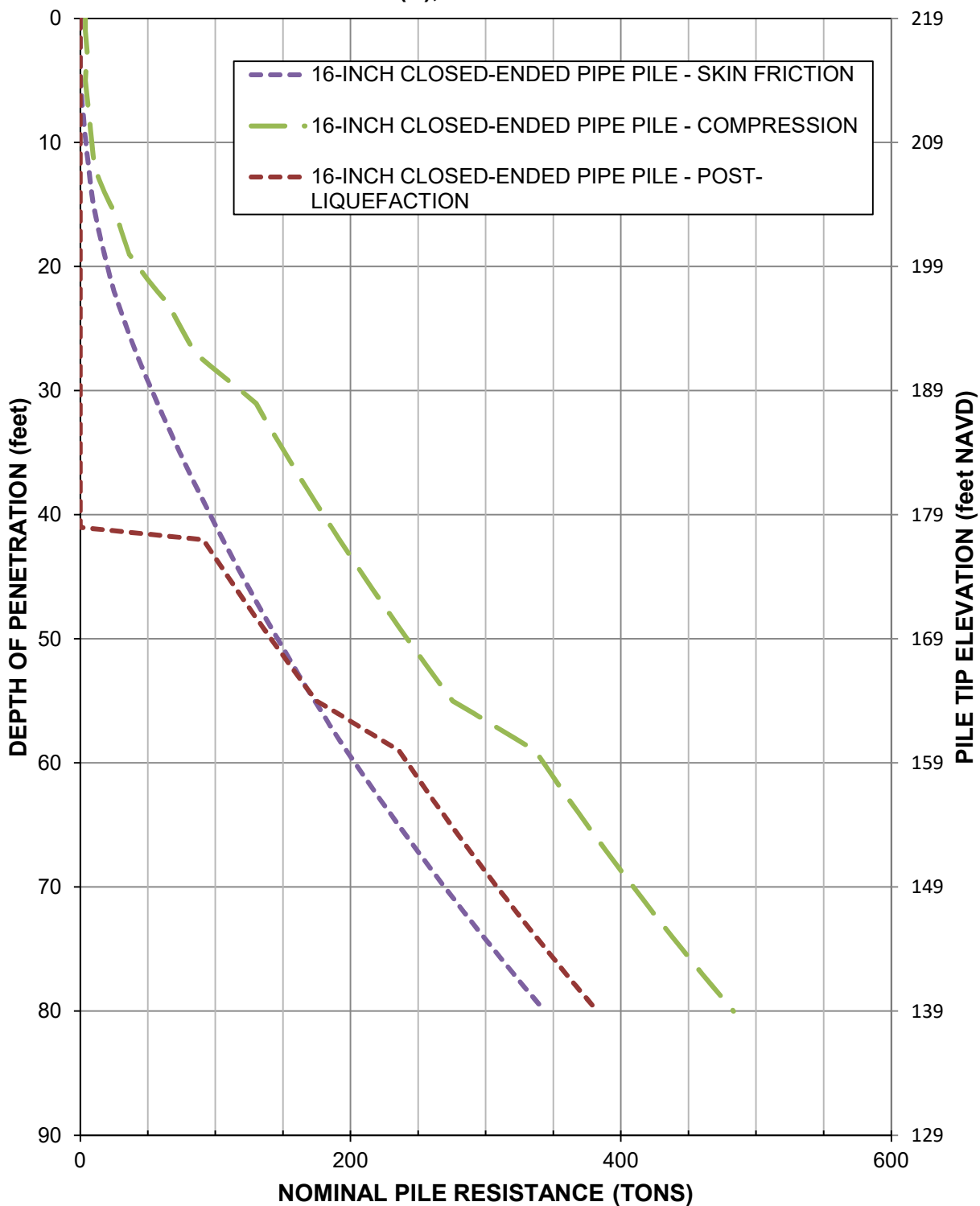
APPENDIX J –NOMINAL AXIAL RESISTANCE CURVES



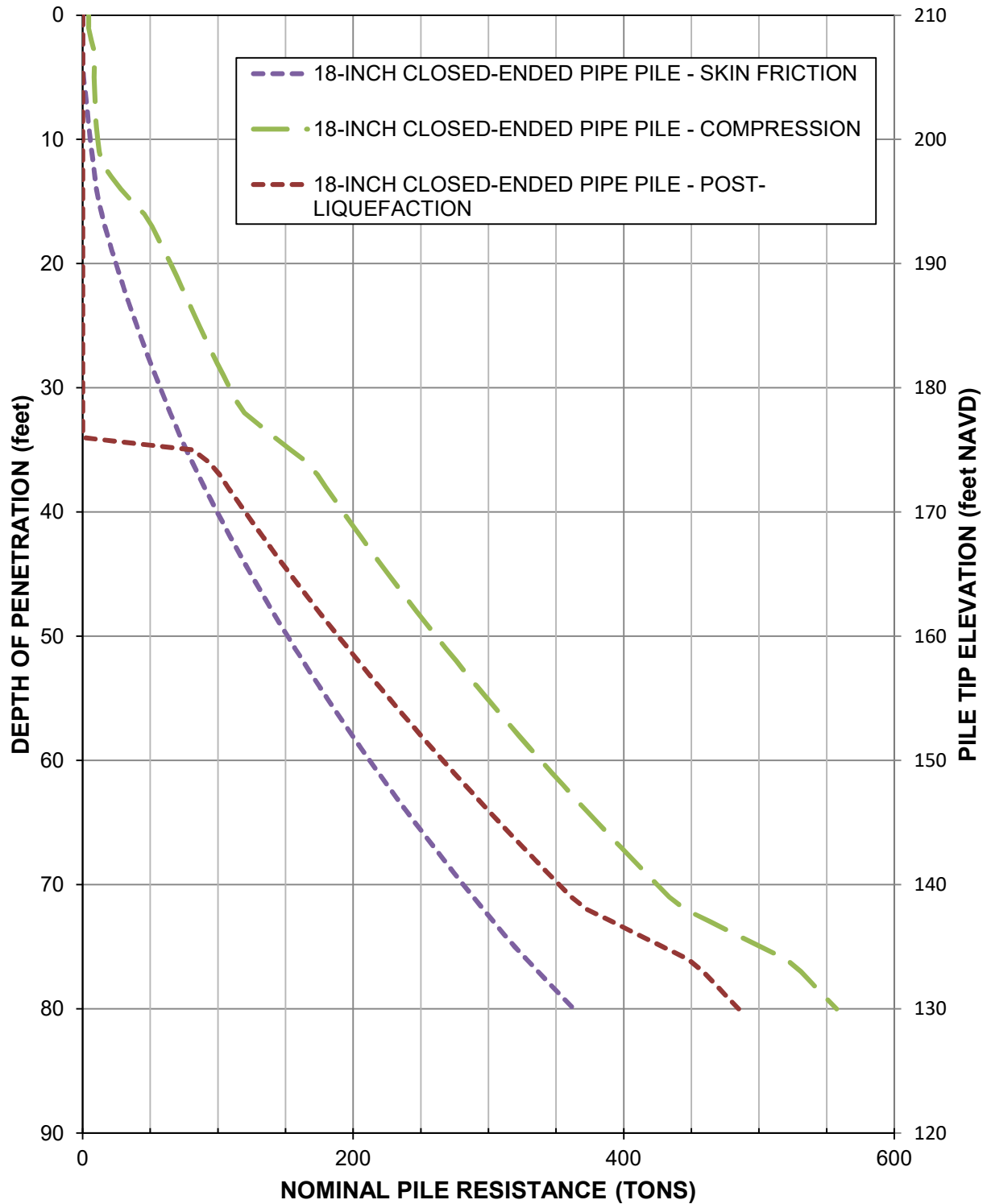
**NOMINAL RESISTANCE CURVES
 14-INCH CLOSED-ENDED PIPE PILE
 ARDOT TYRONZA RIVER & DITCH NO. 6 - BRIDGE 3
