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Geotechnical Report

Report Date:	04/19/2024	**Date Sampled:**	04/19/2024
Project:	240211.1 TO #1 - Community Building - Geotechnical Testing	**Sampled By:**	
Location:	Loyal Shawnee Cultural Center		
Client:	Cherokee Nation		
		Lab No:	2582
Client PO:	883377	**Report No:**	240211.1-1283

TEST RESULTS

1-ec Hinderliter Geotechnical Engineering
Attn: Mark Hinderliter
1-ec CEC Corporation Attn: Brett Cowan
1-ec CEC Corporation Attn: Luke Counts

Respectfully Submitted,
CEC Corporation

LICENSED PROFESSIONAL ENGINEER
BRETT
A. COWAN
21935
OKLAHOMA

Brett Cowan

Brett Cowan, Geotechnical Engineer

04/19/2024

GEOTECHNICAL ENGINEERING REPORT

CN – LOYAL SHAWNEE COMMUNITY CENTER
S 4340 ROAD
WHITE OAK, OKLAHOMA

PROJECT NO. CEC-24-04

HINDERLITER GEOTECHNICAL ENGINEERING

PREPARED FOR:

CEC CORPORATION
OKLAHOMA CITY

DATE:

APRIL 15, 2024

April 15, 2024

CEC Corporation
4555 W Memorial Road
Oklahoma City, OK 73142

Attn: Mr. Brett Cowan, P.E.

Re: Geotechnical Engineering Report
CN-Loyal Shawnee Community Center
White Oak, Oklahoma
HGE Project No. CEC-24-04

Dear Mr. Cowan:

The Geotechnical Engineering Report has been completed for the proposed CN-Loyal Shawnee Community Center near White Oak, Oklahoma. Our services and fee were detailed in HGE Project Scope & Fee Reference: Loyal Shawnee Cultural Center, dated February 21, 2024. Acceptance of the scope and fee and notice to proceed was provided by email correspondence on March 14, 2024.

The purpose of the attached report is to provide a summary of the field investigation and laboratory testing methods used, and provide recommendations for earthwork, and the design and construction of on-grade floor slabs, foundations and pavements. Soil boring results and laboratory test results are provided in the appendices of this report.

Mr. Cowan, please do not hesitate to contact HGE at (405) 942-4090 should you have questions regarding this report.

Respectfully Submitted:
HINDERLITER GEOTECHNICAL ENGINEERING, LLC
Certificate of Authorization No. 5528, Expires 30-June-2025

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P:\HGE\Reports\2024 Geo\April\CEC-24-04 Report

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1.0 EXECUTIVE SUMMARY

The subsurface exploration and geotechnical engineering report are complete for the proposed Cherokee Nation Loyal Shawnee Community Center near White Oak, Oklahoma. We understand a single-story building with a footprint area of 4,000 square feet will be constructed. Associated parking and drive areas will be included in the development. Construction type and foundation loads were unknown at the time of this report; we assume continuous wall loads will be less than 2 kips per linear foot and isolated column loads, if any, will be less than 20 kips. Pavement may be Portland cement concrete, asphaltic cement concrete or a combination of both. Less than 2 feet of cut or fill is expected to be required to balance the earthwork across the site.

Two soil test borings were advanced to depths of 19 feet within the proposed building footprint and two borings were advanced to depths of 5 feet within the parking and drive areas. HGE personnel located the borings in the field based on the documents provided to us by the Client. The approximate location of each boring is displayed on the boring location diagram and is recorded on the boring logs, both attached within Appendix A of this report.

The borings encountered surficial vegetation and topsoil that was underlain by lean clay, lean to fat clay or fat clay soils. The clays were generally stiff to very stiff and brown to olive in color. Borings B-3 and B-4 were terminated within these soils at depths of 5 feet. Within borings B-1 and B-2, the subsurface soils transitioned to brown, soft to moderately hard, poorly cemented, weathered sandstone at respective depths of 9-1/2 feet and 14 feet. Soft to moderately hard, gray, weathered shale was encountered within the borings at depths of approximately 16 feet. These borings were terminated within weathered shale. Subsurface geology appears best described as belonging to the Bluejacket Unit (Pbj).

The borings were monitored for the presence of groundwater while drilling and immediately after boring completion. Groundwater was not encountered within the borings at these times. The borings were backfilled or plugged per OWRB requirements immediately after completion.

Shallow footing foundations can be used to support the proposed building. Drilled pier and grade beam foundations may also be used. On-grade floor slabs can be used with either foundation system but will require over-excavation of the existing subgrade soils and replacement with low volume change material. Crushed aggregate base materials could be used for pavement support. Subgrade stabilization using a calcium-based admixture, such as, hydrated lime is also acceptable. Specific geotechnical recommendations concerning earthwork and the design and construction of on-grade floor slabs, foundations and pavements are presented subsequently in this report.

2.0 PROJECT DESCRIPTION

The existing Cherokee Nation Loyal Shawnee Cultural Center is located on the west side of S4340 Road approximately 0.2 miles south of US-60 near White Oak, Oklahoma. The Community Center will be located north of the existing facility. We understand a single-story building with a footprint area of 4,000 square feet will be constructed. Associated parking and drive areas will be included in the development. Construction type and foundation loads were unknown at the time of this report; we assume continuous wall loads will be less than 2 kips per linear foot and isolated column loads, if any, will be less than 20 kips. Pavement may be Portland cement concrete, asphaltic cement concrete or a combination of both. Less than 2 feet of cut or fill is expected to be required to balance the earthwork across the site.

3.0 SITE EXPLORATION

3.1 Boring Layout & Elevations

Two soil test borings were advanced to depths of 19 feet within the proposed building footprint and two borings were advanced to depths of 5 feet within the parking and drive areas. HGE personnel located the borings in the field based on the documents provided to us by the Client. The approximate location of each boring is displayed on the boring location diagram and is recorded on the boring logs, both attached within Appendix A of this report.

Elevations at the boring locations were determined using a common surveyor's level and grade rod. The finish floor elevation of the existing building, located approximately as displayed on the boring location diagram, was used as an elevation benchmark. The benchmark assigned a relative elevation of 100 feet. Based on this benchmark, elevations at the boring locations ranged from 101.2 feet to 101.9 feet.

Boring locations and elevations should be considered only roughly accurate and not survey quality. Borings are often offset in the field by drill operators to locations accessible to the drill rig or to avoid utilities. Significant offsets are typically noted on the boring location diagram.

3.2 Subsurface Investigation

A truck-mounted, CME-45 rotary drill rig outfitted with 6-inch solid augers was used to advance the boreholes. Representative soil samples were obtained using either the split-barrel or thin-walled tube sampling procedure, generally as detailed in ASTM D 1586 and ASTM D 1587, respectively. ASTM D 1586 is commonly referred to as the Standard Penetration Test (SPT).

The split-barrel sampling process requires a two-piece sampling tube be used to obtain soil samples. A two-inch outside diameter sampling tube is hammered into the bottom of the boring with a 140-pound weight falling 30 inches. The number of blows required to advance the sampling tube the last 12 inches, or less, of an 18-inch sampling interval is recorded as the SPT resistance value, N. The in-situ relative density of granular soils, consistency of cohesive soils, and the hardness of weathered bedrock can be estimated from the N value. The N values recorded for each test are displayed on the attached boring logs adjacent to their relative sampling depths.

An automatic drive hammer was used to advance the split-barrel sampler. A greater mechanical efficiency is achieved using an automatic drive hammer when compared to a conventional safety drive hammer operated with a cathead and rope. The effect of this higher efficiency on the N values has been considered in our interpretation and analysis of the subsurface information provided with this report.

In the thin-walled tube sampling procedure, a seamless steel tube with a sharpened cutting end is pushed into the bottom of the borehole using the hydraulic drill head. The sampler is carefully removed from the borehole with a cohesive soil sample retained inside. The tube is capped to retain moisture, marked for identification and returned to the laboratory for further evaluation.

The drill crew prepared field boring logs as part of the subsurface exploration operations. The split-barrel samples were packaged in plastic bags to reduce moisture loss, labeled for identification and transported to our laboratory for further evaluation. Appendix A of this report contains the final boring logs that represent modifications based on the engineer's observations.

The borings were backfilled or plugged per OWRB requirements after the drilling operations were completed. Groundwater level measurements are included in Section 5.3 of this report.

4.0 LABORATORY EVALUATION

As part of the geotechnical investigation, soil samples obtained from the borings were evaluated for moisture content. Selected samples were evaluated for liquid and plastic limits, and grain size. These test results provide the information required to classify the soils and help to determine their engineering properties. One bulk sample was evaluated for moisture-density relationships and CBR strength. The engineer reviewed all soil descriptions and made modifications based on the materials plasticity, texture, and color along with the laboratory test results.

The laboratory test results and group symbol from the Unified Soil Classification System are displayed on the boring logs included in Appendix A of this report and on the laboratory report sheets located in Appendix B. The following sections provide brief information about some of the tests performed.

4.1 In-Situ Moisture Content

The in-situ moisture content of soil samples was determined in the laboratory in general accordance with specification ASTM D 2216. The results of these tests have been provided in the appropriate column of the boring log. The moisture content is expressed as a percentage and is the ratio of the weight of water in a given amount of soil to the weight of solid particles.

4.2 Liquid & Plastic Limits

The Liquid Limit (LL) and Plastic Limit (PL) of selected soil samples were determined in the laboratory in general accordance with ASTM D 4318. The Liquid Limit (LL) of a soil is the water content at which the soil passes from a liquid state to a plastic state. The Plastic Limit (PL) of a soil is the water content at which the soil passes from a plastic state to a semi-solid state. The Plasticity Index (PI) is the difference between the Liquid Limit and the Plastic Limit (PI = LL – PL). There is a correlation between these limits and experimental shrink / swell data. The results of these tests have been provided in the appropriate column of the boring log.

4.3 Sieve Analysis Test

The amount of material passing the No. 4, No. 10, No. 40 and No. 200 U.S. Standard Sieves was determined in the laboratory in general accordance with ASTM D 6913. Determination of the material grading, combined with the LL, PL and PI provide the information needed to classify the soil according to the Unified Soil Classification System (USCS). The resultant percentage of material passing each sieve has been provided in the appropriate column of the boring log.

4.4 Unconfined Compressive Strength

The in-situ density of cohesive soils sampled using thin-walled tubes was estimated and the unconfined compressive strength of representative specimens was determined per ASTM D 2166. The results of these tests are reported on the boring logs located within Appendix A of this report. Once extruded, the sample was trimmed, measured and weighed. The trimmed specimen was subjected to an axial loading at a constant rate of strain until failure or 15 percent strain occurred. The maximum load at failure or at 15 percent strain is reported herein as the unconfined compressive strength.

4.5 Moisture Density Relationships

The moisture-density relationship of bulk composite soil samples obtained from the borings was determined in general accordance with ASTM D 698 (commonly referred to as the standard-effort Proctor test). Results of this test are included in Appendix B of this report. The maximum dry density and optimum moisture content of the sampled materials are determined from this test. These values are used to determine the target molding properties of CBR specimens. The maximum dry density and optimum moisture are determined by constructing a curve from a plot of density vs. moisture.

