OMES
OFFICE OF MANAGEMENT & ENTERPRISE SERVICES

Amendment of Solicitation

Date of Issuance: 02/12/20 **Solicitation No.** 3450004899

Requisition No. 3450032026 / 20-FM-0064 **Amendment No.** 2

Hour and date specified for receipt of offers is changed: ☒ No ☐ Yes, to: ______________ CST

Pursuant to OAC 260:115-7-30(d), this document shall serve as official notice of amendment to the solicitation identified above. Such notice is being provided to all suppliers to which the original solicitation was sent.

Suppliers submitting bids or quotations shall acknowledge receipt of this solicitation amendment prior to the hour and date specified in the solicitation as follows:

(1) Sign and return a copy of this amendment with the solicitation response being submitted; or,

(2) If the supplier has already submitted a response, this acknowledgement must be signed and returned prior to the solicitation deadline. All amendment acknowledgements submitted separately shall have the solicitation number and bid opening date printed clearly on the front of the envelope.

ISSUED BY and RETURN TO:

U.S. Postal Delivery:

Oklahoma Department of Transportation
Procurement Division Room 3C6
200 NE 21st Street
Oklahoma City, OK 73105

Personal or Common Carrier Delivery:

,OK -

Cheryl Emerson
Contracting Officer

405-522-3209
Phone Number

cemerson@odot.org
E-Mail Address

Description of Amendment:

a. This is to incorporate the following:

Section B9.1. of the Solicitation Package should read the following:

B.9.1. The construction period for this project is One Hundred Eighty (180) calendar days from the issuance of the Notice to Proceed. The Department reserves the right to assess a per day liquidated damages for each day that the Contractor exceeds the One Hundred Eighty (180) calendar days construction period as outlined in ODOT's 2009 Specification 108.09 and Table 108:1.

A copy of the Planholders' List is attached.

Questions and Answers submitted for this project.

The Following Sub-Contractors attended the Site Visit:

Drew Bodden, Pella of Oklahoma, 918-360-3581

Roy Layes – B & L Heat & Air Cond. Inc. – 405-372-8140

Eric Stehm – Frazier Fire, LLC-0918-260-8965

Blaine Menard – Total Demolition – 405-562-2994

Kerry Meek – Accel Electric & Lighting – 405-230-0487

b. All other terms and conditions remain unchanged.

Supplier Company Name (**PRINT**)	Date

Authorized Representative Name (**PRINT**)	Title	Authorized Representative Signature

Solicitation # 3450004899

Planholder's List

Company Name	Contact Name	Phone #	Email Address	City/State/Zip	P or S
Voy Construction	Andy Obrochta	918-994-1160Ex. 2	andy@voyconstruction.com	Tulsa, OK	P
Callahan Steel Buildings	Darin Snow	405-376-4949	dsnow@callahansteel.com	Mustang, OK 73064	P
Magnum Construction	Misti Neufled	918-251-8667	MNeufeld@magnumconstruction.com	Broken Arrow, OK 74013	P
Lambert Construction	Donna Abbott	405-372-1444	donna@lambertconstructionco.com	Stillwater, OK	P
Dennard Construction	Kelly Denard	405-740-2021	jonkdennard@outlook.com	Oklahoma City, OK	P
Downey Contracting, LLC	Dalton McCaskill	405-478-5277	dmccaskill@downeycontracting.com dfuksa@downeycontracting.com	Oklahoma City, OK	P
W L McNatt & Co	Jason Masterson	405-232-7245	jason@wlmcnatt.com	Oklahoma City, OK	P
Rick Scott Construction Inc.		580-762-7027	estimating@rickscottconstruction.com	Ponca City, OK 74601	P
Lopp Construction, LLC	Seth Welliver	405-762-2335	seth@loppconstruction.com	Stillwater, OK 74076	P
Anderson & House Inc.	Cory Muis	405-232-1188	cmuir@ahinc.net	OKC, OK 73106	P
Hoey Construction Company	Philip Goddard	918-665-2624	pgoddard@hoeyconstruction.com	Tulsa, OK 74107	P
DealCorp Solutions Contractor	Tim Deal	405-774-0232	tim@dealcorpokc.com	Norman, OK 73069	P
Bronze Oak, LLC	Jeanie Jo Jackson	918-364-5505	jjackson@bronzeoakgc.com	Broken Arrow, OK 74014	P
Cavins Construction Group	Tiffany Flying Out	405-573-3048	tiffany@cavinsconstruction.com	Norman, OK 73069	P

Solicitation: 3450004899 / 20-FM-0064
Stillwater Residency Addition

QUESTIONS AND ANSWERS:

Question 1: **Since the existing building at the ODOT in Stillwater has a R panel roof, please remove the weather tightness warranty requirement from the specifications. It is an industry standard that metal building manufacturers do not provide a weather tightness warranty for a exposed fastener roof panel.**

Please remove this requirement from the specifications.

Answer: Requirement may be omitted.

