GEOTECHNICAL ENGINEERING REPORT

CHEROKEE NATION OSU BUILDING

TAHLEQUAH, OKLAHOMA

Prepared for:

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45 South 4th Street
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Prepared by:

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PPI PROJECT NUMBER: 255932

January 30, 2019

PALMERTON & PARRISH, INC.

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January 30, 2019

Childers Architect
45 South 4th Street
Fort Smith, AR 72901

Attn: Mr. Shane Boren, AIA, LEED AP
Email: shane@childersarchitect.com

RE: Geotechnical Engineering Report
New Cherokee Nation OSU Building
Tahlequah, Oklahoma
PPI Project Number: 255932

Dear Mr. Boren:

Attached, please find the report summarizing the results of the Geotechnical Investigation conducted for the proposed new Cherokee Nation OSU Building in Tahlequah, Oklahoma. We appreciate this opportunity to be of service. If you have any questions, please don't hesitate to contact this office.

PALMERTON & PARRISH, INC.
By:

Brandon R. Parrish, P.E.
Vice-President

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PALMERTON & PARRISH, INC.
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BRP/BRP/jrh

PALMERTON & PARRISH, INC. INC.

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EXECUTIVE SUMMARY

A Geotechnical Investigation was performed at the site planned for construction of the new Cherokee Nation Oklahoma State University (OSU) Building located to the west of S. Bliss Avenue on the existing W.W. Hastings Hospital campus in Tahlequah, Oklahoma. This project is anticipated to include construction of a new building and access drive to the south. The new structure is anticipated to be three (3) to four (4) stories in height with a partial walk out basement to the south, steel framed, utilize a slab-on-grade floor system (north half) and a basement slab-on-grade (south half), exhibit light to moderate foundation loads, with a footprint measuring approximately 20,000 sq. ft. in plan view. Up to approximately 12 ft. of cut and 8 ft. of fill is anticipated within the building footprint on the south and north sides, respectively, to provide finish subgrade elevation. Based upon project plans, new pavement for a access drive will be located on the south side of the new building.

The existing Physical Therapy Building is located within the footprint of the proposed new structure. This building consists of a single-story, slab-on-grade structure and is anticipated to be demolished prior to construction of the new building.

Ten (10) borings were originally planned to be drilled, but Boring 4 was omitted due to the unknown location of buried utilities. A total of nine (9) sample borings were drilled within or adjacent to the proposed development footprint during the Geotechnical Investigation. All borings were discontinued in natural overburden soils or chert at depths ranging from 9.4 to 30 ft. below the existing ground surface.

Based upon the information obtained from the borings and subsequent laboratory testing, the site is suitable for construction of the proposed new Cherokee Nation OSU Building. Important geotechnical considerations for the project are summarized below. However, users of the information contained in the report must review the entire report for specific details pertinent to geotechnical design considerations.

- The project site primarily consists of the footprint of the existing Physical Therapy Building or grass covered lawn areas;

EXECUTIVE SUMMARY CONTINUED

- Existing fill depths ranging from 1 to 2.5 feet were encountered within the Borings drilled. Deeper existing fill depths, up to approximately 8 ft., are anticipated below the existing Physical Therapy building footprint based upon current site topography and existing finish floor elevation. However, most of the existing fill is anticipated to be removed during site grading;

- The existing fill consisted of chert gravels and sands or gravelly clays. **The origin and method of placement of the existing fill is unknown and for the purposes of this report should be considered uncontrolled;**

- Thin topsoil (~3 inches or less) was encountered with the majority of the borings;

- Overburden soils generally consisted of chert gravels and sands or gravelly clays with zones of solid chert, or chert boulders and cobbles, as typically found in the Tahlequah area. These soils were primarily logged as very stiff or very dense and exhibit significant drilling difficulty when using standard drilling methods;

- Voids underlying and within dense chert zones have been encountered at the project site, primarily along Hospital Drive and Visitors Drive located to the south and southeast of the proposed project site. Loss of drilling fluid return was noted within most borings drilled for the Cherokee Nation OSU Building, indicating fractures and possible voids within the chert stratum. These zones typically ranged from 0.5 to 1 ft. in thickness. However, large voids or caverns, were not noted during the subsurface exploration;

- Foundation loads for the new Cherokee Nation OSU Building may be supported upon shallow foundations bearing upon very stiff or dense natural overburden soils, or controlled fill. All existing fill below proposed new foundations/slabs should be removed and replaced. These recommendations are further discussed in Section 9.0 of this report;

EXECUTIVE SUMMARY CONTINUED

- Foundation loads may also be supported upon deep foundations bearing in very stiff/very dense natural overburden soils/chert or bedrock, although only **one (shallow or deep) foundation type** is recommended for the structure. Deep foundation recommendations are further discussed in Section 10.0 of this report. **However, due to the potential presence of voids within the subsurface and associated potential concrete loss during construction, shallow foundations are the recommended foundation alternate;**

- Due to the stiff and/or dense nature of the existing subgrade soils, sufficient support is anticipated to be provided for any slabs or pavements;

- The project site classifies as a Site Class C in accordance with Section 1613 of the 2015 International Building Code (IBC), as determined by shear wave velocity testing performed at the site during this investigation;

- Excavation and mass earth moving at this project site is anticipated to generally be difficult and variable. Excavation difficulty and rippability of the existing overburden soils at the site is further discussed in Section 8.6 of this report;

- Once basement excavation and undercutting of the existing fill has been performed, it is recommended that the building footprint be scanned using Ground Penetrating Radar (GPR) in search for large shallow subsurface voids. PPI has performed a similar investigation using GPR in the past to the south of the site with success; and

- Palmerton & Parrish, Inc. should be retained for construction observation and construction materials testing. Close monitoring of subgrade preparation work is considered critical to achieve adequate foundation and subgrade performance.

GEOTECHNICAL ENGINEERING REPORT

NEW CHEROKEE NATION OSU BUILDING

TAHLEQUAH, OKLAHOMA

1.0 INTRODUCTION

This is the report of the Geotechnical Investigation performed at the site planned for construction of the new Cherokee Nation OSU Building located west of S. Bliss Avenue on the existing W.W. Hastings Hospital campus in Tahlequah, Oklahoma. This investigation was authorized by a letter proposal prepared by Palmerton & Parrish, Inc. (PPI) dated January 7, 2019 and signed by Mr. Breck Childers, AIA, representing Childers Architect. The approximate site location is shown below for reference.

The purpose of the Geotechnical Investigation was to provide recommendations for foundation design and construction planning, and to aid in site development. Palmerton & Parrish Inc.'s (PPI) scope of services included field and laboratory investigation of the subsurface conditions in the vicinity of the proposed project site, engineering analysis of the collected data, development of recommendations for foundation design and construction planning, and preparation of this engineering report.

2.0 PROJECT DESCRIPTION

Item	Description
Site Layout	See Figure 1: Boring Location Plan
New Cherokee Nation OSU Building	• Three to four-stories in height; • Slab-on-grade (north half) & walk out basement (south half); • Steel framed; • Finish Floor Elevation (ground level) = 908.67; • Basement Elevation = 892.67; • Column loads ranging from 5 to 600 kips; • Wall loads ranging from 0.5 to 3 kips per foot; and • Measure approximately 20,000 sq. ft. in plan view.
Pavement	New pavement for an access drive is also anticipated at the south end of the proposed new structure at the walk out basement elevation.
Anticipated Grading	• 8 ft. fill – North Half; and • 12 ft. cut – South Half.
Retaining Wall	A below grade foundation (retaining) wall will be constructed along the east side (south half) and within the center of the structure separating the slab-on-grade and basement structure.

3.0 SITE DESCRIPTION

Item	Description
Latitude/Longitude (± Center of Project Site)	35.909978° / -94.951009°
Available Historic Aerial Photography	The north half of the existing Physical Therapy Building is believed to have been constructed in approximately 2007 with the southern addition constructed in 2011. The project site is believed to have consisted of grass/wooded areas since prior to construction of the existing Physical Therapy Building. The W.W. Hastings Hospital is believed to have been constructed around the early 1980's.
Current Ground Cover	Building or grass/gravel covered.
Existing Topography	Sloping to the southwest
Drainage Characteristics	Fair to Good.

4.0 BACKGROUND INFORMATION

PPI performed the geotechnical investigation for currently under construction Cherokee Nation Outpatient Health Clinic located to the east of the proposed project site. During this investigation, only minor voids at significant depth were noted in the borings drilled. However, during mass grading for the project, significant voids were noted to the south and southeast of the proposed project site during construction of Hospital and Visitors Drive.

5.0 SUBSURFACE INVESTIGATION

Subsurface conditions were originally planned to be investigated through completion of ten (10) subsurface borings and subsequent laboratory testing. However, one (1) boring was omitted as discussed below. In addition, shear wave velocity testing was also performed for seismic site classification purposes.

5.1 Subsurface Borings

All borings were located within or adjacent to the proposed structure footprint. As previously mentioned, the existing Physical Therapy Building is presently located at the site, which limited the area in which could be accessed during drilling. Borings were identified as Borings 1 through 10 and are shown on Figure 1: Boring Location Plan. Boring locations were selected by PPI based upon recommendations by the Design Team and adjusted to areas accessible by a drill rig. Boring 4 was not drilled due to the unknown location of buried utilities within this area.

Borings drilled were discontinued in chert or natural overburden soils at depths ranging from 9.4 to 30 ft. below the existing ground surface. The Oklahoma One-Call System, as well as hospital maintenance personnel, were notified prior to the investigation to assist in locating buried public and private utilities, respectively. Logs of the borings showing descriptions of soil and rock units encountered, as well as results of field and laboratory tests and a “Key to Symbols” are presented in Appendix I. Surface elevations for each boring are noted on each boring log. Surface elevations were surveyed in the field using the existing Physical Therapy

finish floor elevation as a benchmark and are anticipated to be within +/- 0.5 ft. of actual elevations.

Borings were drilled January 21 through 24, 2019 using a 3.625-inch tricone with wash rotary methods. All borings were drilled by an ATV-mounted CME-1050 drill-rig. Soil samples were collected at 2.5 to 5-ft. centers during drilling using a split spoon sampler while performing the Standard Penetration Test (SPT) in general accordance with ASTM D1586. Please refer to Appendix II for general notes regarding boring logs and additional soil sampling information.

5.2 Laboratory Testing

Collected samples were sealed and transported to the laboratory for further evaluation and visual examination. Laboratory soil testing included the following:

- Moisture Content (ASTM D2216);
- Grain Size Analysis (ASTM D6913);
- Atterberg Limits (ASTM D4318); and
- Pocket Penetrometers.

Laboratory test results are shown on each boring log in Appendix I and are summarized in the following table and grain size analysis results are also presented in Appendix III.

Boring	Depth (ft.)	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Moisture Content (%)	USCS Symbol	% Passing No. 200 Sieve
1	3	-	-	-	15.2	GC	18
2	3.5	-	-	-	19.1	GC	19
3	13.5	-	-	-	21.5	GC	25
6	13.5	86	26	60	40.0	CH	-
8	3.5	-	-	-	12.9	GC	27
9	18	-	-	-	19.2	SC	30
9	23	89	26	63	40.8	CH	-
10	8.5	-	-	-	-	GC	14
10	23.5	65	21	44	28.2	CH	-
Note: Sample classification sometimes differs from general strata description on the boring logs due to relatively small sample size & coarse nature of the strata. See individual boring logs for description of general strata.							

6.0 SITE GEOLOGY

According to the United States Geologic Survey's Geological Map of Oklahoma, the general site is underlain at depth by Mississippian Age deposits primarily of the Keokuk and Reeds Spring formation and the St. Joe Group. Within the site area, the primary rock type is chert with other rock types consisting of limestone and shale. Overburden soils at the site are typically residual having developed through chemical and physical weathering of the underlying parent bedrock, consisting primarily of chert fragments, boulders and clay layers. The boundary between overburden soils and relatively unweathered limestone is usually abrupt.

7.0 GENERAL SITE & SUBSURFACE CONDITIONS

Based upon subsurface conditions encountered within the borings drilled at the project site, generalized subsurface conditions are fairly consistent across the project site, and similar to typical overburden soils found within the Tahlequah area. Surficial materials primarily consist of thin (approximately 3-inches or less) topsoil, overlying very stiff to very dense chert laden lean or fat clays. Oftentimes the percentage of clay is less than 50 percent, and the soils classify as chert gravels or sands. Zones of relatively chert free very stiff fat clays were encountered, but are believed to be isolated. These conditions are presented on each boring log attached in Appendix I. Soil stratification lines on the boring logs indicate approximate boundary lines between different types of

soil and rock units based upon observations made during drilling. In-situ transitions between soil and some rock types are typically gradual.

7.1 Groundwater

Shallow groundwater was not noted within the borings on the date drilled. However, it should be noted that water-based drilling methods were used during field drilling. As a result, obtaining groundwater levels below a couple feet in depth was not possible. Groundwater levels should be expected to fluctuate with changes in site grading, precipitation, and regional groundwater levels. Groundwater may be encountered at shallower depths during wetter periods.

8.0 EARTHWORK

As previously mentioned, up to approximately 8 ft. of fill and 12 ft. of cut is anticipated within the north and south half of the proposed structure footprint, respectively, to provide finish subgrade elevations.

The initial phase of site preparation should include the following:

- Removal of the existing physical therapy building and any existing foundations or slabs within the proposed building footprint. In addition, clearing and grubbing of all vegetative matter should be performed within current lawn/landscape areas. All vegetative matter, including trees/root bulbs and topsoil should be removed from areas scheduled to receive new fill and/or slab/pavement construction;
- Topsoil/vegetative matter stripping on the order of 3-inches should be anticipated in grass covered areas. Topsoil should either be hauled off-site or stockpiled for reuse in lawn and landscape areas only;
- Much of the existing fill material is anticipated to be removed to achieve finish subgrade elevations at the project site. However, several feet of existing fill material is anticipated to be present near the center of the proposed structure, located on the north side of the proposed basement area. Any existing fill remaining after initial site grading should be removed, and properly replaced in accordance with Section 8.0 of this report; and

- Areas scheduled to receive controlled fill should be proof-rolled and approved in accordance with the following section of this report.