4.6 Bearing Ratio Strength

The Bearing Ratio (CBR) strength was determined in general accordance with ASTM D 1883. Once the standard-effort maximum dry density and optimum moisture content are determined, specimens are compacted within CBR molds at varying degrees of compactive effort. A surcharge weight equivalent to the estimated weight of pavement and base is placed on the sample and the entire assembly is immersed in water for four days. While soaking, the swell of each specimen is measured. At the completion of the soaking period, the samples are removed from the water and allowed to drain. The sample, with the surcharge imposed, is subjected to penetration by a 1.95-inch diameter piston moving at a speed of 0.05 in/min. A plot of the load versus penetration curve is constructed. The unit load corresponding to 0.1-inch penetration is recorded as the CBR value. The design CBR value is the value corresponding to 95 percent compaction of the specimen's maximum dry density.

5.0 FINDINGS & RECOMMENDATIONS

Shallow footing foundations can be used to support the proposed building. Drilled pier and grade beam foundations may also be used. On-grade floor slabs can be used with either foundation system but will require over-excavation of the existing subgrade soils and replacement with low volume change material. Crushed aggregate base materials could be used for pavement support. Subgrade stabilization using a calcium-based admixture, such as, hydrated lime is also acceptable. Specific geotechnical recommendations concerning earthwork and the design and construction of on-grade floor slabs, foundations and pavements are presented subsequently in this report.

5.1 Site Conditions

The existing Cherokee Nation Loyal Shawnee Cultural Center is located on the west side of S4340 Road approximately 0.2 miles south of US-60 near White Oak, Oklahoma. Based on available aerial imagery, the site north of the cultural center where the community center will be constructed appears to have been a farm field with no previous development. Adjacent properties appear to be agricultural or undeveloped. The site and surrounding terrain appeared relatively flat to gently rolling

5.2 Subsurface Conditions

The borings encountered surficial vegetation and topsoil that was underlain by lean clay, lean to fat clay or fat clay soils. The clays were generally stiff to very stiff and brown to olive in color. Borings B-3 and B-4 were terminated within these soils at depths of 5 feet. Within borings B-1 and B-2, the subsurface soils transitioned to brown, soft to moderately hard, poorly cemented, weathered sandstone at respective depths of 9-1/2 feet and 14 feet. Soft to moderately hard, gray, weathered shale was encountered within the borings at depths of approximately 16 feet. These borings were terminated within weathered shale. Subsurface geology appears best described as belonging to the Bluejacket Unit (Pbj).

Based on published report[1], the Bluejacket Unit is dominantly tan sandstone with some stringers of gray shale in northern Division Eight. The unit is about 56 feet in northern Mayes County thickening southward to nearly 300 feet in Rogers County. The unit is underlain in Craig County by the thicker Savanna Unit (Psv).

In accordance with publication ASCE7-10 Chapter 20, a Site Class C should be used for foundation design at this site. This site class is based on Standard Penetration Tests conducted within 19 feet of the ground surface, the reported depth of the geologic units, and classification of the subgrade soils.

Results of each boring are included on the boring logs in Appendix A of this report. Every attempt is made to accurately reflect the depths of material change; however, stratification boundaries should be considered approximate. Specific recommendations concerning on-grade floor slabs, foundations and pavements are presented in the following sections of this report.

5.3 Groundwater Conditions

The borings were monitored for the presence of groundwater while drilling and immediately after boring completion. Groundwater was not encountered within the borings at these times. The borings were backfilled or plugged per OWRB requirements immediately after completion.

To obtain more accurate groundwater level information, long-term observations in a well or piezometer that is sealed from the influence of surface water would be needed. Groundwater level fluctuations

[1] 1965; Research & Development Division; Oklahoma Highway Department; Engineering Classification of Geologic Materials – Division Eight

and / or perched water conditions may occur due to seasonal variations in the amount of rainfall and other factors such as drainage characteristics. The possibility of groundwater level fluctuations should be considered during the preparation of construction plans.

5.4 General Site Development

Site preparation for the proposed building and pavements should include removing the existing grass a topsoil to a depth of at least 8 inches. Any rocks greater than 3 inches and any matted vegetation should be removed. Other unsuitable materials encountered during construction operations should be removed and replaced with suitable soil.

After removing the recommended deleterious materials and making required cuts or over-excavations, but before placing concrete, pavement or fill, we recommend the site be proof-rolled to identify any soft or loose areas. Proof-rolling operations should be observed by qualified geotechnical personnel to identify soft or loose areas to be removed or stabilized, and to verify that all unsuitable materials have been removed. Proof-rolling should be performed using a loaded, tandem-axle dump truck having a minimum gross weight of 25 tons, or other equipment having a similar subgrade loading. Proof-rolling should be performed slowly and in overlapping passes, repeating the process in a perpendicular direction. Any areas of rutting or pumping should be removed and replaced with moisture-conditioned, low volume change soil (defined in section 5.6 of this report).

The soils encountered on site are susceptible to becoming soft or loose with the addition of moisture. During periods of rain, the site may become unworkable and difficult to travel across. If wet subgrade conditions are encountered during construction, we recommend reducing the soil's moisture content by aeration. The use of calcium-based admixtures (lime, fly ash, cement kiln dust, etc.) to dry the soil can also be considered.

5.5 On-Grade Slab Subgrade

One factor affecting on-grade slab support is the shrink-swell potential of the subgrade materials due to seasonal variations in the subgrade moisture content. Typically, some increase in moisture content will occur as a result of gradual accumulation of capillary moisture after a slab is constructed. The shrink-swell potential of the soil beneath an on-grade slab is dependent on its plasticity, moisture content, density, confining pressure provided by the weight of the slab and the overburden pressure (including the fill required to develop design grade). Higher plasticity and density and lower confining pressure and moisture content result in greater swell potential of the soils.

The existing near surface soils at the boring locations consist of moderately plastic to highly plastic clay soils for which significant volume changes due to variations in subgrade moisture content could occur. Typically, buildings such as the one proposed for this site are designed to tolerate vertical floor slab movements of approximately 1-inch or less. Based on the soils liquid limit, plastic limit and an expected moisture change zone of about 8 feet, we predict a potential vertical rise (PVR) between 1-1/4 inches and 1-1/2 inches. Therefore, the existing subgrade soils are considered to be unsuitable for support of floor slabs. We recommend over-excavating the existing soils to a depth of 24 inches and constructing a low volume change soil support zone at least 24 inches in thickness. The following recommendations are provided to develop this low volume change soil zone beneath the slab.

After performing any required over-excavation, but before placing any fill, the exposed subgrade should be scarified to a minimum depth of 8 inches and compacted to at least 95 percent of its maximum dry density as determined by test method ASTM D 698 (commonly referred to as the standard Proctor) at a moisture content at optimum or above. Any soft or loose areas observed, or over-saturated, rutting or pumping soils observed during the compaction operation should be removed and replaced with moisture-controlled, low volume change soils.

All fill required to develop the 24-inch soil support zone should consist of suitable low volume change (LVC) fill materials. The LVC pad should extend laterally at least one foot outside the slab footprint for every foot of fill placed. Suitable LVC soils are considered to be clean, cohesive materials with a liquid limit less than 40 and a plasticity index between 5 and 18, or cohesion-less materials with at least 25 percent passing the standard No. 200 sieve. All fill should be placed in lifts not exceeding 9 inches in loose thickness and compacted to at least 95 percent of the material's maximum dry density. LVC fill soils should be placed at a moisture content within two percent of optimum (test method ASTM D 698).

For direct slab support, at least 4 inches of crushed aggregate, such as ODOT Type A Aggregate Base, should be placed and compacted. Aggregate base should be compacted to at least 95 percent of the material's maximum dry density, generally at a moisture content within 2 percent of optimum. Sand is not recommended for direct slab support due to the tendency to loosen when unconfined. The aggregate base can be considered a part of the 24-inch low volume change soil zone.

During compaction operations, the exposed subgrade and each lift of compacted fill should be tested for moisture and density and reworked as necessary until the lift is approved by the geotechnical engineer's representative prior to the placement of additional material. We recommend the scarified surface and each lift of fill be tested for density and moisture content at a rate of one test per 2,500 square feet of compacted area with a minimum of two tests per compacted area. In addition, we recommend one test per lift for every 100 linear feet of compacted utility trench backfill.

The ground surface should be sloped away from on-grade slabs on all sides to prevent water collection. Water should not be allowed to pond near the slab during or after construction. The moisture content of the soil pad should be maintained near optimum until the slab is constructed. We recommend the moisture content of the on-grade slab subgrade be evaluated just before concrete for the slab is placed.

If floor slabs will be covered with materials that are impervious to moisture migration, we recommend taking precautions to minimize the potential for floor covering problems relative to moisture emission. These precautions should include the following: Place a heavy-duty vapor retarder immediately below the floor slabs and seal the retarder at all penetration points. All fill materials should be placed *below* the vapor retarder. Concrete for the floor slabs should have a low slump and should be carefully cured due to the retention of mix water at the base of the slab over the vapor retarder. To maximize effectiveness, floor slab concrete should be water-cured for at least 7 days, which will also reduce the potential for slab edge curling. Lastly, after the building is enclosed and the HVAC is operating, slab moisture emission tests should be performed to confirm that vapor emission levels comply with the floor covering manufacturer's specifications.

SUMMARY OF EARTHWORK FOR ON-GRADE SLABS	
Clear & Grub	Grass and topsoil to at least 8 inches
Over-Excavate	Existing subgrade to a depth of at least 24 inches
Proofroll	Exposed soils at base of over-excavation
Scarify & Recompact	Top 8" at base of over-excavation, compact to 95% MDD at a moisture content of optimum or above
Low Volume Change Fill	24" minimum. Max. lifts 9" loose, compact each lift to 95% MDD within 2% of optimum moisture
Aggregate Base	4" compacted to 95% MDD within 2% of optimum moisture

5.6 Shallow Footing Foundations

Shallow footing foundations could be used to support the proposed building. To provide adequate confinement and protection against frost and shrinking or swelling of the subgrade soils due to moisture changes, footings should bear at a depth of at least 2 feet below the final adjacent subgrade elevation. For the design of foundations bearing at the recommended depth within tested and approved low volume change fill, a net allowable bearing pressure of 2,250 pounds per square foot can be used. This is the pressure at the base of the foundation in excess of the adjacent overburden pressure. A representative of the geotechnical engineer should be retained to evaluate that footings bear on soils suitable for the design bearing pressure prior the placement of concrete.

Continuous formed footings should have a minimum width of at least 16 inches, and isolated column footings should have a minimum width of at least 30 inches. Earth formed trench footings can also be used and should have a minimum width of at least 12 inches.

Care should be taken to prevent wetting or drying of the bearing materials during construction. Any extremely wet or dry material, or any loose or disturbed material in the bottom of the footing excavations, should be removed prior to placing concrete.

Long-term movement is expected to be less than 1-inch for footings bearing within the materials described above and proportioned for the recommended maximum net allowable bearing pressure. Differential movement is not expected to exceed 1/2 of the total value.

5.7 Drilled Pier and Grade Beam Foundations

A drilled pier and grade beam foundation system supported by a combination of end bearing and side resistance within weathered sandstone or weathered shale encountered below depths of 9-1/2 feet to 14 feet could also be used to support the proposed building. For grade beams bearing at least 24 inches below final adjacent grade, void forms are not required.

For drilled piers should penetrating at least 2 pier diameters of 3 feet, whichever is greater, into weathered sandstone or weathered shale, a maximum net allowable end bearing pressure of 20,000 pounds per square foot can be used for design. This is the pressure at the foundation bearing level in excess of the minimum surrounding overburden pressure. Additional support can be achieved using a side resistance value of 2,000 psf for that portion of the pier that extends more than two pier diameters into bedrock. A representative of the geotechnical engineer should be on-site during pier drilling to evaluate that bearing materials suitable for the maximum design bearing pressure are adequately penetrated.