Question 2: **Will the flagpole be reused?**

Answer: I don't believe so. Arch references spec 10 7500. I'm assuming that means new. Plus, there was only one existing. Two are proposed.

Question 3: **Is ODOT expecting to reuse any parking lot lights that are to be taken out?**

Answer: No, we are calling for one new site light pole in a new location.

Question 4: **What will be the turn around time for submittals.**

Answer: Two weeks

Question 5: **Will the awarded Contactor be required to reimburse Studio Architect for the cost of the State Fire Marshal Permit?**

Answer: Yes. Due to the lengthy time that it tends to takes to acquire a State Fire Marshal Permit, Studio Architect initiated the process and the permit is in place. The cost of the permit to be reimbursed is $613.78.

Question 6: **I am wanting to see of you could help me with some clarification on the electrical. Sheet E3.1 shows the new sub panel (A) being fed from the existing MDP (E) with a new 100A breaker.**

The wiring from the MDP to the new sub panel specified under the "feeder schedule" on sheet E3.1 is only rated at 70 Amps. Can you get me clarification that they do want a 100A breaker feeding the 70A service wire or does it need to be a 70A breaker in the MDP?

Answer: New breaker in existing MDP needs to be a 70A. The 70A feeder and 70A main breaker in panel A are correct as shown.

Question 7: **On the aluminum windows - the plans show a window with a center mullion. These window are rather small - do they want a fixed window with a center mullion in the window or just a solid insulated glass?**

Or are they trying to indicate a sliding window that opens?

Answer: New windows shall match existing operable sliding windows.

Question 8. **I've been going through the documents looking for a geotechnical report on the existing area. Can you please provide it?**

Answer: Attached.

Question 9. I am wanting to see of you could help me with some clarification on the electrical. Sheet E3.1 shows the new sub panel (A) being fed from the existing MDP (E) with a new 100A breaker.

The wiring from the MDP to the new sub panel specified under the "feeder schedule" on sheet E3.1 is only rated at 70 Amps. Can you get me clarification that they do want a 100A breaker feeding the 70A service wire or does it need to be a 70A breaker in the MDP?

Answer: **See response to Question 6.**

SUBSURFACE EXPLORATION AND GEOTECHNICAL ENGINEERING REPORT

PROPOSED RESIDENCY EXTENSION
W. AIRPORT ROAD & N. HUSBAND STREET

STILLWATER, OKLAHOMA

PROJECT NO. CEC-18-21

HINDERLITER
GEOTECHNICAL ENGINEERING

PREPARED FOR:

CEC CORPORATION
OKLAHOMA CITY

OCTOBER 23, 2018

HINDERLITER GEOTECHNICAL ENGINEERING

October 23, 2018

CEC Corporation
4555 West Memorial Road
Oklahoma City, OK 73142

Attn: Mr. Chris Snider, P.E., Assoc DBIA

Re: Subsurface Exploration & Geotechnical Engineering Report
Proposed Residency Extension
W. Airport Road & N. Husband Street
Stillwater, Oklahoma
HGE Project No. CEC-18-21

Dear Mr. Snider:

The Subsurface Exploration & Geotechnical Engineering Report has been completed for the proposed Division Four – Stillwater Residency Extension in Stillwater, Oklahoma. Our services and fee were detailed in HGE Project Scope & Fee Reference: Stillwater Residency Addition, dated November 16, 2017. Acceptance of the scope and fee and notice to proceed were provided by issuance of CEC Task Order No. 62, dated September 13, 2018.

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

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

Respectfully:
HINDERLITER GEOTECHNICAL ENGINEERING, LLC
Certificate of Authorization No. 5528, Expires 30-June-2019

LICENSED PROFESSIONAL ENGINEER
MARK
H.
HINDERLITER
10-23-2018
21327
OKLAHOMA

Mark H. Hinderliter, P.E.
Oklahoma No. 21327

P:\HGE\Reports\2018 Geo\October\CEC-18-21 Report

Copies: chris.snider@connectcec.com (pdf report & invoice)

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1.0 EXECUTIVE SUMMARY

The subsurface exploration and laboratory soil testing are complete for the proposed extension of the ODOT Residency building located at the intersection of W. Airport Road and N. Husband Street in Stillwater, Oklahoma. We understand a 1,400 square foot, single-story, pre-engineered metal addition will be constructed. The addition is expected to have an on-grade floor slab and no basement level construction. Minor parking lot work will also be conducted.

Exploration of the subsurface materials at the project site consisted of advancing two soil test borings to depths of 20 feet within the proposed addition footprint. An additional boring was advanced to a depth of 5 feet within the new paving area. The approximate locations of the borings are recorded in the lower right corner of the boring logs and are displayed on the boring location diagram, both included within Appendix A of this report. Soil samples obtained from the borings were returned to the laboratory for further evaluation.

