GEOTECHNICAL EXPLORATION
WEST MEMPHIS RECREATION CENTER
WEST MEMPHIS, ARKANSAS

Prepared for:

PROJECT LUONG
HOUSTON, TEXAS

Prepared by:

UES PROFESSIONAL SOLUTIONS 25, LLC
MEMPHIS, TENNESSEE

Date:

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Project No.:

A25138.00362.001

SAFETY
TEAMWORK
RESPONSIVENESS
INTEGRITY
VALUE
EXCELLENCE

Environmental
Geotechnical Engineering
Materials Testing
Field Inspections & Code Compliance
Geophysical Technology

February 13, 2026

Mr. Jeffry Farr, AIA
Project Luong
820 Gessner Road, Ste 775
Houston, Texas 77024

Re: Geotechnical Exploration
West Memphis Recreation Center
West Memphis, Arkansas
Project No. A25138.00362.001

Dear Mr. Farr:

Presented in this report are the results of the geotechnical exploration performed by UES Professional Solutions 25, LLC (UES) for the referenced project in West Memphis, Arkansas. 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,

UES

Ryan Farrar, P.E.
Geotechnical Manager

RSP/JTM/RTF:rsp/rtf

Copies submitted: Client (email)

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1.0 INTRODUCTION

UES has prepared this geotechnical exploration report for Project Luong for the proposed West Memphis Recreation Center in West Memphis, Tennessee. Our services documented in this report were provided in general accordance with the scope of services documented in our proposal A25138.00362.001 Rev. 1, dated November 7, 2025. Our services were authorized by the Standard Agreement issued November 10, 2025.

The purposes of the geotechnical exploration were to develop a general subsurface profile at the site and prepare recommendations for the geotechnical aspects of the design and construction of the project as defined in our proposal. Our scope of services included site reconnaissance, geotechnical borings, laboratory testing, engineering analyses, and preparation of this report. Unless noted otherwise, all dimensions, measurements, depths, and locations in this report should be considered approximate.

A copy of "Important Information about This Geotechnical-Engineering Report," published by the Geotechnical Business Council of the Geoprofessional Business Association, is included in Appendix A for your review. The publication discusses report limitations and ways to manage risk associated with subsurface conditions.

2.0 SITE DESCRIPTION

The proposed area of construction, herein referred to as the site, is located in the northern portion of the existing Tilden Rogers Sports Complex in West Memphis, Arkansas. The site is located in an area presently occupied by three youth baseball fields, greenspace, and paved pedestrian walkways and parking and drive areas as shown on Figure 1 in Appendix B. The site is bound to the north by West Service Road and a commercial property, to the west by College Boulevard and commercial properties, to the south by a lake and tennis courts, and to the east by additional youth baseball fields, a concession building, and a residential subdivision. The site appears to be relatively level.

3.0 PROJECT INFORMATION

Based on review of the mark-up of an existing conditions plan provided by Arco Murray, we understand the project will consist of the design and construction of an approximately 100,000 square foot recreation center, containing six basketball courts, an eight-lane swimming pool, a family / physical therapy pool, and multi-purpose fitness rooms. Paved parking and drive areas are planned around the proposed development, with a driveway connecting to College Boulevard.

Maximum column and wall loads of 150 kips and 0.3 kips per linear foot (klf), respectively, were provided by the structural engineer for the sports complex building. Additionally, we have assumed maximum column loads of 30 kips for ancillary structures at the site. Grading information was not provided. We have assumed up to 3 feet of grade change will be required to achieve design grades.

4.0 GEOTECHNICAL EXPLORATION

The geotechnical exploration consisted of 11 borings. Seven borings, designated as Borings B-1 through -7, were performed in the proposed structure footprint and four borings, designated as Borings P-1 through -4, were performed in the proposed parking and drive areas. The borings were located in the field by a UES representative. The boring locations shown on Figure 2 in Appendix B are approximate; if elevations or more precise locations are required, the client should retain a registered surveyor to establish boring locations and elevations.

The borings were drilled December 8 through 16, 2025, using an ATV-mounted rotary drill rig (Diedrich D50) and a track mounted rotary drill rig (Geoprobe 7822DT). Hollow-stem auger and wet rotary drilling methods were used as indicated on the boring logs presented in Appendix C. Sampling of the soils was accomplished ahead of the augers at the depths indicated on the boring logs, using 2-inch-outside-diameter (O.D.) split-spoons and 3-inch-O.D., thin-walled Shelby tube samplers in general accordance with the procedures outlined by ASTM D1586 and ASTM D1587, respectively. Standard Penetration Tests (SPTs) were performed using an automatic hammer to obtain the standard penetration resistance, or N-value[1], of the sampled material.

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

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

5.0 LABORATORY REVIEW AND TESTING

Laboratory testing was performed on soil samples to assess engineering and index properties. The soil testing consisted of moisture contents (ASTM D2216), Atterberg limits (ASTM D4318), grain size (sieve) distributions (ASTM D6913), one-dimensional consolidation (ASTM D2435),

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

and unconsolidated-undrained triaxial compression (UU; ASTM D2850). The laboratory test results are presented on the boring logs in Appendix C. The Atterberg limit, grain size distribution, consolidation, and UU test results are also provided in Appendix D.

The boring logs were prepared by a geotechnical engineer from the field logs, visual classifications of the soil samples in the laboratory, and laboratory test results. Terms and symbols used on the boring logs are presented in 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.

6.0 SUBSURFACE CONDITIONS

6.1 Stratigraphy

The ground surface in Borings B-1 through -7, P-1, and P-2 was covered in 3 to 4 inches of topsoil. The ground surface in Borings P-3 and P-4 was covered in 8 inches of asphalt. Below the surficial materials, the soil stratigraphy generally consisted of predominately fine-grained soils overlying predominately coarse-grained soils to the maximum depth of exploration (100 feet). Below the predominately fine-grained soils, coarse-grained soils were encountered and extended to the maximum depth of exploration (100 feet) in Borings B-1 through -7. More specific descriptions of the soil layers are provided below and on the boring logs in Appendix C.

6.1.1 Fine-Grained Soils

Soils classified as low plasticity “lean” clay (CL), silt (ML), and high plasticity “fat” clay (CH) were encountered below the surficial materials and extended to depths of 28.5 to 43.5 in Borings B-1 to -5 and to boring termination depths (10 feet) in Borings B-6, B-7, and P-1 through -4. Moisture contents of the tested samples ranged from 26 to 57 percent. Atterberg limits performed on select samples yielded liquid limits (LL) of 31 to 106 percent and plasticity indices (PI) of 6 to 61 percent. UU tests performed on relatively undisturbed Shelby tube samples yielded undrained shear strengths of 525 to 1,195 pounds per square foot (psf), indicative of medium stiff to stiff consistencies. SPT N-values measured in the fine-grained soils ranged from 2 to 15 bpf, which in our experience is indicative of soft to stiff consistencies.

6.1.2 Coarse-Grained Soils

Soils classified as intermixed sand (SP, SP-SC, and SP-SM), silty sand (SM), and clayey sand (SC) were encountered below the fine-grained soils in Borings B-1 through -5 and extended to the boring termination depths. SPT N-values measured in the lower, fine-grained soils ranged from 4 to 49 bpf, indicative of very loose to dense consistencies.

6.2 Groundwater

Groundwater was encountered in Borings B-2, -3, and -4 at a depth of 15 feet below ground surface. Groundwater was not encountered in the other borings but may have been masked by the use of wet rotary drilling methods in Borings B-1 and B-5. Groundwater levels will vary over time due to the effects of seasonal variations in precipitation or other factors not evident at the time of exploration.

7.0 CONCLUSIONS AND RECOMMENDATIONS

UES has prepared the following conclusions and recommendations based on our understanding of the proposed project, the field and laboratory data presented in this report, engineering analyses, and our experience and judgment. UES should be allowed to review final grading and foundation plans to verify that our recommendations have been properly implemented and are suitable for the final design.

7.1 High Plasticity Soil Concerns

Soft and saturated, high plasticity “fat” clays were encountered in the borings extending to depths of up to approximately 43.5 feet. High plasticity clays typically experience volume change with changes in moisture content. Structure foundations, floor slabs, swimming pool shells, and pavements supported on high plasticity, potentially expansive soils within the drying/wetting zone can undergo distress as the soil shrinks or swells unless these soils are mitigated.

We understand the proposed sports complex building will be supported on deep foundations. For ancillary structures, we recommend the bottoms of footings be separated from the fat clays with a buffer zone. Additionally, we recommend the bottoms of floor slabs, the bottoms of swimming pool excavations (pool shell subgrade), and the base material under new pavement structures be separated from the fat clays with a buffer zone. The buffer zone should consist of suitable fill material and be placed as recommended subsequently in this report. The buffer zone should extend a minimum of 4 feet below the base of footings, 3 feet below the base of floor slabs, 3 feet below the bottom of swimming pool excavations, and 3 feet below pavement base materials. The buffer zone should extend a minimum of 5 feet beyond the building footprints, pool footprints, and parking lots where site constraints allow. For pool walls, the buffer zone should additionally be extended outward in a wedge starting approximately 5 feet back from the wall at a 45-degree angle or flatter, to mitigate lateral pressures from swelling of the high plasticity clay.

The construction of the buffer zone may require filling, undercutting, or a combination thereof. The geotechnical engineer or their representative should be onsite during grading operations to assess the soil types at the bottom of excavations for potential high plasticity clays and determine if additional excavation is necessary.

The proposed methods of fat clay remediation are based on generally accepted standards in the local engineering community. Clay properties, including plasticity, moisture content, unit weight, swell pressure, and mineralogy are variable and could in some circumstances, be conducive to more severe swell pressures and volume change potential than can be mitigated by nominal treatment. Consequently, when building in an area where fat clays are present, the client should realize there is an inherent risk that damage associated with shrink or swell of the soil could occur, even with remedial treatment of the subgrade soils.

Contractors should consider the timing of construction relative to the time of year and weather in their estimates. If construction is performed during relatively cold and wet weather, lime- or cement-treatment of the subgrade could be beneficial to maintain progress during construction. Otherwise, the subgrade could be weakened by softening from saturation by rain and/or snow, leading to delays in reworking the subgrade to prepare it back to its pre-softened condition.

If coarse-grained soils are used as the buffer zone material over the high plasticity clay, the contractor should use measures that prevent water from ponding on top of the fat clay and within the buffer material. Measures can include temporary drainage ditches (bleeder ditches), sump pumps, grading the site to promote drainage away from the construction area, and placing impermeable material over the buffer zone until pavements, slabs, footings, and pool shells are constructed. A geotextile separator may be considered at the interface between the high plasticity clay and the granular buffer material, particularly where soft or wet conditions are encountered during construction.

7.2 Site Preparation and Earthwork

Grading information was not provided. We have assumed up to 3 feet of grade change will be required to achieve design grades. The following paragraphs outline grading recommendations for the site.

7.2.1 Site Preparation

Existing structures, floor slabs, foundations, and pavements from the previous development should be fully demolished and removed prior to continued construction. In general, cut areas and areas to receive new fill are to be stripped of fill soils (if encountered), topsoil, soft soils, and other deleterious materials. Topsoil should be placed in landscape areas or disposed of off-site. Vegetation and tree root-balls should be over-excavated. Contractors should account for the removal of vegetation in their estimates.

7.2.2 Proof-Roll

The exposed subgrade should be proof-rolled with a tandem axle dump truck loaded to approximately 20 kips per axle (or equivalent proof-rolling equipment). The proof-rolling equipment should traverse the exposed subgrade with overlapping passes of the vehicle. This requirement may be waived if the geotechnical engineer determines that proof-rolling would disturb an otherwise acceptable subgrade.

Soft areas or pumping subgrade that develop during proof-rolling should be over-excavated and backfilled with soil compacted to the densities specified subsequently in this report prior to placement of fill or continued construction. The geotechnical engineer or their representative should be onsite to observe proof-roll operations and make recommendations for improvement.

7.2.3 Cut Areas

After excavation, the top 6 inches of the resulting subgrade should be compacted to a minimum of 98% of the maximum dry unit weight as determined by a standard Proctor test (ASTM D698).

7.2.4 Fill Materials, Placement, and Compaction

Fill material should consist of natural soils classifying as silt, lean clay, silty sand, or clayey sand (ML, CL, or SC), have a maximum LL of 45 and a PI of 7 or greater but no more than 20. Such materials should be free from organic matter, debris, or other deleterious materials, and have a maximum particle size of 2 inches. In general, the onsite soils do not meet these criteria.

Fill and backfill should be placed in level lifts, up to 8 inches in loose thickness. For soils that exhibit a well-defined moisture density relationship, each lift should be moisture-conditioned to within the acceptable moisture content range provided in Table 1, and compacted to at least the minimum percent compaction indicated in Table 1. Moisture-conditioning can include: aeration and drying of wetter soils; wetting drier soils; and/or mixing wetter and drier soils into a uniform blend. For granular soils that do not exhibit a well-defined moisture density relationship, the soils should be compacted to at least the minimum relative densities indicated in Table 2. Thinner lifts should be used for lighter compaction equipment.

Table 1. Percent Compaction and Moisture-Conditioning Requirements for Fill and Backfill.

Area	Minimum Percent Compaction[a,b]	Acceptable Moisture Content Range[c]
Structural[d]	95%	±2%
Non-structural	92%	±2%
Pavement subgrades	98%	±2%

[a] In reference to the standard Proctor maximum dry unit weight measured by ASTM D698.

[b] For granular soils that do not exhibit a well-defined moisture-density relationship, refer to Table 2 for minimum relative density requirements.

[c] In reference to optimum moisture content as measured by ASTM D698.

[d] Structural fill and backfill for foundations are defined as fill and backfill located within the zones of influence of structures. The zone of influence of a structure is defined as the area below the footprint of the structure and 1V:1H outward and downward projections from the bearing elevation of the structure.

Table 2. Relative Density Compaction Requirements for Granular Fill and Backfill.

Area	Minimum Relative Density[a,b]
Structural[c]	70%
Non-Structural	70%
Pavement Subgrades	75%

[a] Relative density evaluated from the maximum and minimum index densities measured by ASTM D4253 and D4254, respectively.

[b] For granular soils that exhibit a well-defined moisture density relationship, refer to Table 1 for minimum percent compaction and moisture-conditioning requirements.

[c] Structural fill and backfill for foundations are defined as fill and backfill located within the zones of influence of structures. The zone of influence of a structure is defined as the area below the footprint of the structure and 1V:1H outward and downward projections from the bearing elevation of the structure.

7.2.5 Site Water Management

Managing site water is important in successful performance of the pavement and foundation systems. Water from surface runoff, downspouts, and subsurface drains should be collected and discharged through a storm water collection system. Positive drainage should be established to promote drainage of surface water away from and reduce ponding of water adjacent to these structures.

Maintaining the moisture content of bearing and subgrade soils within the acceptable range provided in Table 1 is important during and after construction. Silty and clayey bearing and 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 and to reduce drying of these soils.

7.2.6 Additional Earthwork Considerations

Trees and other deep-rooted vegetation should not be planted within 1.5 times their projected mature foliage radius from foundations or below-grade walls, as the roots extract moisture from plastic and low-plastic soils alike, causing them to shrink, which can potentially result in foundation settlement. Shrubs and flowerbeds should be located a minimum of 5 feet away from the perimeter of foundations and below-grade walls.

Asphalt, concrete, or fill should not be placed over frozen or saturated soils, and frozen or saturated soils should not be used as compacted fill or backfill. Upon completion of earthwork, disturbed areas should be stabilized.

7.3 2021 IBC/ASCE 7-16 Seismic Site Classification and Seismic Design Parameters

The site lies within the influence of the New Madrid Seismic Zone (NMSZ). It is our understanding the proposed construction will be designed in accordance with the 2021 International Building Code (IBC) and Chapter 20 of the ASCE 7-16. The 2021 IBC/ASCE 7-16 stipulates structures be designed based on an earthquake event with a probability of exceedance of 2% in 50 years. Based on the results of the field and laboratory testing, our experience in the vicinity, and our interpretation of the 2021 IBC/ASCE 7-16, it is our opinion the site class and seismic parameters in Table 3 are applicable for this project.

Table 3. Site Class and Seismic Parameters (2% Probability of Exceedance in 50 Years.

Category/ Parameter	Designation/ Value	Reference
S_S	1.135g[a]	Latitude 35.158019°N / Longitude 90.222537°W
S_1	0.391g[a]	
Seismic Site Class	F[b]	Chapter 20 of ASCE 7-16
F_a	1.046	2021 IBC Table 1613.2.3(1)
F_v	--[c]	2021 IBC Table 1613.2.3(2)
F_{PGA}	1.100	ASCE 7-16 Table 11.8-1
S_{MS}	1.187g	2021 IBC Equation 16-20
S_{M1}	--[c]	2021 IBC Equation 16-21
S_{DS}	0.792g	2021 IBC Equation 16-22
S_{D1}	--[c]	2021 IBC Equation 16-23
PGA	0.677g	ASCE 7-16 Figure 22-7
PGA_M	0.744g	ASCE 7-16 Equation 11.8-1

[a] S_S and S_1 were computed using the web-based U.S. Seismic Design Maps (https://ascehazardtool.org) using the indicated latitude and longitude coordinates of the project site.

[b] Based on ASCE 7-16, sites with liquefiable soils, discussed subsequently in this report, may be classified as Site Class F. However, based on Section 20.3.1 of the ASCE 7-16, if the structure has a fundamental period of vibration equal to or less than 0.5 seconds, the site class may be determined from the definitions provided in Table 20.3-1 of ASCE 7-16 and the corresponding values of F_a and F_v determined from Tables 11.4-1 and 11.4-2. The values provided in this table assume that the proposed structure meets the above criteria.

[c] Refer to ASCE 7-16 Section 11.4.8 exception for the Site Class D and mapped S1 > 0.2g.

Based on Supplement 3 of ASCE 7-16 Section 11.4.8, a site-specific ground motion response analysis (SSGMRA) shall be performed for Site Class F and Site Class D sites with mapped S_1 values greater than 0.2g. An exception to this requirement states that a SSGMRA is not required if the value for S_{M1}, and the resulting value of S_{D1}, are increased by 50%. Should this exception be utilized, the following values may be used:

$S_{M1} = 1.120g$
$S_{D1} = 0.746g$

If a SSGMRA is desired, please contact UES.

7.4 Liquefaction, Dynamic Settlement, and Lateral Spread Potential

Liquefaction Hazard Evaluation. Liquefaction can occur in loose, saturated, cohesionless soil deposits subjected to earthquake motions. A deterministic study was performed to evaluate the liquefaction and dynamic settlement potential at the site using the field and laboratory data. The laboratory data included USCS classification, soil unit weight, and percent fines of soil samples obtained from various strata. Field data included SPT N-values and groundwater level.

An earthquake magnitude (M_W) of 7.7 and a peak ground acceleration of 0.744g at the ground surface were considered for a probability of exceedance in 50 years. The groundwater level for the analyses was assumed to be at depths of 15 feet below ground surface during the seismic event, as encountered in Borings B-2 through B-4.

Subsurface characteristics (as characterized by 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. Safety factors against liquefaction using SPT data were computed using the deterministic method by Idriss and Boulanger (2014) in conjunction with what is known as "The Simplified Method" as introduced by Seed (1971) which subsequent modifications/revisions by other researchers. Results of the SPT-based liquefaction hazard analyses performed for the borings are presented in Table 3.

Table 3. Results of Liquefaction Analyses.

Boring	Liquefiable Zones[a] (feet below ground surface)	Estimated Dynamic Settlement (inches)
B-1	28.5 – 38.5	4
B-2	33.5 – 38.5	2
B-4	38.5 – 43.5	3
B-5	28.5 – 33.5	2

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

Please note 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 other researchers (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 substance liquefaction in relatively deep sand deposits below 50 feet is considered unlikely.

Based on ASCE 7-16, sites with liquefiable soils may be classified as Site Class F. However, based on Section 20.3.1 of the ASCE 7-16, if the structure has a fundamental period of vibration equal to or less than 0.5 second, the site class may be determined from the definitions provided in Table 20.3-1 of the ASCE 7-16. The values provided in Tables 3 and 4 are based on the assumption that that the subject structure meets the above criteria; the project structural engineer should verify this assumption and inform UES should this assumption be incorrect.

Consequences of Liquefaction. Potential consequences of liquefaction include loss of support for shallow foundations, differential settlements, and drag loads on pile foundations. Downdrag occurs as soil strata move downward relative to piles due to settlement of the soil layers. A settlement of 0.4-inch or greater could produce full downdrag on pile foundations according to guidance in Section 3.11.8 of the AASHTO publication “LRFD Bridge Design Specifications”.