An earth auger could be used to penetrate the overburden soils. However, a rock bit will be required to penetrate weathered sandstone and weathered shale. Temporary casing is not expected to be required to prevent caving of the subsurface soils encountered at the site. Infiltration of pier excavations by groundwater is not expected to occur, but should be anticipated. The final decision concerning casing of the pier excavations should be made at the time of construction.

Prior to placing concrete, any water deeper than 2 inches and all sloughed material should be removed from the base of the drilled piers. Concrete placed in small shaft pier excavations deeper than 10 feet should be placed through a tremie or with a concrete pump to prevent segregation of the aggregates. In larger shafts this requirement may be waived as long as concrete is placed through the center of the shaft and is not allowed to strike the excavation sides, tie wires or reinforcing steel. Concrete should have a minimum slump of 6 inches and should be vibrated to insure adequate consolidation. In no event should a pier excavation be allowed to remain open for more than 4 hours.

Long-term settlement for straight shaft piers bearing within suitable soils is expected to be less than 1/2-inch provided piers have a minimum diameter of 18 inches. Differential settlement is not expected to exceed one-half of the total settlement value.

5.8 Pavement Thickness & Subgrade Development

Light-duty parking lot pavements are expected to support passenger automobiles only. Drive and parking areas accessible to delivery trucks or refuse collection trucks should be designed as heavy-duty pavements. Based on laboratory testing of the subgrade soils, a CBR value of 2.1 should be used for the design of pavements at this site. The following table provides minimum pavement thicknesses for both rigid and flexible pavements.

	Light Duty Pavement	**Heavy Duty Pavement**
Rigid Pavement	5” Portland Cement Concrete (3500 psi min.) 6” ODOT Type “A” Aggregate Base or 8” Stabilized Subgrade	7” Portland Cement Concrete (3500 psi min. doweled across joints) 6” ODOT Type “A” Aggregate Base or 8” Stabilized Subgrade
Flexible Pavement	2” Type S4 ACC (PG 64-22 OK) 3” Type S3 ACC (PG 64-22 OK) 6” ODOT Type “A” Aggregate Base or 8” Stabilized Subgrade	2” Type S4 ACC (PG 64-22 OK) 2” Type S4 ACC (PG 64-22 OK) 3” Type S3 ACC (PG 64-22 OK) 6” ODOT Type “A” Aggregate Base or 8” Stabilized Subgrade

It is recommended that reinforced concrete pads be constructed in front of and beneath the refuse storage and pick-up area. The refuse trucks should be parked on rigid Portland cement concrete pavement when the trash receptacles are lifted. The concrete pad should be at least 7 inches thick and fully reinforced.

Before any required fill is placed, the subgrade should be proof-rolled as recommended in Section 5.4 of this report. Once design grades are developed, 6 inches of ODOT Type “A” Aggregate Base should be placed for direct pavement support. Aggregate base materials should be compacted to at least 98 percent of the materials maximum dry density per test method ASTM D 698 at a moisture content within 2 percent of optimum. A separator fabric or geogrid could be placed between the aggregate base and the subgrade soils to provide long-term separation of the materials.

As an alternative to aggregate base, the top 8 inches of the subgrade could be stabilized with an estimated 5 percent hydrated lime. The actual percentage of additive should be determined at the time of construction for the exposed subgrade soils. After final mixing of the lime and adjusting the moisture content of the mixture to within two percent of optimum, the material should be compacted to at least 98 percent of the maximum dry density as determined by test method ASTM D-698.

All fill required to develop final grade lines in the proposed parking and drive areas should consist of low volume change soils that are free of organic matter and debris. Fill should be placed in loose lifts not exceeding 9 inches and should be compacted to at least 95 percent of the maximum dry density at a moisture content within 2 percent of optimum. Any soft or loose areas observed or over-saturated, rutting or pumping soils observed during compaction operations should be removed and replaced.

During compaction operations, each lift of compacted fill should be tested for moisture and density and reworked as necessary until that surface is approved by the geotechnical engineer's representative prior to the placement of additional lifts. We recommend the aggregate base or stabilized subgrade, and each lift of fill be tested for density and moisture content at a rate of one test per 10,000 square feet of compacted area with a minimum of two tests per compacted area. In addition, we recommend one test per lift for every 100 linear feet of compacted utility trench backfill. The moisture content of the aggregate base or stabilized soil should be maintained near optimum during construction. We recommend the moisture content be evaluated immediately before pavement is placed.

Minimizing subgrade saturation is an important factor in maintaining subgrade strength. Water allowed to pond on or adjacent to pavements could saturate the subgrade and cause premature pavement deterioration. The pavement and subgrade should be sloped approximately ¼ inch per foot to provide rapid surface drainage, and positive surface drainage should be maintained away from the edge of the paved areas. Design alternatives that would reduce the risk of subgrade saturation and improve long-term pavement performance include placing a separator fabric between the aggregate base and subgrade soils, crowning the pavement subgrade to drain toward the edges, rather than to the center of the pavement areas and installing surface drains next to any areas where surface water could pond.

Maintenance of the pavement will be required to obtain a satisfactory design life. Maintenance should include crack sealing, surface sealing and patching of any deteriorated areas. In addition, thicker pavement sections could be used to reduce the required maintenance and extend the service life of the pavement.

6.0 CONCLUDING REMARKS

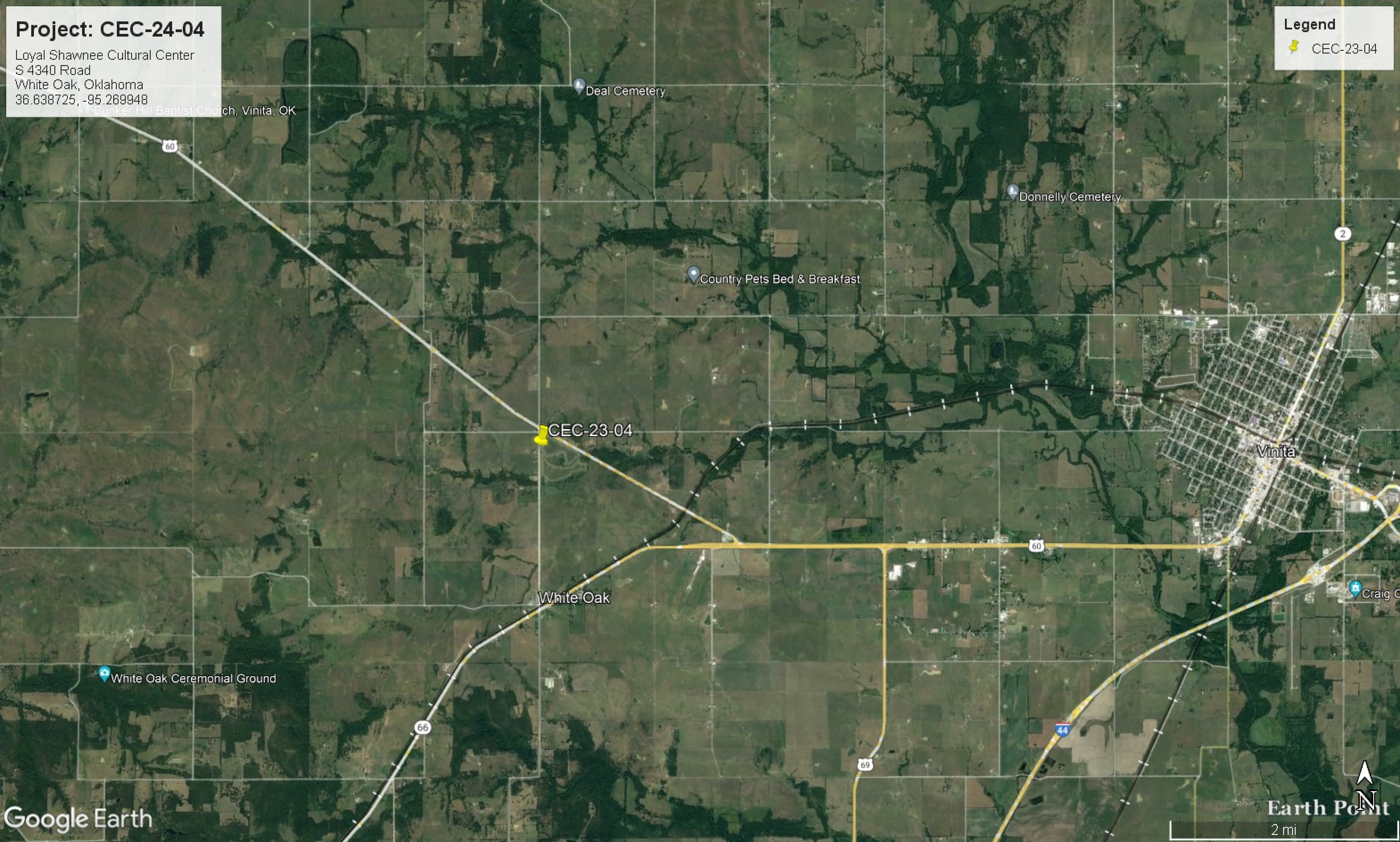
Recommendations provided in this report are based on information from discrete borings (generally 4 to 8 inches in diameter) and, in some cases, from an engineer's general surficial observations. All site conditions cannot be detailed based on a limited number of borings and increasing the number of borings so that all site conditions can be defined is generally not practical. Variations in site conditions between boring locations should be expected and, on occasion, revised recommendations will be required. Hinderliter Geotechnical Engineering, LLC (HGE) should be retained to review final plans and specifications so that comments can be provided regarding the implementation of recommendations provided in this report. HGE should also be retained to provide monitoring of site construction.

This report provides recommendations concerning site construction and, while it may provide limited analysis of soil corrosiveness and / or contaminant content, is not an Environmental Site Assessment (ESA). If the Owner is concerned about environmental and / or biological assessment, a separate study specifically focused on environmental issues should be undertaken.