After the initial phase is complete, it is recommended that all building, pavement and undercut bottoms be proof-rolled to assure a stable subgrade. Proof-rolling consists essentially of rolling the ground surface with a fully loaded tandem axle dump truck or similar heavy rubber-tired construction equipment and noting any areas which rut or deflect during rolling. All soft subgrade areas, if any, identified during proof-rolling should be undercut and replaced with compacted fill as outlined below. Proof-rolling, undercutting and replacement should be monitored by a representative of PPI. **Although anticipated to be minimal, the depth and areal extent of undercutting soft subgrade areas will be largely dependent upon the time of year and related soil moisture conditions. If construction is initiated during or immediately following wetter months, the requirement for undercutting soft surficial soils below planned cut depths should be anticipated and reflected in the contract documents.**

After evaluation by proof-rolling and approval, the subgrade should be scarified to a depth of at least 8 inches, adjusted to within the optimum moisture content ranges and compacted to specified density, provided below (See Section 8.3). Placement of controlled fill may then proceed.

8.1 Fill Material Types

Fill Type[1]	USCS Classification	Acceptable Location for Placement
On-Site Soils / Imported Fill	Gravelly CL or CL-CH, GC, SC, SW or GW	All locations and elevations
Low Volume Change (LVC) Engineered Fill[2]	Gravelly CL, GC, or SC (LL < 50)	Within 2 ft. below bottom of slab elevation
On-Site Soils	CH[3]	Should **not** be placed within the upper 2 ft. beneath foundations, floor slabs and pavements.

1. Controlled, compacted fill should consist of approved materials that are free of organic matter and debris and contain maximum rock size of 12 inches, or the lift thickness, whichever is less. Any particles larger than 12 inches should be reduced in size to permit placement or removed from the fill embankment. Frozen material should not be used and fill should not be placed on a frozen subgrade. A sample of each material type should be submitted to the Geotechnical Engineer for evaluation prior to its use.
2. Low plasticity cohesive soil or granular soil having a liquid limit of less than 50%, contain at least 15% fines retained on the No. 200 sieve, and preapproved by the Geotechnical Engineer.
3. CH Clays with Liquid Limit equal to or above 50 are considered suitable for use as controlled fill only if the percentage of rock fragments exceeds 35% or if placed 2 ft. below shallow foundations, slab or pavement areas.

8.2 Acceptable LVC Material

LVC material is recommended within 2 ft below bottom of floor slab elevation. Potential sources of LVC material are as follows:

- Import from an off-site borrow area complying with Table 8.1; and
- On-site soils, classifying as CL, SC or GC may be segregated during footing or floor slab undercutting procedures or general earthwork procedures.

Most soil types present at the project site classify as LVC fill material, except CH material. Topsoil strippings or material containing organics should not be used as LVC material.

8.3 Compaction Requirements

Item	Description
Subgrade Scarification Depth	At least 8 inches
Fill Lift Thickness	12-inches (loose) using the minimum compactor referenced below
Compaction Requirements[1]	Six (6) passes (3 each direction) minimum using a self-propelled vibratory compactor with a minimum drum diameter of 48-inches for granular soils, or 95% Standard Proctor Density (ASTM D698) for materials containing sufficient fines content.
Moisture Content	• ± 2% optimum moisture for CL, SC, GC, GW & SW Soil Types; and • 0 to 4% above optimum for CH Soil Types.
Field Density Testing Frequency (if material type allows)	• Building Areas – One (1) test every 2500 sq. ft. per fill lift; • Pavement Areas – One (1) test every 5000 sq. ft. per fill lift; and • No less than three (3) tests per each fill lift.

1. We recommend that engineered fill (including scarified compacted subgrade) be tested for moisture content and compaction during placement. Should the results of the in-place density tests indicate the specified moisture or compaction limits have not been met, the area represented by the test should be reworked and retested as required until the specified moisture and compaction requirements are achieved.

8.4 Site Drainage

Discharge from roof downspouts should be collected and diverted well away from the building perimeter. Rapid, efficient runoff away from the building should also be provided. In addition, landscaping requiring frequent watering should be prohibited adjacent to building foundations.

8.5 Excavations

Based upon the subsurface conditions encountered during this investigation, the on-site soils typically classify as Type B in accordance with OSHA regulations. Temporary excavations in soils classifying as Type B with a total height of less than 20 ft. should be cut no steeper than 1H:1V in accordance with OSHA guidelines. **Confirmation of soil classification during construction, as well as construction safety (including shoring, if required), is the responsibility of the contractor.**

8.6 Rippability

As mentioned throughout this report, the overburden soils at the project site primarily consist of very dense clayey gravels with chert cobbles and boulders with isolated areas consisting of clays with a reduced chert content. Significant difficulty was experienced when drilling the geotechnical borings within this chert laden material. Based upon this information, the overburden soils are anticipated to be rippable with dozers, but with difficulty. In addition, areas resistant to ripping consisting of large chert boulders, requiring other removal methods (pneumatic breakers) should be anticipated. The Earthwork Contractor should review the attached boring logs when assessing excavation difficulty at this site. Mass grading at this site is anticipated to occur at a slower rate as compared to sites where overburden soils are primarily fine grained (silts and clays).

8.7 Expansive Soils

Due to the overburden soils primarily consisting of clayey gravels and gravelly clays, significant shrink/swell behavior is not anticipated. If relatively chert free fat clay zones are encountered at footing bottom and finish subgrade elevation, they should be undercut 2 ft., or to gravelly clays/clayey gravels, whichever is shallower, and replaced with LVC fill material. Although isolated zones of fat clays were encountered during drilling, they are not the primary material anticipated within footing, floor slab and pavement subgrades. In any event, soil subgrades should not be allowed to become dry and dessicate prior to concrete placement.

8.8 Utility Trenches

New utility trenches servicing the new structures are anticipated to be required. These trenches are often times sources of moisture migration into the structure. A relatively impervious material (clay with little rock, etc.) should be placed within the utility trench, surrounding the utility immediately outside the structure to reduce the potential for moisture migration into the structure via utility trenches. The “trench plug” should extend out from the structure a minimum of 5 ft. horizontally, and be placed in a controlled manner in accordance with Section 8.3 above.

9.0 FOUNDATIONS

As previously mentioned, the new Cherokee Nation OSU Building is anticipated to exhibit light to moderate foundation loads (column loads ranging from approximately 5 to 600 kips with wall loads ranging from 0.5 to 3 kips per foot). Recommendations for both shallow foundations and deep foundations are provided in the following sections. Due to primarily dense/stiff consistency of the existing overburden soils, as well as the potential concrete loss during placement of deep foundation elements due to potential deeper subsurface voids, shallow foundations are the preferred foundation alternate. **Regardless, only one foundation type is recommended to reduce the potential for differential settlement.**

9.1 Shallow Foundations

Foundation loads at this project site may be supported upon stiff or dense natural overburden soils or controlled fill placed in accordance with Section 8.0 of this report, following removal and replacement of the existing fill within the building footprint. Recommendations for shallow foundation design and construction are provided in the following table.

Description	Column (Spread Footing)	Wall (Continuous Footing)
Net Allowable Bearing Pressure[1]	4,000 psf	3,500 psf
Minimum Dimensions	2.5 ft.	1.5 ft.
Recommended Bearing Depth (Natural Soils or Controlled Fill)[2]	Depth sufficient to achieve minimum frost protection	
Minimum Embedment Below Finished Grade for Frost Protection & Variation in Soil Moisture[3]	2.0 ft.	
Allowable Passive Pressure[4]	230 pcf (equivalent fluid pressure)	
Coefficient of Sliding Friction[5]	0.26 (natural soils or controlled fill)	

1. The recommended net allowable bearing pressure is the pressure in excess of the minimum surrounding overburden pressure at the footing base elevation. The recommended pressure considers that all unsuitable and/or soft or loose soils, if encountered, are undercut and replaced with tested and approved new engineered fill. Footing excavations should be free of loose and disturbed material, debris, and water when concrete is placed. **This bearing pressure assumes an existing fill material placed below foundations consist of properly placed clayey gravels or gravelly clays.**
2. PPI should be retaining to observe footing bottoms prior to placing concrete.
3. For perimeter footings and footings beneath unheated areas.
4. Allowable passive pressure value considers a Factor of Safety of about 2. Passive pressure value applies to undisturbed native clay or properly compacted fill. If formed footings are constructed, the space between the formed side of a footing and excavation sidewall should be cleaned of all loose material, debris, and water and backfilled with tested and approved fill compacted to at least 95% of the material's Standard Proctor dry density. Passive resistance should be neglected for the upper 2.5 ft. of the soil below the final adjacent grade due to strength loss from freeze/thaw and shrink/swell.
5. Coefficient of friction value is an allowable value assuming a Factor of Safety equal to approximately 2. This value is applicable for on-site clayey gravels and gravelly clays.

9.1.1 Uplift Capacity of Shallow Foundations

Resistance of shallow spread footings to uplift (U_p) may be based upon the dead weight of the concrete footing structure (W_C) and the weight of soil backfill contained in an inverted cone or pyramid directly above the footings (W_S). The following parameters may be used in design:

Description	Weights
Weight of Concrete (W_c)	150 pcf
Weight of Soil Resistance (W_s)	100 pcf

The base of the cone or pyramid should be the top of the footing and the pyramid or cone sides should form an angle of 30 degrees with the vertical. Allowable uplift capacity (U_p) should be computed as the lesser of the two (2) equations listed below:

$$U_P = (W_S/2.0) + (W_C/1.25) \text{ or } U_P = (W_S + W_C)/1.5$$

9.1.2 Construction Considerations for Shallow Foundations

It is essential that footing bottoms should not be allowed to become dry and desiccate prior to concrete placement to help reduce the potential for shrink/swell behavior. Footings should be clean and free of standing water, debris, and loose soil at the time of concrete placement. Footing/mat excavations should be observed by a representative of PPI prior to placement of reinforcing steel and concrete placement.

9.1.3 Ground Improvement

Due to the dense to very dense overburden soils at the project site, ground improvement using aggregate piers is not believed to be an economic foundation system. Due to the significant drilling difficulties associated with very dense and large size chert, installation of aggregate piers is anticipated to be costly, as well as time consuming and provide little increase in allowable bearing capacity.

9.2 Deep Foundations

Deep foundations are also considered a viable foundation alternate. Several methods of deep foundation support were evaluated for this site. However, due to site specific conditions such as deep bedrock, potential for deeper subsurface voids and very dense overburden soils consisting of chert that are resistant to typical drilling methods, only two (2) deep foundation alternates have been recommended. The two (2) deep foundation system alternates include:

- Drilled piers bearing in dense natural overburden soils/chert or limestone bedrock; or
- Predrilled driven piling bearing in dense natural overburden soils/chert or limestone bedrock.

9.2.1 Drilled Piers

Foundation recommendations for each alternate are provided in the following sections. As previously mentioned, bedrock (limestone) was not encountered within a depth of 30 ft. at the project site. Previous borings drilled by PPI for the adjacent Cherokee Nation Outpatient Health Center did not encounter limestone

within 50 and possibly greater than 90 ft. at the project site. Since limestone is not anticipated to be encountered within a practical depth, drilled piers bearing in dense natural overburden soils/chert are recommended, if utilized for building support. Based upon the borings drilled at the project site, a minimum drilled pier depth of at least 25 ft. is recommended. Drilled pier depth will also be dictated by the required compressive load at each drilled pier and the amount of skin friction utilized in the design. The following subsections provide drilled pier recommendations.

9.2.2 Drilled Pier Design Recommendations

Description	Value
Foundation Type	Straight shaft drilled piers
Bearing Material[1]	Dense to very dense natural clayey chert or very stiff cherty clay overburden soils
Minimum Pier Penetration	25 ft. below existing finish grade elevation
Maximum Net Allowable Bearing Pressure[2]	20 ksf (overburden soils)
Maximum Allowable Skin Friction – Axial Compression[3]	1.0 ksf (overburden soils)
Maximum Allowable Skin Friction – Uplift[4]	1.0 ksf (overburden soils)
Group Effects – Axial Capacity	Piers should be installed with a center-to-center spacing of at least three (3) pier diameters. Group effects can be neglected and the total capacity of the pier group taken as the sum of the individual per capacities, provided that the adjacent piers are spaced at least three (3) pier diameters (center-to-center).
Group Effects – Lateral Capacity	When piers are installed close together, the lateral capacity of the group is not equivalent to the lateral capacity of an isolated individual pier times the number of piers in the group. Only those piers that are unobstructed by the other piers in the direction of the force develop full capacity. For pier groups with a pier spacing of three (3) pier diameters center-to-center, a multiplier of 0.8 should be used for the lead row of piers, 0.4 for the 2nd row and 0.3 for the 3rd and subsequent rows. The efficiency of the pier group is dependent upon the pier layout in the group, but would typically be on the order of 75 percent of a single pier for a pier spacing of three (3) pier diameters. The pier group effect increases significantly for closer spacing, resulting in lower efficiency.
Minimum Shaft Diameter[5]	30-inches
Minimum Grade Beam Bearing Depth	24-inches below final exterior adjacent grade
Estimated Total Settlement	1-inch or less
Estimated Differential Settlement	½-inch or less

1. **Due to variations in the depth and quality of the dense to very dense overburden soils across the site, the Geotechnical Engineer or his representative should be present during pier drilling to verify that unsuitable bearing strata is <u>not</u> present within the pier bottom.**
2. This is the pressure at the base of the foundation in excess of the adjacent overburden pressure. The allowable bearing pressure has a Factor of Safety of approximately 3.
3. The allowable skin friction has a Factor of Safety of approximately 2.
4. The allowable skin friction values have a Factor of Safety of approximately 2.
5. Sufficient steel reinforcement should be placed to provide adequate structural integrity.

9.2.3 Lateral Loading for Drilled Piers

It is anticipated that resistance of the foundations to lateral loading and the associated lateral deflection will be evaluated using finite difference computer models based on the horizontal modulus of subgrade reaction (K_h). The following values may be used in the analysis for this site.