Liquefaction Mitigation Considerations. Liquefaction hazard mitigation can be accomplished using compaction piles (large displacement piles) or proprietary ground improvement techniques such as earthquake drains. Proprietary ground improvement techniques are typically performed by specialty firms on a design/build basis.

Lateral Spread. Lateral spreading is triggered and sustained by earthquake ground motions and the occurrence of liquefaction. Lateral spread is dependent on the location of the site in reference to a nearby free-facing slope. Additionally, lesser lateral spreading may occur to gradual slopes that span over greater distances. Lateral load analyses are beyond the scope of our exploration. Additional information and analyses are required to perform lateral load analyses. Please contact UES if lateral load analyses are required.

7.5 Ancillary Shallow Foundations

We understand the proposed sports complex building will be supported on intermediate or deep foundations. We have assumed that ancillary structures at the site will be supported on shallow foundations. We have assumed maximum column loads of 30 kips for ancillary structures. If the actual loading exceeds those assumed, UES should be contacted to perform additional settlement analyses.

The bottoms of foundations should be separated from the high plasticity clays with a 4-foot-thick buffer zone as discussed in Section 7.1 of this report. Provided the site is prepared as recommended in Sections 7.1 and 7.2 of this report, shallow foundations can be proportioned using a maximum net allowable bearing pressure of 1,800 and 1,400 pounds per square foot (psf) for spread and strip footings, respectively. Total and differential settlement of shallow foundations are anticipated to be 1 inch and ¾ of an inch, respectively, for loads of 30 kips or less. These recommendations are based on the conditions that foundation plans are forwarded to UES for review and the foundation excavations are observed by the geotechnical engineer or their representative. Additional recommendations may be required based on the results of the foundation observations.

Footing excavations should be made with a smooth-edged backhoe bucket, and foot traffic in the bottom of the excavation should be minimized. Footing excavations should be extended through deleterious materials and/or zones of soft soil, if encountered; the over-excavation can be backfilled with compacted fill, lean concrete, and or flowable fill.

For exterior footings, we recommend that the footings bear a minimum of 18 inches below finish grade. An additional 6 inches of embedment is recommended if the erosion of the cover material is not controlled. Drainage should be maintained away from the foundations throughout the life of the structure. Water should never be allowed to pond against the footings.

7.6 Ground Improvement

Ground improvement techniques may be considered to support the project structure. Ground improvement systems may be designed to increase the allowable bearing capacity of the improved ground to values on the order of 3,000 to 5,000 psf while limiting total settlement to approximately 1 inch or less, subject to final design and loading conditions.

Ground improvement methods, such as aggregate piers (AP), rigid inclusions, or other proprietary systems, are typically designed and installed by a specialty design/build contractor using data from this report and specific loads and layouts of the structures. Installation of AP or rigid inclusion systems increases the lateral stress in surrounding soil, thereby further stiffening the stabilized composite soil mass. The results of the AP or rigid inclusions system installation is a strengthening and stiffening of subsurface soils that then support structural loads.

It should be noted that ground improvement systems are not remedial measures for the high plasticity and highly expansive soils encountered at the site. The recommendations for remediation of these soils, discussed in Section 7.1, should be followed regardless of the installation of ground improvement systems.

7.7 Augercast Pile Foundations

Augercast piles (ACP) have been evaluated for support of proposed sports complex. We have evaluated 16-, 18-, and 24-inch diameter ACP ranging in depth from 45 to 70 feet below ground surface. If other deep foundation types or sizes are to be considered, UES should be contacted to perform additional analyses.

The ACP should extend through zones of soft or loose soils into stable soil strata. Recommended maximum ultimate axial capacities for augercast piles (ACP) are presented in Table 4. It should be noted that the provided axial capacities are ultimate and do not have a factor of safety applied. We recommend load testing be performed as described subsequently.

The structural engineer should apply appropriate factors of safety (FOS) when designing ACP foundations. Provided that the load testing program outlined in Section 7.7.5 is performed, we recommend FOS of 2.0, 3.0, and 1.5 be applied to the ultimate capacities in compression, uplift, and post-liquefaction cases, respectively.

Table 4. Recommended Ultimate Axial Capacity – Augercast Piles.

Pile Diameter (inches)	Embedment Length[a] (feet)	Ultimate Axial Capacity (kips)			Anticipated Maximum Drag Load Due to Liquefaction (kip)
		Compression	Uplift	Post-Liquefaction Compression[b]	
16	45	116	74	42	78
	50	158	108	85	
	55	200	142	127	
	60	241	175	168	
	65	282	207	209	
	70	321	238	248	
18	45	134	85	52	88
	50	182	123	100	
	55	230	161	147	
	60	276	199	194	
	65	322	235	239	
	70	366	270	283	
24	45	196	119	86	118
	50	259	170	170	
	55	322	221	221	
	60	383	272	272	
	65	443	321	321	
	70	501	368	368	

[a] Accounts for liquefaction-induced loss of skin friction and downdrag within the upper 50 feet.

We recommend a pile spacing of three pile diameters center-to-center. For closer spacing, the capacity should be evaluated for group effects. Foundation plans should be forwarded to UES to evaluate whether pile lengths should be revised based on the actual cutoff elevation and for evaluation of pile group capacity and settlement. The structural engineer should verify the structural capacity of the piles based on the requirements of the applicable code. If a different configuration or higher load capacities are required, UES should be notified so that the required analyses can be performed in a timely manner.

7.7.1 Downdrag

When soil surrounding pile foundations moves downward (settles) with respect to the pile, negative skin friction occurs and drag forces are exerted on the pile by a means of friction. Drag loads will accumulate with depth along impacted piles causing additional stress, strain, and displacement response. A maximum stress will occur in the pile at the bottom of the liquefiable zone. Below the liquefiable zone, soil side friction contributes to resistance to axial loads.

We understand that liquefaction mitigation will be performed at the site. If liquefaction is mitigated, loss of support due to liquefaction and the associated drag loading will not occur. Therefore, it is recommended that if liquefaction mitigation is performed, the provided

Post-liquefaction axial capacities of the deep foundation systems considered accounting for downdrag are presented in Table 4. For the purposes of the downdrag analyses, the bottom of the liquefiable zone was limited to 50 feet below existing ground surface.

7.7.2 ACP Uplift Resistance

Ultimate uplift capacities were calculated for ACP and are presented in Table 4. The ultimate capacities include the effective weight of the pile plus side resistance. ACP resisting net uplift loads should be provided with a full-depth tension reinforcement bar; high-capacity mechanical splices may be required.

7.7.3 ACP Groups

The center-to-center pile spacing in a group should be at least three pile diameters. In such cases, the group axial capacity can be computed as the number of piles times the capacity of a single pile.

7.7.4 ACP Construction Considerations

ACP are constructed by displacing soil with a hollow-stem auger and pumping grout through the auger stem as it is withdrawn. The capacity and structural integrity of ACP are influenced significantly by the installation technique. Piles should be installed using a continuous hollow-stem auger, and cement grout should be pumped continuously during withdrawal of the auger. If groundwater is encountered during construction, auger withdrawal should be accomplished so that a positive head of grout (minimum 5 feet) is always maintained on the tip of the auger. The pump should be equipped with a functional pressure gage and stroke counter or other means of accurately measuring the quantity of grout.

Because of the above construction considerations, it is essential that the work be observed by a representative of UES. The representative should observe the pressures used to pump the grout as well as the withdrawal rate of the auger to determine that the pile is being properly constructed. In addition, pile depths and abnormalities encountered during reinforcement should be recorded.

7.7.5 Static Load Testing

At least one pile compression test should be performed for each pile type/size that is selected. The testing should be performed in accordance with ASTM D1143 using the quick loading procedure. Piles should be tested to a minimum of two times the allowable static compression load.

If the piles are to support net uplift loads, at least one tension load test is required. The test should be performed in accordance with ASTM D3689. Piles should be tested to a minimum of three times the allowable uplift load.

Load tests are required to verify the recommended pile capacity and should not be used to increase the design pile capacity. The piles used in the load tests should not be used for support of structures. UES should be consulted regarding the location of test piles.

7.7.6 Pile Foundation Settlement

Once foundation plans have been completed, they should be forwarded to UES for pile settlement analyses.

7.7.7 Pile Lateral Loading

Piles will deflect under applied lateral loads and external moments. The magnitude of the deflection will depend upon the proposed pile type, embedment, arrangement, pile cap geometry and construction, and the applied axial and lateral loading.

A lateral load test is recommended if substantial lateral loads are to be supported by vertical piles, which is typically performed in accordance with ASTM D3966 standards. A lateral load analysis is not included in this scope of work.

7.8 Floor Slabs

The bottoms of floor slabs should be separated from the high plasticity clays with a minimum 3-foot-thick buffer zone as discussed in Section 7.1 of this report. The slab-on-grade floors should be supported on stable subgrade or compacted fill. The subgrade should be prepared as recommended in Sections 7.1 and 7.2 of this report. The floor slab should be underlain by a minimum 4-inch-thick layer of granular material to serve as a capillary break and a base of support. The granular material layer should be compacted per the requirements of Table 2. The top 8 inches of clayey floor slab subgrade should be compacted and moisture-conditioned per the requirements presented in Table 1 prior to placing the granular layer.

Care should be taken during slab-on-grade construction to not allow the subgrade to become desiccated or saturated. Additionally, consideration should be given to the timing of construction relative to the time of year and weather. If slab construction is performed during relatively cold and wet weather, lime- or cement-treatment of the subgrade could be beneficial to maintain progress during construction. Otherwise, the subgrade could be weakened by softening from saturation by rain and/or snow, leading to delays in reworking the subgrade to prepare it back to its pre-softened condition.

It is recommended control joints be provided within the concrete slab-on-grade floors. These joints should be sealed to mitigate surface water infiltration until the building is enclosed. The floor slab should be structurally separated from walls, columns, footings, and penetrations to allow independent movement of the floor. Alternatively, floor slabs that are not structurally independent should be designed to allow for differential movements of that normally occur between the floor slabs, columns, and foundation walls.

A 6- to 15-mil plastic sheet should be placed below the floor to reduce the potential for moisture to permeate the slab and the potential for mold growth within the building. Some designers prefer not to place a vapor barrier directly beneath the concrete floor because it could affect the curing of the concrete, resulting in “curling” of the slab. This concern can be addressed by embedding the vapor barrier in or below the crushed rock layer below the slab.

7.9 Swimming Pool Shells

Swimming pool shells should be supported on stable subgrade or compacted fill that has been properly mitigated to address the presence of high plasticity clays. The bottoms of swimming pool excavations (pool shell subgrade) should be separated from the high plasticity clays with a minimum 3-foot-thick buffer zone, as discussed in Section 7.1 of this report.

The pool subgrade should be prepared in accordance with the recommendations presented in Sections 7.1 and 7.2 of this report. Where fine-grained soil is used as the buffer zone material, a minimum 4-inch-thick layer of clean granular material may be placed directly beneath the pool shell to provide a uniform bearing surface and facilitate construction. Where granular material is used as the buffer zone, a geotextile separation fabric is recommended at the interface between high plasticity clay and the granular buffer material to reduce migration of fines and maintain long-term performance.

For below grade walls, we recommend that in situ high plasticity clays be removed and replaced in a wedge drawn upward and away from the edge of the pool wall footing, starting approximately 5 feet back from the wall, at a 45-degree angle or flatter, to mitigate potential swelling and shrinkage pressures associated with potentially expansive soils.

Swimming pool shells should be structurally designed to accommodate potential differential movement and uplift forces associated with potentially expansive soils and groundwater conditions. Pool walls should incorporate the 5-foot setback wedge/backfill where high plasticity clay is present to mitigate lateral pressures. The pool designer should evaluate the need for hydrostatic relief systems or underdrainage based on final excavation conditions.

Waterproofing of the swimming pool shell is the responsibility of the pool designer and contractor. The waterproofing system should be compatible with the prepared subgrade, wedge/backfill, and buffer zone, and designed to accommodate site-specific conditions including potential groundwater, drainage, and expansive soils. The pool designer may have additional recommendations to be implemented during construction. The geotechnical engineer should be notified if the pool design includes waterproofing elements that interact directly with the subgrade, backfill wedge, or buffer zone materials.

7.10 Lateral Earth Pressures

We understand below-grade walls are planned for the swimming pool. We have assumed the below-grade walls will be up to 10 feet tall. UES should be contacted if below-grade wall heights are greater than those reported.

The following paragraphs outline below-grade wall recommendations for the swimming pool only. We are not aware of any other below-grade structures at the site. Once wall plans have been finalized, plans should be forwarded to UES for review.

7.10.1 Static Lateral Earth Pressures

Walls can be designed for static active earth pressures if the top of the wall is permitted to tilt laterally after construction approximately 0.5 percent of the wall height for sandy retained soil, and 2 percent of the wall height for clay retained soil. These walls are referred to as yielding walls. Rigid walls and walls with fixed heads, referred to as non-yielding walls, should be designed for at-rest earth pressures in the static condition, unless the structural design permits independent rotation.

Presented in Table 5 are general design soil parameters for calculating load force, assuming level ground at the top and toe and a vertical wall backside. Other wall configurations will have different lateral earth pressures; UES should be contacted to revise these recommendations should inclined ground existing at the top or toe of the wall or if wall inclinations other than vertical are to be considered. We recommend that the in situ, high plasticity clays be removed and replaced in a wedge drawn upward and away from the edge of the wall footing, starting approximately 5 feet back from the wall, at a 45-degree angle or flatter, to mitigate potential swelling and shrinkage pressures associated with expansive soils.

Table 5. Static Lateral Earth Pressure Parameters.

Soil Type Behind Wall	**Moist Unit Weight, γ (pcf)**	**Φ, (°)**	**Static Lateral Earth Pressure Parameter**	
			Active Coefficient, K_a	**At-Rest Coefficient, K_0**
High Plasticity Fat Clay (CH; in situ; not recommended behind wall)	118	--	--	--
Lean Clay (CL)	122	28	0.36	0.53
Poorly Graded Sand or Gravel (SP/GP) (e.g., 1-inch-clean)	125	34	0.28	0.44
Well-Graded Gravel-Sand (GW/SW) (e.g., 1-inch-minus)	135	36	0.26	0.41

The static lateral loading force for yielding and non-yielding below-grade walls adjacent to level ground may be estimated using the following equation:

$$P = \frac{1}{2} K\gamma H^2$$

Where:

P = Static lateral earth pressure (pounds per foot)
K = Earth pressure coefficient (active or at-rest)
γ = Moist unit weight of retained soil (pcf)
H = Wall height in feet (bottom of foundation to top of wall)

Static earth pressures from the retained soil are triangular and the equivalent force can be assumed to act on the wall at a distance equal to one-third of the wall height, measured from the base of the wall. The static lateral pressure associated with uniform surcharge loads are rectangular and the equivalent force should be considered to act horizontally at the midpoint of the wall.

For the above equation to be valid for sand or gravel backfill, the backfill should be placed in a wedge drawn upward and away from the edge of the wall footing, starting approximately 5 feet back from the wall, at a 45-degree angle or flatter, to reduce the influence of swell and shrinkage pressures from expansive soils. Further, soft soils encountered during excavations should be removed prior to placement of backfill. Design drawings should reflect this requirement.

An adequate subsurface drainage system should be installed behind the base of the wall to prevent hydrostatic loading on the wall. Full hydrostatic pressures should be used in computing the lateral loads on walls whenever the retained material cannot be drained. Granular backfill behind the wall should be capped with at least 1 foot of compacted cohesive soil to reduce surface water infiltration behind the wall.

7.10.2 Sliding

A sliding coefficient of 0.35 may be used to evaluate sliding for walls bearing on fine-grained soils. A sliding coefficient of 0.50 may be used to evaluate sliding for walls bearing on coarse-grained soils placed a minimum of 2 feet below the base of the wall foundation. Passive resistance should be neglected for exterior footings unless the walls will have a shear key. If the base will include a shear key, UES should be contacted to provide additional data

7.11 Pavements

A project-specific pavement design was not performed as vehicle loads and traffic patterns were not provided. The following pavement recommendations are based on the separation of pavement base materials from the high plasticity clays with a minimum 3-foot-thick buffer zone as discussed in Section 7.1 of this report. Pavements are to be placed on stable in-situ soil or compacted fill. The pavement subgrade should be proof-rolled and prepared as recommended in the Sections 7.1 and 7.2 of this report. Once the subgrade is prepared, it should be promptly paved to protect it from the weather, as the naturally occurring soils of the area are susceptible to changes in moisture content.

7.11.1 Flexible Pavements

The flexible pavement recommendations provided herein are based on the following assumed parameters for the 1993 AASTHO pavement design method.

Table 6. Assumed AASHTO Flexible Pavement Design Parameters.

Parameter	Light-Duty Pavement	Heavy-Duty Pavement
Reliability	90%	
Standard Deviation	0.49	
CBR	4.0[a]	
Soil Resilient Modulus, M_R	6,000 psi	
Estimated Equivalent Single-Axle Loads (ESALs) Over the Design Life of the Facility	20,500	268,900
Drainage Coefficient	1.0	
Initial Serviceability	4.2	
Terminal Serviceability	2.0	
Structural Number, SN	2.0	3.0

[a] Based on construction of a 3-foot-thick buffer zone below new pavement bases.

To arrive at the layer thicknesses presented herein, a drainage coefficient of 1.0 was assumed. Recommendations for structural numbers (SN) of 2 and 3 have been provided for light- and heavy-duty sections, respectively; the required SN will be dependent on the number of equivalent single-axle loads (ESALs) estimated for the design life of the facility. Two alternative pavement sections for the considered SN's have been provided for consideration and cost evaluation.

Table 7. Flexible Pavement Thickness Recommendations.

Layer Type	ARDOT Pavement Coefficient[a]	Light-Duty Pavement SN = 2.0		Heavy-Duty Pavement SN = 3.0	
		Layer Thicknesses (inches)		Layer Thicknesses (inches)	
		Alt. 1	**Alt. 2**	**Alt. 1**	**Alt. 2**
ACHM[b] Surface Course	0.44	2.0	2.0	2.0	2.0
ACHM[b] Base Course	0.36	--	--	2.0	2.0
Soil Cement Base	0.20	6.0	--	7.0	--
Crushed Stone Base	0.14	--	8.0	--	10.0[c]

[a] The materials should meet the requirements set forth in the applicable sections of the Roadway Design Manual, latest edition, published by the Arkansas Department of Transportation (ARDOT).

[b] ACHM = Asphalt Concrete Hot Mix.

[c] To be placed in multiple lifts.

7.11.2 Rigid Pavements

The alternative rigid pavement recommendations provided herein are based on the following assumed parameters for the 1993 AASHTO pavement design method.

Table 8. Assumed AASHTO Rigid Pavement Design Parameters.

Parameter	Light-Duty Pavement	Heavy-Duty Pavement
Reliability	90%	
Standard Deviation	0.35	
Composite Modulus of Subgrade Reaction (Accounting for Loss of Support)	50 pci	
Estimated Single-Axle Loads (ESALs) Over the Design Life of the Facility	65,100	384,100
Initial Serviceability	4.5	
Terminal Serviceability	2.0	
Concrete 28-Day Strength	4,000 psi	
Concrete Modulus of Elasticity	3,600,000 psi	
Concrete Modulus of Rupture	500 psi	
Load Transfer Coefficient	4.0	
Subbase Minimum Elastic Modulus	50,000 psi	

Table 9. Rigid Pavement Thickness Recommendations.

Layer Type	Light-Duty Pavement		Heavy-Duty Pavement	
	Layer Thicknesses (inches)		Layer Thicknesses (inches)	
	Alt. 1	Alt. 2	Alt. 1	Alt. 2
Portland Cement Concrete (PCC)	6.0	6.0	8.0	8.0
Soil Cement Subbase	7.0	--	7.0	--
Crushed Stone Subbase	-	7.0	--	7.0

Improved performance of rigid pavements can be achieved by utilizing load transfer (dowel) bars. The diameter of the dowel bars is typically equal to one-eighth of the slab thickness (1 inch is recommended for heavy-duty pavements). The length of the bars should be a minimum of 18 inches (2 feet is preferable; 1 foot on each side of the joint). The spacing of the dowel bars is typically 12 inches on center.

At expansion joints, the dowels should accommodate the lateral movement of the slab due to expansion/contraction. At contraction joints, the depth of the reservoir should be a minimum of ½ of the joint width. The spacing of contraction joints depends upon the thickness of the slab. UES should be contacted to assist with joint spacing once the pavement thickness is decided.