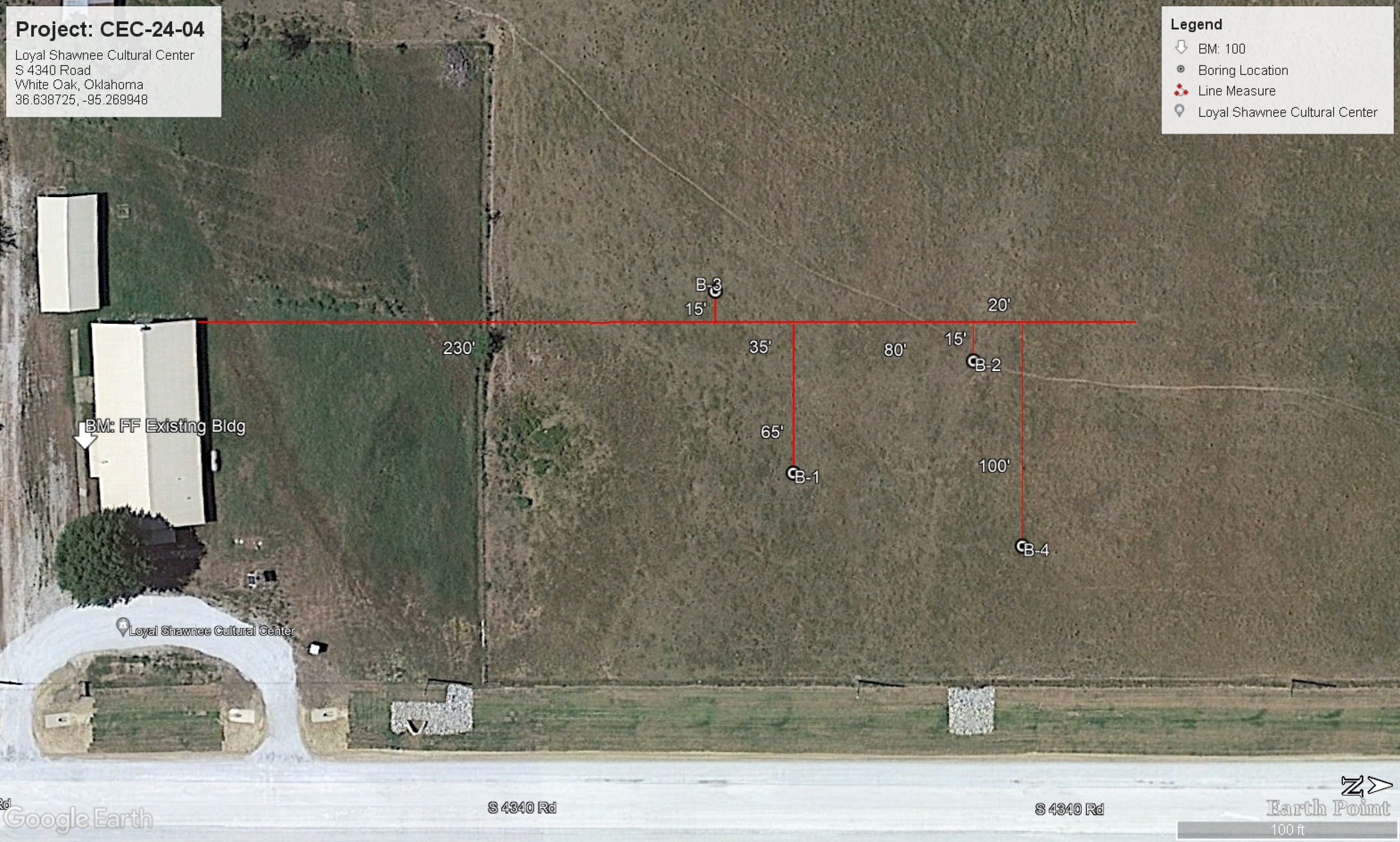
This report has been prepared specifically for the referenced project and for the exclusive use of our Client. Third-party reliance may be granted upon specific written request of the Client. This report has been prepared using locally specific and generally accepted geotechnical engineering practices based on structural information provided by the Client and information gained from the site. No warranties are implied or granted regarding site recommendations not specifically discussed in this report.

APPENDIX A

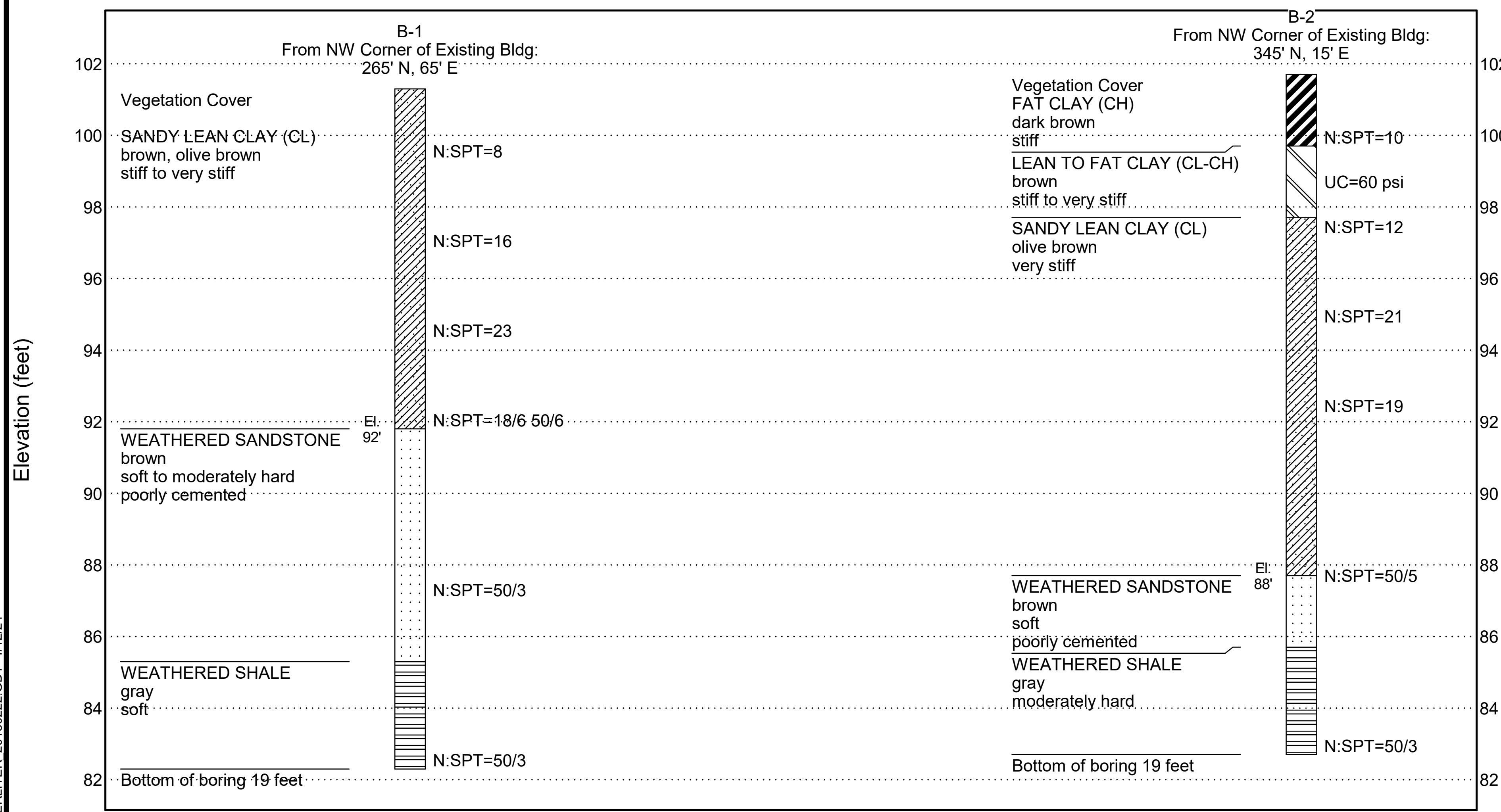
SITE VICINITY MAP
BORING LOCATION DIAGRAM
SUBSURFACE FENCE DIAGRAM
BORING LOGS



Project: CEC-24-04
Loyal Shawnee Cultural Center
S 4340 Road
White Oak, Oklahoma
36.638725, -95.269948
Legend
CEC-23-04
Bunker Hill Baptist Church, Vinita, OK
Deal Cemetery
Donnelly Cemetery
Country Pets Bed & Breakfast
CEC-23-04
Vinita
White Oak
White Oak Ceremonial Ground
60
2
66
69
44
Google Earth
Earth Point
2 mi
N



Project: CEC-24-04
Loyal Shawnee Cultural Center
S 4340 Road
White Oak, Oklahoma
36.638725, -95.269948
Legend
BM: 100
Boring Location
Line Measure
Loyal Shawnee Cultural Center
B-3
15'
20'
230'
35'
80'
15'
B-2
BM: FF Existing Bldg
65'
B-1
100'
B-4
Loyal Shawnee Cultural Center
S 4340 Rd
S 4340 Rd
N
Google Earth
Earth Point
100 ft



Hinderliter Geotechnical Engineering
4071 NW 3rd Street
Oklahoma City, OK 73107
Telephone: (405) 942-4090
Website: HinderliterGeo.com

LEGEND:
N:SPT=Standard Penetration Test
TCP=Texas Cone Penetrometer
R=Recovery
RQD=Rock Quality Designation
UC=Unconfined Compressive Strength

SUBSURFACE FENCE DIAGRAM

Project: CN - Loyal Shawnee Community Center

Location: S 4340 Rd - White Oak, OK 74301

Number: CEC-24-04

US FENCE CEC-24-04.GPJ DT HINDERLITER 20180222.GDT 4/12/24

LOG OF BORING B-1

Hinderliter Geotechnical Engineering
4071 NW 3rd Street
Oklahoma City, OK 73107
Telephone: (405) 942-4090
Website: HinderliterGeo.com

CLIENT: CEC Corporation
PROJECT: CN - Loyal Shawnee Community Center
LOCATION: S 4340 Rd - White Oak, OK 74301
NUMBER: CEC-24-04

DATE(S) DRILLED: 3/19/24

DRILLING METHOD(S):
CME-45 truck-mounted drill. 6" solid flight augers. SPT penetration testing & sampling.

GROUNDWATER INFORMATION:
No groundwater encountered prior to boring termination.

SURFACE ELEVATION: 101.3

FIELD DATA / LABORATORY DATA

DEPTH (FT)	SAMPLES	N: BLOWS/FT P: TONS/SQ FT T: BLOWS R: % RQD: %	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (LL)	PLASTIC LIMIT (PL)	PLASTICITY INDEX (PI)	DRY DENSITY POUNDS/CU.FT	MINUS NO. 4 SIEVE (%)	MINUS NO. 10 SIEVE (%)	MINUS NO. 40 SIEVE (%)	MINUS NO. 200 SIEVE (%)	DESCRIPTION OF STRATUM
0–1												Vegetation Cover
1–2.5	X	N = 8	19.1									SANDY LEAN CLAY (CL) brown, olive brown stiff to very stiff
3.5–5	X	N = 16	18.5	35	14	21		100	100	96	62.6	
6–7.5	X	N = 23	13.7									
8.5–10	X	N = 18/6 50/6	14.5									
9.5–16												WEATHERED SANDSTONE brown soft to moderately hard poorly cemented
13.5–14.5	X	N = 50/3	8.8									
16–19												WEATHERED SHALE gray soft
18.5–19	X	N = 50/3	7.4									
19												Bottom of boring 19 feet

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - TXDOT CONE PENETRATION RESISTANCE
R - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:
Approximate Boring Location:
From NW Corner of Existing Bldg:
265' N, 65' E

LOG OF BORING B-2

HGE
Hinderliter Geotechnical Engineering
4071 NW 3rd Street
Oklahoma City, OK 73107
Telephone: (405) 942-4090
Website: HinderliterGeo.com

CLIENT: CEC Corporation
PROJECT: CN - Loyal Shawnee Community Center
LOCATION: S 4340 Rd - White Oak, OK 74301
NUMBER: CEC-24-04

DATE(S) DRILLED: 3/19/24

DRILLING METHOD(S):
CME-45 truck-mounted drill. 6" solid flight augers. SPT penetration testing & sampling. Steel tube sampling.

GROUNDWATER INFORMATION:
No groundwater encountered prior to boring termination.