The borings encountered approximately 7 inches of asphaltic cement concrete underlain by lean to fat clays and lean clays with varying amounts of sand. Soil color tended to be reddish-brown with some thin gray seams. Soils were soft to medium stiff near the surface, becoming stiff to very stiff below depths of 6 feet. At depths of 9 feet to 11 feet the subsurface materials transitioned to reddish-brown, hard, shaley lean clays which persisted to depths of 19 feet to 19-1/2 feet. Moderately hard, weathered shale was tagged at these depths which persisted to the boring termination depths of approximately 20 feet within borings B-1 and B-2. Boring B-3 was terminated at 5 feet. Subsurface geology appears best described as belonging to the Wellington-Admire (Pwa) unit.

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 plugged per OWRB requirements immediately after boring completion.

Based on the subsurface conditions encountered at the time of our investigation, shallow footing or drilled pier and grade beam foundations may be used to support the proposed building addition. On-grade floor slabs may be used with either option. However, due to the presence of moderately to highly plastic clay soils, over-excavation and replacement of a portion of the existing soils with low volume change soils will be required for support of on-grade slabs. This report provides recommendations concerning earthwork and the design and construction of on-grade floor slabs, foundations and pavements.

2.0 PROJECT DESCRIPTION

The existing ODOT Residency is located at 3613 N. Husband in Stillwater, Oklahoma. We understand a 1,400 square foot, single-story, pre-engineered metal addition will be constructed. The addition is expected to have an on-grade floor slab and no basement level construction. Minor parking lot work will also be conducted. Pavements may be either asphaltic cement concrete or Portland cement concrete.

3.0 SITE EXPLORATION

3.1 Boring Layout & Elevations

Exploration of the subsurface materials at the project site consisted of advancing two soil test borings to depths of approximately 20 feet within the footprint of the proposed building addition. One additional boring was advanced to 5 feet within the proposed parking area. The locations of the borings are recorded in the lower right corner of the boring logs and are displayed on the boring location diagram, both included in Appendix A of this report. Borings were located in the field from existing site features using a measuring wheel; right angles were estimated.

Elevations at the boring locations were determined using a common surveyor's level and grade rod. The sidewalk next to the building, located approximately as displayed on the boring location diagram, was used as an elevation benchmark and was assigned a relative elevation of 100 feet. Based on the benchmark, elevations at the boring locations were 98 feet and 99 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 logs.

3.2 Subsurface Investigation

A truck-mounted, CME-45 rotary drill rig outfitted with 4-inch solid augers was used to advance the boreholes. Representative soil samples were obtained using either the split-barrel or the thin-walled tube sampling procedure as detailed in ASTM D 1586 or ASTM D 1587. 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. The tube is carefully removed from the boring and the ends are capped and taped. The tube is returned to the laboratory for extrusion and testing of the relatively undisturbed soil sample.

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 boring was backfilled or grouted 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. A geotechnical engineer selected representative samples for further laboratory analysis. These tests were chosen to help the engineer classify the soils and to provide their engineering properties. These laboratory tests include Liquid and Plastic Limits (commonly referred to as Atterberg Limits) and Washed Sieve Analysis. Samples obtained within thin-walled tubes were subjected to density and unconfined compressive strength evaluations. 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 an estimated group symbol from the Unified Soil Classification System are provided next to their representative sample locations in the appropriate column of the boring logs. 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 logs. 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 results of these tests have been provided in the appropriate column of the boring logs. 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.

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 1140. 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 logs.

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.

5.0 FINDINGS & RECOMMENDATIONS

Moderately to highly plastic clay soils are present at this site. Recommendations are made herein to mitigate damage that could occur to floor slabs and foundations due to shrinking and swelling of the clay soils as changes in moisture content occur. However, even when these precautions are taken damage could still occur due to excessive pressures placed on the structure by swelling soils. Vegetation with a high demand for water should not be planted close to the buildings due to the potential for shrinkage of the highly plastic clays as the vegetation acquires the available moisture.

5.1 Existing Site Conditions

The proposed addition will extend west approximately 30 feet from the west side of the existing building. At the time of our investigation, that space was occupied by an entry way, sidewalk, a rock bed and asphaltic cement pavement. Existing grades appeared to fall gently from east to west. Surrounding properties were generally commercial, industrial or municipal.

5.2 Subsurface Conditions

The borings encountered approximately 7 inches of asphaltic cement concrete underlain by lean to fat clays and lean clays with varying amounts of sand. Soil color tended to be reddish-brown with some thin gray seams. Soils were soft to medium stiff near the surface, becoming stiff to very stiff below depths of 6 feet. At depths of 9 feet to 11 feet the subsurface materials transitioned to reddish-brown, hard, shaley lean clays which persisted to depths of 19 feet to 19-1/2 feet. Moderately hard, weathered shale was tagged at these depths which persisted to the boring termination depths of approximately 20 feet within borings B-1 and B-2. Boring B-3 was terminated at 5 feet. Subsurface geology appears best described as belonging to the Wellington-Admire (Pwa) unit.

Based on published reports[1], the Wellington-Admire unit consists of shale, sandstone, limestone and siltstone. Most of the unit is shale which is reddish colored and clayey to silty in texture. The total thickness of the unit is difficult to ascertain but is several hundred feet thick. The shales underlie valleys and slopes or form gently rolling hills.