 STR. & APPRS. (S), POINSETT COUNTY - BENT 1**



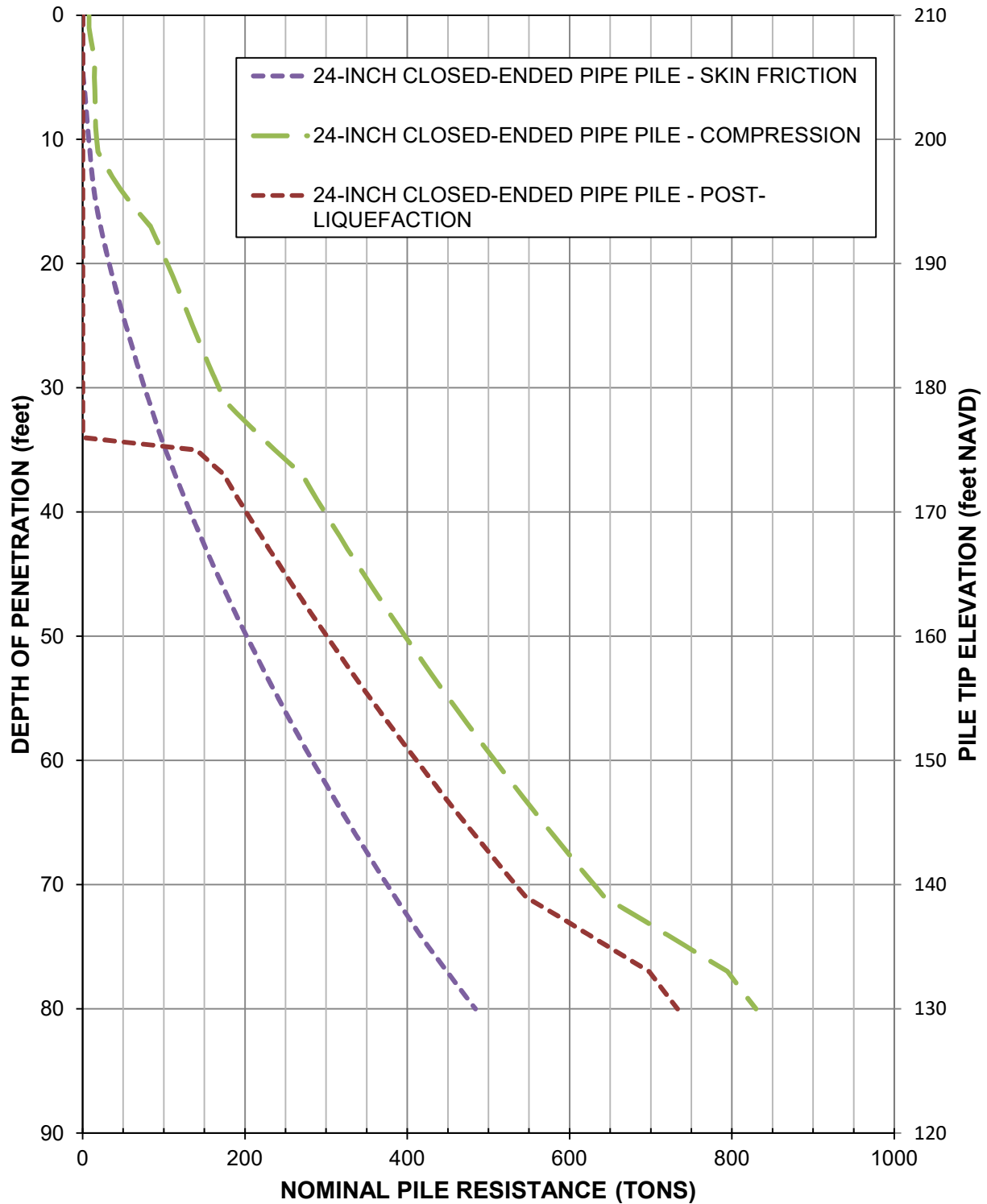
**NOMINAL RESISTANCE CURVES
 16-INCH CLOSED-ENDED PIPE PILE
 ARDOT TYRONZA RIVER & DITCH NO. 6 - BRIDGE 3
 STR. & APPRS. (S), POINSETT COUNTY - BENT 1**



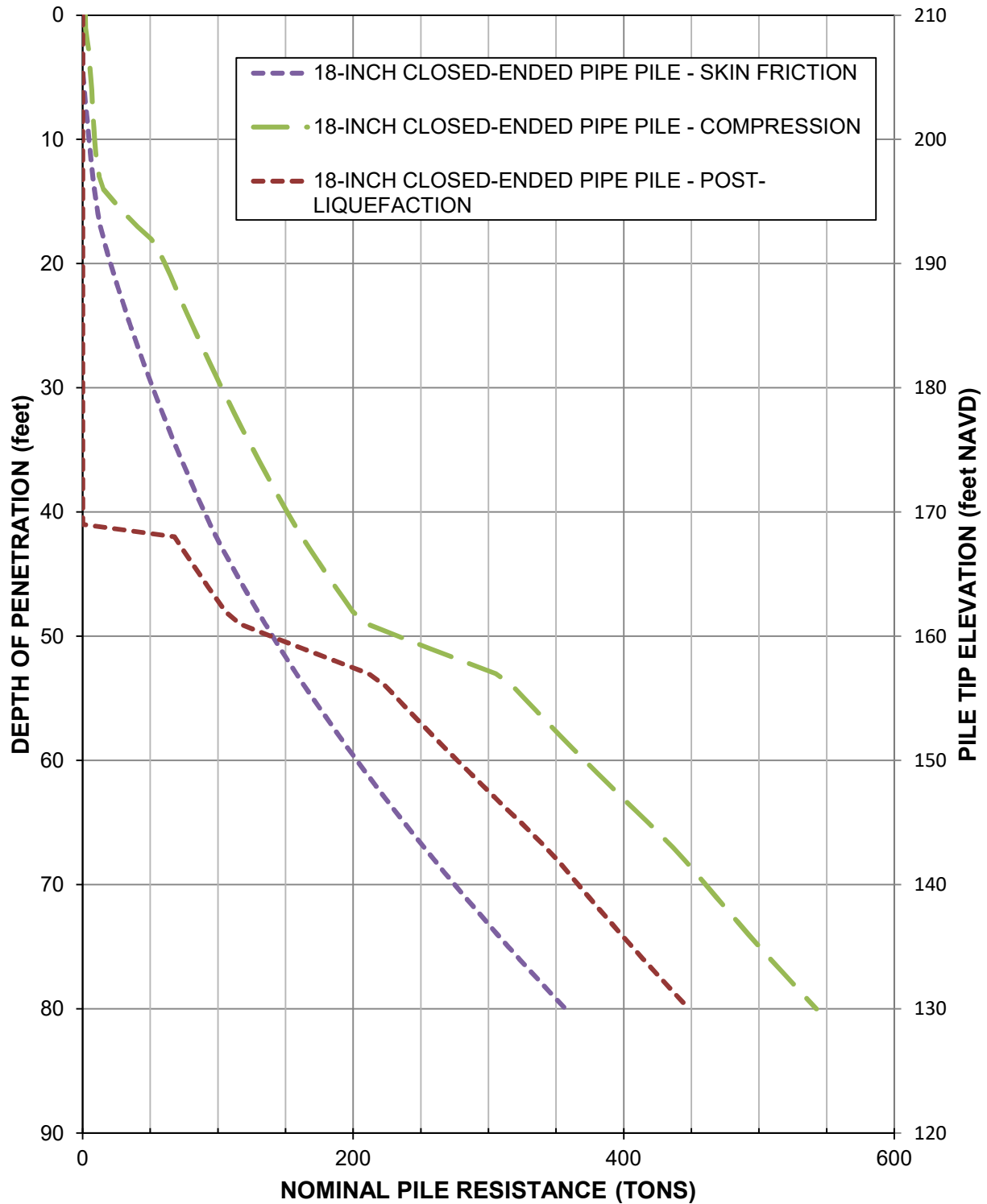