SURFACE ELEVATION: 101.7

FIELD DATA / LABORATORY DATA

DEPTH (FT)	N: BLOWS/FT P: TONS/SQ FT T: BLOWS R: % RQD: %	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT (LL)	PLASTIC LIMIT (PL)	PLASTICITY INDEX (PI)	DRY DENSITY POUNDS/CU.FT	MINUS NO. 4 SIEVE (%)	MINUS NO. 10 SIEVE (%)	MINUS NO. 40 SIEVE (%)	MINUS NO. 200 SIEVE (%)	DESCRIPTION OF STRATUM
0–2	N = 10	22.3	50	13	37		100	100	96	71.8	Vegetation Cover; FAT CLAY (CH), dark brown, stiff
2–4	UC=60 psi	19.4	48	22	26	108.6	100	100	99	86.2	LEAN TO FAT CLAY (CL-CH), brown, stiff to very stiff
4–14	N = 12	20.1									SANDY LEAN CLAY (CL), olive brown, very stiff
	N = 21	13.2									
	N = 19	16.6									
14–16	N = 50/5	11.7									WEATHERED SANDSTONE, brown, soft, poorly cemented
16–19	N = 50/3	6.0									WEATHERED SHALE, gray, moderately hard
19											Bottom of boring 19 feet

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - TXDOT CONE PENETRATION RESISTANCE
R - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:
Approximate Boring Location:
From NW Corner of Existing Bldg:
345' N, 15' E

LOG OF BORING B-3

HGE
Hinderliter Geotechnical Engineering
4071 NW 3rd Street
Oklahoma City, OK 73107
Telephone: (405) 942-4090
Website: HinderliterGeo.com

CLIENT: CEC Corporation
PROJECT: CN - Loyal Shawnee Community Center
LOCATION: S 4340 Rd - White Oak, OK 74301
NUMBER: CEC-24-04

DATE(S) DRILLED: 3/19/24

DRILLING METHOD(S):
CME-45 truck-mounted drill. 6" solid flight augers. SPT penetration testing & sampling.

GROUNDWATER INFORMATION:
No groundwater encountered prior to boring termination.

SURFACE ELEVATION: 101.2

FIELD DATA				LABORATORY DATA									
SOIL SYMBOL	DEPTH (FT)	SAMPLES	N: BLOWS/FT P: TONS/SQ FT T: BLOWS R: % RQD: %	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT LL	ATTERBERG LIMITS: PLASTIC LIMIT PL	ATTERBERG LIMITS: PLASTICITY INDEX PI	DRY DENSITY POUNDS/CU.FT	MINUS NO. 4 SIEVE (%)	MINUS NO. 10 SIEVE (%)	MINUS NO. 40 SIEVE (%)	MINUS NO. 200 SIEVE (%)	DESCRIPTION OF STRATUM
	0–1												Vegetation Cover
	1–2	X	N = 9	24.6	51	22	29		100	100	98	77.5	FAT CLAY with SAND (CH) olive brown stiff
	3.5–5	X	N = 13	15.8									
	5												Bottom of boring 5 feet

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - TXDOT CONE PENETRATION RESISTANCE
R - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:
Approximate Boring Location:
From NW Corner of Existing Bldg:
230' N, 15' W

LOG OF BORING B-4

Hinderliter Geotechnical Engineering
4071 NW 3rd Street
Oklahoma City, OK 73107
Telephone: (405) 942-4090
Website: HinderliterGeo.com

CLIENT: CEC Corporation
PROJECT: CN - Loyal Shawnee Community Center
LOCATION: S 4340 Rd - White Oak, OK 74301
NUMBER: CEC-24-04

DATE(S) DRILLED: 3/19/24

DRILLING METHOD(S):
CME-45 truck-mounted drill. 6" solid flight augers. SPT penetration testing & sampling.

GROUNDWATER INFORMATION:
No groundwater encountered prior to boring termination.

SURFACE ELEVATION: 101.9

FIELD DATA / LABORATORY DATA

SOIL SYMBOL	DEPTH (FT)	SAMPLES	N: BLOWS/FT P: TONS/SQ FT T: BLOWS R: % RQD: %	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	DRY DENSITY POUNDS/CU.FT	MINUS NO. 4 SIEVE (%)	MINUS NO. 10 SIEVE (%)	MINUS NO. 40 SIEVE (%)	MINUS NO. 200 SIEVE (%)	DESCRIPTION OF STRATUM
	0												Vegetation Cover
	1–2	X	N = 17	25.8									SANDY LEAN CLAY (CL) olive brown very stiff
	3–5	X	N = 18	16.5	34	15	19		100	100	94	64.4	
	5												Bottom of boring 5 feet

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - TXDOT CONE PENETRATION RESISTANCE
R - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:
Approximate Boring Location:
From NW Corner of Existing Bldg:
365' N, 100' E

APPENDIX B

MOISTURE CONTENT & VISUAL CLASSIFICATION
PARTICLE SIZE DISTRIBUTION REPORTS
UNCONFINED COMPRESSION TEST
MOISTURE-DENSITY TEST DATA
BEARING RATIO TEST REPORT

CEC

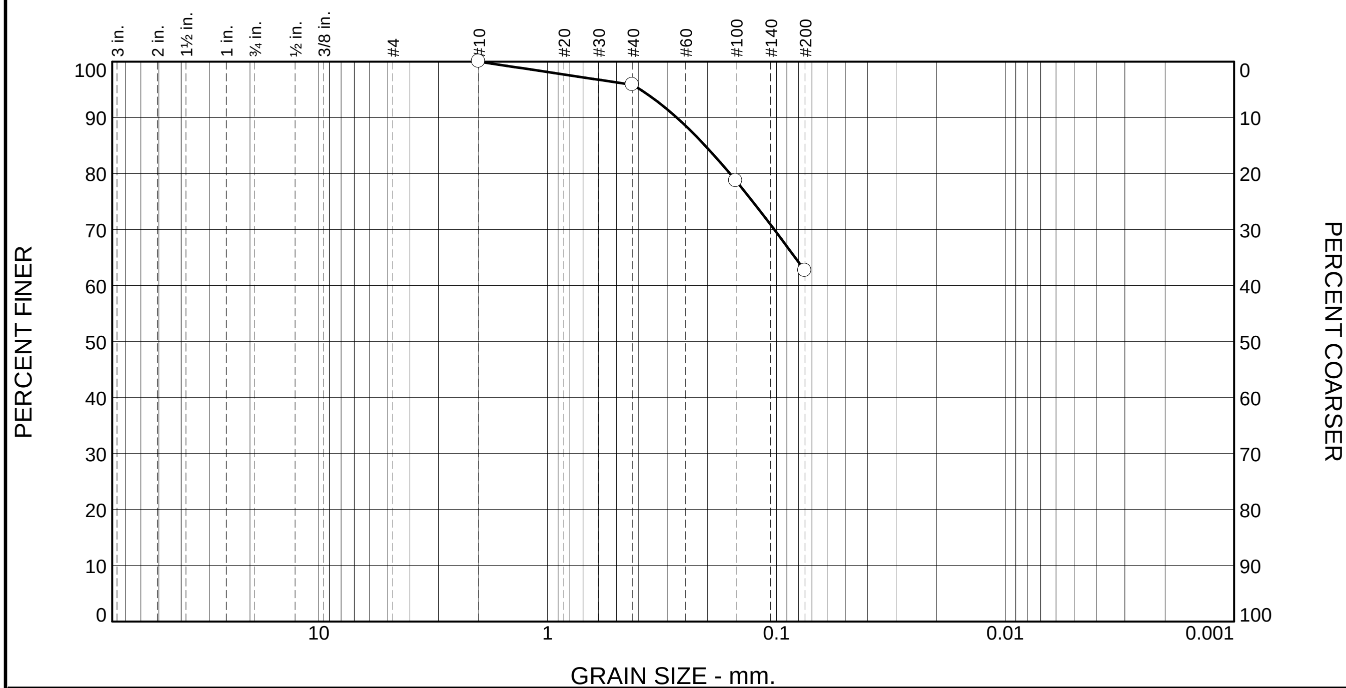
Moisture Content & Visual Classification

Project No : CEC-24-04
Project : ***Loyal Shawnee Cultural Center***
Client : CEC

Sample Date: March 20, 2024
Sampled By: HGE
Date Received: March 20, 2024
Tested By: Soils Lab

Sample ID	Depth	Color	Description	Pan Mass	Pan + Wet	Pan + Dry	% Moisture
B-1 S-1		Brown	Lean Clay	30.5	158.1	137.6	**19.1%**
B-1 S-2		Olive Brown	Sandy-Lean Clay	405.3	1316.7	1174.1	**18.5%**
B-1 S-3		Brown	Lean Clay with Silt	31.8	158.8	143.5	**13.7%**
B-1 S-4		Olive Brown	Lean Clay with Sand	31.7	177.7	159.2	**14.5%**
B-1 S-5		Olive Brown	Silty Sand	30.2	171.1	159.7	**8.8%**
B-1 S-6		Olive Brown	Silty Sand	30.7	147.2	139.2	**7.4%**
B-2 S-1		Dark Brown	Fat Clay	403.2	1214.7	1067.0	**22.3%**
B-2 S-2		Brown	Lean Clay with Sand	30.7	163.2	141.0	**20.1%**
B-2 S-3		Brown	Lean Clay with Sand	31.0	164.2	148.7	**13.2%**
B-2 S-4		Olive Brown	Lean Clay	30.8	158.8	140.6	**16.6%**
B-2 S-5		Olive Brown	Silty Sand	31.0	177.8	162.4	**11.7%**
B-2 S-6		Brown	Silty Sand	30.9	161.7	154.3	**6.0%**
B-3 S-1		Olive Brown	Fat Clay with Sand	402.0	1019.9	898.0	**24.6%**
B-3 S-2		Olive Brown	Lean Clay	31.8	203.2	179.8	**15.8%**
B-4 S-1		Olive Brown	Lean Clay	30.7	166.9	139.0	**25.8%**
B-4 S-2		Olive Brown	Sandy-Lean Clay	392.2	1353.4	1217.6	**16.5%**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	4.2	33.2	62.6	

Test Results (AASHTO T 27 & AASHTO T 11)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#10	100.0		
#40	95.8		
#100	78.7		
#200	62.6		

* (no specification provided)

Material Description

Olive-Brown Sandy Lean Clay

Atterberg Limits (ASTM D 4318)

PL= 14 **LL=** 35 **PI=** 21

Classification

USCS (D 2487)= CL **AASHTO (M 145)=** A-6(10)