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 20 feet of the ground surface and the reported depth of the geologic unit.

[1] 1967; Engineering Classification of Geologic Materials – Division Four; Oklahoma Department of Transportation

A graphic log of each boring is included 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.

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 plugged per OWRB requirements immediately after boring 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 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 extension should include removing the existing entry way including foundations, the sidewalk, rock garden and existing ACC pavements. Any other unsuitable materials encountered during construction operations should be removed.

After removing the recommended deleterious materials and making any required excavations, but before placing fill or paving, we recommend the exposed grade be proof-rolled to identify any soft or loose areas. Proof-rolling operations should be observed by a representative of the geotechnical engineer 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 and placed as recommended in Section 5.5 of this report).

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 to highly plastic clay soils for which significant volume changes due to variations in subgrade moisture content could occur. Typically, a building addition such as the one proposed for this site is designed to tolerate vertical floor slab movements of 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) approaching 1-3/4 inches. Therefore, the existing subgrade soils are considered to be unsuitable for direct support of floor slabs. We recommend over-excavating the existing subgrade soils to a depth of at least 18 inches and placing at least 30 inches of low volume change soil. If design grades will not allow for 30 inches of low volume change soil, over-excavations should be extended deeper. The following recommendations are provided to develop this low volume change soil zone beneath the slab.

After performing the required over-excavations, 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 30-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 lean, 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 6 inches and should be compacted to at least 95 percent of the material's maximum dry density. Fill soils should be placed at a moisture content within two percent of optimum (test method ASTM D 698). Additionally, the top 4 inches of the low volume change soil support zone should consist of ODOT Type A Aggregate Base, placed and compacted as recommended above.

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.

On-Grade Slab Subgrade Summary	
Remove	Entry way, foundations, sidewalk, rock garden, 7" ACC pavement
Over-Excavate	Minimum 18 inches depending on design grades
Proofroll & Repair	Entire over-excavated area
Scarify & Recompact	8" compact to 95% at optimum or above
Low Volume Change Zone	30" in 6-inch lifts compacted to 95% at ± 2%

5.6 Shallow Footing Foundations

Shallow footing foundations can be used to support the proposed residency addition. To provide protection against frost heave and shrinking or swelling of the subgrade soils due to moisture changes, footings should bear at a depth of at least 2-1/2 feet below the final adjacent subgrade elevation. Footings bearing within undisturbed, native soils or within tested and approved fill, a maximum net allowable bearing pressure of 1,500 psf can be used for foundation design. This is the pressure at the base of the foundation in excess of the adjacent overburden pressure and is calculated using a safety factor of approximately 2.4. A representative of the geotechnical engineer should be retained to evaluate that footings bear on soils suitable for the design bearing pressures 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 2/3 of the total value.

5.7 Drilled Pier & Grade Beam Foundations

A drilled pier and grade beam foundation system supported by a combination of end bearing and side resistance within the shaley lean clays could also be used to support the proposed building addition. Straight-shaft drilled piers should extend into the shaley lean clays encountered below elevation 90. For drilled piers penetrating at least 2 pier diameters of 3 feet, whichever is greater, below elevation 90, a maximum net allowable end bearing pressure of 12,500 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 800 psf for that portion of the pier that extends more than two pier diameters below elevation 90. 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.

Void forms beneath grade beams are not required if the base of the grade beams extend at least 30 inches below exterior, adjacent grade.

An earth auger could be used to penetrate the overburden soils. However, a rock bit may be required to penetrate the shaley clays. We anticipate that temporary casing will not 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 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-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. Based on our experience with similar soils, a CBR value of 3 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
Rigid Pavement	5" Portland Cement Concrete (3500 psi min.) 6" ODOT Type A Aggregate Base or 8" Stabilized Subgrade
Flexible Pavement	2" Type S4 ACC (PG 70-28 OK) 3" Type S3 ACC (PG 64-22 OK) 6" ODOT Type A Aggregate Base or 8" Stabilized Subgrade

Before aggregate base is placed or the subgrade soils are stabilized, the exposed 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 100 percent of the materials maximum dry density per test method AASHTO T 99 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 4 percent hydrated lime or 12 percent fly ash. The actual percentage of additive should be determined at the time of construction for the exposed subgrade soils. After final mixing of the additive and adjusting the moisture content of the mixture to within two percent of optimum, the material should be compacted to at least 100 percent of the maximum dry density as determined by test method AASHTO T 99.