Refer to AASTHO or American Concrete Pavement Association for further recommendations.

7.12 Utility Construction

Settlement of trench backfill can result in unsightly depressions and localized pavement failures. The magnitude of settlement can be reduced by mechanically compacting the trench backfill. Select granular backfill can be used for pipe bedding and minimum cover for utilities. The remainder of the utility trenches should be backfilled with flowable fill or compacted clayey soils up to the design subgrade elevation to reduce the potential for water collecting in these trenches and being absorbed by the surrounding clays, causing heave of foundations, slabs, pavement, etc.

Granular bedding and backfill that exhibits a well-defined moisture density relationship should be compacted and moisture-conditioned per the requirements presented in Table 1; otherwise, the granular material should be compacted to at least the minimum relative densities indicated in Table 2 in Section 7.2 of this report.

Utility trench backfill should be placed in 6- to 8-inch thick lifts with each lift compacted to at least the specified degree of compaction. Thinner lifts should be used for lighter compaction equipment. The backfill should not be flushed with water in an attempt to obtain compaction.

For utilities within the perimeters of the proposed building, one of the following options can be implemented to further reduce the potential for water collecting in the utility trenches:

1. Use flowable fill in place of granular bedding and pipe zone backfill around utility pipes. Provisions should be implemented during construction to keep the pipes from floating in the flowable fill until the flowable fill sets.

2. The bottom of the utility excavation should generally be sloped to drain to a collection pipe (underdrain) in the bottom of the utility excavation at its downstream end. The collection pipe should then connect to an outlet, such as the proposed storm sewer system.

3. The granular bedding and pipe zone backfill should be capped with at least 1 foot of compacted clay backfill prior to the granular bedding and backfill collecting water. Additionally, concrete dams or anti-seepage collars should be provided where the utility crosses beneath the exterior footings of the proposed building. These dams or collars should extend at least 6 inches beyond the sides and bottoms of the utility trenches into the in-situ soils to stop water from migrating underneath the building. If groundwater seepage is observed in the utility excavations, this option should not be implemented, but rather one of the other two options.

Prior to placing the bedding and utilities within the utility trench, soft, saturated, and compressible material should be removed from the bottom of the trench to expose stiff soils.

8.0 RECOMMENDED ADDITIONAL SERVICES

The conclusions and recommendations given in this report are based on: UES' 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 UES, we recommend that UES be included in the final design and construction process, and be retained to review the project plans and specifications to confirm that the recommendations given in this report have been correctly implemented. We recommend that UES 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 that UES 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.

9.0 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 that the information is provided for factual data only, and not as a warranty of subsurface conditions presented in this report.

UES has attempted to conduct the services reported herein in a manner consistent with that 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 UES' 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, UES must be allowed to review them to assess their impact on the findings, conclusions, and/or design recommendations given in this report. UES 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. UES should be retained to perform construction observation and continue its geotechnical engineering service using observational methods. UES cannot assume liability for the adequacy of its recommendations when they are used in the field without UES 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 <u>not</u> 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 <u>Not</u> 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 <u>not</u> 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 <u>not</u> building-envelope or mold specialists.***

GEOPROFESSIONAL
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ASSOCIATION

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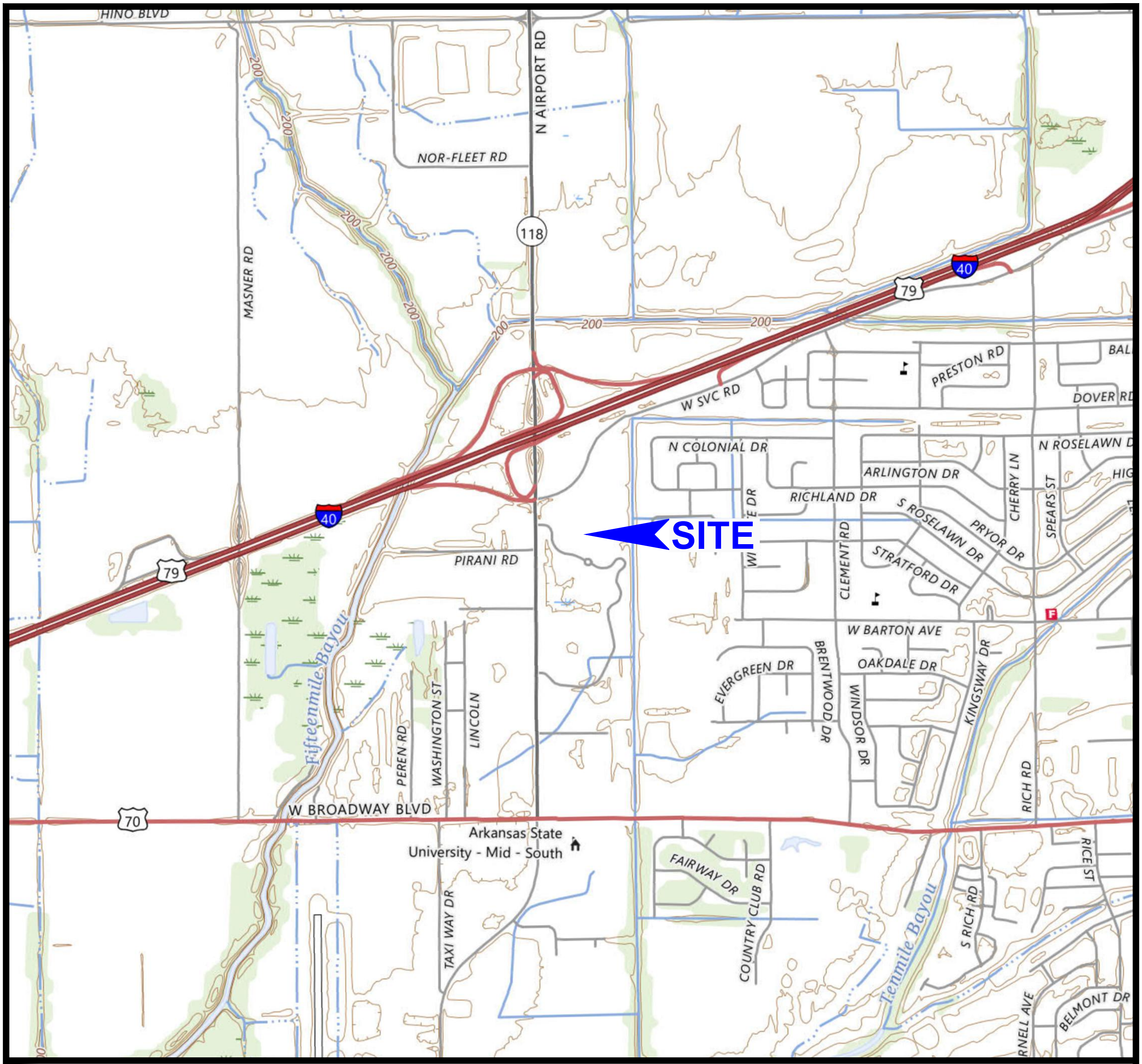
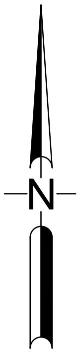
APPENDIX B – FIGURES



NOTES

1. Plan adapted from a 7.5 minute U.S.G.S. map for West Memphis, Arkansas-Tennessee quadrangle, last revised in 2024.



Drawn By: WAH	Ck'd By: jdm	App'vd By: rtf
Date: 1-14-26	Date: 1-16-26	Date: 2-13-26

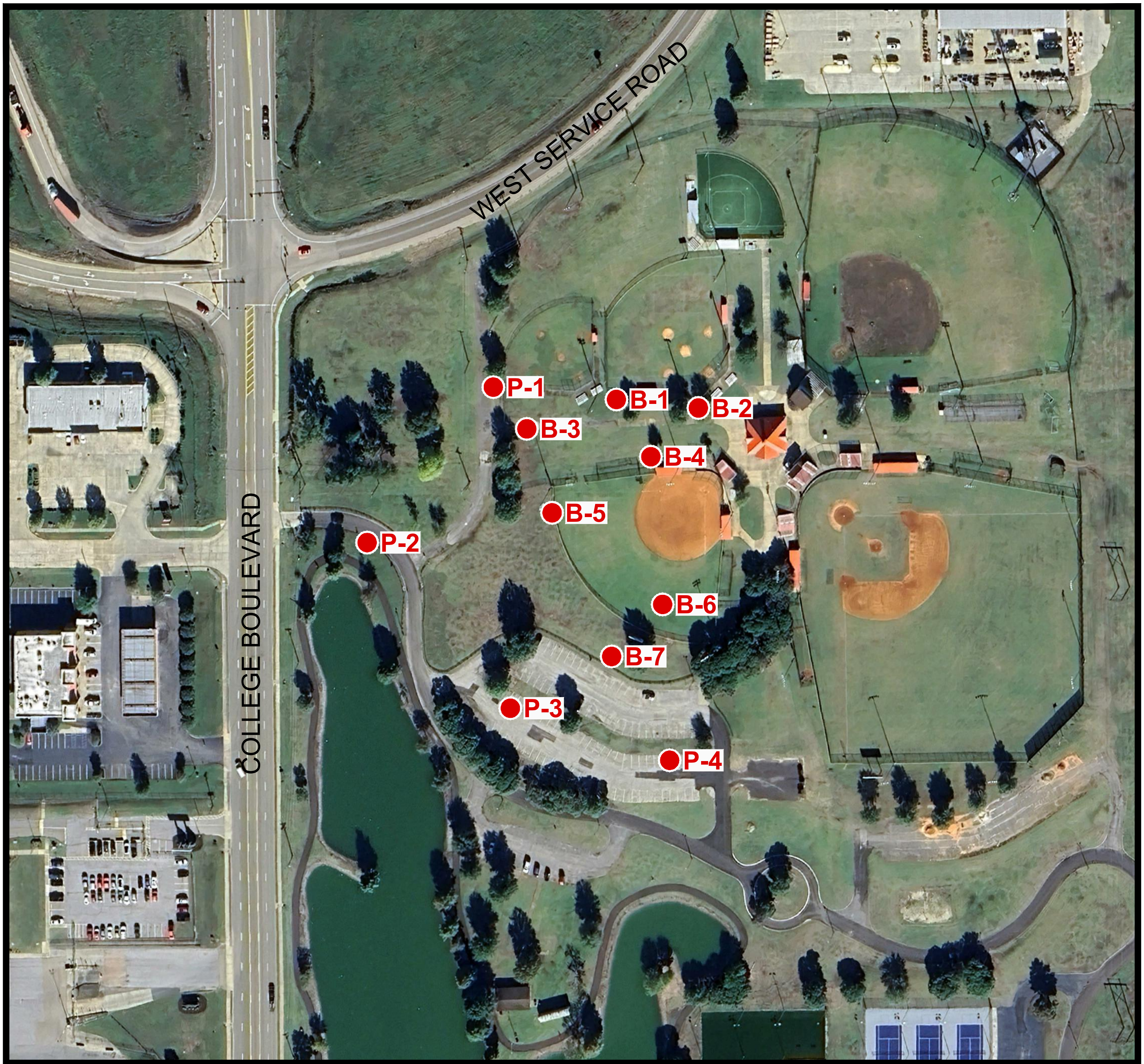
UES™

Proposed Recreation Center
West Memphis, Arkansas

SITE LOCATION
AND TOPOGRAPHY

Project Number
A25138.00362.001

FIGURE 1



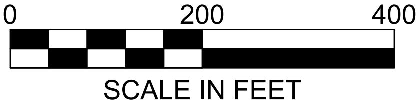
NOTES

1. Plan adapted from a November 9, 2025 aerial photograph courtesy of Google Earth.

2. Borings were located in the field with reference to site features and are shown approximate only.

LEGEND

● Boring Location



N

Drawn By: WAH	Ck'd By: jdm	App'vd By: rtf
Date: 1-14-26	Date: 1-16-26	Date: 2-13-26

UES™

Proposed Recreation Center
West Memphis, Arkansas

AERIAL PHOTOGRAPH OF SITE AND BORING LOCATIONS

Project Number
A25138.00362.001

FIGURE 2

APPENDIX C – BORING INFORMATION

Boring Logs

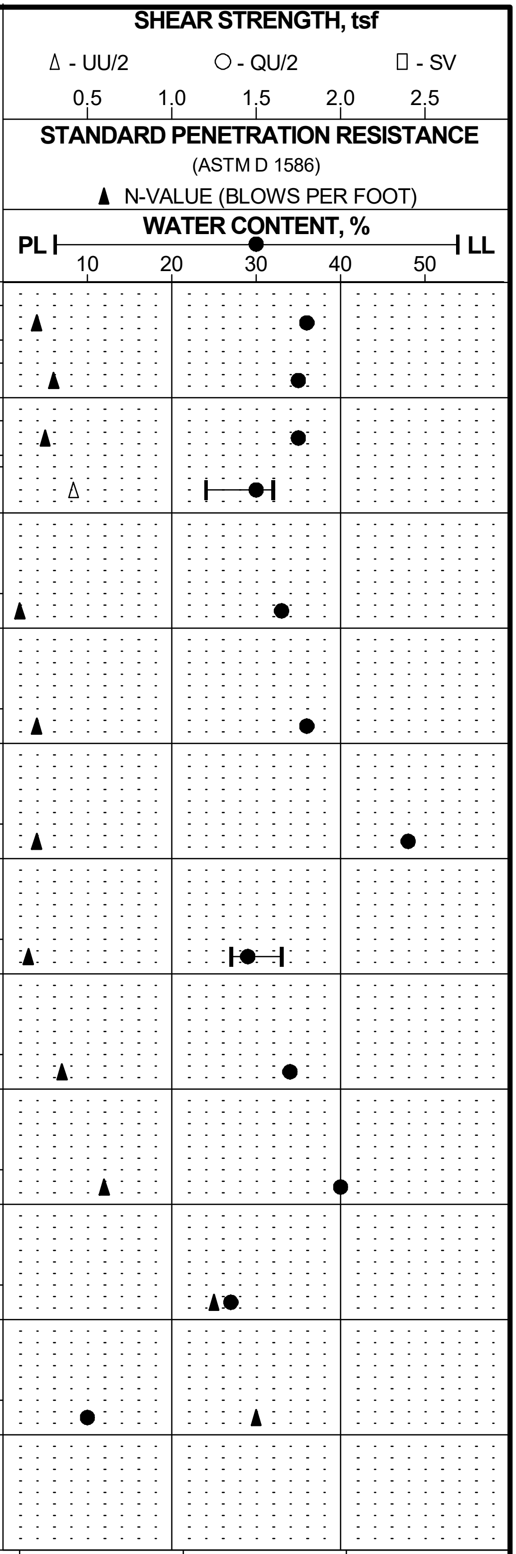
Boring Log Terms and Symbols

Surface Elevation: N/A

Datum N/A

Completion Date: 12/15/25

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Topsoil: 4 inches			
	Soft to medium stiff, gray, FAT CLAY, trace sand - CH		2-1-3	SS1
5			2-2-4	SS2
			2-2-3	SS3
10	Medium stiff, gray SILT, some sand - (ML) 78% passing No. 200 sieve		94	ST4
15	Soft, gray, silty, LEAN CLAY - CL		1-1-1	SS5
20	Soft, gray, silty, FAT CLAY - CH		2-2-2	SS6
25			2-2-2	SS7
30	Soft to medium stiff, gray SILT - (ML)		1-1-2	SS8
35			6-4-3	SS9
40	Stiff, gray, silty, FAT CLAY, some sand - CH		4-5-7	SS10
45	Medium dense, gray and brown SAND, trace silt - SP-SM little black organics/charcoal		5-11-14	SS11
50			6-13-17	SS12
	Boring terminated at 50 feet.			



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

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM

WASHBORING FROM 15 FEET

AKM DRILLER CNM LOGGER

Diedrich D50 DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 94 %

REMARKS: Boring located in proposed building footprint.

Drawn by: SAS	Checked by: RSP	App'vd. by: RTF
Date: 12/16/25	Date: 1/12/26	Date: 1/15/26

UES™

West Memphis Recreation Center
West Memphis, Arkansas

LOG OF BORING: B-1

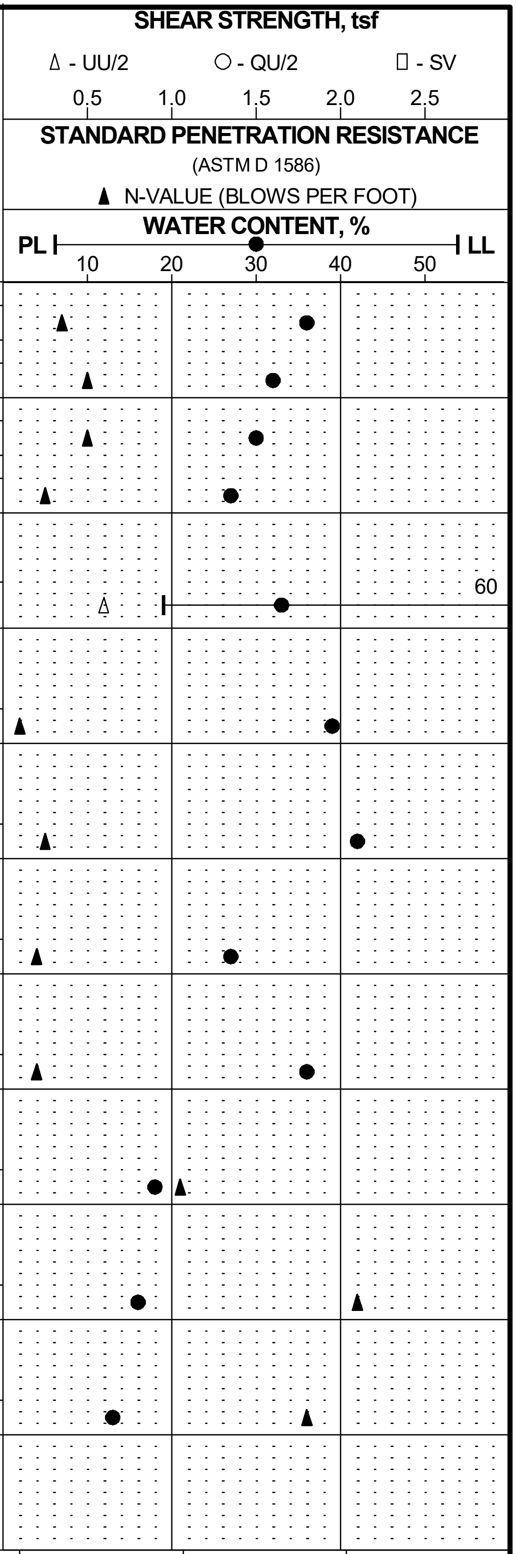
Project No. A25138.00362.001

Surface Elevation: N/A

Datum N/A

Completion Date: 12/15/25

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Topsoil: 4 inches		
	Medium stiff to stiff, gray, FAT CLAY, trace sand - CH	1-3-4	SS1
	trace organics	3-4-6	SS2
5			
	trace sand and organics	2-4-6	SS3
	Medium stiff, gray, sandy, LEAN CLAY - CL	2-2-3	SS4
10			
	Soft to stiff, gray, FAT CLAY - (CH)	88	ST5
15	∇		
20		2-1-1	SS6
	Medium stiff, gray, sandy, LEAN CLAY - CL trace organics	2-2-3	SS7
25			
	Soft, gray, sandy, FAT CLAY - CH	2-1-3	SS8
30			
	Very loose to medium dense, gray, SILTY SAND - SM	2-1-3	SS9
35			
	40% passing No. 200 sieve	10-8-13	SS10
40			
	Dense, gray and brown SAND - SP	10-17-25	SS11
45			
50		10-13-23	SS12
	Boring terminated at 50 feet.		



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

GROUNDWATER DATA

ENCOUNTERED AT 15 FEET ∇

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM

WASHBORING FROM 15 FEET

AKM DRILLER CNM LOGGER

Diedrich D50 DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 94 %

REMARKS: Boring located in proposed building footprint.