Please note that the table states to ignore lateral support for the depth of 0 to 1 pier diameter or 2.5 ft., whichever is shallower. This notation is intended to account for the fact that near surface soils are significantly disturbed during drilled shaft excavation, which generally reduces the lateral support provided. Designers should use their judgment and make an appropriate reduction of soil strength parameters in this zone.

Values summarized in the table below are based upon published correlations and field and laboratory data collected during this subsurface investigation. **Values shown below are ultimate values representative of in-situ soil properties, and do not include a Factor of Safety.** These values may be used to compute resistance to lateral loading of the overburden soils. The appropriate Factor of Safety should be chosen by the designer.

Pier Depth	Unit Weight (pcf)	Static K_h (pci)	Cyclic K_h (pci)	e_{50}
*0-1 Pier Diameter	Ignore	-	-	-
*1 Pier Diameter to Bottom of Pier	125	1000	400	0.005
*Lateral parameters for the upper 1 pier diameter, or 2.5 ft., whichever is shallower, should be ignored.				

The above values were measured or based upon published correlations with anticipated soil strength and classification tests. **PPI can perform a site/structure specific lateral loading analysis once foundation type and loading has been determined, if desired.**

9.2.4 Drilled Pier Construction Recommendations

Drilled piers should have a straight shaft and should be founded at least 25 ft. below the existing ground surface bearing in dense to very dense natural

overburden soils/chert. **Overburden soils/chert are considered very resistant to typical auger methods. In any event, the drilled pier contractor should anticipate the use of rock augers, rock core barrels and potentially down the hole hammers with a heavy-duty drill rig in order to excavate the drilled piers to a minimum depth of at least 25 ft.**

Based upon the results of this investigation, the drilled pier contractor should be prepared to mobilize casing due to potential caving gravel and boulder sidewalls. Casing may be extracted as the shaft concrete is placed. Drilled pier bottoms should be well cleaned of all loose soil and rock fragments at the time of concrete placement. No more than 2 to 3 inches of water should be present in the bottom of piers when concrete is introduced into the shaft. **The drilled pier contractor should also anticipate minor to moderate concrete loss in small voids/cracks within the boulders and cobbles within the overburden soils, and/or possible large voids. Concrete over-run related to sloughing or caving of the shaft sidewalls should also be anticipated.**

9.2.5 Drilled Pier Load Test

An on-site load test of a production drilled pier is not considered a requirement. Isolated piers or pier groups may encounter differing conditions as compared to this report. It is recommended that the contractor bid form include a cost to perform such a load test in the event differing subsurface conditions are encountered during drilled pier installation. Pier load tests, if required, should be performed in accordance with ASTM D1143 and ASTM D3689 for compressive and tensile capacity.

9.3 Driven Piles

Another deep foundation alternate considered applicable at the project site is driven piling. Design recommendations for driven H-Piles are presented in the table below.

Description	Value
Foundation Type[1]	Steel H Piles w/End Protection
Bearing Material	Dense to very dense or stiff natural overburden soils/chert
Minimum Pile Penetration[2]	25 ft. below existing ground surface
Allowable Pile Capacity – Axial Compression	If driven to practical refusal, the allowable stress of the pile cross section controls the pile capacity. Compressive stress developed in the steel section should not exceed 9 kips per square inch (ksi) for 36 ksi grade steel and 12.5 ksi for 50 ksi grade steel sections.
Allowable Skin Friction – Uplift[3]	0.5 ksf (overburden soils)
Group Effects – Axial Capacity	Driven piles should be installed with a center-to-center spacing of at least three (3) pile widths. Group effects can be neglected and the total capacity of the pile group taken as the sum of the individual pile capacities provided that adjacent piles are spaced at least three (3) pile widths (center-to-center). Design of the piling as structural members should be in accordance with applicable building codes.
Group Effects – Lateral Capacity	When piles are installed close together, the lateral capacity of the group is not equivalent to the lateral capacity of an isolated individual pile times the number of piles in that group. Only those piles that are unobstructed by the other piles in the direction of the force develop full capacity. For pile groups with a pile spacing of three (3) pile widths center-to-center, a multiplier of 0.8 should be used for the lead row of piles, 0.4 for the 2nd row, and 0.3 for the 3rd and consecutive rows. The efficiency of the pile group is dependent upon the pile layout in the group, but would typically be on the order of 75 percent of a single pile for a pile spacing of three (3) pile widths. The pile group effect increases significantly for closer spacing, resulting in a lower efficiency.
Minimum Pile Cap & Grade Beam Bearing	24-inches below final exterior adjacent grade.
Estimated Total Settlement	1-inch or less
Estimated Differential Settlement	½-inch or less

1. Because of the relatively high driving resistance expected from the overburden soils/chert, steel H-piles with end protection are recommended so that the anticipated high driving stresses can be endured. Driven piles will develop their capacity from end bearing and side resistance in the very dense overburden soils below the pre-bore depth.
2. The pile should be driven to practical refusal, which should occur after penetrations of 1 to several feet into very dense overburden soils below the minimum 25 ft. of depth. We recommend that the pile installation be monitored by a representative of PPI.
3. The allowable skin friction has a Factor of Safety of approximately 2 and applies to the non-pre-bored depth only. Skin friction within the pre-bore depth should be ignored.

9.3.1 H-Pile Driving Criteria & Pre-Boring

Specifications for end bearing H-Piles should clearly state that end-bearing piles should be driven to refusal. Prior to driving structural steel piles, the contractor should review the boring logs to determine the depth at which impenetrable overburden soils may be anticipated. In addition, the contractor should submit a hammer wave equation to be evaluated and used during PDA testing (see below). The contractor should be attentive to the physical conditions associated with pile refusal. Pile refusal should be determined by on-site PDA testing. Pile refusal depth is anticipated to be highly variable. Pile refusal is anticipated to occur within approximately 5 ft. or less below the prebore depth due to very dense gravels encountered within the borings.

As stated above, pile driving refusal should be defined during PDA testing with an approved hammer. An approved hammer shall be defined as a hammer that develops the minimum hammer energy that is no less than any of the following:

1. 3.0 ft-lb/lb times the total pile weight in pounds, including mandrel, if used;
2. 32 ft-lb/kip times the minimum nominal axial compressive resistance in kips, divided by the pile batter factor, β, if applicable; and
3. 8,000 ft-lb.

In order to achieve full pile development and to ensure the pile reaches the intended very dense bearing stratum, pre-boring pile locations to a minimum depth of 25 ft. minimum is recommended. The pre-bored hole may be filled with sand prior to or following pile driving.

9.3.2 Driven Pile Construction Observation & PDA Testing

Construction surveillance activities should be provided throughout pile installation. Specific information regarding pile driving should be maintained in daily log form. The daily log form should include hammer type, energy, operating characteristics, driving time, delays, and other pertinent information. Complete pile driving records should be kept for the Project. Care should be exercised to monitor pile hammer operation to verify actual hammer energy.

In addition, PDA Testing (or dynamic load testing using a Pile Driving Analyzer) is recommended to confirm that damage to the pile has not occurred during driving, **and that the pile will carry the design load**. It is possible for piles to be driven down the side of a large chert boulder, resulting in pile deflection and subsequent damage. PDA testing would be especially useful in this case. A minimum of five (5) PDA tests or piles within a footprint of 50,000 sq. ft. is recommended spread over the structure footprint prior to production pile installation. A firm that has significant experience in PDA testing and that PPI has significant work experience with is listed below for your use, if desired.

Foundation Testing & Consulting, LLC
Mr. Casey Jones, P.E., P.G. - President
16500 Lucille Street
Overland Park, Kansas 66221
Ph: 913-626-8499
Email: cj@FTandC.com

9.3.3 Lateral Loadings for Driven Piles

The lateral loading parameters provided in Section 9.2.3 above may be used during foundation design utilizing driven piling.

9.4 Settlement Potential

Due to the overburden soils primarily consisting of dense to very dense chert sands, gravels, and occasional gravelly clays, settlement potential of the natural overburden soils is anticipated to be minimal. To essentially eliminate the potential for foundation settlement, foundations should bear in bedrock. However, due to the deep depth of limestone bedrock anticipated at this site, bearing upon bedrock is not considered practical. If shallow or deep foundations are constructed using the above foundation design parameters provided, total settlements on the order of 1-inch or less and differential settlements on the order of 0.5-inches or less are anticipated.

10.0 SEISMIC CONSIDERATIONS

Code Used	Site Classification
2015 International Building Code (IBC)[1]	C[2]

1. In general accordance with the *2015 International Building Code*, Section 1613
2. Based upon an average Shear Wave Velocity of 1,772 feet per second within the top 100 ft. of depth computed during site shear wave velocity testing performed on 1/15/19 by PPI.

According to the 2015 IBC, the Mapped Spectral Response Acceleration parameters for short period (F_a) and the 1-second period (F_v) for the project site are presented below.

Mapped Spectral Response Parameter	F_a	F_v
Value	1.2	1.7

Values are based upon a Site Class C, S_s = 15.2%, S_1 = 8.1% using Tables 11.4 (1 & 2) from ASCE 7-10

The seismic site classification presented above was determined using shear wave velocity testing. Shear wave velocity testing was performed along one (1) array, or line, situated within the southeast corner of the proposed building footprint. Shear wave velocity testing was performed in substantial conformance with industry standards using surface seismic methods, more specifically Multi-Channel Analysis of Surface Waves (MASW).

Surface waves are a type of seismic wave whose propagation is confined to the near surface medium. The depth of subsurface penetration of a surface wave is directly proportional to its wavelength. In a non-homogeneous medium, surface waves are dispersive, meaning each wavelength has a characteristic velocity resulting from subsurface heterogeneities.

MASW Combined Active and Passive method was utilized to obtain the average shear wave velocity for the top 100 ft. (Vs 100) at the project site. This method was selected to increase the range of frequency to be analyzed therefore increasing the depth of investigation. Active method captures a dispersion curve at relatively higher frequencies than the Passive method. Combing the dispersion curves for each method allows for a more reliable identification of the fundamental mode dispersion curve utilized in

calculating the shear wave velocity. Please refer to Figure 3 for the graphical shear wave velocity vs. depth output.

11.0 FLOOR SLABS

A slab-on-fill floor system is considered appropriate at the **north half** of the project site based upon subsurface conditions encountered and future site grading. Listed below are key considerations for design purposes of the floor slab.

- Prior to placement of controlled fill, if any, natural soils should be scarified, moisture content adjusted and re-compacted in accordance with Sections 8.0 of this report;
- Any fat clays containing little to no sand/gravel content present at slab subgrade elevation, if present, should be undercut and replaced in accordance with Section 8.7 above; and
- Prior to slab placement, soil moisture should be adjusted and maintained within the parameters specified in Section 8.0 of this report.

Placement of 4 or more inches of compacted free-draining granular base course below slabs that are not below grade is recommended to limit moisture rise through slabs and to improve slab support, particularly at joints. An impervious moisture barrier consisting of 6-mil plastic sheeting or equivalent should be provided in accordance with the 2012 IBC. Use of a 10-mil vapor barrier is recommended below all slab areas with an intended use sensitive to slab moisture.

11.1 Modulus of Subgrade Reaction

The floor slab by be designed with the modulus of subgrade reaction presented in the table below.

Bearing Material	Bearing Material Thickness (inches)	Modulus of Subgrade Reaction (pci)
LVC Fill Material and Natural Soils	N/A	175
Dense Graded Aggregate Base	6	275
Dense Graded Aggregate Base	12	350
Dense Graded Aggregate Base	18	425

12.0 BELOW GRADE SLABS

All slabs that are below exterior grade are considered below grade slabs. This condition is anticipated within the south half of the project site within the basement area. **In addition, any elevator pits, recessed mats, floor depressions, etc., are considered below grade slabs and the following recommendations do apply to these areas.**

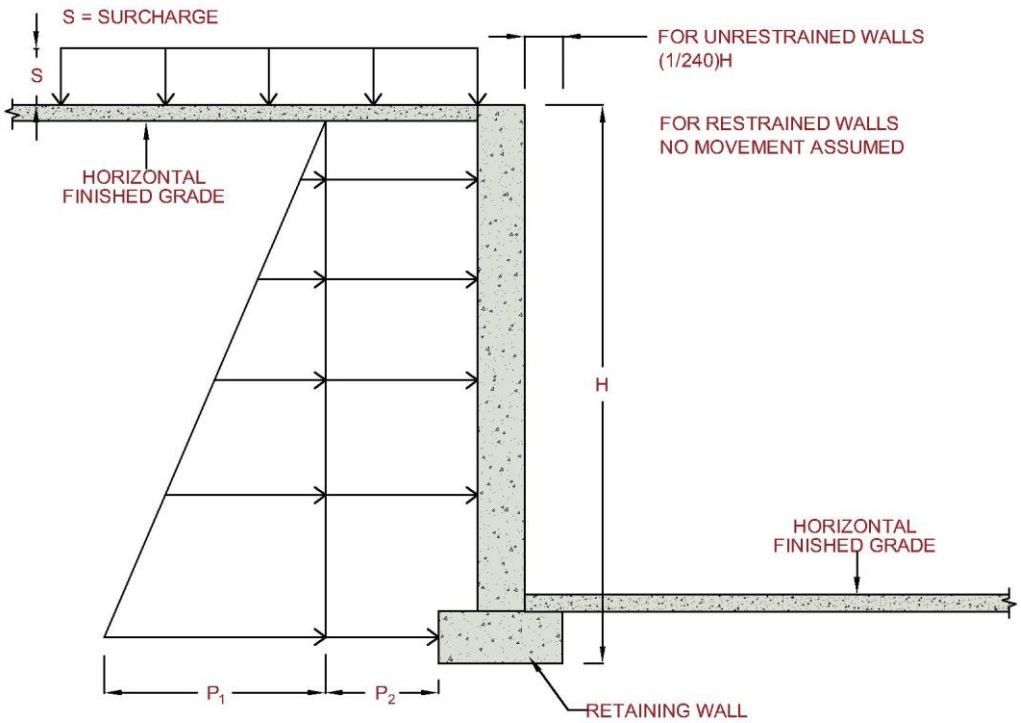
Although shallow groundwater was not encountered within the borings drilled, site earthwork can, and often does, manipulate the shallow groundwater regime. In view of the possibility for perched groundwater at the project site, it is recommended that any portions of the structure below exterior grade, as described above, be designed and constructed recognizing the possibility of shallow groundwater. A French drain system should be installed under the below grade floor slabs to limit hydrostatic pressure below the slab. A drainage system constructed with coarse free-draining gravel with a minimum 6-inch thickness and perforated pipes wrapped in filter fabric and installed on 30-ft. centers below the free draining gravel is considered adequate. Groundwater collected by these perforated pipe drains should be removed to free discharge by gravity flow. If gravity flow cannot be provided a sump and pump system consisting of a wet well with a duplex pump arrangement is recommended. At least one (1) pump should turn on when groundwater levels are more than 24-inches below finish floor elevation.