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 lifts not exceeding 9 inches in loose thickness and 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 by the project owner or design engineer 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

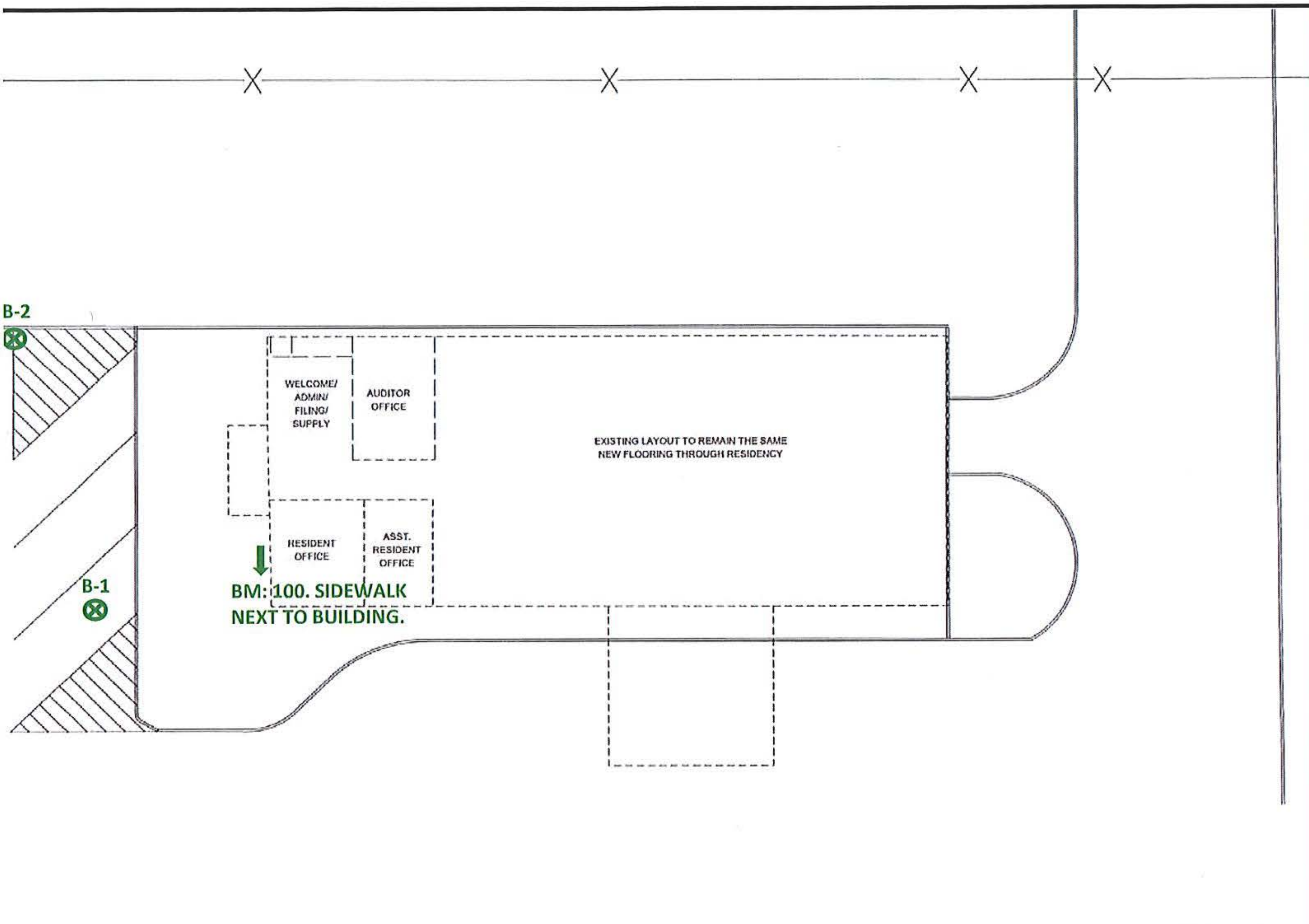
SITE VICINITY MAP
APPROXIMATE BORING LOCATIONS
SUBSURFACE FENCE DIAGRAM
BORING LOGS