Coefficients

D_{90}= 0.2728 D_{85}= 0.2049 D_{60}=

D_{50}= D_{30}= D_{15}=

D_{10}= C_u= C_c=

Remarks

Date Received: 3/20/24 **Date Tested:** 3/26/24

Tested By: N. Johnson

Checked By: R. Saldana

Title: Soil Lab Coordinator

Location: B-1 S-2

Date Sampled: 3/20/24

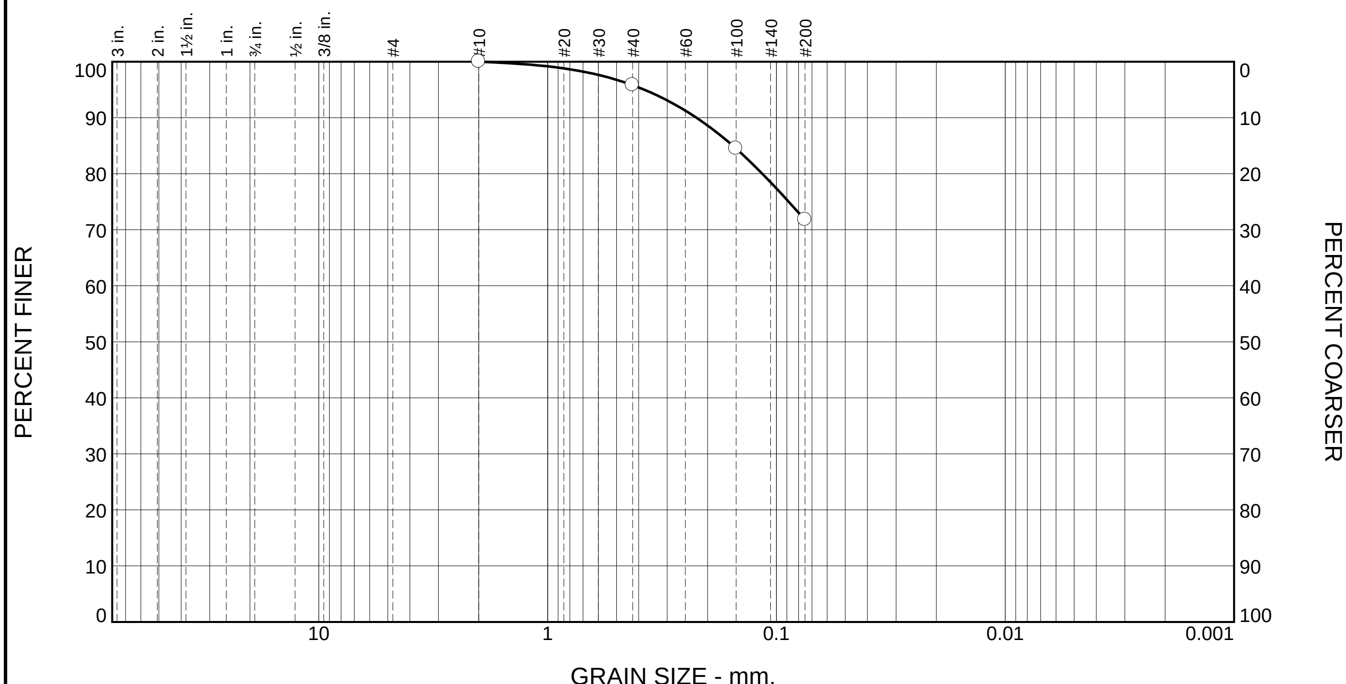
CEC materials testing

Client: CEC

Project: Loyal Shawnee Cultural Center

Project No: CEC-24-04 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	4.2	79.6	16.2	

Test Results (AASHTO T 27 & AASHTO T 11)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#10	100.0		
#40	95.8		
#100	84.5		
#200	71.8		

* (no specification provided)

Material Description

Dark Brown Clayey Sand

Atterberg Limits (ASTM D 4318)

PL= 13 **LL=** 50 **PI=** 37

Classification

USCS (D 2487)= CH **AASHTO (M 145)=** A-7-6 (25)

Coefficients

D_{90}= 0.2308 D_{85}= 0.1556 D_{60}= 0.1113
D_{50}= 0.1010 D_{30}= 0.0845 D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 3/20/24 **Date Tested:** 3/26/24

Tested By: A. Taylor

Checked By: R. Saldana

Title: Soil Lab Coordinator

Location: B-2 S-1 **Date Sampled:** 3/20/24

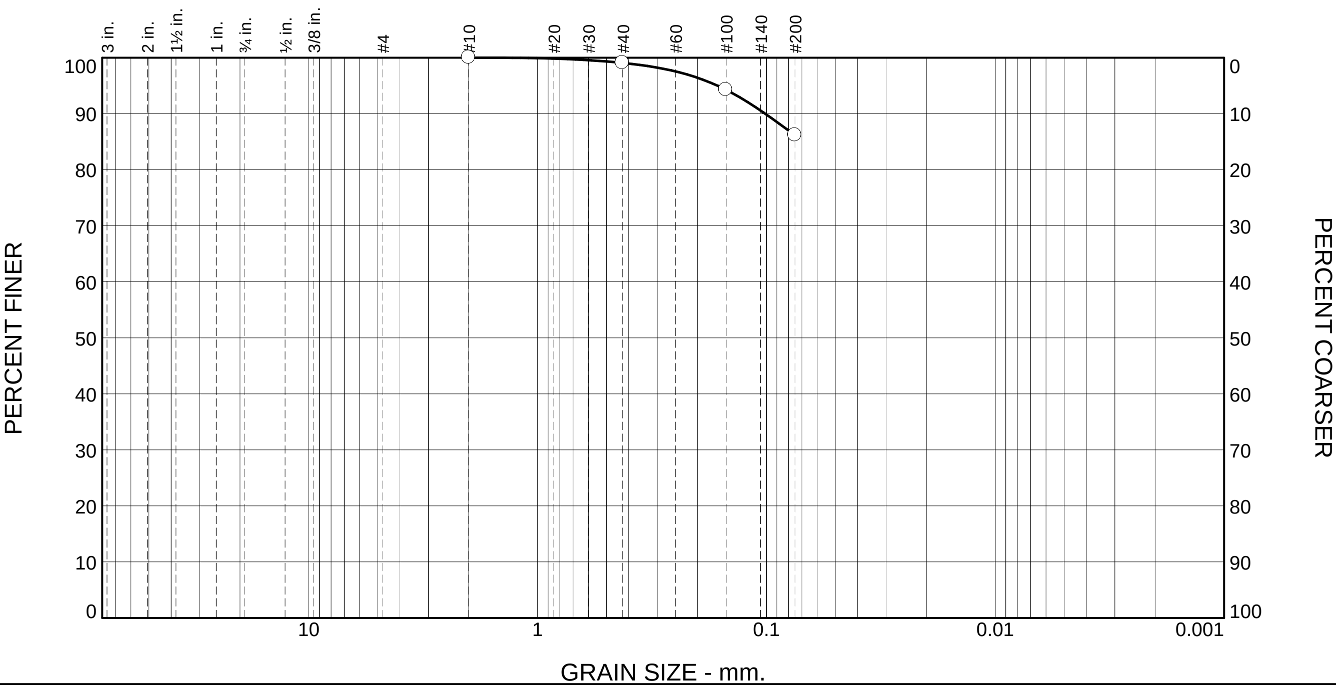
CEC materials testing

Client: CEC

Project: Loyal Shawnee Cultural Center

Project No: CEC-24-04 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.9	12.9	86.2	

Test Results (AASHTO T 27 & AASHTO T 11)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#10	100.0		
#40	99.1		
#100	94.3		
#200	86.2		

* (no specification provided)

Material Description

Brown Lean Clay

Atterberg Limits (ASTM D 4318)

PL= 22 **LL=** 48 **PI=** 26

Classification

USCS (D 2487)= CL **AASHTO (M 145)=** A-7-6(24)

Coefficients

D_{90}= 0.1013 D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 3/20/24 **Date Tested:** 3/27/24

Tested By: A. Taylor

Checked By: R. Saldana

Title: Soil Lab Coordinator

Location: Shelby Tube B2
Depth: 2' - 4"

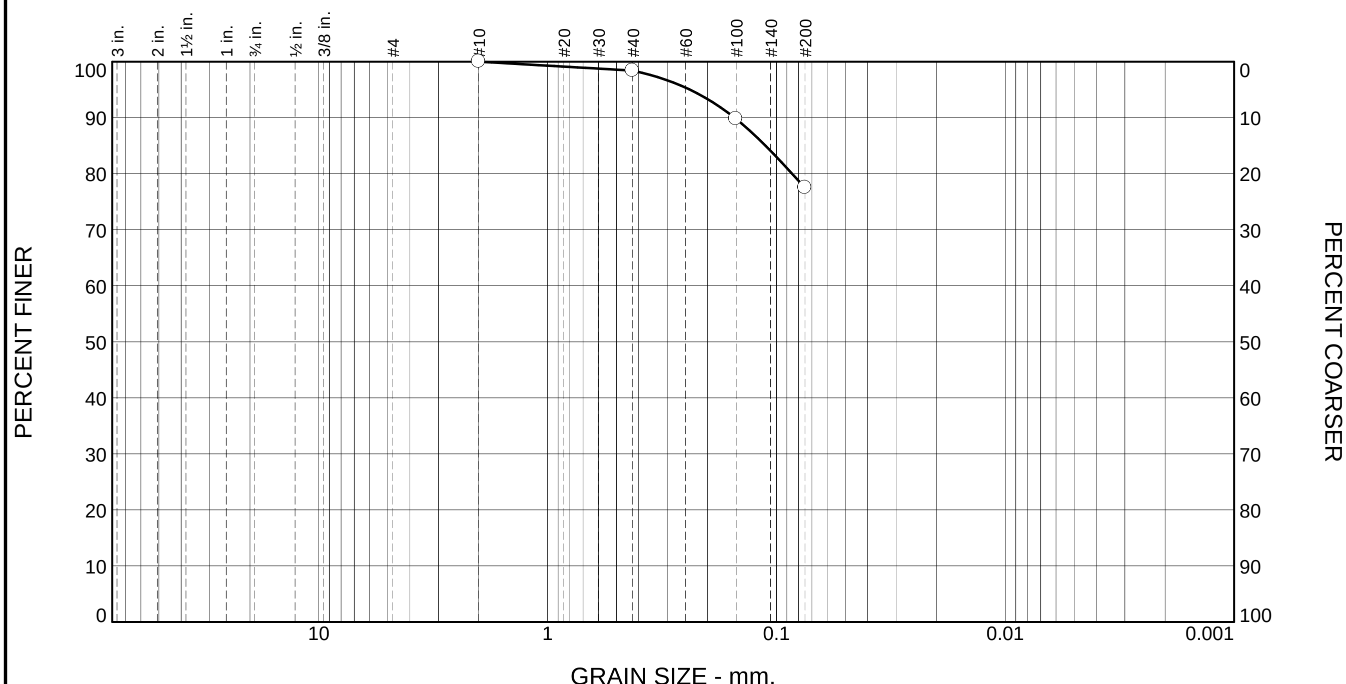
Date Sampled: 3/20/24

CEC materials testing

Client: CEC
Project: Loyal Shawnee Cultural Center

Project No: CEC-24-04 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	1.6	20.9	77.5	

Test Results (AASHTO T 27 & AASHTO T 11)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#10	100.0		
#40	98.4		
#100	89.8		
#200	77.5		

* (no specification provided)

Material Description

Olive Brown Fat Clay with Sand

Atterberg Limits (ASTM D 4318)

PL= 22 **LL=** 51 **PI=** 29

Classification

USCS (D 2487)= CH **AASHTO (M 145)=** A-7-6(23)

Coefficients

D_{90}= 0.1524 D_{85}= 0.1117 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 3/20/24 **Date Tested:** 3/26/24