A French drain should be installed underneath all below grade slabs. Lateral drain pipes installed on 30-ft. centers should be at least 4-inches in diameter, with perimeter collector pipes at least 6-inches in diameter. An impervious moisture barrier consisting of 6-mil. plastic sheeting or equivalent should be provided below all slab areas. A minimum 10-mil plastic sheeting is recommended beneath all slab areas with an intended use sensitive to slab moisture. Soil moisture should not be allowed to dry and desiccate or be saturated by inundation prior to slab placement.

12.1 Retaining Wall Backfill & Drainage

A foundation drain is recommended to be installed around the portion of the perimeter where the below grade slab is at or below exterior grade level in

accordance with Section 1805 of the 2015 IBC. Below grade wall backfill should consist of free-draining crushed stone or alternatively, may consist of gravelly clays or clayey gravels. Crushed stone, if selected, must be imported from a quarry source whereas on-site soils suitable for wall backfill could probably be segregated and stockpiled during excavation. Depending upon the type of backfill selected and degree of wall restraint, the following table of lateral earth pressures are considered appropriate for wall design. **If a building floor slab is planned over the wall backfill, use of an imported free draining stone should be separated from the earth face of the excavation by using a nonwoven filter fabric.**



EQUIVALENT FLUID PRESSURES, P_1 (Drained Backfill Only)				
Type of Backfill	Level Backfill		Sloped Backfill (2H:1V)*	
	Restrained Walls (Using K_o)	Unrestrained Walls (Using K_a)	Restrained Walls (Using K_o)	Unrestrained Walls (Using K_a)
Compacted On-Site GC, GW, SC & CL Soils	70 pcf	45 pcf	80 pcf	55 pcf
Clean Crushed Stone	50 pcf	35 pcf	60 pcf	45 pcf
Rock Fill (Free-Draining)	50 pcf	35 pcf	60 pcf	45 pcf
*For backfill sloped other than 2H:1V, interpolate between values presented above for level and sloped backfill. NOTE: Structural design of unrestrained walls should permit wall rotation at top of wall equal to 1/240th of wall height.				

SURCHARGE PRESSURE, P_2		
Type of Backfill	Level Backfill	
	Restrained Walls (Using K_o)	Unrestrained Walls (Using K_a)
Compacted On-Site GC, GW, SC & CL Soils	0.58 (S)	0.38 (S)
Clean Crushed Stone	0.42 (S)	0.29 (S)
Rock Fill (Free-Draining)	0.42 (S)	0.29 (S)

If crushed stone backfill is selected and wall design in accordance with the above equivalent fluid pressures, the crushed stone backfill should be placed within a boundary projecting 30 degrees from the vertical commencing at a point 1 ft. out from the base of wall. Regardless of the backfill type selected, an impervious moisture barrier should be applied to the below grade wall. In addition, if lean clay backfill is selected, a geosynthetic drainage mat should be applied to the wall to assure removal of subsurface water. A perforated pipe should be laid at the base of wall to collect and remove subsurface water either from free-draining crushed stone or drainage mats. Flow line of the perforated pipe should be laid below partial basement finished floor. Again, groundwater collected should be removed by gravity

flow to free discharge. If this is not possible, groundwater may be removed by pumping. An exterior sump pit with dual pumping arrangement is recommended.

Please refer to Section 9.1 above for retaining wall foundation design parameters constructed in natural overburden soils or controlled fill material placed in accordance with Section 8.0 of this report.

13.0 SOIL CORROSIVITY

Bulk samples collected during drilling of previous borings adjacent to the site were tested for Oxidation Reduction Potential, Resistivity, Percent Solids, Sulfide, Chloride, Sulfate, Conductivity and pH were performed to determine corrosivity and resistivity of the soils at the project site. Results of this testing are presented in the table below:

Test	Results (2.5' to 6.5')	Method
Oxidation Reduction Potential (mV)	127	SM 2580 B-(2009)
Resistivity (ohm/cm)	3860	SM 2510 B-(1997)
Percent Solids (%)	89.1	SM 2540 G-(1997)
Sulfide	Absent	Commission Analytical Reactions
Chloride (mg/L)	Not Detected	EPA 300.0
Sulfate (mg/L)		EPA 300.0
Conductivity (µS/cm)	259	SM 2510 B-(1997)
pH (SU)	5.48	SM 4500-H+B-(2011)

Based upon the results of the corrosion and resistivity testing, the risk of sulfate and chloride exposure to concrete, reinforcing steel and other steel elements is minimal according to ACI guidelines. In general, the on-site soils are considered only slightly corrosive, mainly due to resistivity. Standard-of-practice regarding general protection against corrosion of buried metallic elements from slightly corrosive soils at this site is recommended. Based upon the above results, Type I or II cement is considered satisfactory for use at the project site.

14.0 PAVEMENT

It is anticipated that any new pavements associated with this project will be constructed of either an asphaltic concrete wearing surface placed over a base or a rigid Portland

Cement Concrete pavement over a granular base. Prior to pavement placement, preparation of the pavement subgrade should be performed in accordance with Section 8.0 of this report.

14.1 Flexible Pavement

If asphaltic paving is selected, the aggregate base may be a granular compacted crushed limestone with a gradation and quality conforming to the requirements of the Oklahoma Department of Transportation (ODOT), Standard Specification 703.01 for Type A aggregate. The maximum lift thickness for the granular base is 4 in. Granular base thicknesses in excess of 4 inches should be placed in multiple lifts with each lift being of approximately equal thickness. The granular base should be compacted to at least 100% of Standard Proctor Compaction (ASTM D-698).

Asphaltic concrete, both base and surface, should conform to the applicable requirements of ODOT Standard Specification 708. Asphaltic concrete should be compacted to 92 to 96% of Maximum Theoretical Specific Gravity (ASTM D-2041). Substitution of an appropriate Superpave Mix Design, SP 190C or SP 250C, can be used in place of the bituminous base. SP 190C or SP 125C may be used for the surface. All bituminous mix designs should have been prepared or verified within 6 months of the date of placement on the project.

14.2 Rigid Pavement

If rigid concrete paving is selected, a minimum 4-in. thickness granular base compacted to 100% of Standard Proctor should be placed on the prepared subgrade. The Portland Cement Concrete (PCC) mix should have a minimum 28-day compressive strength of 4000 pounds per square inch (psi). Concrete should be placed at a low slump (1 to 3 inches) and have an entrained air content of 5 to 7%. If an increased slump is desired, use of Super Plasticizer is recommended.

14.3 Pavement Subgrade CBR

Based upon the relatively high SPT-N values obtained during drilling, the natural soil deposits, as well as controlled fill originating from on-site should exhibit stiff

subgrades for pavement construction. A CBR value equal to 6.0 for the natural subgrade soils, or natural overburden soils that have been properly recompacted is recommended to be used in pavement design.

14.4 Pavement Thickness

Typical pavement design for this type of development would generally generate a Structural Number of 3.0 to 3.5 within heavy duty areas and 2.4 to 2.6 within light duty areas, depending on the subgrade conditions. The following table presents corresponding typical flexible and rigid pavement thickness using the general Structural Numbers.

Pavement Type	Anticipated Traffic Frequency	Asphaltic Surface (in.)	Asphaltic Base (in.)	Concrete Thickness (in.)	Aggregate Base (in.)
Flexible Pavement	Heavy Duty	3.0	4.0	-	6.0
	Medium Duty	2.0	3.0	-	6.0
	Light Duty	2.0	2.0	-	6.0
Rigid Pavement	Heavy Duty	-	-	7.0	4.0
	Medium Duty	-	-	6.0	4.0
	Light Duty	-	-	5.0	4.0

15.0 GROUND PENETRATING RADAR

As previously mentioned, subsurface voids have been documented immediately south and southeast of the project site encountered during construction of Hospital and Visitors Drive. In addition to the borings drilled during this investigation, performing ground penetrating radar (GPR) within the footprint of the proposed building footprint is recommended to potentially locate large shallow subsurface voids, if present. GPR should be performed following excavation within the building footprint as required to achieve proposed basement elevation and following removal of the existing fill material. PPI can perform these additional services if requested.

16.0 CONSTRUCTION OBSERVATION & TESTING

The construction process is an integral design component with respect to the geotechnical aspects of a project. Since geotechnical engineering is influenced by

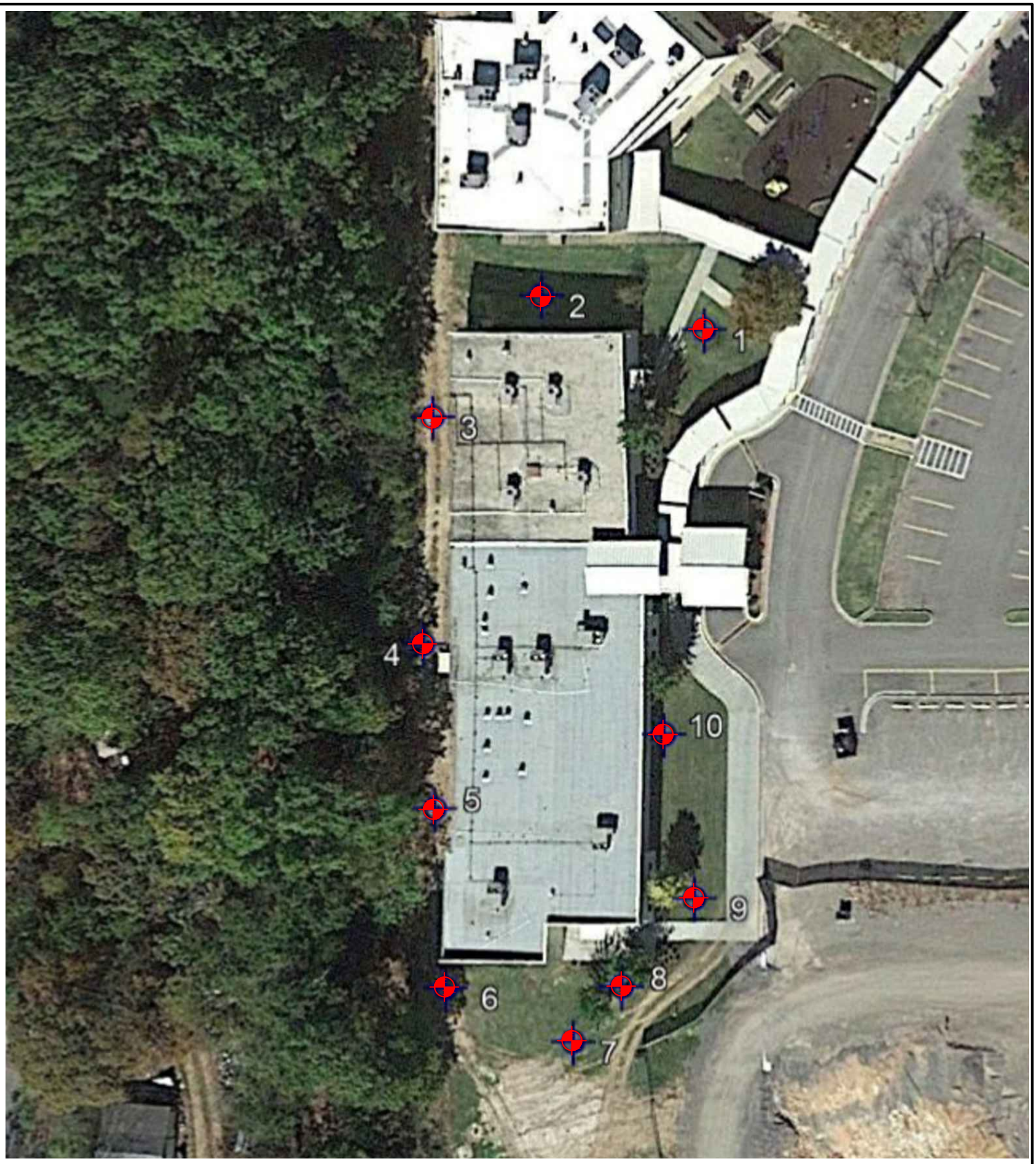
variable depositional and weathering processes and because we sample only a small portion of the soils affecting the performance of the proposed structures, unanticipated or changed conditions can be disclosed during grading. Proper geotechnical observation and testing during construction is imperative to allow the Geotechnical Engineer the opportunity to evaluate assumptions made during the design process. Therefore, we recommend that PPI be kept apprised of design modifications and construction schedule of the proposed project to observe compliance with the design concepts and geotechnical recommendations, and to allow design changes in the event that subsurface conditions or methods of construction differ from those assumed while completing this study. We recommend that during construction all earthwork be monitored by a representative of PPI, including site preparation, placement of all engineered fill and trench backfill, and all foundation excavations as outlined below.

- An experienced Geotechnical Engineer or Engineering Technician of PPI should observe the subgrade throughout the proposed project site immediately following stripping to evaluate the native clay, identify areas requiring additional undercutting, and evaluate the suitability of the exposed surface for fill placement;
- An experienced Engineering Technician of PPI should monitor and test all fill placed within the building and pavement areas to determine whether the type of material, moisture content, and degree of compaction are within recommended limits. **Refer to Section 8.3 for recommendations regarding Field Density (compaction) testing frequency**;
- An experienced Technician or Engineer of PPI should observe and test all footing excavations. Where unsuitable bearing conditions are observed, remedial procedures can be established in the field to avoid construction delays; and
- The condition of the subgrade should be evaluated immediately prior to construction of the building floor slabs to determine whether the moisture content and relative density of the subgrade soils are as recommended.