P

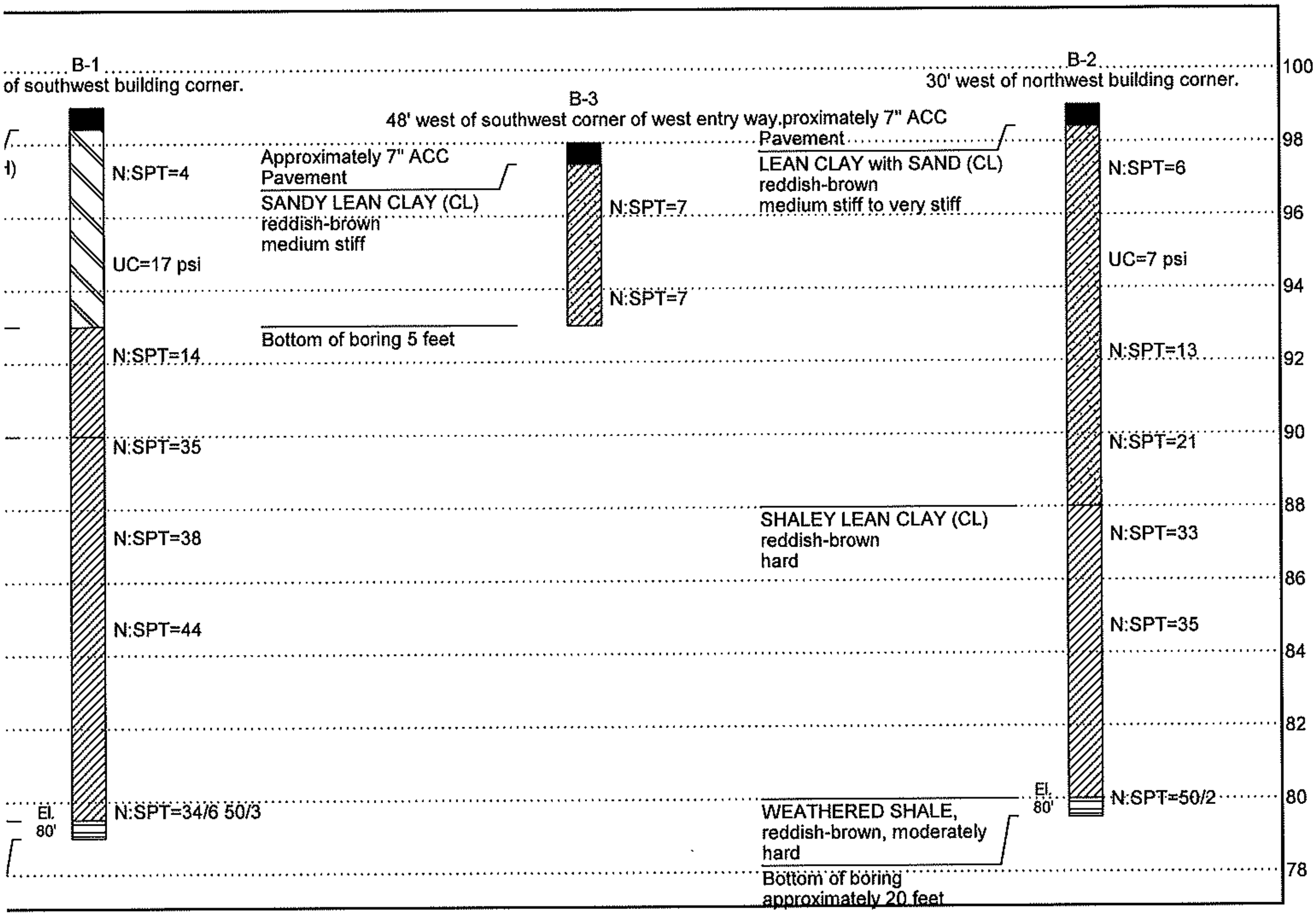
HINDERLITER GEOTECHNICAL ENGINEERING

STILLWATER RESIDENCY EXTENSION STILLWATER, OKLAHOMA

ENGINEER	MHH	SCALE	NTS
DRAWN BY	MHH	PROJECT NO.	CEC-18-21
REVISIONS		DATE	OCTOBER 2018



NG	HINDERLITER GEOTECHNICAL ENGINEERING	**STILLWATER RESIDENCY EXTENSION STILLWATER, OKLAHOMA**	ENGINEER	SCALE
			MHH	NTS
			DRAWN BY	PROJECT NO.
			MHH	CEC-18-21
			REVISIONS	DATE
				OCTOBER 2018



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942-4090
erGeo.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: Stillwater Residency Extension

Location: W. Airport Rd. & Husband, Stillwater, OK

Number: CEC-18-21

LOG OF BORING B-1

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

CLIENT: CEC Corporation
PROJECT: Stillwater Residency Extension
LOCATION: W. Airport Rd. & Husband, Stillwater, OK
NUMBER: CEC-18-21

DATE(S) DRILLED: 10/1/18

DRILLING METHOD(S):
Solid flight augers. SPT penetration testing & sampling.

GROUNDWATER INFORMATION:
No groundwater encountered.

SURFACE ELEVATION: 99

DEPTH (FT)	N: BLOWS/FT P: TONS/SQ FT T: BLOWS R: % RQD: %	MOISTURE CONTENT (%)	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
											Approximately 7" ACC Pavement
	N = 4 P = .5	18.9									LEAN TO FAT CLAY (CL-CH) reddish-brown soft to stiff
5	P = 1.0 UC=17 psi	21.1	48	15	33	105.9		100	98	85.8	
	N = 14 P = 4.25	14.7									LEAN CLAY (CL) reddish-brown with thin gray seams stiff
10	N = 35	13.0									SHALEY LEAN CLAY (CL) reddish-brown hard
	N = 38	15.3									
15	N = 44	11.2									
20	N = 34/6 50/3	11.8									WEATHERED SHALE, reddish-brown, moderately hard
											Bottom of boring 20 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:
18' west of southwest building corner.

LOG OF BORING B-2

HGE
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Oklahoma City, OK 73107
Telephone: (405) 942-4090
Website: HinderliterGeo.com

CLIENT: CEC Corporation
PROJECT: Stillwater Residency Extension
LOCATION: W. Airport Rd. & Husband, Stillwater, OK
NUMBER: CEC-18-21
DATE(S) DRILLED: 10/1/18

DRILLING METHOD(S):
Solid flight augers. SPT penetration testing & sampling.