Tested By: A. Taylor

Checked By: R. Saldana

Title: Soil Lab Coordinator

Location: B-3 S-1

Date Sampled: 3/20/24

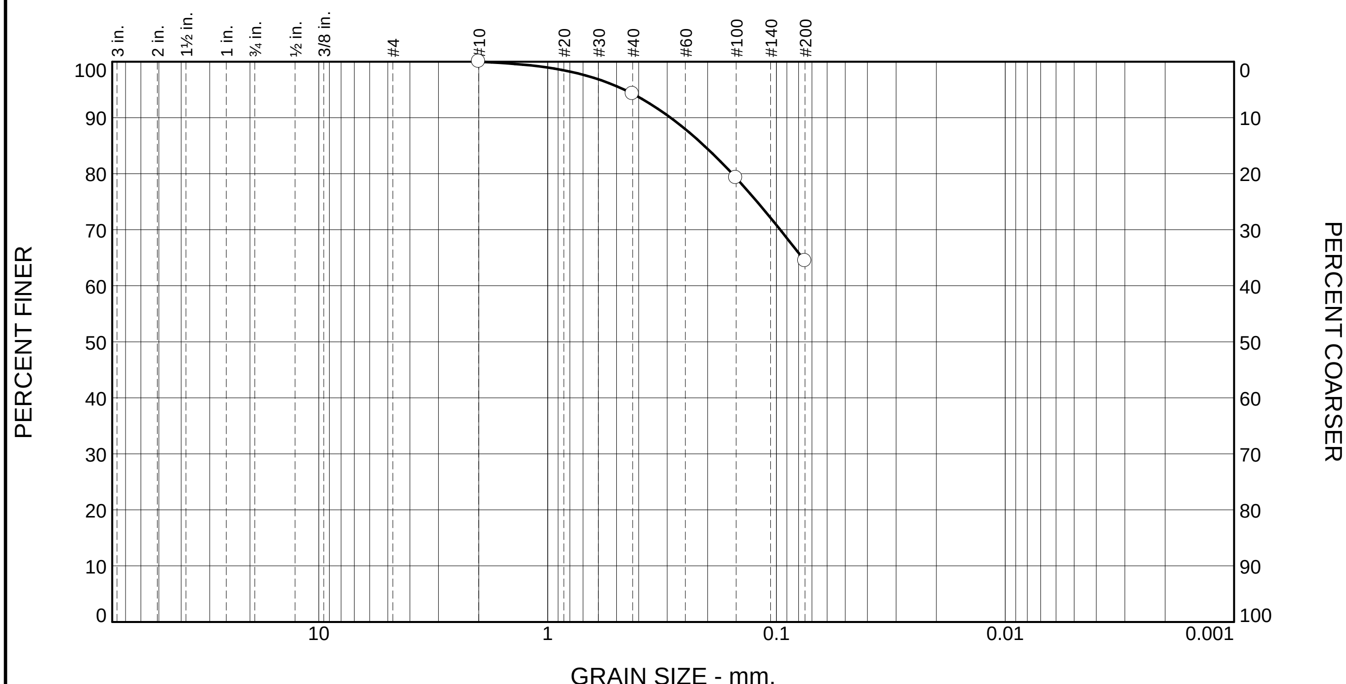
CEC materials testing

Client: CEC

Project: Loyal Shawnee Cultural Center

Project No: CEC-24-04 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	5.7	29.9	64.4	

Test Results (AASHTO T 27 & AASHTO T 11)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#10	100.0		
#40	94.3		
#100	79.3		
#200	64.4		

* (no specification provided)

Material Description

Olive Brown Sandy Lean Clay

Atterberg Limits (ASTM D 4318)

PL= 15 **LL=** 34 **PI=** 19

Classification

USCS (D 2487)= CL **AASHTO (M 145)=** A-6(9)

Coefficients

D_{90}= 0.2902 D_{85}= 0.2069 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Sulfate PPM=

Date Received: 3/20/24 **Date Tested:** 3/26/24

Tested By: A. Taylor

Checked By: R. Saldana

Title: Soil Lab Coordinator

Location: B-4 S-2

Date Sampled: 3/20/24

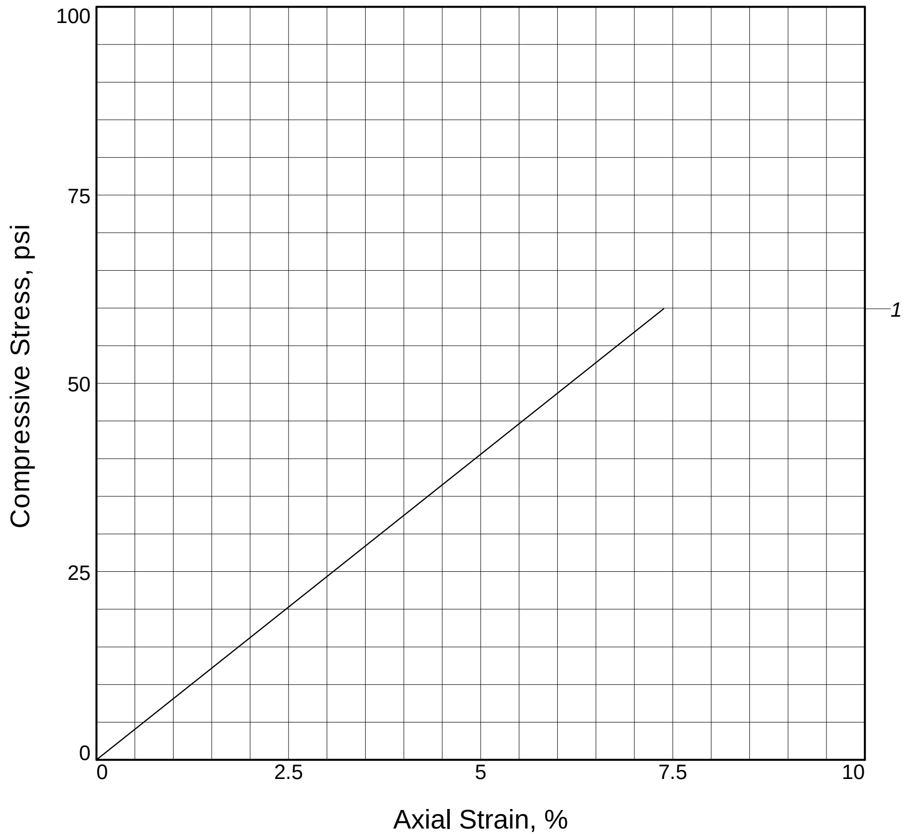
CEC materials testing

Client: CEC

Project: Loyal Shawnee Cultural Center

Project No: CEC-24-04 **Figure**

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psi	59.90			
Undrained shear strength, psi	29.95			
Failure strain, %	7.4			
Strain rate, in./min.	0.040			
Water content, %	19.4			
Wet density, pcf	129.6			
Dry density, pcf	108.6			
Saturation, %	98.0			
Void ratio	0.5232			
Specimen diameter, in.	2.84			
Specimen height, in.	5.83			
Height/diameter ratio	2.05			

Description: Brown Lean Clay

LL = 48	**PL =** 22	**PI =** 26	**Assumed GS=** 2.65	**Type:** Shelby Tube

Project No.: CEC-24-04
Date Sampled: 3/20/24
Remarks:

Figure ________

Client: CEC

Project: Loyal Shawnee Cultural Center

Location: Shelby Tube B2
Depth: 2' - 4"

CEC materials testing

Tested By: R. Saldana **Checked By:** R. Saldana

MOISTURE DENSITY TEST DATA

3/31/2024

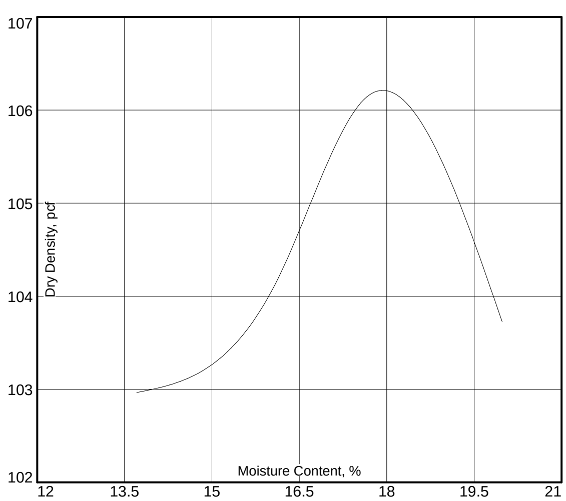
Client: CEC
Project: Loyal Shawnee Cultural Center
Project Number: CEC-24-04
Location: Bulk Material
Description: Brown Lean Clay with Sand
Sample Date: 3/20/24
Date Received: 3/20/24
Grain Size Test Method: AASHTO T 27
#200 Wash Method: AASHTO T 11
Preparation Method: Dry
Rammer Type: Manual
USCS: CL **AASHTO:** A-6(18)
LL: 40 **PI:** 25
Testing Remarks: Sulfate PPM=0
Tested By: A. Taylor **Test Date:** 3/25/24
Checked By: R. Saldana **Title:** Soil Lab Coordinator
Percent passing #4 sieve: 100.0

Test Data and Results

Test Specification:

Type of Test: AASHTO T 99-15 Method A Standard

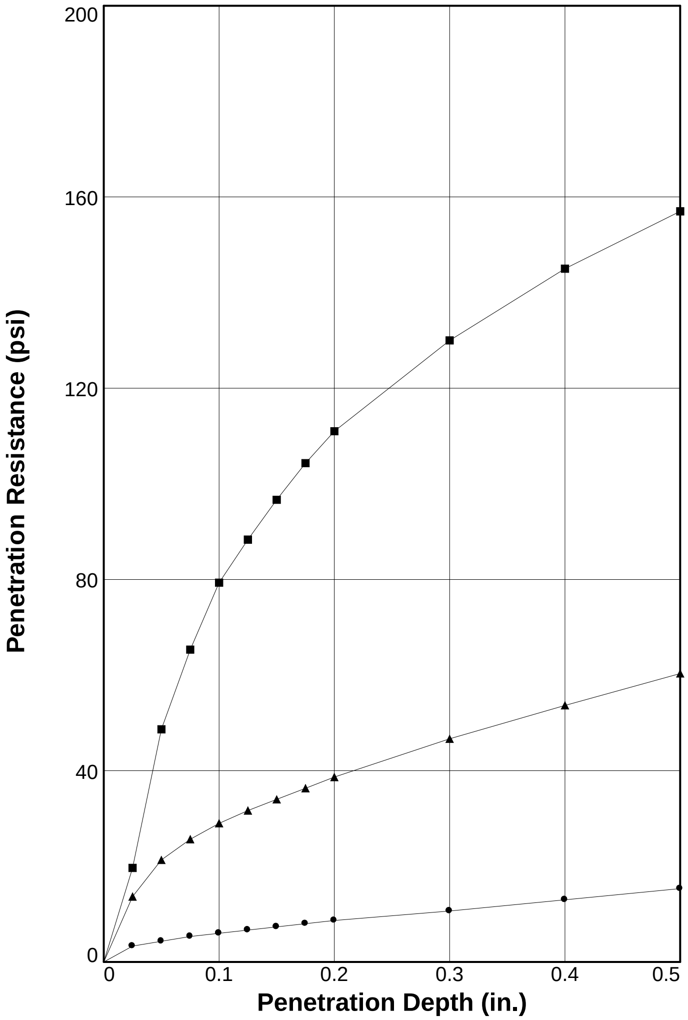
Mold Dia: 3.99 **Hammer Wt.:** 5.5 lb. **Drop:** 12 in. **Layers:** Three **Blows per Layer:** 25