17.0 REPORT LIMITATIONS

This report has been prepared in accordance with generally accepted practices of other consultants undertaking similar studies at the same time and in the same geographical area. Palmerton & Parrish, Inc. observed that degree of care and skill generally exercised by other consultants under similar circumstances and conditions. Palmerton & Parrish's findings and conclusions must be considered not as scientific certainties, but as opinions based on our professional judgment concerning the significance of the data gathered during the course of this investigation. Other than this, no warranty is implied or intended.

FIGURES



Note: Boring 4 not drilled due to unknown location of buried utilities

LEGEND

Boring Location

NTS

Project: Cherokee Nation OSU Building - Tahlequah, Oklahoma
Client: Childers Architect

Boring Location Plan

DATE: January 30, 2019

Project Number: 255932

PALMERTON & PARRISH, INC.
GEOTECHNICAL AND MATERIALS ENGINEERS/
MATERIALS TESTING LABORATORIES / ENVIRONMENTAL SERVICES

FIGURE 1

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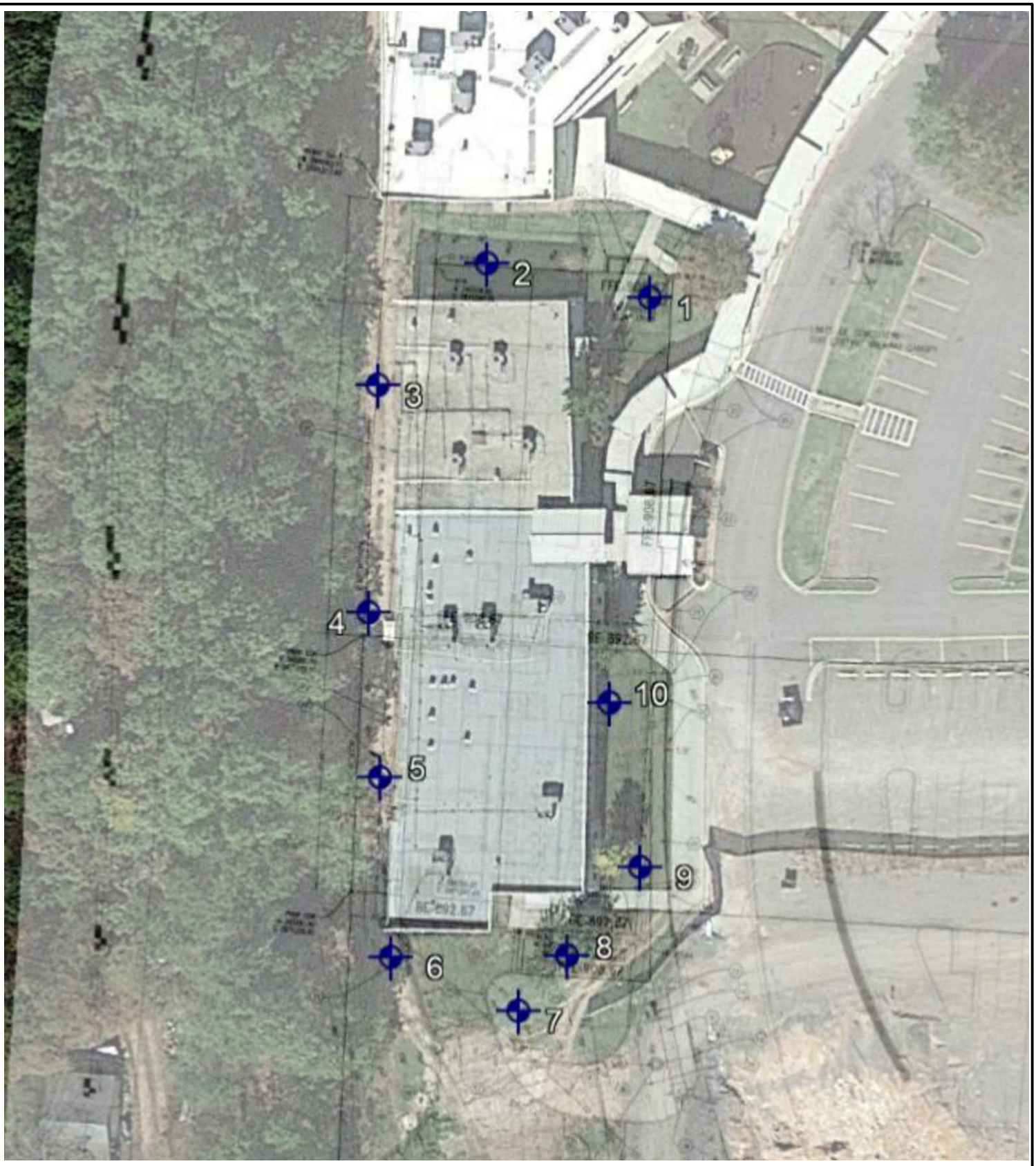

Note: Boring 4 not drilled due to unknown location of buried utilities

LEGEND

Boring Location

NTS

Project: Cherokee Nation OSU Building - Tahlequah, Oklahoma
Client: Childers Architect

Boring Location Plan - w/ Building Footprint

DATE: January 30, 2019

Project Number: 255932

PALMERTON & PARRISH, INC.
GEOTECHNICAL AND MATERIALS ENGINEERS/
MATERIALS TESTING LABORATORIES / ENVIRONMENTAL SERVICES

FIGURE 2

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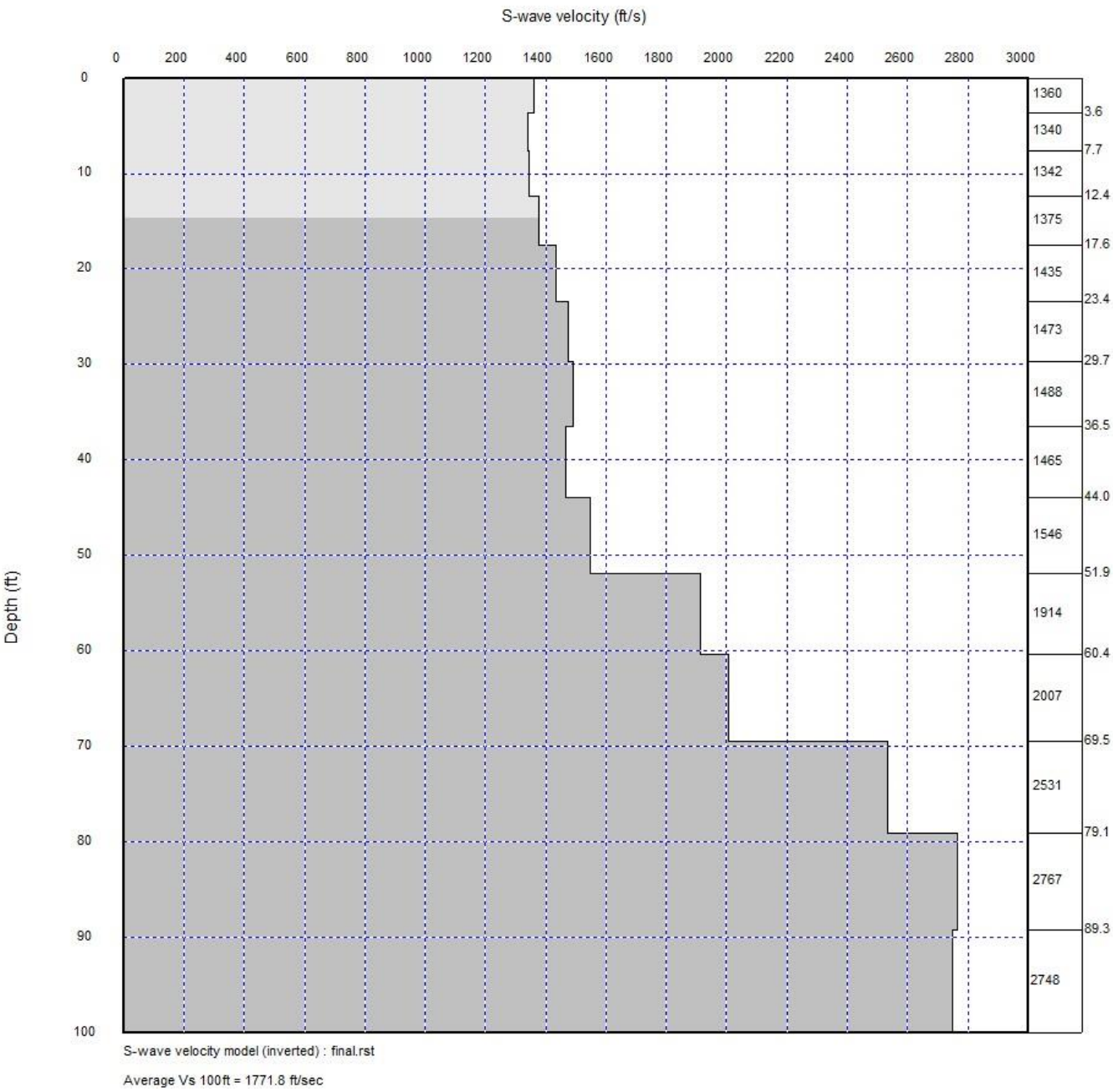
Figure 3 - 1D Shear Wave Velocity Profile



APPENDIX I

BORING LOGS & KEY TO SYMBOLS

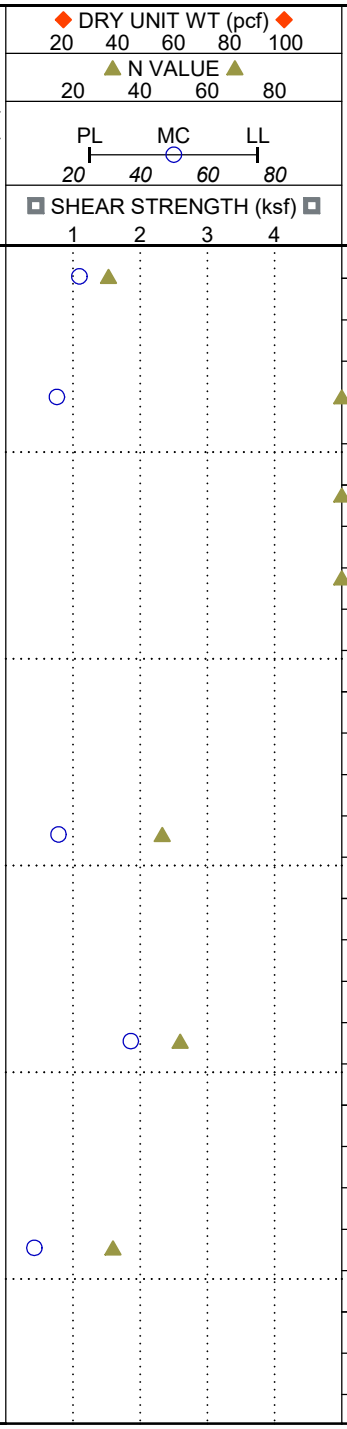
4168 W. Kearney St.
Springfield, MO
Telephone: (417) 864-6000
Fax: (417) 864-6004

GEOTECHNICAL BORING LOG

BORING NUMBER 1

CLIENT Childers Architect
PROJECT NAME Cherokee Nation OSU Building
PROJECT NO. 255932
PROJECT LOCATION Tahlequah, OK
DATE STARTED 1/23/19 **COMPLETED** 1/23/19
SURFACE ELEVATION 905.8 ft **BENCHMARK EL.**
DRILLER MR **DRILL RIG** CME 1050
GROUND WATER LEVELS
HAMMER TYPE Auto
AT TIME OF DRILLING None
LOGGED BY BC **CHECKED BY** BP
AT END OF DRILLING
NOTES

DEPTH (ft)	DRILLING METHOD	STRATA SYMBOL	MATERIAL DESCRIPTION (Unified Soil Classification System)	SAMPLE TYPE NUMBER	RECOVERY % (RQD %)	CORRECTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	ELEVATION (ft)
0	ROTARY - 3 5/8" Tricone		TOPSOIL, Brown, Medium Stiff, Moist, Grass Covered (0.3 ft)	SPT 1		13-15-16 (31)		905
			CLAYEY CHERT GRAVEL, w/ Sand, Brown Tan, Medium Dense to Very Dense, Moist (GC)	SPT 2		27-53-67/4"		
5			(5.5 ft) CHERT GRAVEL, COBBLES & BOULDERS, Brown, Tan & White, Very Dense, Slightly Moist	SPT 3		67/1"		900
				SPT 4		67/1"		
			-Area of Possible Voids (8.75' to 9.5'). Water Loss During Drilling. (9.5 ft)					
10			CLAYEY CHERT GRAVEL, w/ Sand & Cobbles, Brown Tan, Dense to Very Dense, Moist (GC)					895
				SPT 5		12-13-33 (46)		
15								890
				SPT 6		28-36-16 (52)		
20								885
				SPT 7		13-11-21 (32)		
25			(28.5 ft)					880



Bottom of borehole at 28.5 feet.