Drawn by: SAS	Checked by: RSP	App'vd. by: RTF
Date: 12/16/25	Date: 1/12/26	Date: 1/15/26

UES™

West Memphis Recreation Center
West Memphis, Arkansas

LOG OF BORING: B-2

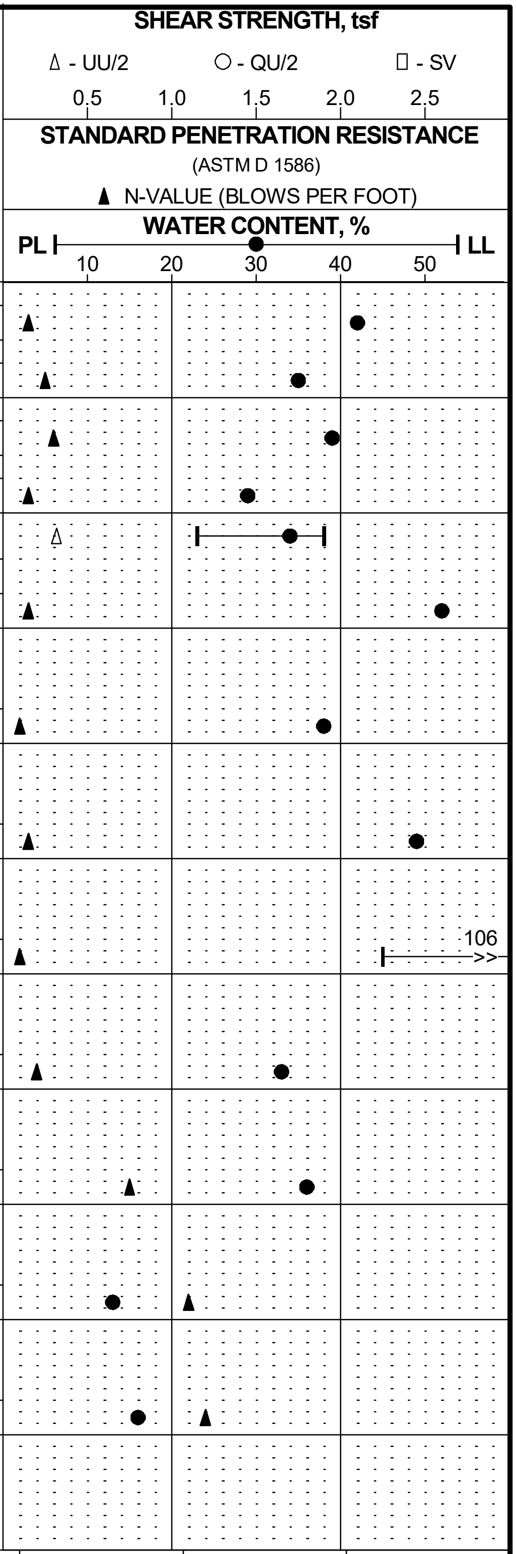
Project No. A25138.00362.001

Surface Elevation: N/A

Datum N/A

Completion Date: 12/16/25

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Topsoil: 3 inches Soft to medium stiff, gray, silty, FAT CLAY - CH		1-1-2	SS1
5			1-2-3	SS2
			2-3-3	SS3
			2-1-2	SS4
10	Medium stiff, brown, silty, LEAN CLAY - (CL)		90	ST5
15	Soft, gray, FAT CLAY - CH	∇	2-1-2	SS6
20	Soft, gray, LEAN CLAY - CL		0-0-2	SS7
25	Soft, gray, FAT CLAY - (CH)		2-1-2	SS8
30			1-1-1	SS9
35	Soft to stiff, gray, silty, LEAN CLAY - CL		1-2-2	SS10
40			2-5-10	SS11
45	Medium dense, gray and brown SAND, trace silt and clay - SP-SM trace organics		7-9-13	SS12
50			8-10-14	SS13
	Boring terminated at 50 feet.			



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

GROUNDWATER DATA

ENCOUNTERED AT 15 FEET ∇

DRILLING DATA

___ AUGER 3 3/4" HOLLOW STEM

WASHBORING FROM 15 FEET

AKM DRILLER BCP LOGGER

Diedrich D50 DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 94 %

REMARKS: Boring located in proposed building footprint.

Drawn by: SAS	Checked by: RSP	App'vd. by: RTF
Date: 12/17/25	Date: 1/12/26	Date: 1/15/26

UES™

West Memphis Recreation Center
West Memphis, Arkansas

LOG OF BORING: B-3

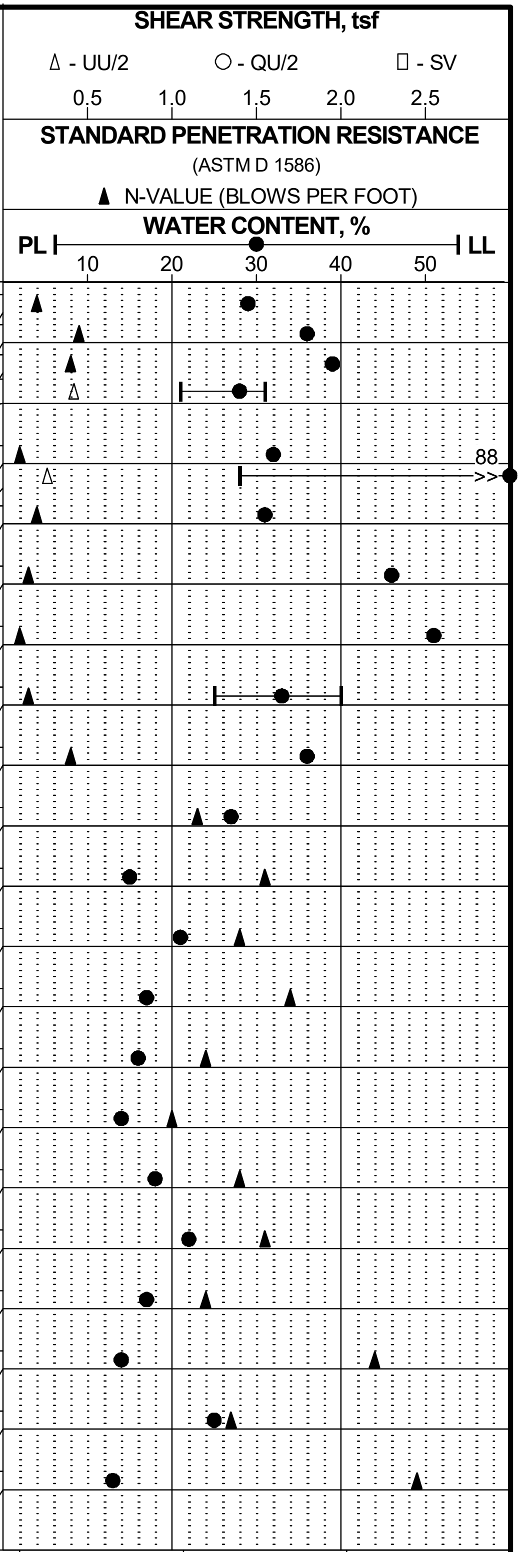
Project No. A25138.00362.001

Surface Elevation: **N/A**

Datum **N/A**

Completion Date: **12/13/25**

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Topsoil: 4 inches		
	Soft to stiff, gray, silty, LEAN CLAY, trace sand - (CL)	1-1-3	SS1
5	trace organics	2-4-5	SS2
	trace organics	2-4-4	ST3
10		97	ST4
15	Soft to medium stiff, gray, FAT CLAY - (CH) ▽	1-1-1	SS5
	97% passing No. 200 sieve	65	ST6
20		2-2-2	SS7
25		1-1-2	SS8
30	Soft, gray, silty, LEAN CLAY - (CL)	1-1-1	SS9
35		1-1-2	SS10
40	Medium stiff, gray, clayey SILT, little sand - ML	2-4-4	SS11
45	Medium dense to dense, gray and brown SAND, trace silt and clay - SP-SM 8% passing No. 200 sieve	8-11-12	SS12
50		10-16-15	SS13
55	trace gravel	8-11-17	SS14
60		12-15-19	SS15
65	trace gravel	9-12-12	SS16
70	Medium dense, gray and brown SAND, little gravel, trace silt and clay - SP-SC 8% passing No. 200 sieve	11-11-9	SS17
75	Medium dense to dense, gray and brown SAND - SP trace black organics	10-16-12	SS18
80		13-15-16	SS19
85	trace gravel	8-11-13	SS20
90	Dense, gray SAND, trace clay and gravel - SP	15-18-26	SS21
95	Medium dense, gray, SILTY SAND - SM	11-12-15	SS22
100	Very dense, gray SAND, trace gravel - SP	19-23-26	SS23
	Boring terminated at 100 feet.		



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

GROUNDWATER DATA

ENCOUNTERED AT 15 FEET ▽

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM

WASHBORING FROM 20 FEET

AKM DRILLER KRF LOGGER

Diedrich D50 DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 94 %

REMARKS: Boring located in proposed building footprint.

Drawn by: SAS	Checked by: RSP	App'vd. by: RTF
Date: 12/16/25	Date: 1/12/26	Date: 1/15/26

UES™

West Memphis Recreation Center
West Memphis, Arkansas

LOG OF BORING: B-4

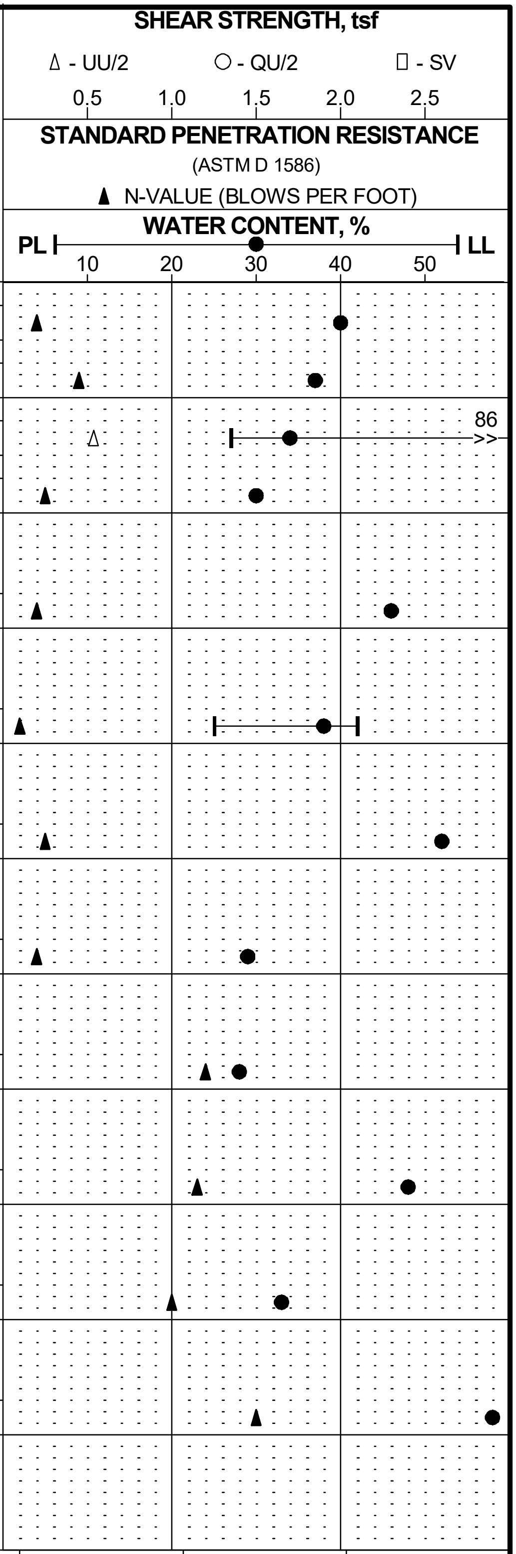
Project No. A25138.00362.001

Surface Elevation: N/A

Datum N/A

Completion Date: 12/12/25

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Topsoil: 4 inches		
	Soft to stiff, gray, FAT CLAY, trace sand - (CH)	2-1-3	SS1
5		2-4-5	SS2
		87	ST3
10	Medium stiff, gray, silty, LEAN CLAY - CL	2-3-2	SS4
15	Soft, gray, FAT CLAY, trace sand - CH	2-1-3	SS5
20	Soft, gray, silty, LEAN CLAY - (CL)	1-1-1	SS6
25	Medium stiff, gray, FAT CLAY, trace sand - CH	2-2-3	SS7
30	Soft to very stiff, gray, sandy, clayey SILT - ML 70% passing No. 200 sieve	2-1-3	SS8
35		3-9-15	SS9
40	Medium dense, gray and brown, silty, CLAYEY SAND - SC	8-12-11	SS10
45	Medium dense, gray and brown SAND, trace silt and clay - SP-SM	7-8-12	SS11
50		11-13-17	SS12
	Boring terminated at 50 feet.		



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

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM

WASHBORING FROM 15 FEET

AKM DRILLER JRM LOGGER

Diedrich D50 DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 94 %

REMARKS: Boring located in proposed building footprint.

Drawn by: SAS	Checked by: RSP	App'vd. by: RTF
Date: 12/16/25	Date: 1/12/26	Date: 1/15/26

UES ™

West Memphis Recreation Center
West Memphis, Arkansas

LOG OF BORING: B-5

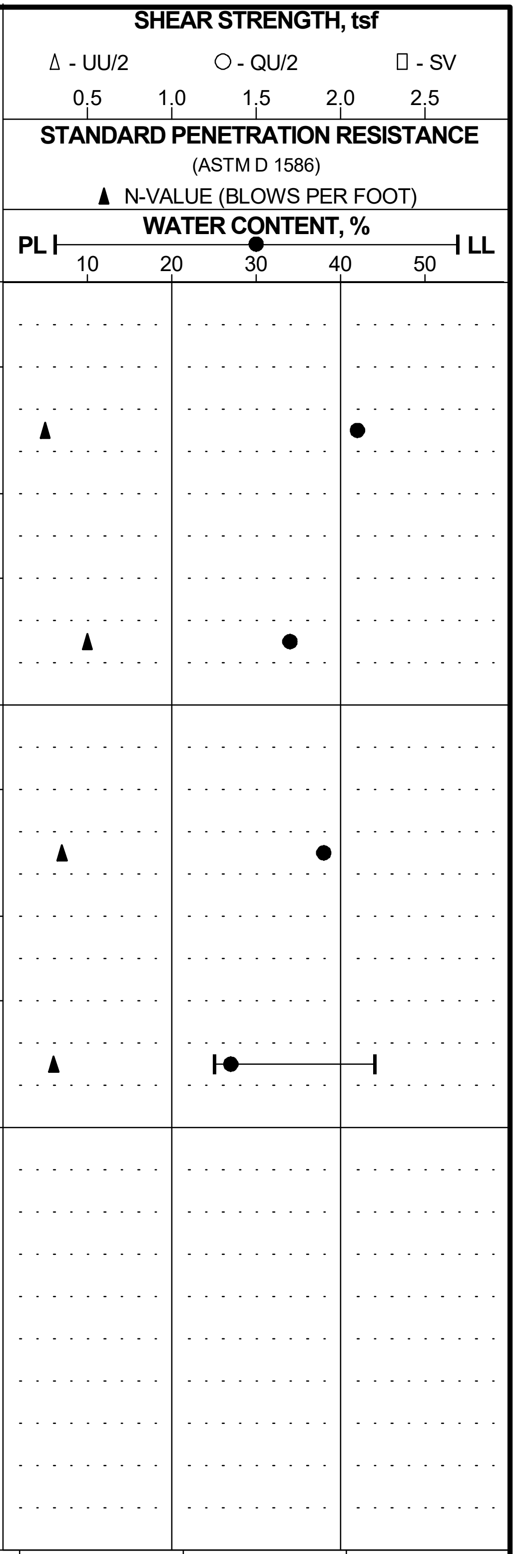
Project No. A25138.00362.001

Surface Elevation: N/A

Datum N/A

Completion Date: 12/8/25

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Topsoil: 4 inches			
	Medium stiff to stiff, gray, FAT CLAY - CH			
			2-2-3	SS1
5			2-4-6	SS2
			3-3-4	SS3
	Medium stiff, gray, silty, LEAN CLAY, trace sand - (CL)		2-3-3	SS4
10	Boring terminated at 10 feet.			



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

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 1/4" HOLLOW STEM

WASHBORING FROM __ FEET

RF DRILLER DBR LOGGER

Geoprobe 7822DT DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 60 %

REMARKS: Boring located in proposed building footprint.

Drawn by: SAS	Checked by: RSP	App'vd. by: RTF
Date: 12/11/25	Date: 1/12/26	Date: 1/15/26

UES™

West Memphis Recreation Center
West Memphis, Arkansas

LOG OF BORING: B-6

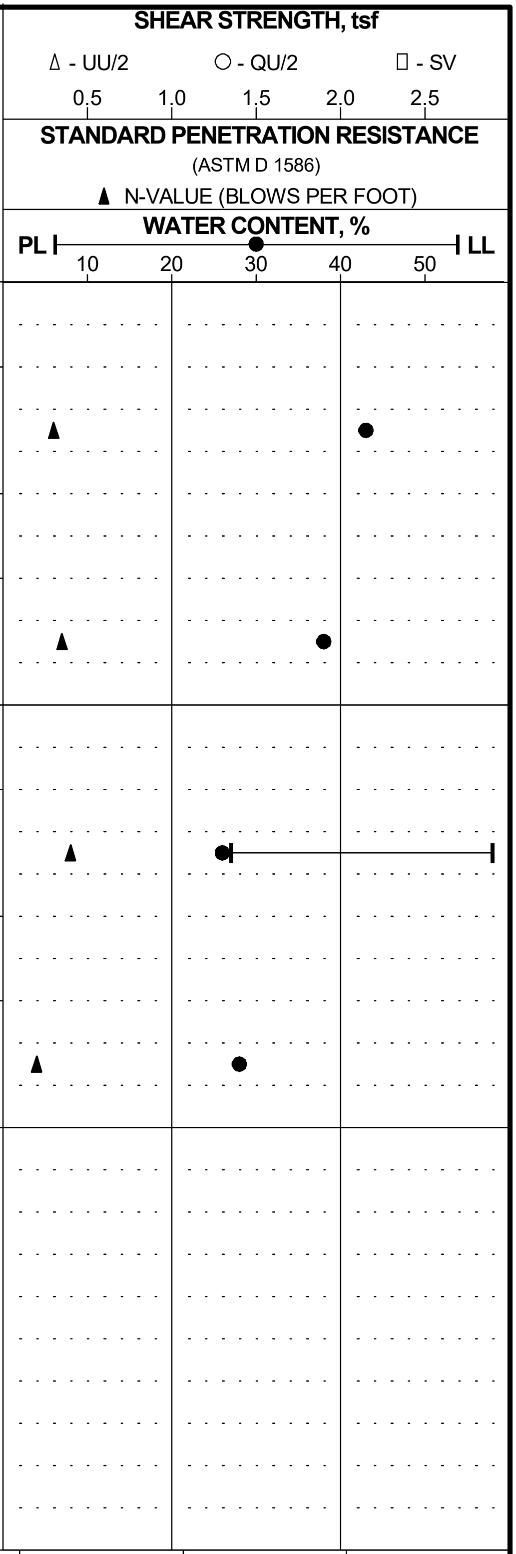
Project No. A25138.00362.001

Surface Elevation: N/A

Datum N/A

Completion Date: 12/8/25

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Topsoil: 4 inches			
	Medium stiff, gray, FAT CLAY - (CH)			
			2-3-3	SS1
	trace sand			
			2-3-4	SS2
5				
			2-4-4	SS3
	Soft, gray, silty, LEAN CLAY, trace sand - CL			
			2-2-2	SS4
10	Boring terminated at 10 feet.			



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

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 1/4" HOLLOW STEM
WASHBORING FROM __ FEET
RF DRILLER DBR LOGGER
Geoprobe 7822DT DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 60 %

REMARKS: Boring located in proposed building footprint.