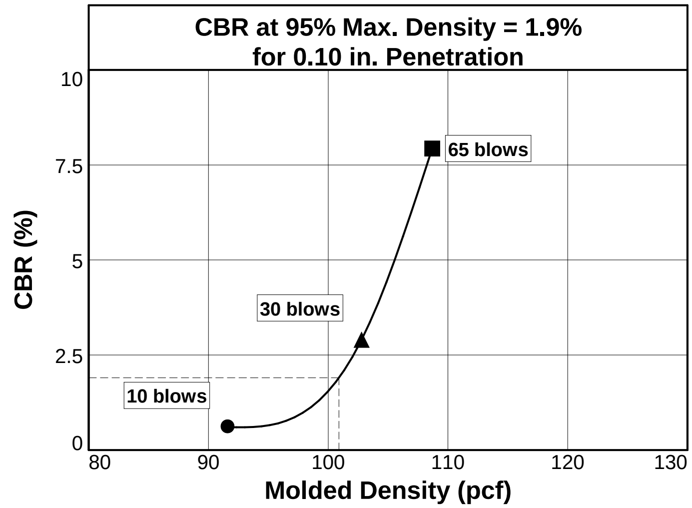


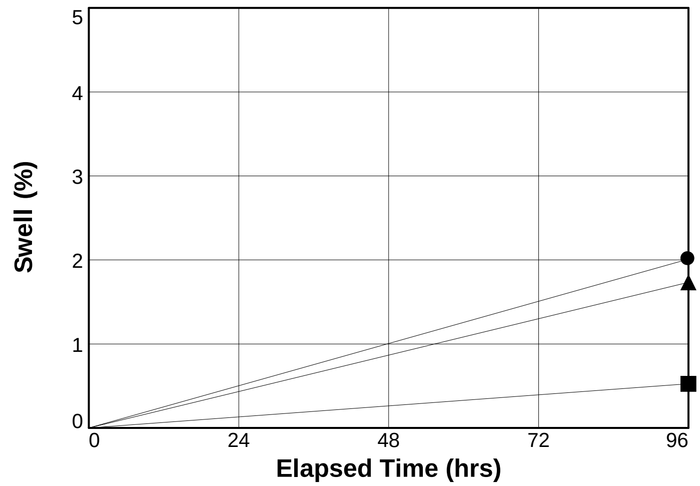
Point No.	1	2	3	4
Wt. M+S	6005.5	6066.0	6122.7	6116.2
Wt. M	4247.7	4247.7	4247.7	4247.7
Wt. W+T	148.9	166.1	156.9	154.5
Wt. D+T	134.7	147.4	137.9	133.8
Tare	31.1	31.8	30.3	30.2
Moist.	13.7	16.2	17.7	20.0
Dry Den.	103.0	104.2	106.1	103.7

Test Results: **Max. Dry Den.=** 106.2 pcf **Opt. Moist.=** 17.9%

BEARING RATIO TEST REPORT
ASTM D1883-16







	Molded			Soaked			CBR (%)		Linearity Correction (in.)	Surcharge (lbs.)	Max. Swell (%)
	Density (pcf)	Percent of Max. Dens.	Moisture (%)	Density (pcf)	Percent of Max. Dens.	Moisture (%)	0.10 in.	0.20 in.			
1 ○	91.7	86.3	18.5	89.9	84.6	27.1	0.6	0.6	0.000	10	2
2 △	102.8	96.8	18.1	101.0	95.1	21.8	2.9	2.6	0.000	10	1.7
3 □	108.7	102.4	18.9	108.1	101.8	18.9	7.9	7.4	0.000	10	0.5

Material Description	USCS	Max. Dens. (pcf)	Optimum Moisture (%)	LL	PI
Brown Lean Clay with Sand	CL	106.2	17.9	40	25

Project No: CEC-24-04

Project: Loyal Shawnee Cultural Center

Location: Bulk Material

Date: 3/20/24

CEC materials testing

Test Description/Remarks:

Figure ________

APPENDIX C

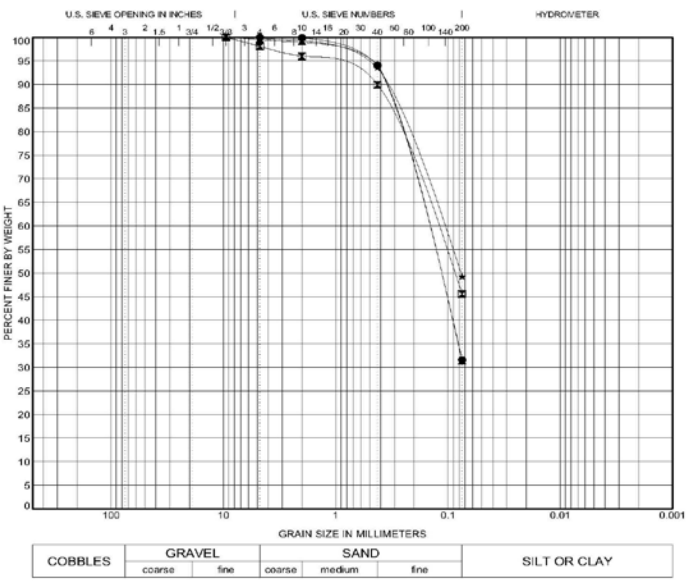
GENERAL NOTES ON SOIL CLASSIFICATION
GENERAL NOTES ON ROCK CLASSIFICATION

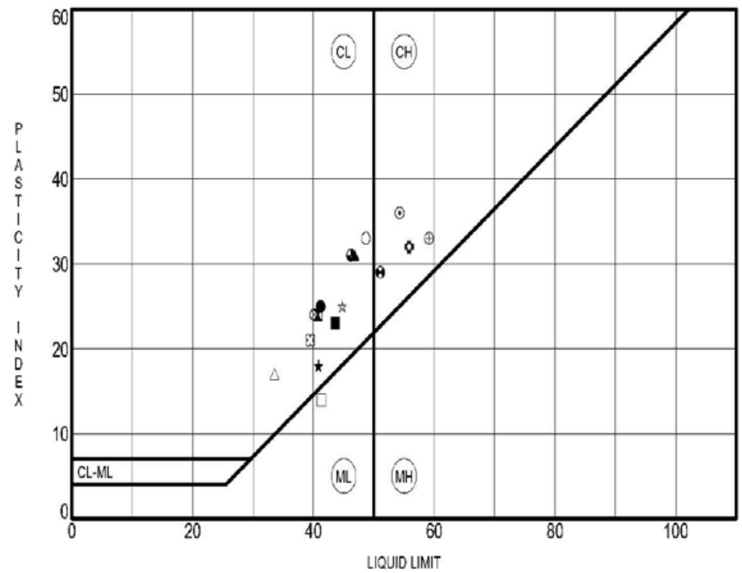
HINDERLITER
GEOTECHNICAL ENGINEERING

GENERAL NOTES ON SOIL CLASSIFICATION

Hinderliter Geotechnical Engineering classifies soils in accordance with the Unified Soil Classification System (USCS). In some cases, the AASHTO Classification System is also used.

USCS soil classifications are derived from soil grain size and material plasticity. Materials with more than 50 percent passing the No. 200 U.S. Sieve (aperture opening = 0.075 mm) are considered to be fine-grained soils (silts or clays). Materials with less than 50 percent passing the No. 200 sieve are considered to be coarse-grained soils (sands, gravels, etc). Coarse-grained soils are classified by the USCS System by plotting the Grain Size in Millimeters vs. Percent Finer by Weight. Depending on the grain size, the materials are classified as cobbles, gravel, sand, or silt / clay. Material plasticity is determined from the Liquid Limit test and the Plastic Limit test. The Liquid Limit (LL) of a soil is the point where, when mixed with water, a pat of soil transitions from a liquid state to a plastic state. The Plastic Limit (PL) is the point where the soil transitions from a plastic state to a solid state. The difference between the LL and PL is known as the Plasticity Index (PI).





Most naturally-occuring materials have some portion of fine-grained and coarse-grained materials. Modifiers are used to describe the relative percentage of minor-occurring materials in the following fashion:

Fine-Grained Soil Modifiers			Coarse-Grained Soil Modifiers	
Modifier	Percentage of Dry Weight		Modifier	Percentage of Dry Weight
Trace	< 15		Trace	< 5
With	15 - 29		With	5-12
Sandy, Gravelly, etc.	> 30		Silty, Clayey, etc.	> 12

The consistency of fine-grained soils and the relative density of coarse-grained soils is generally included on the boring logs as part of the material description. Consistency and relative density are generally defined as follows:

Fine-Grained Soils			Coarse-Grained Soils	
Unconfined Compressive Strength, Qu, psf	Consistency	Standard Penetration Test, N, blows / foot	Standard Penetration Test, N, blows / foot	Relative Density
< 500	Very Soft	< 2	0 - 3	Very Loose
500 - 1000	Soft	2 - 4	4 - 9	Loose
1000 - 2000	Medium	5 - 7	10 - 29	Medium Dense
2000 - 4000	Stiff	8 - 15	30 - 49	Dense
4000 - 8000	Very Stiff	16 - 30	50+	Very Dense
8000+	Hard	30+		

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General Notes on Soil Classification

GENERAL NOTES ON ROCK CLASSIFICATION

Sedimentary Rock Classification

Sedimentary rock is classified based on material composition, weathering and hardness. Depending on how samples are obtained, a measure of the degree of jointing can also be determined. Sedimentary rock is composed of clay, silt and/or sand sized particles and is often named based on the soil classification of the deposited material, such as sandstone or siltstone. Limestone, chert and shale are also sedimentary rock types.

Shale

In general, the reddish shales of western and central Oklahoma or Texas tend to be highly weathered and soft. They are composed of cemented clays but frequently contain lesser amounts of silt, sand or caliche. In eastern Oklahoma, Texas and Missouri the shales tend to be dark in color, usually gray, less weathered and harder.

Sandstone

Reddish sandstones in western and central Oklahoma and Texas tend to be highly weathered and soft. These sandstones often have relatively high clay or silt contents. Sandstones in eastern Oklahoma, Texas and Missouri tend to be brownish and hard. Sandstones may be described according to degree of cementation; well-cemented, cemented or poorly-cemented.

Limestone

Generally light colored and hard, limestone reacts readily with hydrochloric acid due to its calcium carbonate content.

Sedimentary rock can be evaluated by sampling and testing or by in-situ evaluation methods. Frequently, soft sedimentary rock is evaluated using penetration testing methods such as the split-barrel (SPT) method or through use of a Texas Cone (TC). Hard rock is often cored and evaluated by cutting or scratching, or by unconfined compressive strength measurements. In-situ methods, such as the Pressuremeter, can also be used.

SPT "N" Values (50 blows / 6" or less)	Hardness	Texas Cone "T" Values (100 blows / 6" or less)
50/6", 50/5"	Soft	100/3" or more
50/4", 50/3"	Moderately Hard	100/2", 100/1"
50/2" or less	Hard, often cored	100/1" or less

Sedimentary rock is generally cored in 5-foot or 10-foot increments or runs. Rock Core Recovery (R) is measured and expressed as a percentage of the total run. The Rock Quality Designation (RQD), defined as in-tact pieces of core 4 inches or more in length, is also measured and expressed as a percentage of the total core run.

RQD (%)	Empirical Quality
90-100	Excellent
75-90	Good
50-75	Fair
25-50	Poor
Below 25	Very Poor

Rock Core Hardness:

Soft - Can be broken by hand or carved with a knife.
Moderately Hard - Can be scratched with a penny.
Hard - Can be scratched with a knife.
Very Hard - Cannot be scratched with a knife.

Layering or Bedding:

Fissile - Splits along closely spaced planes 1/16" or less.
Thin Bedded - Beds 2 inches to 2 feet.
Thick Bedded - Beds 2 feet to 4 feet.
Massive - Beds greater than 4 feet.

Joints, Faults or Fractures:

Very Low Jointing - More than 6-1/2 feet between discontinuities.
Low Jointing - 2 feet to 6-1/2 feet.
Medium Jointing - 8 inches to 2 feet.
High Jointing - 2-1/2 inches to 8 inches.
Very High Jointing - Less than 2-1/2 inches.