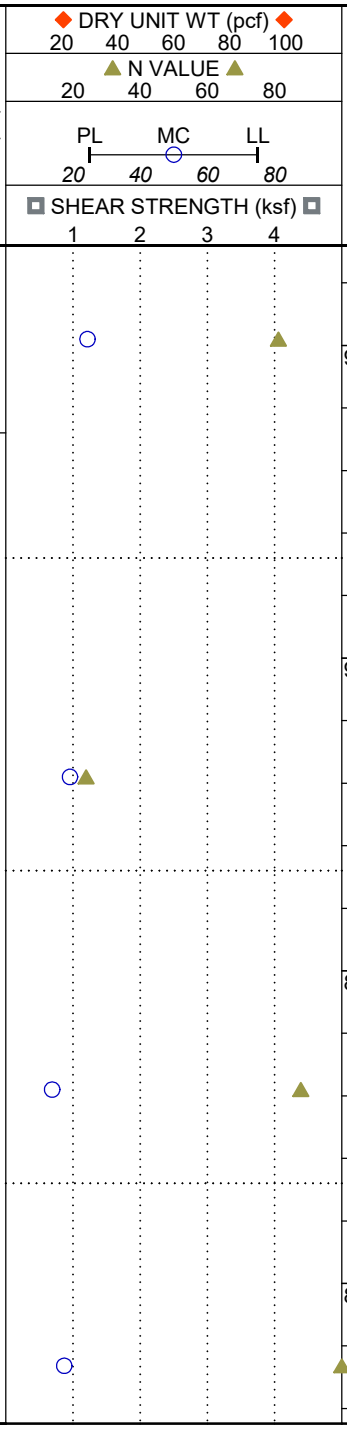
4168 W. Kearney St.
Springfield, MO
Telephone: (417) 864-6000
Fax: (417) 864-6004

GEOTECHNICAL BORING LOG

BORING NUMBER 2

CLIENT Childers Architect
PROJECT NAME Cherokee Nation OSU Building
PROJECT NO. 255932
PROJECT LOCATION Tahlequah, OK
DATE STARTED 1/24/19 **COMPLETED** 1/24/19
SURFACE ELEVATION 903.3 ft **BENCHMARK EL.**
DRILLER MR **DRILL RIG** CME 1050
GROUND WATER LEVELS
HAMMER TYPE Auto
AT TIME OF DRILLING None
LOGGED BY BC **CHECKED BY** BP
AT END OF DRILLING
NOTES

DEPTH (ft)	DRILLING METHOD	STRATA SYMBOL	MATERIAL DESCRIPTION (Unified Soil Classification System)	SAMPLE TYPE NUMBER	RECOVERY % (RQD %)	CORRECTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	ELEVATION (ft)
0.0	ROTARY - 3 5/8" Tricone		TOPSOIL, Brown, Medium Stiff, Moist, Grass Covered 0.5 ft					
			CLAYEY CHERT GRAVEL, w/ Sand & Cobbles, Red Brown Tan, Medium Dense to Very Dense, Moist (GC)	SPT 1		7-16-65 (81)	1	902.5
2.5								900.0
				SPT 2		20-12-12 (24)		
5.0			-Area of Possible Voids (6' to 7'). Water Loss During Drilling.					897.5
				SPT 3		40-47-41 (88)		
7.5			8.5 ft					895.0
			CHERT GRAVEL, COBBLES & BOULDERS, Brown, Tan & White, Very Dense, Slightly Moist 9.4 ft	SPT 4		5-67/5"		



Bottom of borehole at 9.4 feet.

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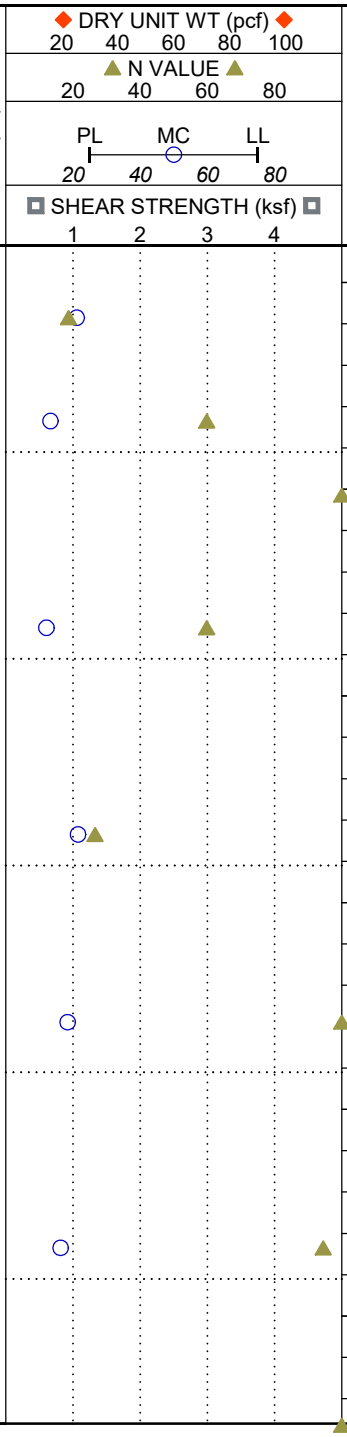
GEOTECHNICAL BORING LOG

BORING NUMBER 3

CLIENT Childers Architect
PROJECT NAME Cherokee Nation OSU Building
PROJECT NO. 255932
PROJECT LOCATION Tahlequah, OK
DATE STARTED 1/24/19 **COMPLETED** 1/24/19
SURFACE ELEVATION 900.9 ft **BENCHMARK EL.**
DRILLER MR **DRILL RIG** CME 1050
GROUND WATER LEVELS
HAMMER TYPE Auto
AT TIME OF DRILLING None
LOGGED BY BC **CHECKED BY** BP
AT END OF DRILLING
NOTES

DEPTH (ft)	DRILLING METHOD	STRATA SYMBOL	MATERIAL DESCRIPTION (Unified Soil Classification System)	SAMPLE TYPE NUMBER	RECOVERY % (RQD %)	CORRECTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	ELEVATION (ft)
0	ROTARY - 3 5/8" Tricone		FILL - GRAVELLY LEAN CLAY, Brown, Stiff, Moist (CL)	SPT 1		17-11-8 (19)		900
2.5 ft			CLAYEY CHERT GRAVEL, w/ Sand & Cobbles, Brown Tan, Medium Dense to Very Dense, Moist (GC)	SPT 2		13-23-37 (60)		
5.5 ft			CHERT GRAVEL, COBBLES & BOULDERS, w/ Occasional Stiff Fat Clay Layers, Red Brown, Tan & White, Dense to Very Dense, Moist	SPT 3		67/1"		895
5				SPT 4		31-28-32 (60)		
10			-Area of Possible Voids (12' to 12.5'). Water Loss During Drilling.	SPT 5		11-7-20 (27)		890
15				SPT 6		40-67/1"		885
20				SPT 7		33-48-47 (95)		880
25								875
28.5 ft			Bottom of borehole at 28.5 feet.	SPT 8		67/1"		



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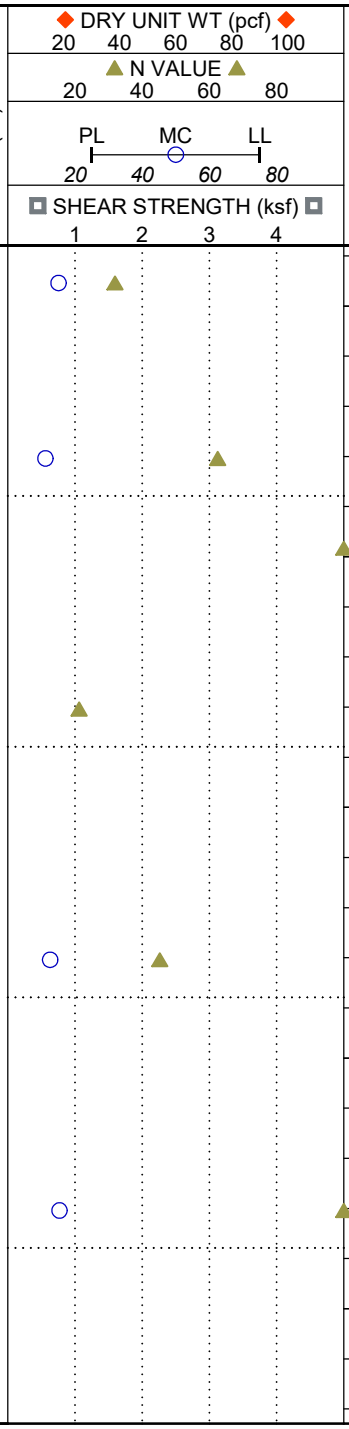
GEOTECHNICAL BORING LOG

BORING NUMBER 5

CLIENT Childers Architect
PROJECT NAME Cherokee Nation OSU Building
PROJECT NO. 255932
PROJECT LOCATION Tahlequah, OK
DATE STARTED 1/24/19 **COMPLETED** 1/24/19
SURFACE ELEVATION 896.21 ft **BENCHMARK EL.**
DRILLER MR **DRILL RIG** CME 1050
GROUND WATER LEVELS
HAMMER TYPE Auto
AT TIME OF DRILLING None
LOGGED BY BC **CHECKED BY** BP
AT END OF DRILLING
NOTES

DEPTH (ft)	DRILLING METHOD	STRATA SYMBOL	MATERIAL DESCRIPTION (Unified Soil Classification System)	SAMPLE TYPE NUMBER	RECOVERY % (RQD %)	CORRECTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	ELEVATION (ft)
0	ROTARY - 3 5/8" Tricone		FILL - CLAYEY CHERT GRAVEL, Red Tan, Dense Moist (GC) 1.0 ft	SPT 1		16-19-13 (32)		
			CLAYEY CHERT GRAVEL, w/ Sand & Cobbles, Brown Tan, Medium Dense to Very Dense, Moist (GC)					895
				SPT 2		29-27-36 (63)		
5				SPT 3		67/1"		890
				SPT 4		13-8-13 (21)		
10			-Area of Possible Voids (11.5' to 12.5'). Water Loss During Drilling.					885
				SPT 5		20-23-23 (46)		
15			17.0 ft					880
			CHERT GRAVEL, COBBLES & BOULDERS, w/ Occasional Stiff Fat Clay Layers, Red Brown, Tan & White, Dense to Very Dense, Moist	SPT 6		27-60-47 (107)		
20			23.5 ft					875

Bottom of borehole at 23.5 feet.



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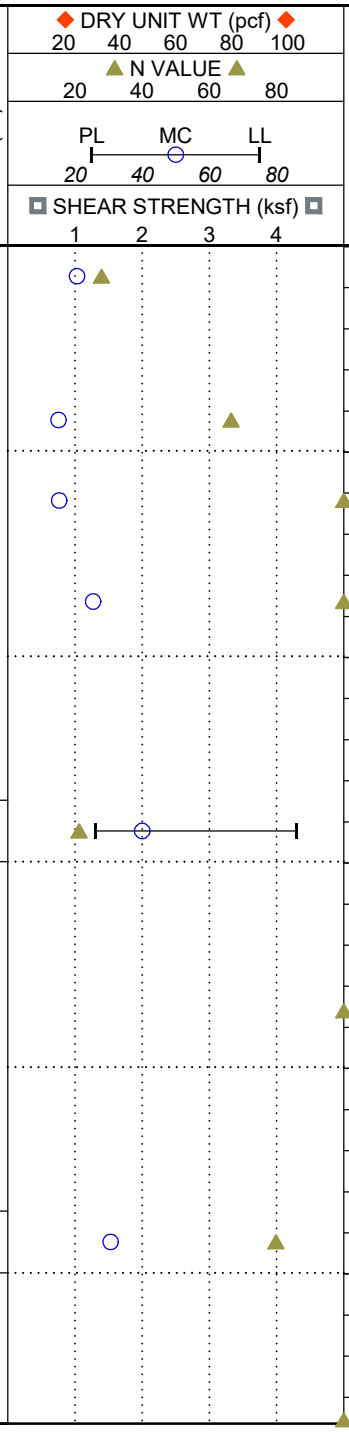
GEOTECHNICAL BORING LOG

BORING NUMBER 6

CLIENT Childers Architect
PROJECT NAME Cherokee Nation OSU Building
PROJECT NO. 255932
PROJECT LOCATION Tahlequah, OK
DATE STARTED 1/24/19 **COMPLETED** 1/24/19
SURFACE ELEVATION 895.03 ft **BENCHMARK EL.**
DRILLER MR **DRILL RIG** CME 1050
GROUND WATER LEVELS
HAMMER TYPE Auto
AT TIME OF DRILLING None
LOGGED BY BC **CHECKED BY** BP
AT END OF DRILLING
NOTES

DEPTH (ft)	DRILLING METHOD	STRATA SYMBOL	MATERIAL DESCRIPTION (Unified Soil Classification System)	SAMPLE TYPE NUMBER	RECOVERY % (RQD %)	CORRECTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	ELEVATION (ft)
0	ROTARY - 3 5/8" Tricone		CLAYEY CHERT GRAVEL, w/ Sand, Brown Tan, Medium Dense, Moist (GC)	SPT 1		13-16-12 (28)		895
				SPT 2		13-27-40 (67)		
5			5.5 ft CHERT GRAVEL, COBBLES & BOULDERS, w/ Occasional Stiff Fat Clay Layers, Red Brown, Tan & White, Very Dense, Slightly Moist	SPT 3		67/5"		890
				SPT 4		67/4"		
10			-Area of Possible Voids (12' to 13'). Water Loss During Drilling. 13.5 ft					885
			FAT CLAY, Trace Gravel, Red Tan, Very Stiff, Moist (CH)	SPT 5		8-8-13 (21)	3	
15			16.0 ft CHERT GRAVEL, COBBLES & BOULDERS, w/ Occasional Stiff Fat Clay Layers, Red Brown, Tan & White, Very Dense, Slightly Moist					880
				SPT 6		67/3"		
20								875
				SPT 7		29-47-33 (80)	2.25	
25								870
			28.7 ft	SPT 8		67/2"		

Bottom of borehole at 28.7 feet.