**NOMINAL RESISTANCE CURVES
 18-INCH CLOSED-ENDED PIPE PILE
 ARDOT TYRONZA RIVER & DITCH NO. 6 - BRIDGE 3
 STR. & APPRS. (S), POINSETT COUNTY - BENT 2**



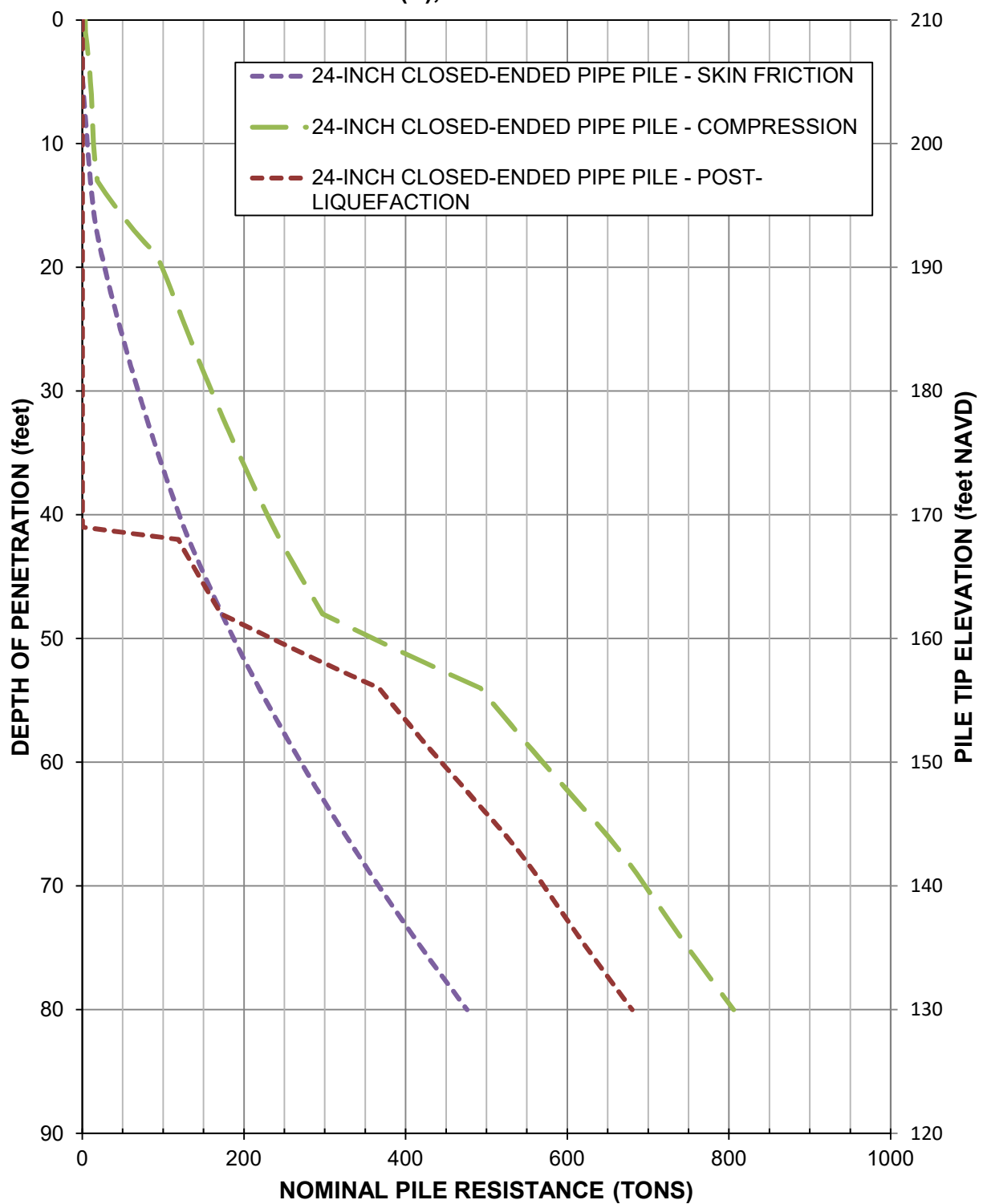
**NOMINAL RESISTANCE CURVES
 24-INCH CLOSED-ENDED PIPE PILE