Drawn by: SAS	Checked by: RSP	App'vd. by: RTF
Date: 12/11/25	Date: 1/12/26	Date: 1/15/26

UES™

West Memphis Recreation Center
West Memphis, Arkansas

LOG OF BORING: B-7

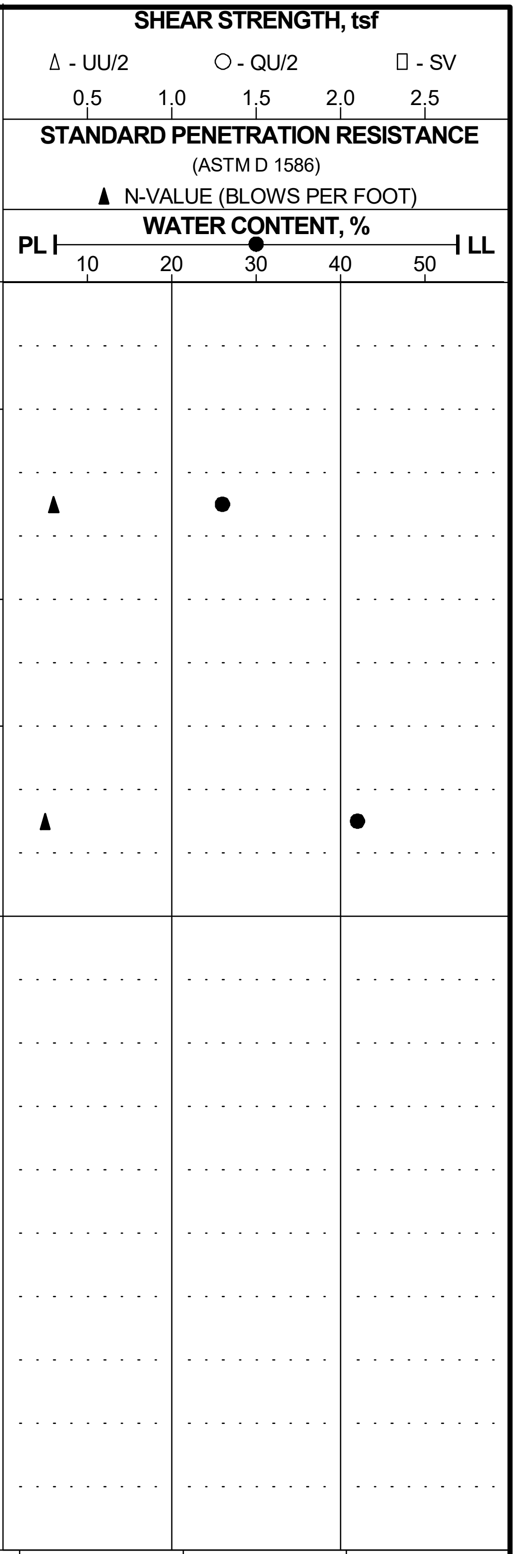
Project No. A25138.00362.001

Surface Elevation: N/A

Datum N/A

Completion Date: 12/8/25

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Topsoil: 4 inches			
	Medium stiff, gray, FAT CLAY, trace sand - CH			
	some silt and gravel			
			6-4-2	SS1
			2-2-3	SS2
5	Boring terminated at 5 feet.			



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

LOG OF BORING 2020 JDM A25138.00362.001.GPJ GTINC 0638301.GPJ 1/16/26

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 1/4" HOLLOW STEM
WASHBORING FROM __ FEET
RF DRILLER DBR LOGGER
Geoprobe 7822DT DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 60 %

REMARKS: Boring located in proposed parking and drive areas.

Drawn by: SAS	Checked by: RSP	App'vd. by: RTF
Date: 12/11/25	Date: 1/12/26	Date: 1/15/26

UES™

West Memphis Recreation Center
West Memphis, Arkansas

LOG OF BORING: P-1

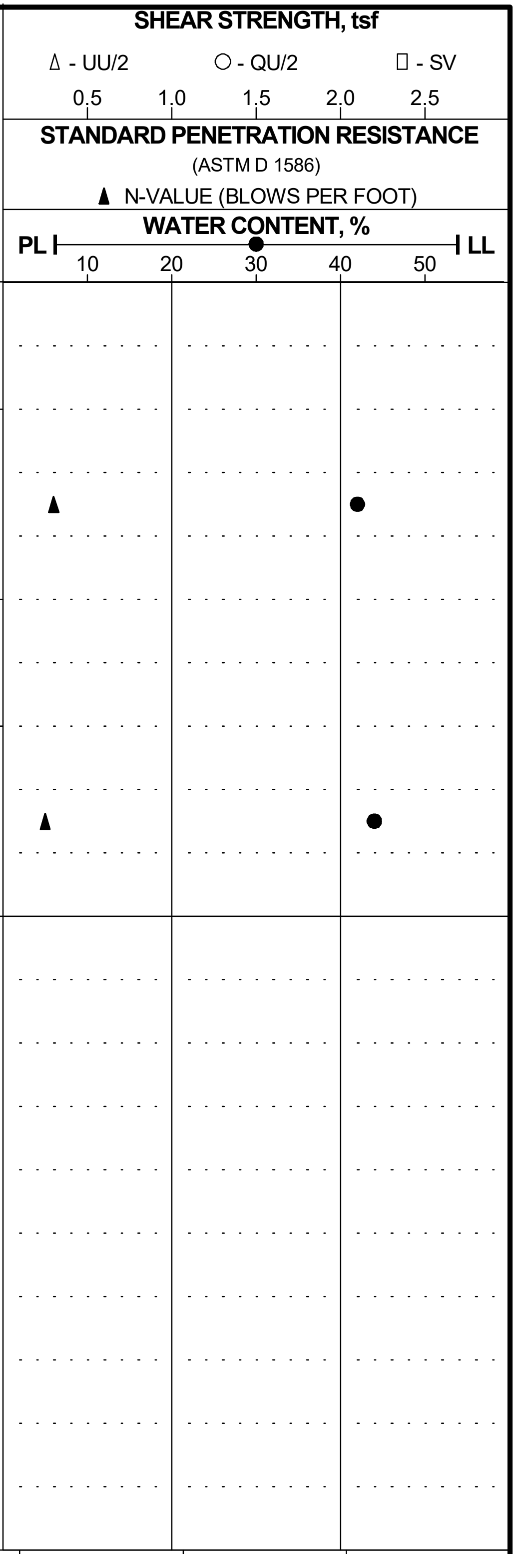
Project No. A25138.00362.001

Surface Elevation: N/A

Datum N/A

Completion Date: 12/8/25

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Topsoil: 4 inches	░		
	Medium stiff, gray, FAT CLAY, trace sand - CH	▨	2-3-3	SS1
		▨	2-2-3	SS2
5	Boring terminated at 5 feet.			



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

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 1/4" HOLLOW STEM

WASHBORING FROM __ FEET

RF DRILLER DBR LOGGER

Geoprobe 7822DT DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 60 %

REMARKS: Boring located in proposed parking and drive areas.

Drawn by: SAS	Checked by: RSP	App'vd. by: RTF
Date: 12/11/25	Date: 1/12/26	Date: 1/15/26

UES™

West Memphis Recreation Center
West Memphis, Arkansas

LOG OF BORING: P-2

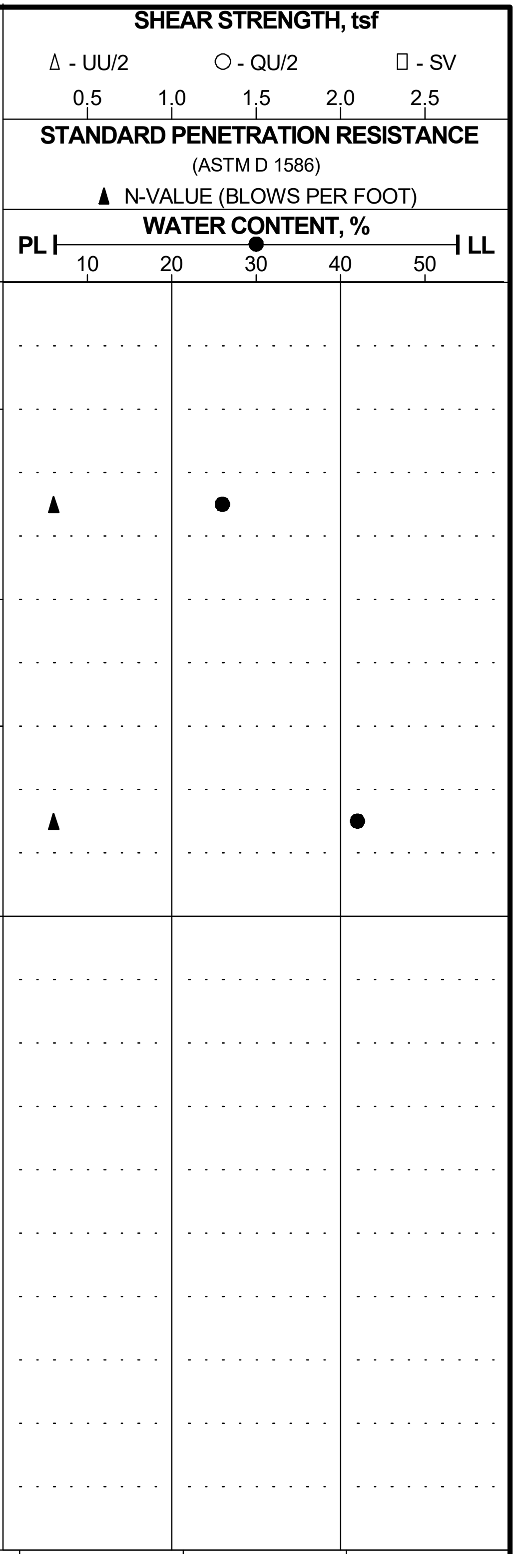
Project No. A25138.00362.001

Surface Elevation: **N/A**

Datum **N/A**

Completion Date: **12/8/25**

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Asphalt: 8 inches	■		
	Medium stiff, gray, silty, LEAN CLAY - CL	▨	2-3-3	SS1
	Medium stiff, gray, FAT CLAY - CH	▨	2-3-3	SS2
5	Boring terminated at 5 feet.			



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

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 1/4" HOLLOW STEM

WASHBORING FROM __ FEET

RF DRILLER DBR LOGGER

Geoprobe 7822DT DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 60 %

REMARKS: Boring located in proposed parking and drive areas.

Drawn by: SAS	Checked by: RSP	App'vd. by: RTF
Date: 12/11/25	Date: 1/12/26	Date: 1/15/26

UES™

West Memphis Recreation Center
West Memphis, Arkansas

LOG OF BORING: P-3

Project No. A25138.00362.001

Surface Elevation: N/A

Datum N/A

Completion Date: 12/8/25

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Asphalt: 8 inches	■		
	Medium stiff, gray, silty, LEAN CLAY, trace gravel and sand - CL	▨	6-4-3	SS1
	Medium stiff, gray, FAT CLAY - CH	▨	2-3-3	SS2
5	Boring terminated at 5 feet.			

SHEAR STRENGTH, tsf

∆ - UU/2 ○ - QU/2 ▯ - SV

0.5 1.0 1.5 2.0 2.5

STANDARD PENETRATION RESISTANCE

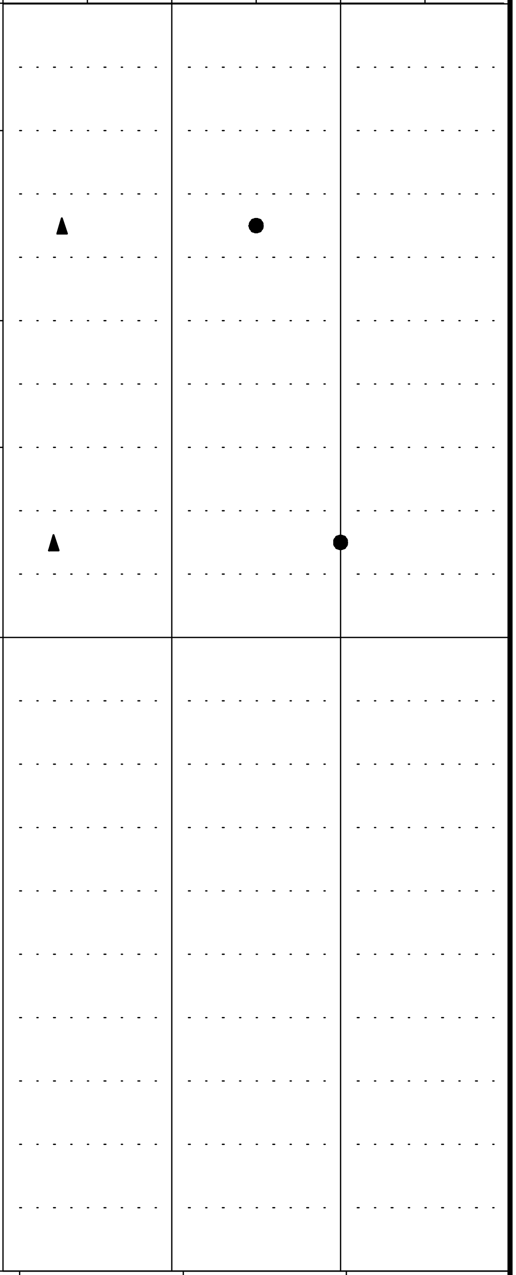
(ASTM D 1586)

▲ N-VALUE (BLOWS PER FOOT)

WATER CONTENT, %

PL **LL**

10 20 30 40 50



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

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 1/4" HOLLOW STEM

WASHBORING FROM __ FEET

RF DRILLER DBR LOGGER

Geoprobe 7822DT DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 60 %

REMARKS: Boring located in proposed parking and drive areas.

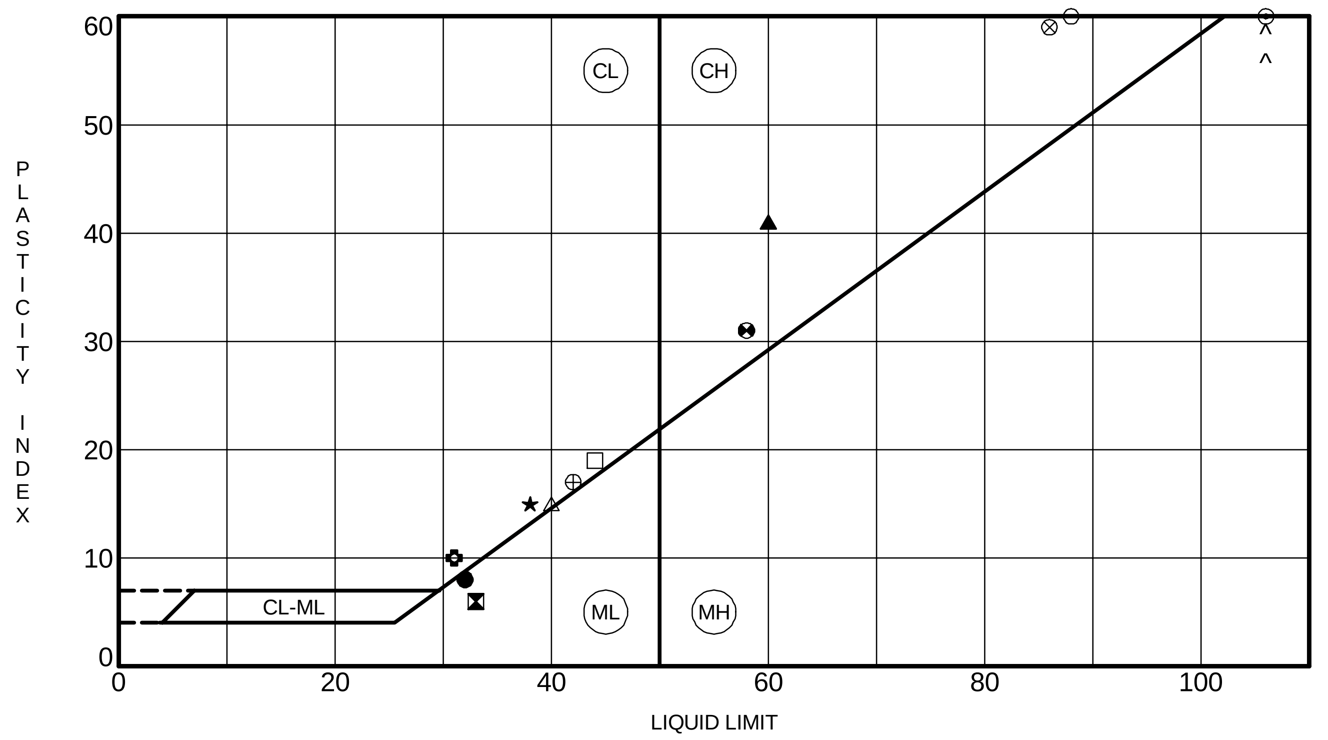
Drawn by: SAS	Checked by: RSP	App'vd. by: RTF
Date: 12/11/25	Date: 1/12/26	Date: 1/15/26

UES™

West Memphis Recreation Center
West Memphis, Arkansas

LOG OF BORING: P-4

Project No. A25138.00362.001

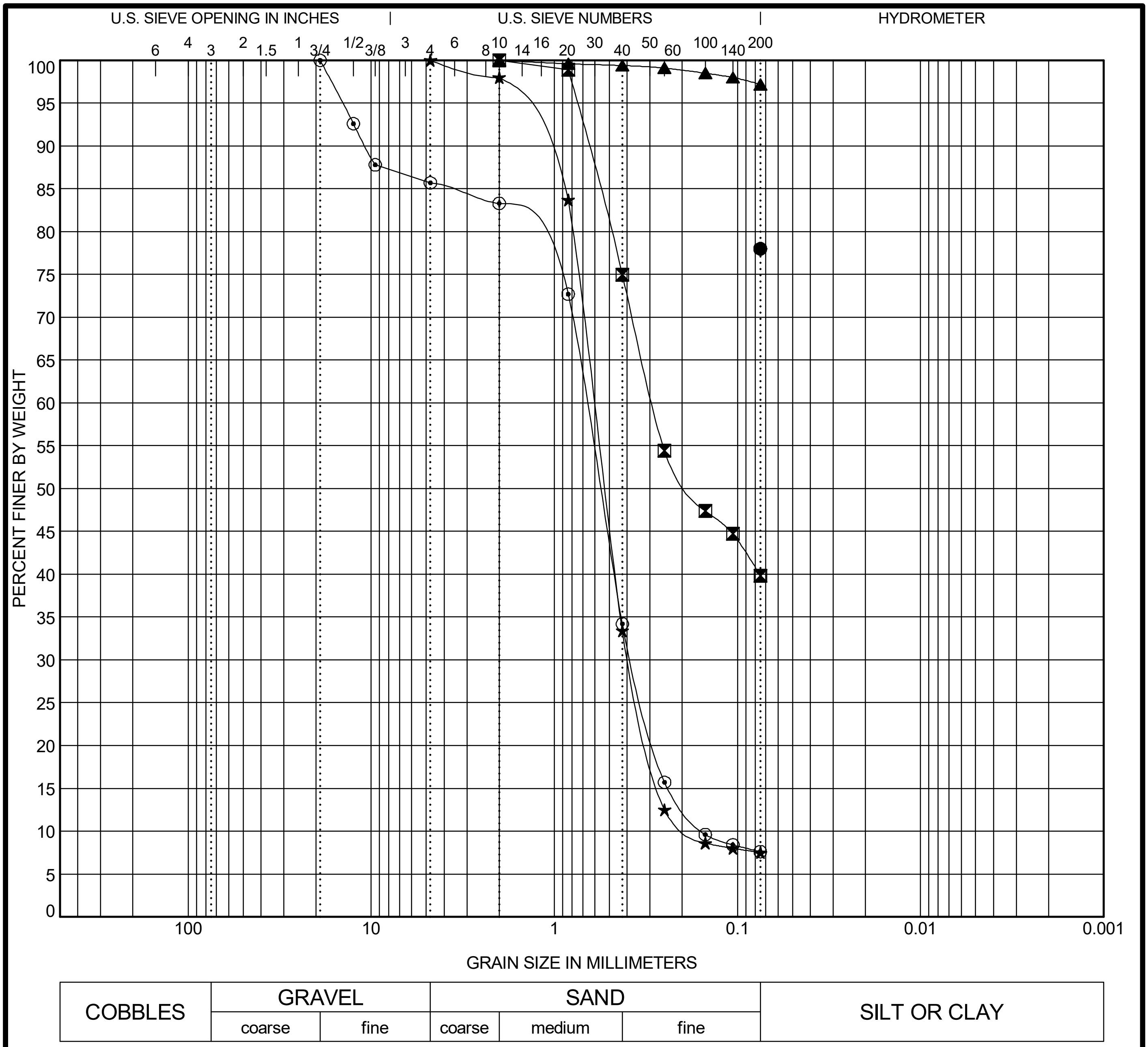


	Specimen Identification		LL	PL	PI	Fines	Classification
●	**B-1**	**8.0**	**32**	**24**	**8**	**78**	**SILT with SAND(ML)**
⧗	**B-1**	**28.5**	**33**	**27**	**6**		**SILT(ML)**
▲	**B-2**	**13.0**	**60**	**19**	**41**		**FAT CLAY(CH)**
★	**B-3**	**10.0**	**38**	**23**	**15**		**LEAN CLAY(CL)**
⊙	**B-3**	**28.5**	**106**	**45**	**61**		**FAT CLAY(CL)**
✿	**B-4**	**8.0**	**31**	**21**	**10**		**LEAN CLAY(CL)**
○	**B-4**	**15.0**	**88**	**28**	**60**	**97**	**FAT CLAY(CH)**
△	**B-4**	**33.5**	**40**	**25**	**15**		**LEAN CLAY(CL)**
⊗	**B-5**	**6.0**	**86**	**27**	**59**		**FAT CLAY(CH)**
⊕	**B-5**	**18.5**	**42**	**25**	**17**		**LEAN CLAY(CL)**
□	**B-6**	**8.5**	**44**	**25**	**19**		**LEAN CLAY(CL)**
⊛	**B-7**	**6.0**	**58**	**27**	**31**		**FAT CLAY(CH)**