GROUNDWATER INFORMATION:
No groundwater encountered.

SURFACE ELEVATION: 99

DEPTH (FT)	FIELD DATA (N: BLOWS/FT, P: TONS/SQ FT, T: BLOWS, R: %, RQD: %)	MOISTURE CONTENT (%)	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
											Approximately 7" ACC Pavement
	N = 6 P = 2.0	20.0									LEAN CLAY with SAND (CL) reddish-brown medium stiff to very stiff
	P = 1.5 UC=7 psi	18.0	38	14	24	109.3	100	99	92	76.2	
5											
	N = 13 P = 4.25	16.3									
	N = 21	15.4									
10											
	N = 33	16.8									SHALEY LEAN CLAY (CL) reddish-brown hard
	N = 35	11.7									
15											
	N = 50/2	8.4									WEATHERED SHALE, reddish-brown, moderately hard
											Bottom of boring approximately 20 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:
30' west of northwest building corner.

LOG OF BORING B-3

HGE
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CLIENT: CEC Corporation
PROJECT: Stillwater Residency Extension
LOCATION: W. Airport Rd. & Husband, Stillwater, OK
NUMBER: CEC-18-21
DATE(S) DRILLED: 10/1/18

DRILLING METHOD(S):
Solid flight augers. SPT penetration testing & sampling.

GROUNDWATER INFORMATION:
No groundwater encountered.

SURFACE ELEVATION: 98

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
													Approximately 7" ACC Pavement
		X	N = 7 P = 2.75	14.9	27	13	14			100	98	69.0	SANDY LEAN CLAY (CL) reddish-brown medium stiff
		X	N = 7 P = 2.0	18.8									
	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:
48' west of southwest corner of west entry way.

APPENDIX B

SUMMARY OF LABORATORY RESULTS
SOIL CLASSIFICATION

Borehole	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Size (mm)	%<#200 Sieve	Class-ification	Water Content (%)	Dry Density (pcf)	Satur-ation (%)	Void Ratio
B-1	1.0							18.9			
B-1	3.5	48	15	33	2	86	CL-CH	21.1	105.9	98	0.579
B-1	6.0							14.7			
B-1	8.5							13.0			
B-1	11.0							15.3			
B-1	13.5							11.2			
B-1	18.5							11.8			
B-2	1.0							20.0			
B-2	3.5	38	14	24	4.75	76	CL	18.0	109.3	91	0.530
B-2	6.0							16.3			
B-2	8.5							15.4			
B-2	11.0							16.8			
B-2	13.5							11.7			
B-2	18.5							8.4			
B-3	1.0	27	13	14	2	69	CL	14.9			
B-3	3.5							18.8			

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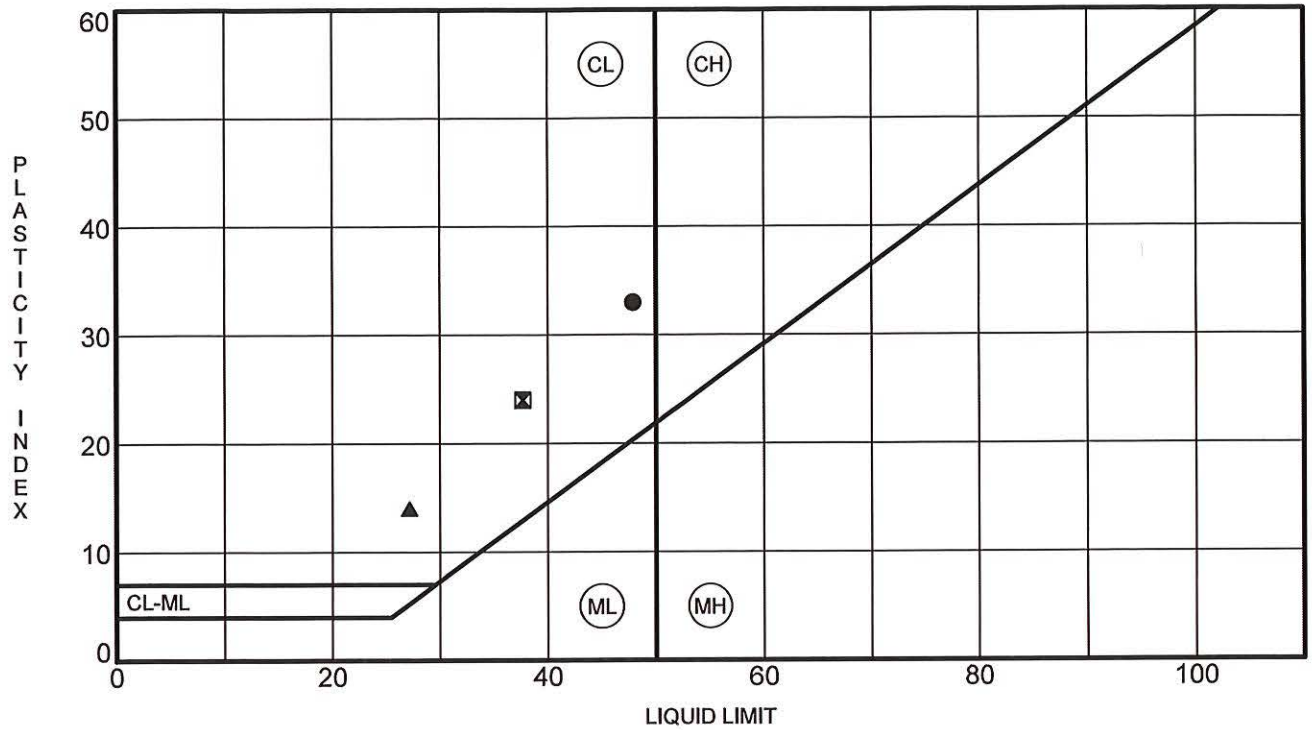
Website: HinderliterGeo.com