 ARDOT TYRONZA RIVER & DITCH NO. 6 - BRIDGE 3
 STR. & APPRS. (S), POINSETT COUNTY - BENT 2**



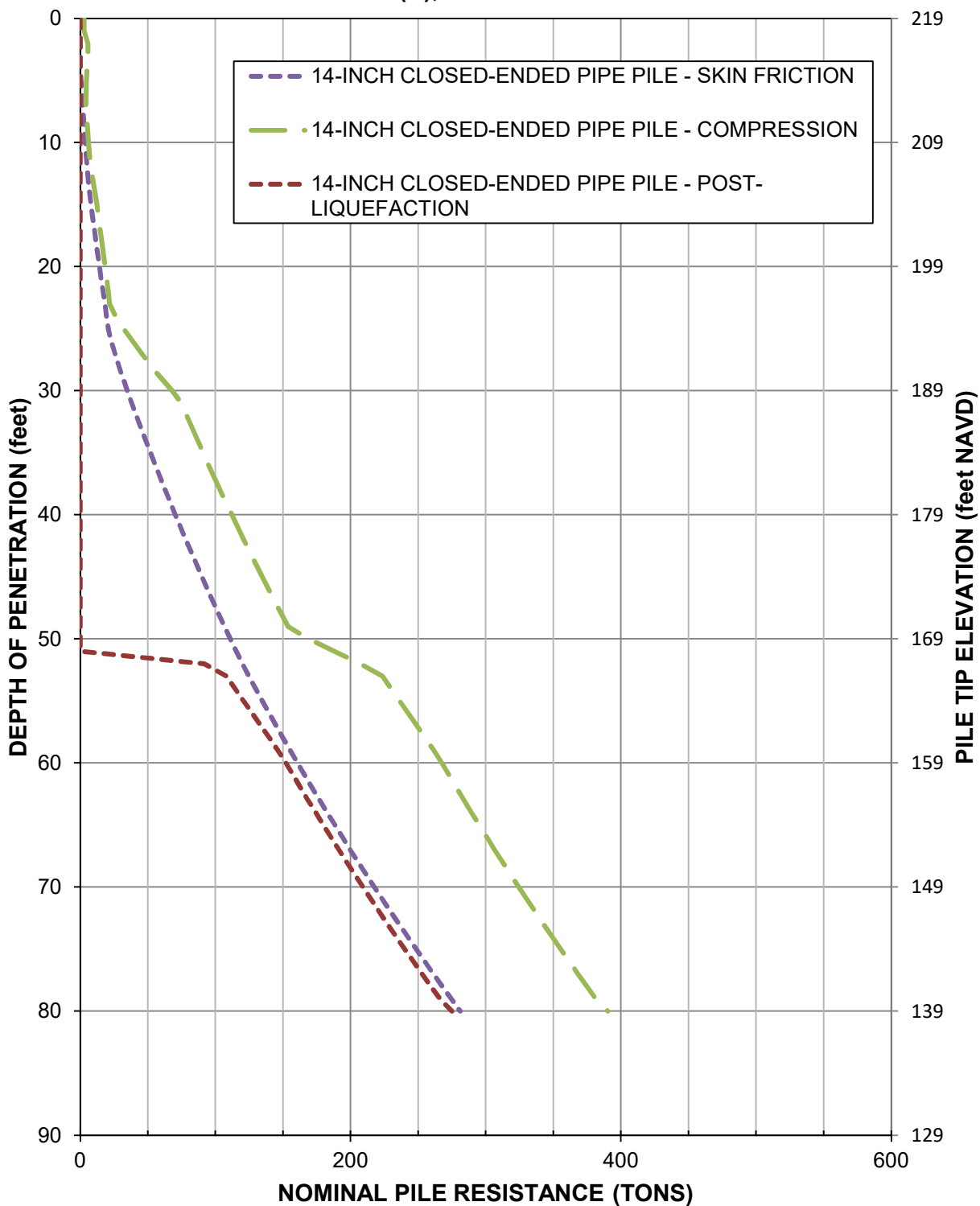
**NOMINAL RESISTANCE CURVES
 18-INCH CLOSED-ENDED PIPE PILE
 ARDOT TYRONZA RIVER & DITCH NO. 6 - BRIDGE 3
 STR. & APPRS. (S), POINSETT COUNTY - BENT 3**



**NOMINAL RESISTANCE CURVES
 24-INCH CLOSED-ENDED PIPE PILE
 ARDOT TYRONZA RIVER & DITCH NO. 6 - BRIDGE 3
 STR. & APPRS. (S), POINSETT COUNTY - BENT 3**



**NOMINAL RESISTANCE CURVES
 14-INCH CLOSED-ENDED PIPE PILE
 ARDOT TYRONZA RIVER & DITCH NO. 6 - BRIDGE 3
 STR. & APPRS. (S), POINSETT COUNTY - BENT 4**



**NOMINAL RESISTANCE CURVES
 16-INCH CLOSED-ENDED PIPE PILE
 ARDOT TYRONZA RIVER & DITCH NO. 6 - BRIDGE 3
 STR. & APPRS. (S), POINSETT COUNTY - BENT 4**