UES™

ATTERBERG LIMITS RESULTS

West Memphis Recreation Center
West Memphis, Arkansas
A25138.00362.001



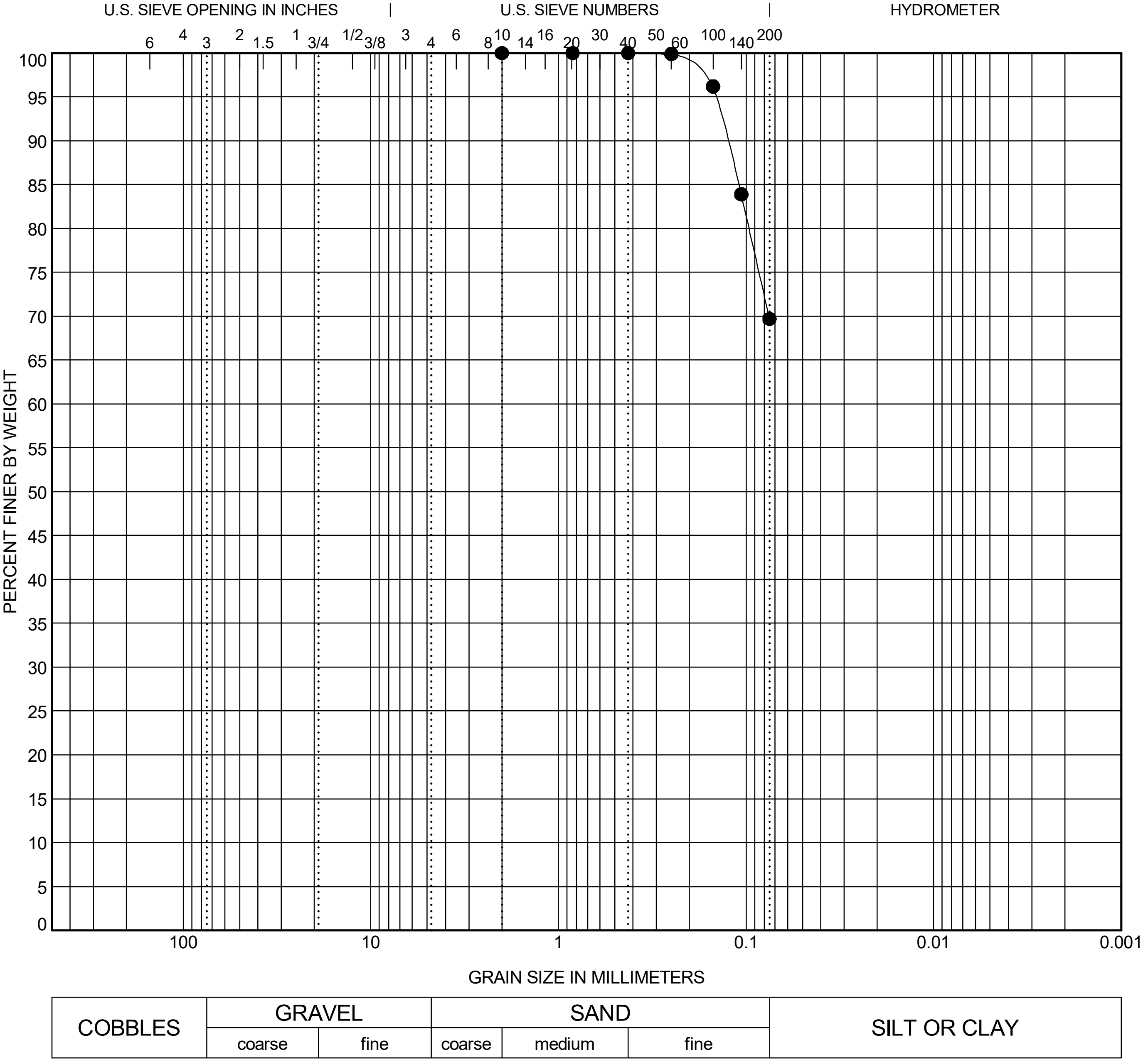
	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	**B-1**	**8.0**	**SILT with SAND(ML)**	**32**	**24**	**8**		
☒	**B-2**	**38.5**	**SILTY SAND(SM)**					
▲	**B-4**	**15.0**	**FAT CLAY(CH)**	**88**	**28**	**60**		
★	**B-4**	**43.5**	**SAND with SILT(SP-SM)**				**1.38**	**3.38**
⊙	**B-4**	**68.5**	**SAND with CLAY(SP-SC)**				**1.36**	**4.33**

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-1**	**8.0**	**0.075**				**0.0**	**0.0**	**78.0**	
☒	**B-2**	**38.5**	**2**	**0.289**			**0.0**	**60.2**	**39.8**	
▲	**B-4**	**15.0**	**2**				**0.0**	**2.8**	**97.2**	
★	**B-4**	**43.5**	**4.75**	**0.609**	**0.39**	**0.18**	**0.0**	**92.5**	**7.5**	
⊙	**B-4**	**68.5**	**19**	**0.671**	**0.377**	**0.155**	**14.3**	**78.1**	**7.6**	

UES™

GRAIN SIZE DISTRIBUTION

West Memphis Recreation Center
West Memphis, Arkansas
A25138.00362.001



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	B-5	28.5	SANDY SILT(ML)					

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-5	28.5	2				0.0	30.3	69.7	

UES™

GRAIN SIZE DISTRIBUTION

West Memphis Recreation Center
West Memphis, Arkansas
A25138.00362.001

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Project Luong
DATE: 1/6/2026
PROJECT NO.: A25138.00362.000
PROJECT: WM Entertainment District Rec Center
LOCATION: West Memphis,AR

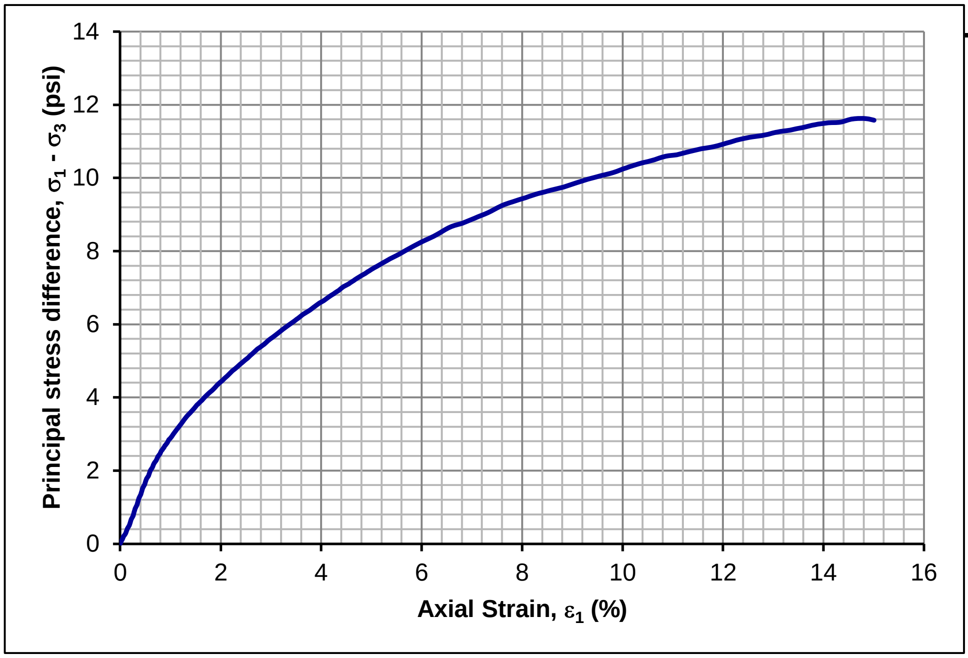
BORING NO.: B-1
SAMPLE NO.: ST-1
DEPTH (ft.): 8.0-10.0
SAMPLE OBTAINED BY: Shelby Tube
CONDITION: Undisturbed
SAMPLE DESCRIPTION: Medium stiff, brown SILT, some sand - (ML)

LIQUID LIMIT (%): 32
PLASTIC LIMIT (%): 24
PLASTICITY INDEX (%): 8
USCS: ML

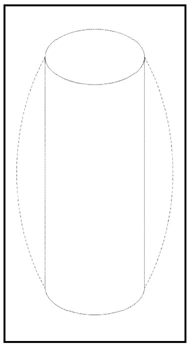
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)
LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.82
HEIGHT (in.):	5.82
HEIGHT TO DIAMETER RATIO:	2.07
WET UNIT WEIGHT (pcf):	123.3
DRY UNIT WEIGHT (pcf):	93.8
VOID RATIO:	0.83
MOISTURE CONTENT (%)*:	31.5
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):	**11.6**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**5.3**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**16.9**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**1,670**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**835**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**1,475**



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 : Project Luong
PROJECT NO.: A25138.00362.000
PROJECT: WM Entertainment District Rec Center
LOCATION: West Memphis,AR

DATE: 1/7/2026

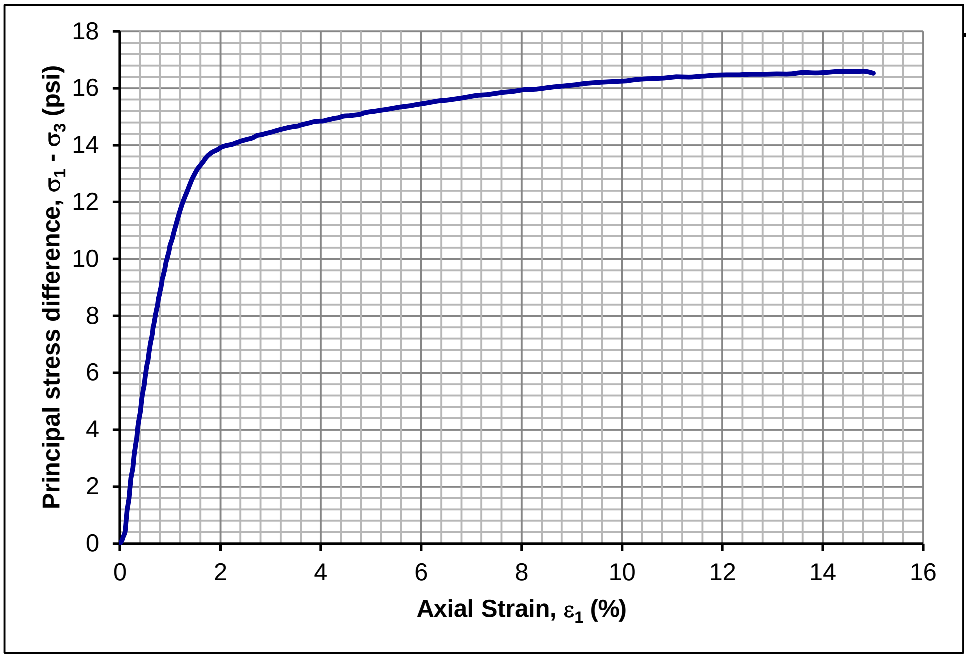
BORING NO.: B-2
SAMPLE NO.: ST-5
DEPTH (ft.): 13.0-15.0
SAMPLE OBTAINED BY: Shelby Tube
CONDITION: Undisturbed
SAMPLE DESCRIPTION: Stiff, gray, FAT CLAY - (CH)

LIQUID LIMIT (%): 60
PLASTIC LIMIT (%): 19
PLASTICITY INDEX (%): 41
USCS: CH

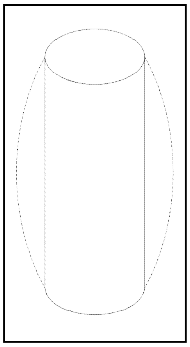
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)
LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.85
HEIGHT (in.):	5.76
HEIGHT TO DIAMETER RATIO:	2.02
WET UNIT WEIGHT (pcf):	118.8
DRY UNIT WEIGHT (pcf):	88.3
VOID RATIO:	0.94
MOISTURE CONTENT (%)*:	34.6
DEGREE OF SATURATION (%):	100.0

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**32.5**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**14.4**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**16.6**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**8.2**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**24.8**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**2,390**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**1,195**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**2,340**



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 : Project Luong
PROJECT NO.: A25138.00362.000
PROJECT: WM Entertainment District Rec Center
LOCATION: West Memphis,AR

DATE: 1/8/2026

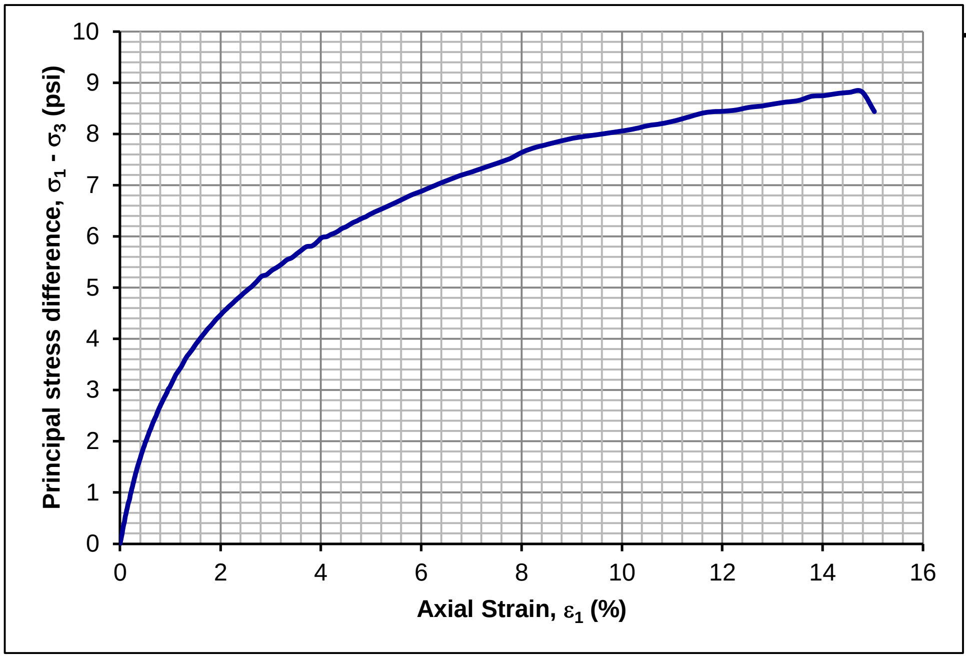
BORING NO.: B-3
SAMPLE NO.: ST-5
DEPTH (ft.): 10.0-12.0
SAMPLE OBTAINED BY: Shelby Tube
CONDITION: Undisturbed
SAMPLE DESCRIPTION: Medium stiff, brown, silty, LEAN CLAY - (CL)

LIQUID LIMIT (%): 38
PLASTIC LIMIT (%): 23
PLASTICITY INDEX (%): 15
USCS: CL

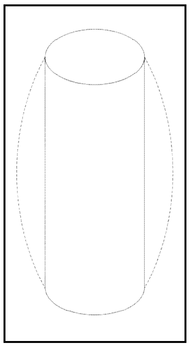
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)
LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.82
HEIGHT (in.):	5.84
HEIGHT TO DIAMETER RATIO:	2.07
WET UNIT WEIGHT (pcf):	120.2
DRY UNIT WEIGHT (pcf):	90.0
VOID RATIO:	0.91
MOISTURE CONTENT (%)*:	33.6
DEGREE OF SATURATION (%):	100.0

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**34.3**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**14.8**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, σ_1 - σ_3 (psi):	**8.8**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**6.4**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**15.2**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**1,270**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**635**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**1,160**



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 : Project Luong
DATE: 1/6/2026
PROJECT NO.: A25138.00362.000
PROJECT: WM Entertainment District Rec Center
LOCATION: West Memphis,AR

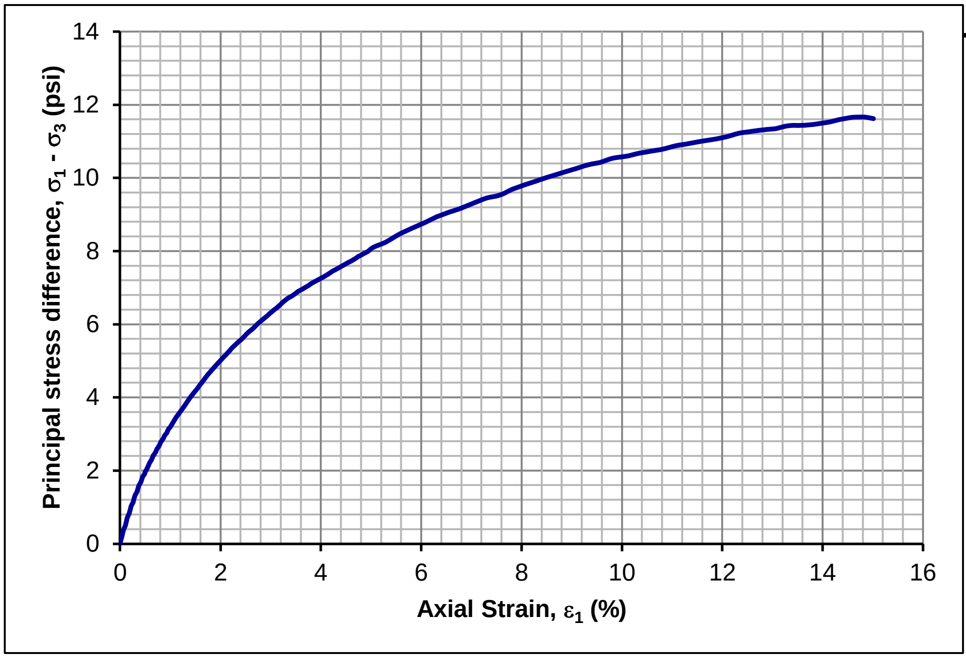
BORING NO.: B-4
SAMPLE NO.: ST-4
DEPTH (ft.): 8.0-10.0
SAMPLE OBTAINED BY: Shelby Tube
CONDITION: Undisturbed
SAMPLE DESCRIPTION: Medium stiff, brown, silty, LEAN CLAY - (CL)

LIQUID LIMIT (%): 31
PLASTIC LIMIT (%): 21
PLASTICITY INDEX (%): 10
USCS: CL

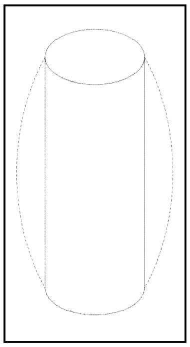
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)
LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.84
HEIGHT (in.):	5.87
HEIGHT TO DIAMETER RATIO:	2.06
WET UNIT WEIGHT (pcf):	123.1
DRY UNIT WEIGHT (pcf):	97.1
VOID RATIO:	0.77
MOISTURE CONTENT (%)*:	26.8
DEGREE OF SATURATION (%):	96.2

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**28.1**
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):	**11.7**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**5.3**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**16.9**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**1,680**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**840**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**1,525**



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 : Project Luong
DATE: 1/6/2026
PROJECT NO.: A25138.00362.000
PROJECT: WM Entertainment District Rec Center
LOCATION: West Memphis,AR

BORING NO.: B-4
SAMPLE NO.: ST-6
DEPTH (ft.): 15.0-17.0
SAMPLE OBTAINED BY: Shelby Tube
CONDITION: Undisturbed
SAMPLE DESCRIPTION: Medium stiff, gray, FAT CLAY - (CH)