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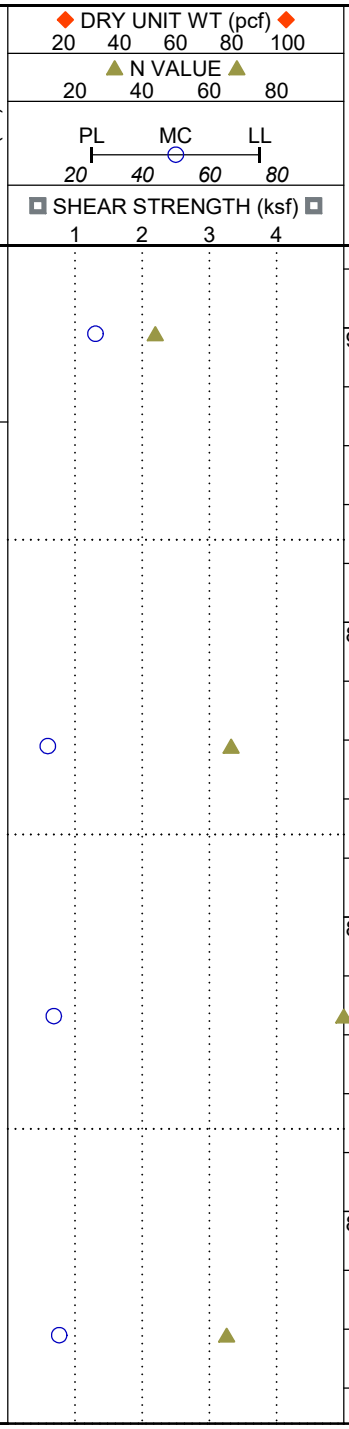
GEOTECHNICAL BORING LOG

BORING NUMBER 7

CLIENT Childers Architect
PROJECT NAME Cherokee Nation OSU Building
PROJECT NO. 255932
PROJECT LOCATION Tahlequah, OK
DATE STARTED 1/23/19 **COMPLETED** 1/23/19
SURFACE ELEVATION 900.7 ft **BENCHMARK EL.**
DRILLER MR **DRILL RIG** CME 1050
GROUND WATER LEVELS
HAMMER TYPE Auto
AT TIME OF DRILLING None
LOGGED BY BC **CHECKED BY** BP
AT END OF DRILLING
NOTES

DEPTH (ft)	DRILLING METHOD	STRATA SYMBOL	MATERIAL DESCRIPTION (Unified Soil Classification System)	SAMPLE TYPE NUMBER	RECOVERY % (RQD %)	CORRECTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	ELEVATION (ft)
0.0	ROTARY - 3 5/8" Tricone		FILL - CLAYEY CHERT GRAVEL, w/ Sand, Brown, Dense, Moist (GC)	SPT 1		7-25-19 (44)	2.25	900.0
1.0 ft			CLAYEY CHERT GRAVEL, w/ Sand, Brown Tan, Very Dense, Moist (GC)					
2.5								897.5
				SPT 2		20-21-45 (66)		
5.0								895.0
6.0 ft			CHERT GRAVEL, COBBLES & BOULDERS, w/ Occasional Stiff Fat Clay Layers, Red Brown, Tan & White, Dense to Very Dense, Moist	SPT 3		67/1"		
			-Area of Possible Voids (6.75' to 7.75'). Water Loss During Drilling.					
7.5								892.5
8.5 ft			CLAYEY CHERT GRAVEL, w/ Sand, Brown Tan, Very Dense, Moist (GC)	SPT 4		40-40-25 (65)		
10.0 ft								
10.0			Bottom of borehole at 10.0 feet.					



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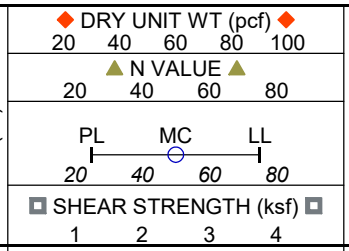
GEOTECHNICAL BORING LOG

BORING NUMBER 8

CLIENT Childers Architect
PROJECT NAME Cherokee Nation OSU Building
PROJECT NO. 255932
PROJECT LOCATION Tahlequah, OK
DATE STARTED 1/23/19 **COMPLETED** 1/23/19
SURFACE ELEVATION 903.3 ft **BENCHMARK EL.**
DRILLER MR **DRILL RIG** CME 1050
GROUND WATER LEVELS
HAMMER TYPE Auto
AT TIME OF DRILLING None
LOGGED BY BC **CHECKED BY** BP
AT END OF DRILLING
NOTES

DEPTH (ft)	DRILLING METHOD	STRATA SYMBOL	MATERIAL DESCRIPTION (Unified Soil Classification System)	SAMPLE TYPE NUMBER	RECOVERY % (RQD %)	CORRECTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	ELEVATION (ft)
0	ROTARY - 3 5/8" Tricone		CLAYEY CHERT GRAVEL, Brown, Medium Dense, Moist (GC) 1.0 ft	SPT 1		4-11-11 (22)	1.25	
			CLAYEY CHERT GRAVEL, w/ Sand, Brown Tan, Very Dense, Moist (GC)	SPT 2		12-43-64 (107)		900
5			6.0 ft CHERT GRAVEL, COBBLES & BOULDERS, w/ Occasional Stiff Fat Clay Layers, Red Brown, Tan & White, Very Dense, Slightly Moist	SPT 3		40-67/4"		895
10			-Area of Possible Voids (13' to 13.5'). Water Loss During Drilling. 14.0 ft	SPT 4		8-7-13 (20)		890
15			CLAYEY CHERT GRAVEL, w/ Sand, Brown Tan, Very Dense, Moist (GC)	SPT 5		13-33-31 (64)	1	885
20				SPT 6		17-53-25 (78)	2	880
25			28.6 ft	SPT 7		67/1"		875

Bottom of borehole at 28.6 feet.



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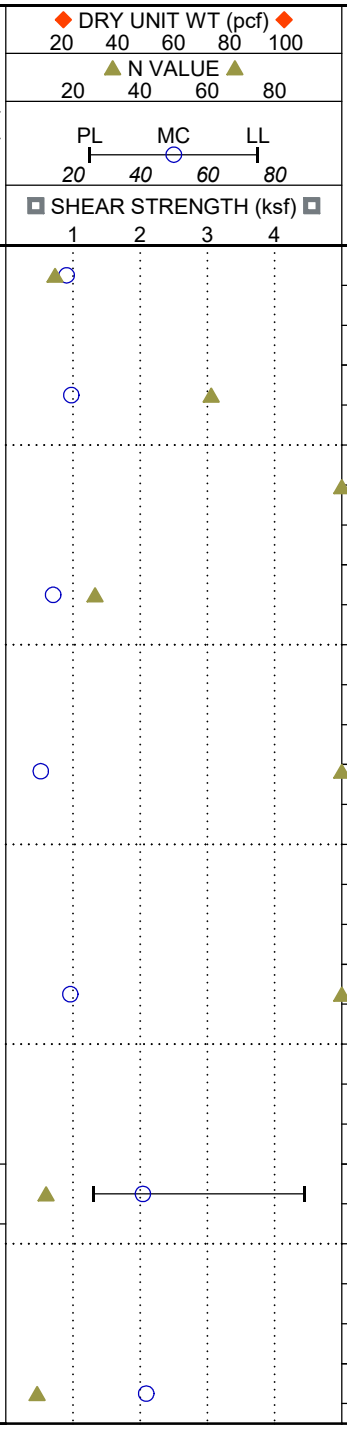
GEOTECHNICAL BORING LOG

BORING NUMBER 9

CLIENT Childers Architect
PROJECT NAME Cherokee Nation OSU Building
PROJECT NO. 255932
PROJECT LOCATION Tahlequah, OK
DATE STARTED 1/22/19 **COMPLETED** 1/22/19
SURFACE ELEVATION 903.0 ft **BENCHMARK EL.**
DRILLER MR **DRILL RIG** CME 1050
GROUND WATER LEVELS
HAMMER TYPE Auto
AT TIME OF DRILLING None
LOGGED BY BC **CHECKED BY** BP
AT END OF DRILLING
NOTES

DEPTH (ft)	DRILLING METHOD	MATERIAL DESCRIPTION (Unified Soil Classification System)	SAMPLE TYPE NUMBER	RECOVERY % (RQD %)	CORRECTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	ELEVATION (ft)
0	ROTARY - 3 5/8" Tricone	FILL - LEAN CLAY, w/ Gravel, Brown, Stiff, Moist (CL)	SPT 1		20-13-1 (14)		
		1.0 ft CLAYEY CHERT GRAVEL, w/ Sand, Brown Tan, Very Dense, Moist (GC)					
			SPT 2		17-40-21 (61)		900
5		5.5 ft CHERT GRAVEL, COBBLES & BOULDERS, w/ Occasional Stiff Fat Clay Layers, Red Brown, Tan & White, Very Dense, Slightly Moist	SPT 3		67/1"		
		-Area of Possible Voids (7.75' to 8'). Water Loss During Drilling.	SPT 4		23-17-9 (26)		895
10							
			SPT 5		67/4"		890
15		17.0 ft CLAYEY CHERT SAND, w/ Gravel, Red Tan, Very Dense, Moist (SC)					
			SPT 6		40-64-40 (104)		885
20		22.0 ft FAT CLAY, Scattered Chert Layers, Red Tan, Stiff, Moist (CH)					
			SPT 7		7-4-8 (12)	1	880
25							
		29.5 ft	SPT 8		4-5-4 (9)		875



Bottom of borehole at 29.5 feet.

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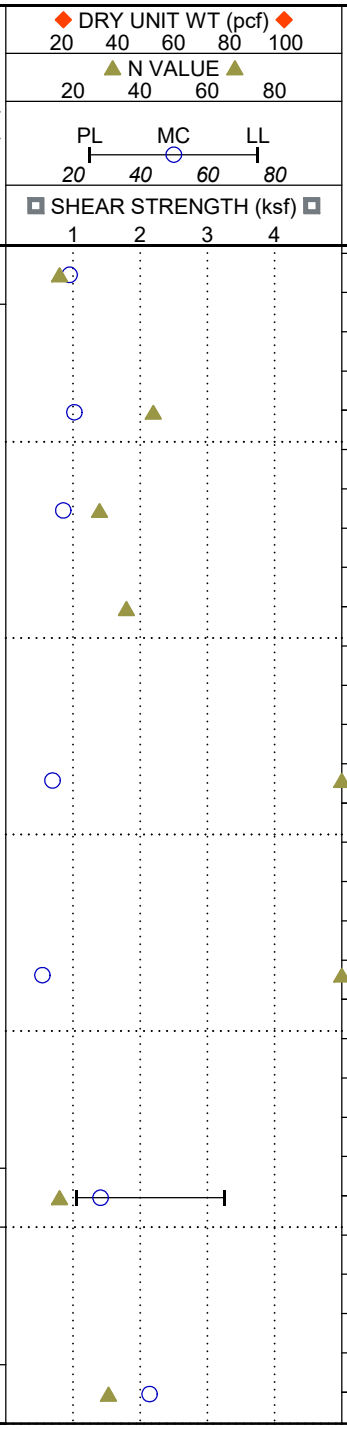
GEOTECHNICAL BORING LOG

BORING NUMBER 10

CLIENT Childers Architect
PROJECT NAME Cherokee Nation OSU Building
PROJECT NO. 255932
PROJECT LOCATION Tahlequah, OK
DATE STARTED 1/21/19 **COMPLETED** 1/22/19
SURFACE ELEVATION 904.2 ft **BENCHMARK EL.**
DRILLER MR **DRILL RIG** CME 1050
GROUND WATER LEVELS
HAMMER TYPE Auto
AT TIME OF DRILLING None
LOGGED BY BC **CHECKED BY** BP
AT END OF DRILLING
NOTES

DEPTH (ft)	DRILLING METHOD	STRATA SYMBOL	MATERIAL DESCRIPTION (Unified Soil Classification System)	SAMPLE TYPE NUMBER	RECOVERY % (RQD %)	CORRECTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	ELEVATION (ft)
0	ROTARY - 3 5/8" Tricone		FILL - LEAN CLAY, Scattered Gravel, Brown, Stiff, Moist (CL) 1.0 ft	SPT 1		4-8-8 (16)	2.25	
			CLAYEY CHERT GRAVEL, w/ Sand, Brown Tan, Very Dense, Moist (GC)					
				SPT 2		15-19-25 (44)		900
5			-Area of Possible Voids (5' to 6'). Water Loss During Drilling.					
				SPT 3		9-11-17 (28)		
				SPT 4		40-13-23 (36)		895
10			11.5 ft					
			CHERT GRAVEL, COBBLES & BOULDERS, w/ Occasional Stiff Fat Clay Layers, Red Brown, Tan & White, Very Dense, Slightly Moist					
				SPT 5		67/3"		890
15								
				SPT 6		67/2"		885
20			-Area of Possible Voids (20' to 22'). Water Loss During Drilling.					
			22.5 ft					
			FAT CLAY, Scattered Gravel, Red Tan, Stiff, Moist (CH)					
				SPT 7		8-8-8 (16)	1.5	880
25								
			30.0 ft	SPT 8		8-8-23 (31)	2	875
30			Bottom of borehole at 30.0 feet.					



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KEY TO SYMBOLS

CLIENT Childers Architect

PROJECT NAME Cherokee Nation OSU Building

PROJECT NO. 255932

PROJECT LOCATION Tahlequah, OK

LITHOLOGIC SYMBOLS
(Unified Soil Classification System)

- CH: USCS High Plasticity Clay
- CHERT: Chert
- FILL: Fill (made ground)
- GC: USCS Clayey Gravel
- TOPSOIL: Topsoil

SAMPLER SYMBOLS

- Standard Penetration Test

WELL CONSTRUCTION SYMBOLS

ABBREVIATIONS

LL - LIQUID LIMIT (%)
PI - PLASTIC INDEX (%)
W - MOISTURE CONTENT (%)
DD - DRY DENSITY (PCF)
NP - NON PLASTIC
-200 - PERCENT PASSING NO. 200 SIEVE
PP - POCKET PENETROMETER (TSF)

TV - TORVANE
PID - PHOTOIONIZATION DETECTOR
UC - UNCONFINED COMPRESSION
ppm - PARTS PER MILLION

- Water Level at Time Drilling, or as Shown
- Water Level at End of Drilling, or as Shown
- Water Level After 24 Hours, or as Shown

APPENDIX II

GENERAL NOTES

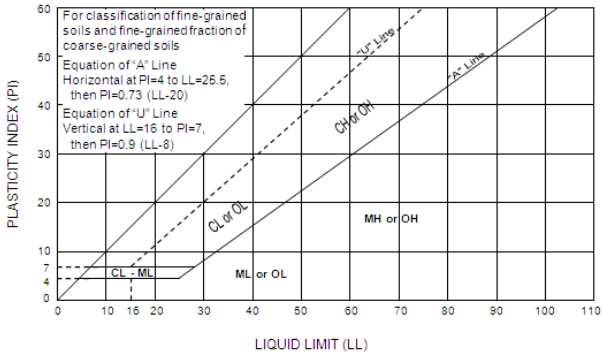
PALMERTON & PARRISH, INC.