Summary of Laboratory Results

Project: Stillwater Residency Extension

Location: W. Airport Rd. & Husband, Stillwater, OK

Number: CEC-18-21



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**B-1**	**3.5**	**48**	**15**	**33**	**86**	**LEAN TO FAT CLAY(CL-CH)**
⊠	**B-2**	**3.5**	**38**	**14**	**24**	**76**	**LEAN CLAY with SAND(CL)**
▲	**B-3**	**1.0**	**27**	**13**	**14**	**69**	**SANDY LEAN CLAY(CL)**

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SOIL CLASSIFICATION

Project: Stillwater Residency Extension

Location: W. Airport Rd. & Husband, Stillwater, OK

Number: CEC-18-21

APPENDIX C

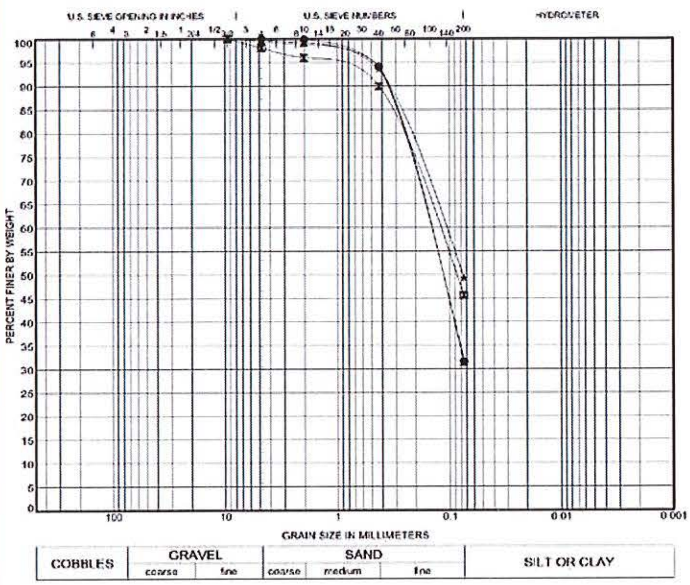
GENERAL NOTES ON SOIL CLASSIFICATION
GENERAL NOTES ON ROCK CLASSIFICATION

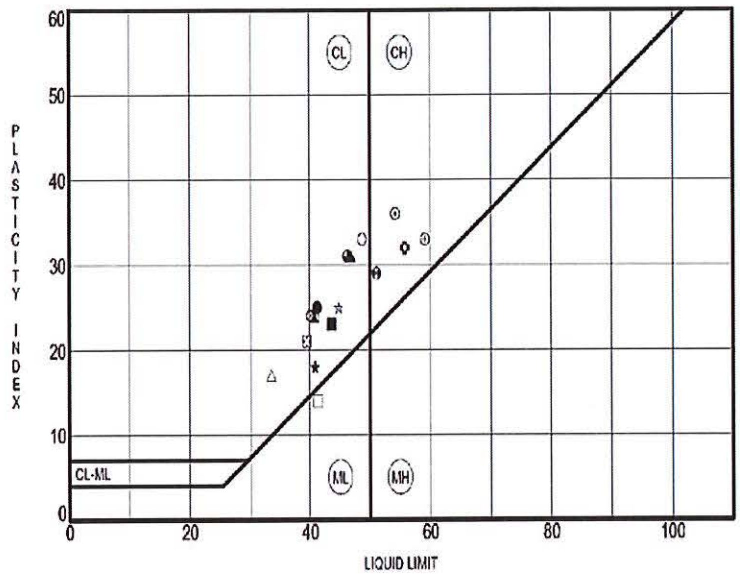
HINDERLITER
GEOTECHNICAL ENGINEERING

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	
Modifier	Percentage of Dry Weight
Trace	< 15
With	15 - 29
Sandy, Gravelly, etc.	> 30

Coarse-Grained Soil Modifiers	
Modifier	Percentage of Dry Weight
Trace	< 5
With	5-12
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

HINDERLITER
GEOTECHNICAL ENGINEERING

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.