LIQUID LIMIT (%): 88
PLASTIC LIMIT (%): 28
PLASTICITY INDEX (%): 60
USCS: CH

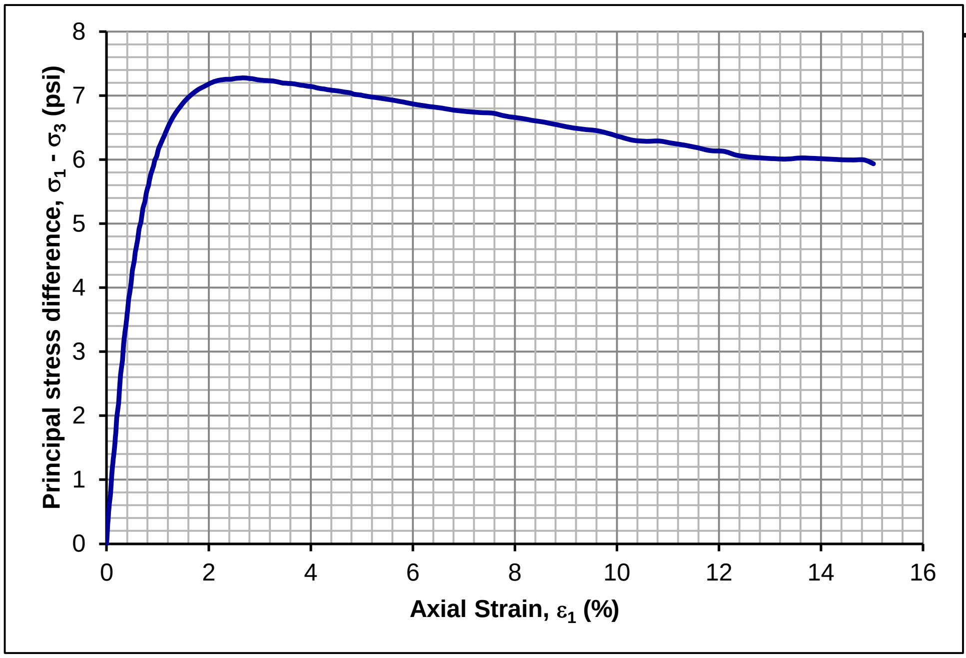
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)
LOAD CELL NO.:

INITIAL SAMPLE DATA

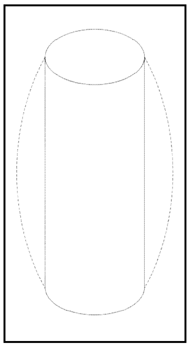
AVERAGE DIAMETER (in.):	2.84
HEIGHT (in.):	5.87
HEIGHT TO DIAMETER RATIO:	2.07
WET UNIT WEIGHT (pcf):	102.9
DRY UNIT WEIGHT (pcf):	64.9
VOID RATIO:	1.64
MOISTURE CONTENT (%)*:	58.6
DEGREE OF SATURATION (%):	98.0

FAILURE DATA***

MOISTURE CONTENT AFTER FAILURE (%)**:	**60.1**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**2.6**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**7.3**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**9.3**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**16.6**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**1,050**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**525**
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 : Project Luong
DATE: 1/10/2026
PROJECT NO.: A25138.00362.001
PROJECT: WM Entertainment District Rec Center
LOCATION: West Memphis,AR

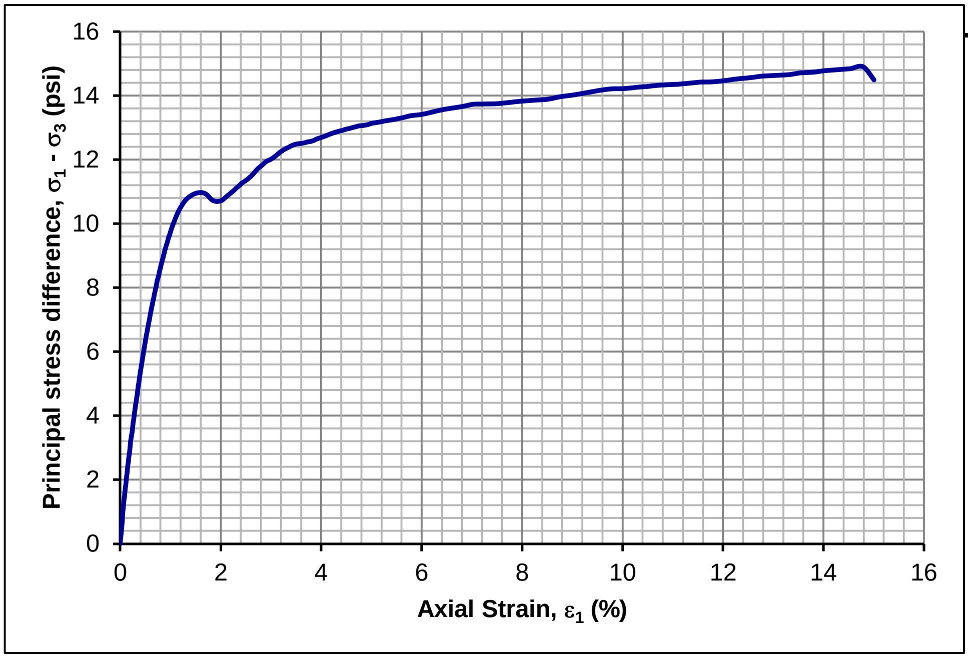
BORING NO.: B-5
SAMPLE NO.: ST-3
DEPTH (ft.): 6.0-8.0
SAMPLE OBTAINED BY: Shelby Tube
CONDITION: Undisturbed
SAMPLE DESCRIPTION: Stiff, brown, FAT CLAY - (CH)

LIQUID LIMIT (%): 86
PLASTIC LIMIT (%): 27
PLASTICITY INDEX (%): 59
USCS: CH

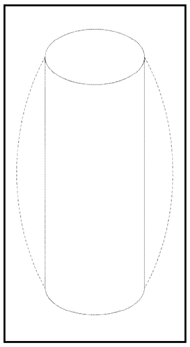
SPECIFIC GRAVITY OF SOLIDS: 2.72 (Assumed)
LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.87
HEIGHT (in.):	5.71
HEIGHT TO DIAMETER RATIO:	1.99
WET UNIT WEIGHT (pcf):	115.8
DRY UNIT WEIGHT (pcf):	87.3
VOID RATIO:	0.94
MOISTURE CONTENT (%)*:	32.6
DEGREE OF SATURATION (%):	93.9

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**33.9**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**14.8**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**14.9**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**3.5**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**18.4**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**2,150**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**1,075**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**2,050**



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.

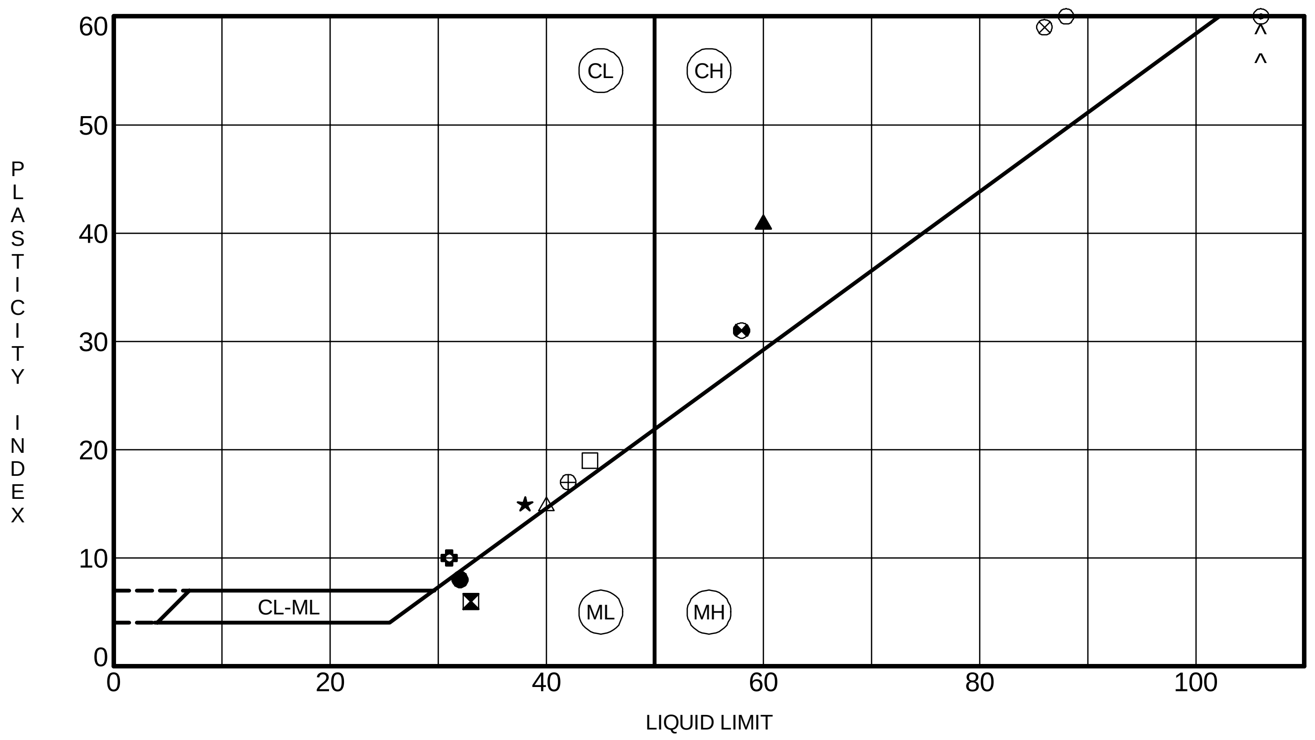
APPENDIX D – LABORATORY TEST DATA

Atterberg Limits Results

Grain Size Distribution

Unconsolidated-Undrained Triaxial Compression

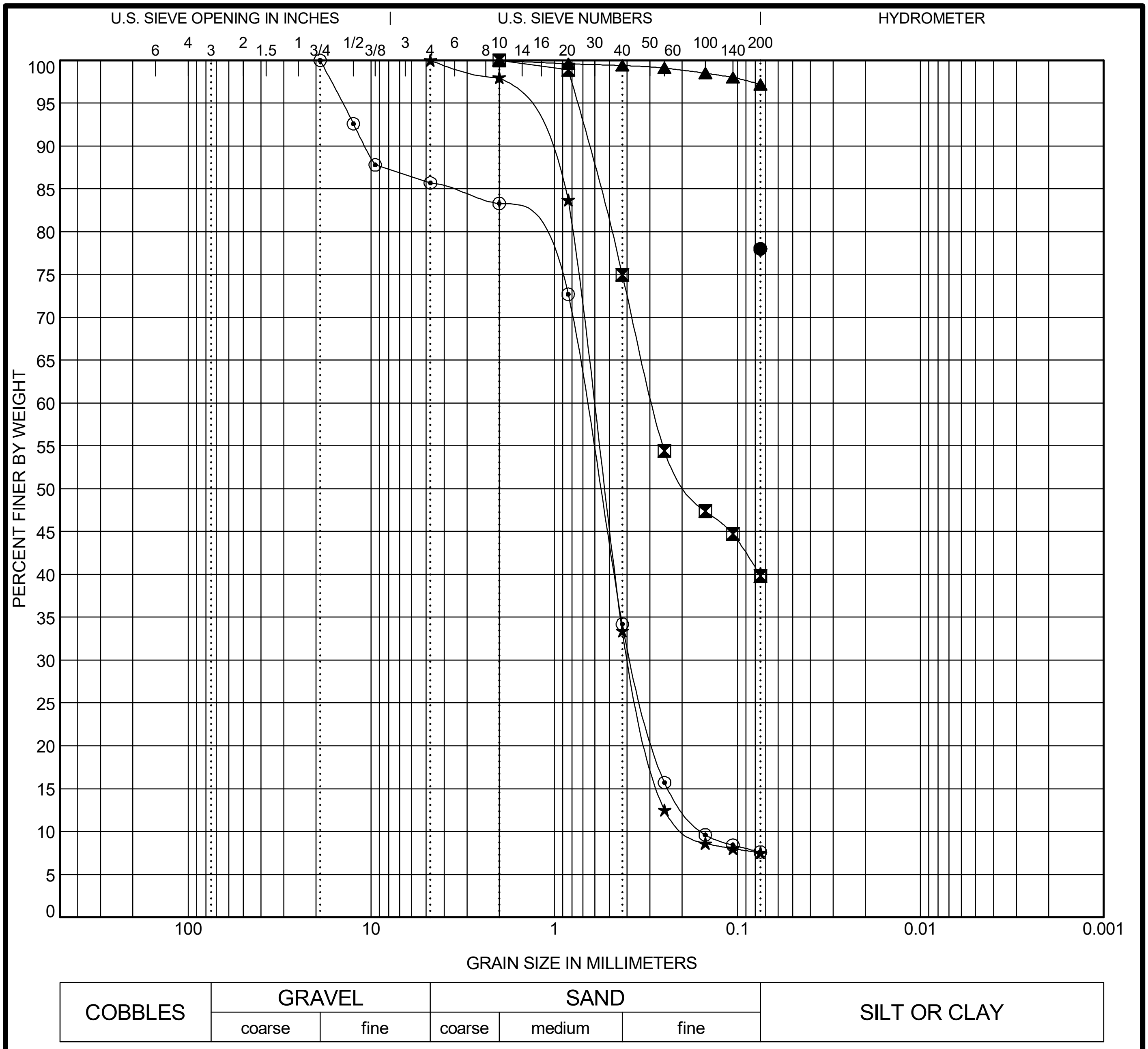
One-Dimensional Consolidation



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**B-1**	**8.0**	**32**	**24**	**8**	**78**	**SILT with SAND(ML)**
⧗	**B-1**	**28.5**	**33**	**27**	**6**		**SILT(ML)**
▲	**B-2**	**13.0**	**60**	**19**	**41**		**FAT CLAY(CH)**
★	**B-3**	**10.0**	**38**	**23**	**15**		**LEAN CLAY(CL)**
⊙	**B-3**	**28.5**	**106**	**45**	**61**		**FAT CLAY(CL)**
✿	**B-4**	**8.0**	**31**	**21**	**10**		**LEAN CLAY(CL)**
○	**B-4**	**15.0**	**88**	**28**	**60**	**97**	**FAT CLAY(CH)**
△	**B-4**	**33.5**	**40**	**25**	**15**		**LEAN CLAY(CL)**
⊗	**B-5**	**6.0**	**86**	**27**	**59**		**FAT CLAY(CH)**
⊕	**B-5**	**18.5**	**42**	**25**	**17**		**LEAN CLAY(CL)**
□	**B-6**	**8.5**	**44**	**25**	**19**		**LEAN CLAY(CL)**
⊛	**B-7**	**6.0**	**58**	**27**	**31**		**FAT CLAY(CH)**

ATTERBERG LIMITS RESULTS

West Memphis Recreation Center
West Memphis, Arkansas
A25138.00362.001



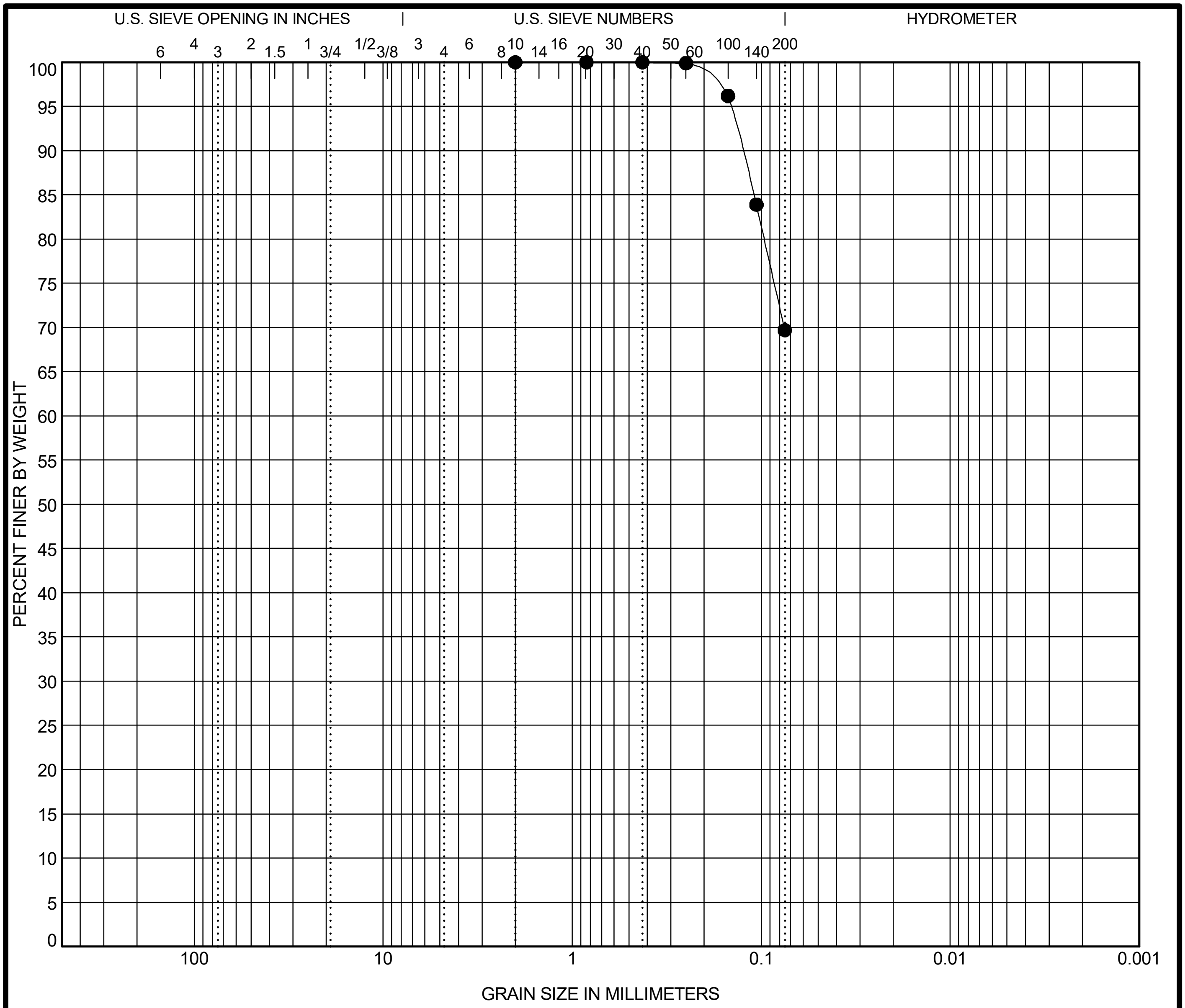
Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**B-1**	**8.0**	**SILT with SAND(ML)**	**32**	**24**	**8**		
☒	**B-2**	**38.5**	**SILTY SAND(SM)**					
▲	**B-4**	**15.0**	**FAT CLAY(CH)**	**88**	**28**	**60**		
★	**B-4**	**43.5**	**SAND with SILT(SP-SM)**				**1.38**	**3.38**
⊙	**B-4**	**68.5**	**SAND with CLAY(SP-SC)**				**1.36**	**4.33**

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-1**	**8.0**	**0.075**				**0.0**	**0.0**	**78.0**	
☒	**B-2**	**38.5**	**2**	**0.289**			**0.0**	**60.2**	**39.8**	
▲	**B-4**	**15.0**	**2**				**0.0**	**2.8**	**97.2**	
★	**B-4**	**43.5**	**4.75**	**0.609**	**0.39**	**0.18**	**0.0**	**92.5**	**7.5**	
⊙	**B-4**	**68.5**	**19**	**0.671**	**0.377**	**0.155**	**14.3**	**78.1**	**7.6**	

UES™

GRAIN SIZE DISTRIBUTION

West Memphis Recreation Center
West Memphis, Arkansas
A25138.00362.001



<table>
<tr><td rowspan="2">COBBLES</td><td colspan="2">GRAVEL</td><td colspan="3">SAND</td><td rowspan="2">SILT OR CLAY</td></tr>
<tr><td>coarse</td><td>fine</td><td>coarse</td><td>medium</td><td>fine</td></tr>
</table>

Specimen Identification	Classification	LL	PL	PI	Cc	Cu
● **B-5** **28.5**	**SANDY SILT(ML)**					

Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● **B-5** **28.5**	**2**				**0.0**	**30.3**	**69.7**	

UES™

GRAIN SIZE DISTRIBUTION

West Memphis Recreation Center
West Memphis, Arkansas
A25138.00362.001

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
ASTM D2850

CLIENT : Project Luong
PROJECT NO.: A25138.00362.000
PROJECT: WM Entertainment District Rec Center
LOCATION: West Memphis,AR