GENERAL NOTES

SOIL PROPERTIES & DESCRIPTIONS

COHESIVE SOILS

Consistency	Unconfined Compressive Strength (Qu)	Pocket Penetrometer Strength	N-Value
	(psf)	(tsf)	(blows/ft)
Very Soft	<500	<0.25	0-1
Soft	500-1000	0.25-0.50	2-4
Medium Stiff	1001-2000	0.50-1.00	5-8
Stiff	2001-4000	1.00-2.00	9-15
Very Stiff	4001-8000	2.00-4.00	16-30
Hard	>8000	>4.00	31-60
Very Hard			>60



Group Symbol	Group Name
CL –	Lean Clay
ML –	Silt
OL –	Organic Clay or Silt
CH –	Fat Clay
MH –	Elastic Silt
OH –	Organic Clay or Silt
PT –	Peat
CL-CH –	Lean to Fat Clay

Plasticity		Moisture	
Description	Liquid Limit (LL)	Descriptive Term	Guide
Lean	<45%	Dry	No indication of water
Lean to Fat	45-49%	Moist	Indication of water
Fat	≥50%	Wet	Visible water

Fine Grained Soil Subclassification	Percent (by weight) of Total Sample
Terms: SILT, LEAN CLAY, FAT CLAY, ELASTIC SILT	PRIMARY CONSTITUENT
Sandy,gravelly, abundant cobbles, abundant boulders	>30-50]
with sand, with gravel, with cobbles, with boulders	>15-30] – secondary coarse grained constituents
scattered sand, scattered gravel, scattered cobbles, scattered boulders	5-15]
a trace sand, a trace gravel, a few cobbles, a few boulders	<5]
The relationship of clay and silt constituents is based on plasticity and normally determined by performing index tests. Refined classifications are based on Atterberg Limits tests and the Plasticity Chart.	

NON-COHESIVE (GRANULAR) SOILS

RELATIVE DENSITY	N-VALUE
Very Loose	0-4
Loose	5-10
Medium Dense	11-24
Dense	25-50
Very Dense	≥51

MOISTURE CONDITION	
Descriptive Term	Guide
Dry	No indication of water
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table.

**GRAIN SIZE IDENTIFICATION		
Name	Size Limits	Familiar Example
Boulder	12 in. or more	Larger than basketball
Cobbles	3 in. to 12 in.	Grapefruit
Coarse Gravel	¾-in. to 3 in.	Orange or lemon
Fine Gravel	No. 4 sieve to ¾-in.	Grape or pea
Coarse Sand	No. 10 sieve to No. 4 sieve	Rock salt
Medium Sand	No. 40 sieve to No. 10 sieve	Sugar, table salt
Fine Sand*	No. 200 sieve to No. 40 sieve	Powdered sugar
Fines	Less than No. 200 sieve	
*Particles finer than fine sand cannot be discerned with the naked eye at a distance of 8 in.		

Coarse Grained Soil Subclassification	Percent (by weight) of Total Sample
Terms: GRAVEL, SAND, COBBLES, BOULDERS	PRIMARY CONSTITUENT
Sandy,gravelly, abundant cobbles, abundant boulders	>30-50]
with gravel, with sand, with cobbles, with boulders	>15-30] – secondary coarse grained constituents
scattered gravel, scattered sand, scattered cobbles, scattered boulders	5-15]
a trace gravel, a trace sand, a few cobbles, a few boulders	<5]
Silty (MH & ML)*, clayey (CL & CH)*	<15]
(with silt, with clay)*	5-15] – secondary fine grained constituents
(trace silt, trace clay)*	<5]
*Index tests and/or plasticity tests are performed to determine whether the term "silt" or "clay" is used.	

*Modified after Ref. ASTM D2487-93 & D2488-93
**Modified after Ref. Oregon DOT 1987 & FHWA 1997
***Modified after Ref. AASHTO 1988, DM 7.1 1982, and Oregon DOT 1987

PALMERTON & PARRISH, INC.

GENERAL NOTES

BEDROCK PROPERTIES & DESCRIPTIONS

ROCK QUALITY DESIGNATION (RQD)	
Description of Rock Quality	*RQD (%)
Very Poor	< 25
Poor	25-50
Fair	50-75
Good	75-90
Excellent	90-100

*RQD is defined as the total length of sound core pieces 4 in. or greater in length, expressed as a percentage of the total length cored. RQD provides an indication of the integrity of the rock mass and relative extent of seams and bedding planes.

SCALE OF RELATIVE ROCK HARDNESS		
Term	Field Identification	Approx. Unconfined Compressive Strength (tsf)
Extremely Soft	Can be indented by thumbnail	2.6-10
Very Soft	Can be peeled by pocket knife	10-50
Soft	Can be peeled with difficulty by pocket knife	50-260
Medium Hard	Can be grooved 2 mm deep by firm pressure of knife	260-520
Moderately Hard	Requires one hammer blow to fracture	520-1040
Hard	Can be scratched with knife or pick only with difficulty	1040-2610
Very Hard	Cannot be scratched by knife or sharp pick	>2610

DEGREE OF WEATHERING	
Slightly Weathered	Rock generally fresh, joints stained and discoloration extends into rock up to 25mm (1 in), open joints may contain clay, core rings under hammer impact.
Weathered	Rock mass is decomposed 50% or less, significant portions of rock show discoloration and weathering effects, cores cannot be broken by hand or scraped by knife.
Highly Weathered	Rock mass is more than 50% decomposed, complete discoloration of rock fabric, core may be extremely broken and gives clunk sound when struck by hammer, may be shaved with a knife.

GRAIN SIZE (TYPICALLY FOR SEDIMENTARY ROCKS)		
Description	Diameter (mm)	Field Identification
Very Coarse Grained	>4.76	
Coarse Grained	2.0-4.76	Individual grains can easily be distinguished by eye.
Medium Grained	0.42-2.0	Individual grains can be distinguished by eye.
Fine Grained	0.074-0.42	Individual grains can be distinguished by eye with difficulty.
Very Fine Grained	<0.074	Individual grains cannot be distinguished by unaided eye.

VOIDS	
Pit	Voids barely seen with naked eye to 6mm (¼-in)
Vug	Voids 6 to 50mm (¼ to 2 in) in diameter
Cavity	50 to 6000mm (2 to 24 in) in diameter
Cave	>600mm

BEDDING THICKNESS	
Very Thick Bedded	> 3' thick
Thick Bedded	1' to 3' thick
Medium Bedded	4" to 1' thick
Thin Bedded	1¼" to 4" thick
Very Thin Bedded	½" to 1¼" thick
Thickly Laminated	⅛" to ½" thick
Thinly Laminated	⅛" or less (paper thin)

DRILLING NOTES

Drilling and Sampling Symbols

NQ – Rock Core (2-in. diameter)
HQ – Rock Core (3 in. diameter)
HSA – Hollow Stem Auger
CFA – Continuous Flight (Solid Stem) Auger
SS – Split Spoon Sampler
ST – Shelby Tube
WB – Wash Bore or Mud Rotary
TP – Test-Pit
HA – Hand Auger

Soil Sample Types

Shelby Tube Samples: Relatively undisturbed soil samples were obtained from the borings using thin wall (Shelby) tube samplers pushed hydraulically into the soil in advance of drilling. This sampling, which is considered to be undisturbed, was performed in accordance with the requirements of ASTM D 1587. This type of sample is considered best for the testing of "in-situ" soil properties such as natural density and strength characteristics. The use of this sampling method is basically restricted to soil containing little to no chert fragments and to softer shale deposits.

Split Spoon Samples: The Standard Penetration Test is conducted in conjunction with the split-barrel sampling procedure. The "N" value corresponds to the number of blows required to drive the last 1 foot of an 18-in. long, 2-in. O.D. split-barrel sampler with a 140 lb. hammer falling a distance of 30 in. The Standard Penetration Test is carried out according to ASTM D-1586.

Water Level Measurements

Water levels indicated on the boring logs are levels measured in the borings at the times indicated. In permeable materials, the indicated levels may reflect the location of groundwater. In low permeability soils, shallow groundwater may indicate a perched condition. Caution is merited when interpreting short-term water level readings from open bore holes. Accurate water levels are best determined from piezometers.

Automatic Hammer

Palmerton and Parrish's CME's are equipped with automatic hammers. The conventional method used to obtain disturbed soil samples used a safety hammer operated by company personnel with a cat head and rope. However, use of an automatic hammer allows a greater mechanical efficiency to be achieved in the field while performing a Standard Penetration resistance test based upon automatic hammer efficiencies calibrated using dynamic testing techniques.

*Modified after Ref. ASTM D2487-93 & D2488-93
**Modified after Ref. Oregon DOT 1987 & FHWA 1997
***Modified after Ref. AASHTO 1988, DM 7.1 1982, and Oregon DOT 1987

APPENDIX III

GRAIN SIZE ANALYSIS

4168 W. Kearney St.
Springfield, MO 65803
Telephone: (417) 864-6000
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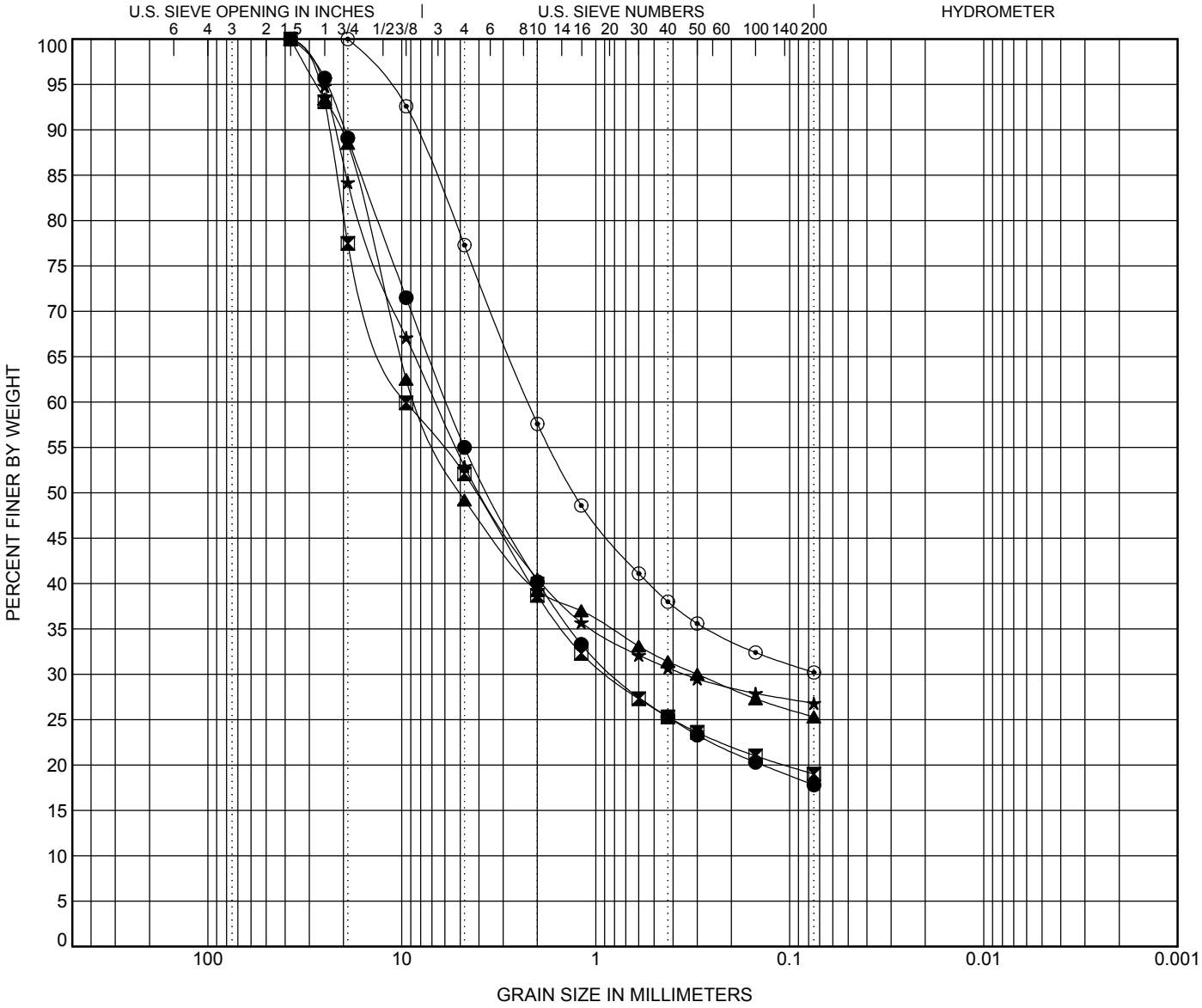
GRAIN SIZE DISTRIBUTION

CLIENT Childers Architect

PROJECT NAME Cherokee Nation OSU Building

PROJECT NO. 255932

PROJECT LOCATION Tahlequah, OK



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BOREHOLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	1	3.0	**CLAYEY GRAVEL with SAND(GC)**					
⊠	2	3.5	**CLAYEY GRAVEL with SAND(GC)**					
▲	3	13.5	**CLAYEY GRAVEL with SAND(GC)**					
★	8	3.5	**CLAYEY GRAVEL with SAND(GC)**					
⊙	9	18.0	**CLAYEY SAND with GRAVEL(SC)**					

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	1	3.0	**37.5**	**5.86**	**0.777**		**45.0**	**37.2**	**17.8**	
⊠	2	3.5	**37.5**	**9.537**	**0.868**		**47.9**	**33.1**	**19.0**	
▲	3	13.5	**37.5**	**8.339**	**0.3**		**50.8**	**23.9**	**25.3**	
★	8	3.5	**37.5**	**6.718**	**0.347**		**47.1**	**26.1**	**26.8**	
⊙	9	18.0	**19**	**2.222**			**22.7**	**47.1**	**30.2**	

APPENDIX IV

IMPORTANT INFORMATION REGARDING YOUR GEOTECHNICAL REPORT

Important Information about This

Geotechnical-Engineering Report

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

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

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

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

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled*. No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

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

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

This Report May Not Be Reliable

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

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

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

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

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

This Report's Recommendations Are Confirmation-Dependent

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

This Report Could Be Misinterpreted

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

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

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

Give Constructors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

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