DATE: 1/6/2026

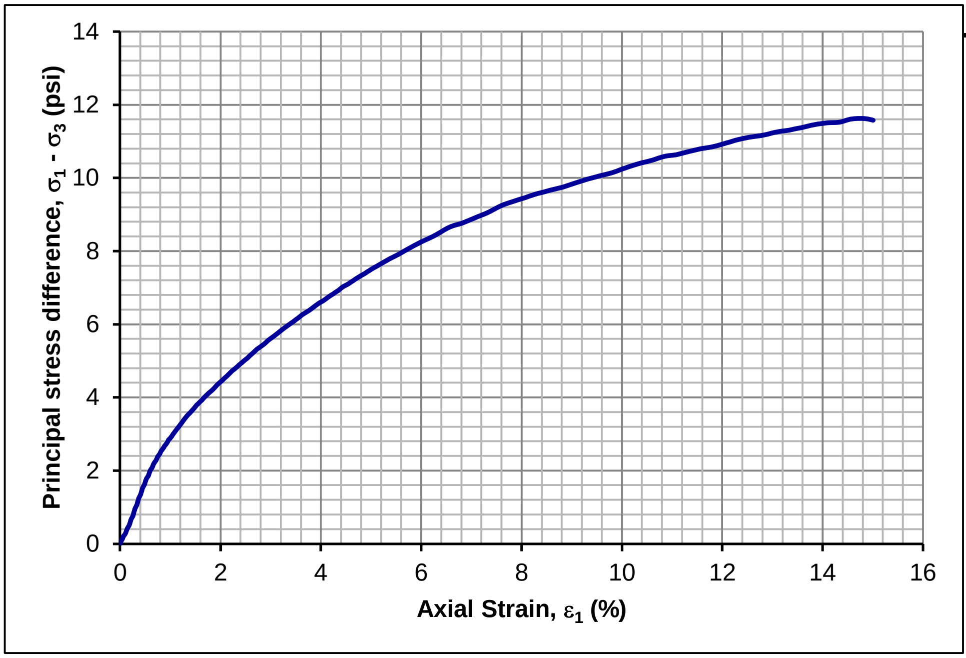
BORING NO.: B-1
SAMPLE NO.: ST-1
DEPTH (ft.): 8.0-10.0
SAMPLE OBTAINED BY: Shelby Tube
CONDITION: Undisturbed
SAMPLE DESCRIPTION: Medium stiff, brown SILT, some sand - (ML)

LIQUID LIMIT (%): 32
PLASTIC LIMIT (%): 24
PLASTICITY INDEX (%): 8
USCS: ML

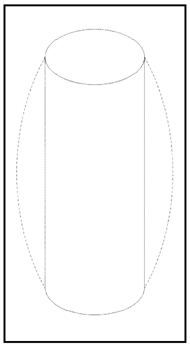
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)
LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.82
HEIGHT (in.):	5.82
HEIGHT TO DIAMETER RATIO:	2.07
WET UNIT WEIGHT (pcf):	123.3
DRY UNIT WEIGHT (pcf):	93.8
VOID RATIO:	0.83
MOISTURE CONTENT (%)*:	31.5
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):	**11.6**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**5.3**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**16.9**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**1,670**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**835**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**1,475**



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 : Project Luong
DATE: 1/7/2026
PROJECT NO.: A25138.00362.000
PROJECT: WM Entertainment District Rec Center
LOCATION: West Memphis,AR

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

LIQUID LIMIT (%): 60
PLASTIC LIMIT (%): 19
PLASTICITY INDEX (%): 41
USCS: CH

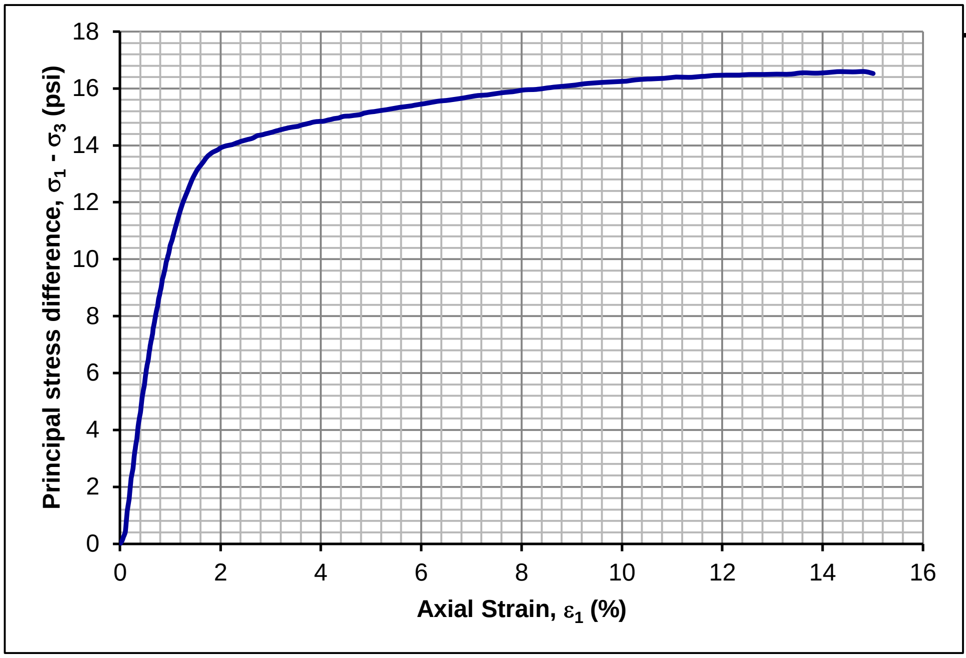
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)
LOAD CELL NO.:

INITIAL SAMPLE DATA

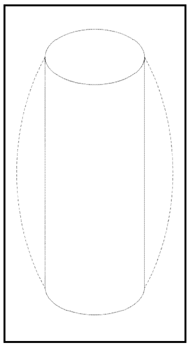
AVERAGE DIAMETER (in.):	2.85
HEIGHT (in.):	5.76
HEIGHT TO DIAMETER RATIO:	2.02
WET UNIT WEIGHT (pcf):	118.8
DRY UNIT WEIGHT (pcf):	88.3
VOID RATIO:	0.94
MOISTURE CONTENT (%)*:	34.6
DEGREE OF SATURATION (%):	100.0

FAILURE DATA***

MOISTURE CONTENT AFTER FAILURE (%)**:	**32.5**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**14.4**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**16.6**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**8.2**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**24.8**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**2,390**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**1,195**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**2,340**



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 : Project Luong
PROJECT NO.: A25138.00362.000
PROJECT: WM Entertainment District Rec Center
LOCATION: West Memphis,AR

DATE: 1/8/2026

BORING NO.: B-3
SAMPLE NO.: ST-5
DEPTH (ft.): 10.0-12.0
SAMPLE OBTAINED BY: Shelby Tube
CONDITION: Undisturbed
SAMPLE DESCRIPTION: Medium stiff, brown, silty, LEAN CLAY - (CL)

LIQUID LIMIT (%): 38
PLASTIC LIMIT (%): 23
PLASTICITY INDEX (%): 15
USCS: CL

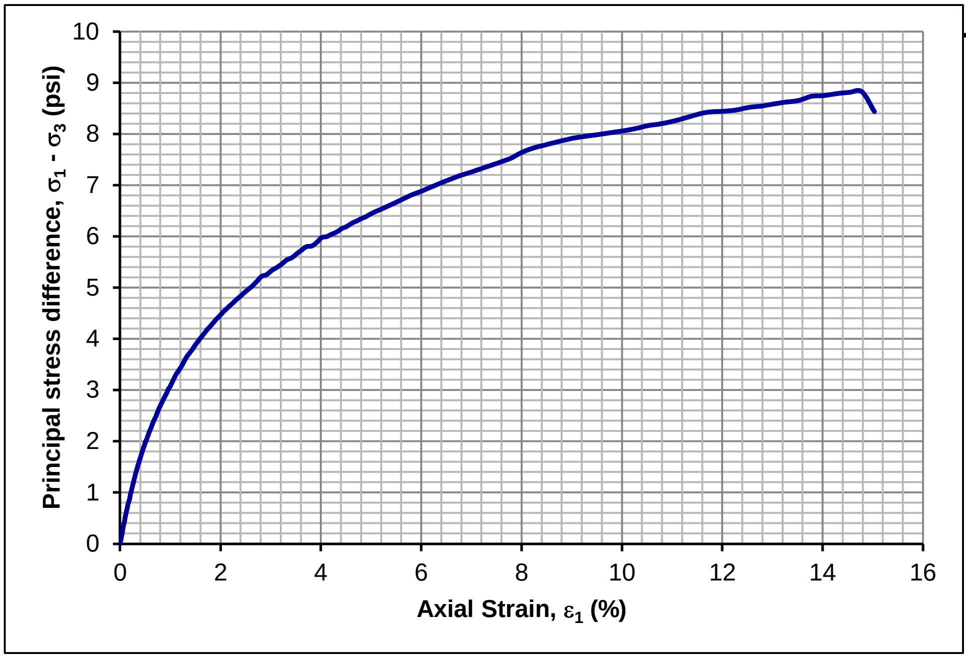
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)
LOAD CELL NO.:

INITIAL SAMPLE DATA

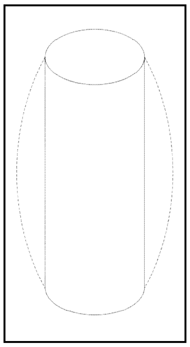
AVERAGE DIAMETER (in.):	2.82
HEIGHT (in.):	5.84
HEIGHT TO DIAMETER RATIO:	2.07
WET UNIT WEIGHT (pcf):	120.2
DRY UNIT WEIGHT (pcf):	90.0
VOID RATIO:	0.91
MOISTURE CONTENT (%)*:	33.6
DEGREE OF SATURATION (%):	100.0

FAILURE DATA***

MOISTURE CONTENT AFTER FAILURE (%)**:	**34.3**
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):	**8.8**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**6.4**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**15.2**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**1,270**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**635**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**1,160**



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 : Project Luong
DATE: 1/6/2026
PROJECT NO.: A25138.00362.000
PROJECT: WM Entertainment District Rec Center
LOCATION: West Memphis,AR

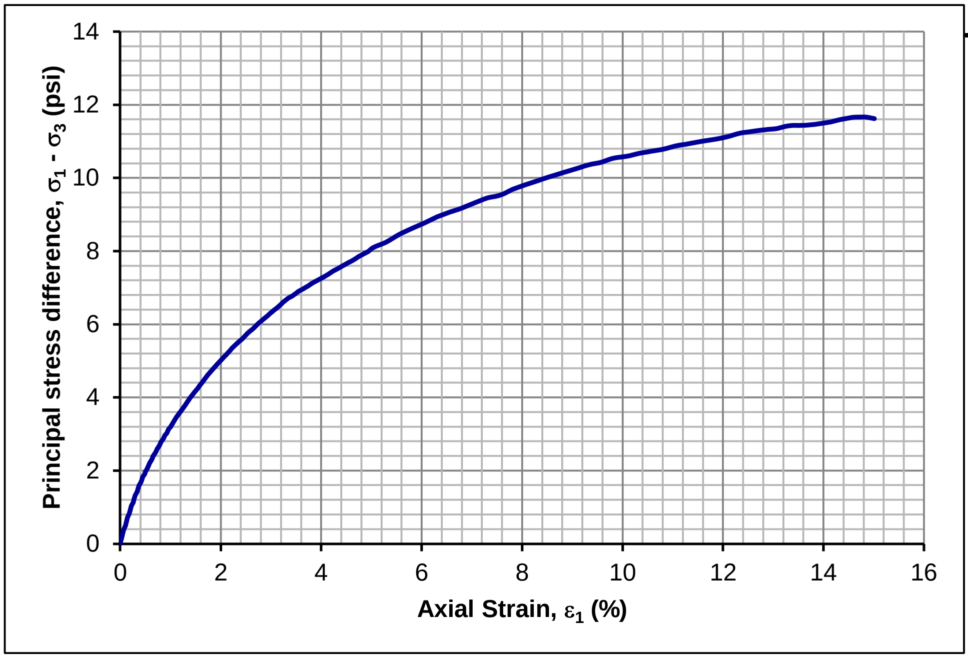
BORING NO.: B-4
SAMPLE NO.: ST-4
DEPTH (ft.): 8.0-10.0
SAMPLE OBTAINED BY: Shelby Tube
CONDITION: Undisturbed
SAMPLE DESCRIPTION: Medium stiff, brown, silty, LEAN CLAY - (CL)

LIQUID LIMIT (%): 31
PLASTIC LIMIT (%): 21
PLASTICITY INDEX (%): 10
USCS: CL

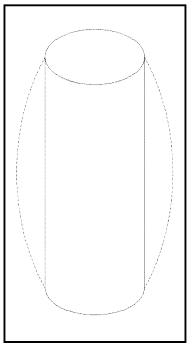
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)
LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.84
HEIGHT (in.):	5.87
HEIGHT TO DIAMETER RATIO:	2.06
WET UNIT WEIGHT (pcf):	123.1
DRY UNIT WEIGHT (pcf):	97.1
VOID RATIO:	0.77
MOISTURE CONTENT (%)*:	26.8
DEGREE OF SATURATION (%):	96.2

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**28.1**
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):	**11.7**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**5.3**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**16.9**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**1,680**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**840**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**1,525**



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 : Project Luong
PROJECT NO.: A25138.00362.000
PROJECT: WM Entertainment District Rec Center
LOCATION: West Memphis,AR

DATE: 1/6/2026

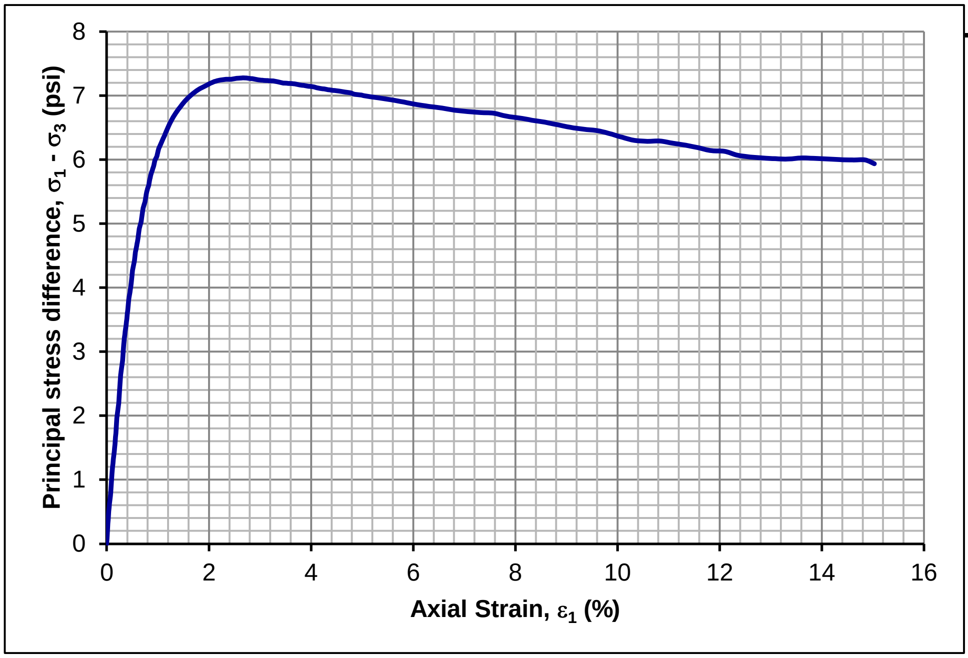
BORING NO.: B-4
SAMPLE NO.: ST-6
DEPTH (ft.): 15.0-17.0
SAMPLE OBTAINED BY: Shelby Tube
CONDITION: Undisturbed
SAMPLE DESCRIPTION: Medium stiff, gray, FAT CLAY - (CH)

LIQUID LIMIT (%): 88
PLASTIC LIMIT (%): 28
PLASTICITY INDEX (%): 60
USCS: CH

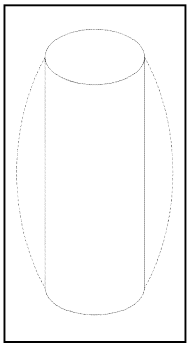
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed)
LOAD CELL NO.:

INITIAL SAMPLE DATA	
AVERAGE DIAMETER (in.):	2.84
HEIGHT (in.):	5.87
HEIGHT TO DIAMETER RATIO:	2.07
WET UNIT WEIGHT (pcf):	102.9
DRY UNIT WEIGHT (pcf):	64.9
VOID RATIO:	1.64
MOISTURE CONTENT (%)*:	58.6
DEGREE OF SATURATION (%):	98.0

FAILURE DATA***	
MOISTURE CONTENT AFTER FAILURE (%)**:	**60.1**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**2.6**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**7.3**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**9.3**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**16.6**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**1,050**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**525**
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 : Project Luong
DATE: 1/10/2026
PROJECT NO.: A25138.00362.001
PROJECT: WM Entertainment District Rec Center
LOCATION: West Memphis,AR

BORING NO.: B-5
SAMPLE NO.: ST-3
DEPTH (ft.): 6.0-8.0
SAMPLE OBTAINED BY: Shelby Tube
CONDITION: Undisturbed
SAMPLE DESCRIPTION: Stiff, brown, FAT CLAY - (CH)

LIQUID LIMIT (%): 86
PLASTIC LIMIT (%): 27
PLASTICITY INDEX (%): 59
USCS: CH

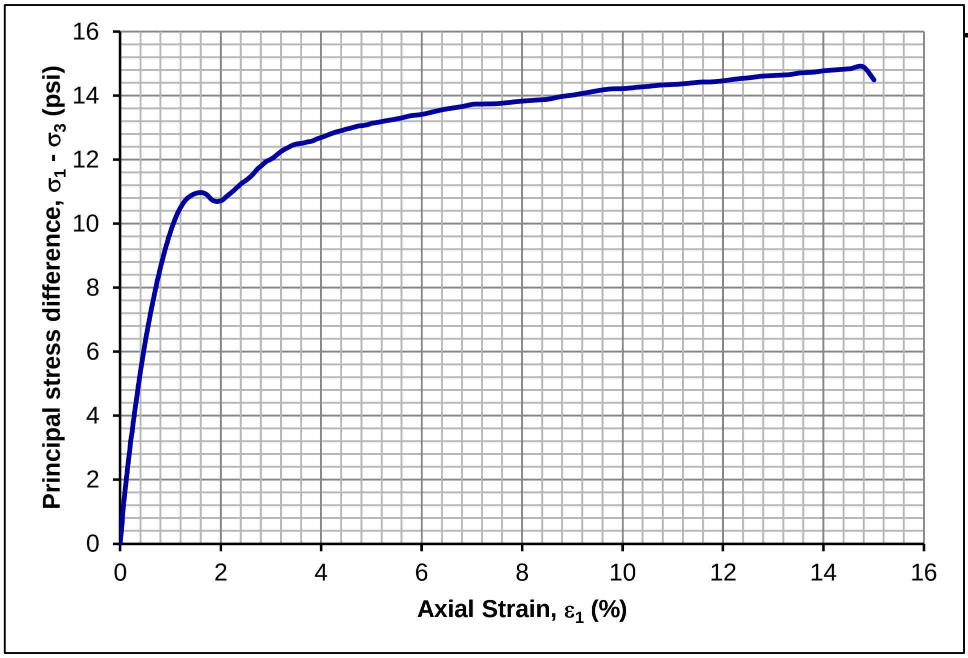
SPECIFIC GRAVITY OF SOLIDS: 2.72 (Assumed)
LOAD CELL NO.:

INITIAL SAMPLE DATA

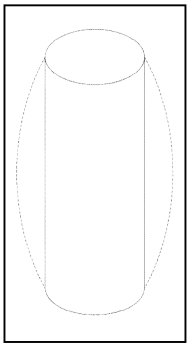
AVERAGE DIAMETER (in.):	2.87
HEIGHT (in.):	5.71
HEIGHT TO DIAMETER RATIO:	1.99
WET UNIT WEIGHT (pcf):	115.8
DRY UNIT WEIGHT (pcf):	87.3
VOID RATIO:	0.94
MOISTURE CONTENT (%)*:	32.6
DEGREE OF SATURATION (%):	93.9

FAILURE DATA***

MOISTURE CONTENT AFTER FAILURE (%)**:	**33.9**
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.0**
AXIAL STRAIN AT FAILURE (%):	**14.8**
PRINCIPAL STRESS DIFFERENCE AT FAILURE, $\sigma_1 - \sigma_3$ (psi):	**14.9**
MINOR PRINCIPAL STRESS AT FAILURE, σ_3 (psi):	**3.5**
MAJOR PRINCIPAL STRESS AT FAILURE, σ_1 (psi):	**18.4**
UNDRAINED COMPRESSIVE STRENGTH, U_u (psf):	**2,150**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**1,075**
LIMITING UNDRAINED COMP. STRESS @ 10% STRAIN (psf):	**2,050**



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.