GEOTECHNICAL ENGINEERING REPORT

Kelley Elementary School Shelter Addition
1900 N. Janeway Avenue,
Moore, Oklahoma 73164

PSI Project No. 05462609

PREPARED FOR:

AGP - the Abla Griffin Partnership LLC
201 North Broadway, Suite 210
Moore, Oklahoma 73160

November 14, 2022

BY:

PROFESSIONAL SERVICE INDUSTRIES, INC.
11825 S. Portland Avenue
Oklahoma City, OK 73107
Phone: (405) 735–6052

Professional Services Industries, Inc.
11825 S. Portland Avenue
Oklahoma City, OK 73107
Phone: (405) 735–6052

November 14, 2022

AGP - the Abla Griffin Partnership LLC
201 North Broadway, Suite 210
Moore, Oklahoma 73160

Attn.: Mr. Michael L. Abla, AIA
Mabla@theAGP.net
405.735.3477

Re: Geotechnical Engineering Report
Kelley Elementary School Shelter Addition
1900 N. Janeway Avenue,
Moore, Oklahoma
PSI Project No. 05462609

Professional Service Industries, Inc. (PSI), an Intertek company, is pleased to submit this Geotechnical Engineering Report for the referenced project. This report includes the results from the field exploration and laboratory testing along with recommendations for use in preparation of the appropriate design and construction documents for this project.

PSI appreciates the opportunity to provide this Geotechnical Engineering Report and looks forward to continuing participation during the design and construction phases of this project. PSI also has great interest in providing materials testing and inspection services during the construction of this project and will be glad to meet with you to further discuss how we can be of assistance as the project advances.

If there are questions pertaining to this report, or if PSI may be of further service, please contact us at your convenience.

Respectfully submitted,
Professional Services Industries, Inc.
Certificate of Authorization No. 1111; Expires 06/30/2023

LICENSED PROFESSIONAL ENGINEER
THOMAS M. VICK
32361
OKLAHOMA

Stephen Schnabel, E.I.
Staff Geotechnical Engineer

Thomas M. Vick, P.E., PMP *11/14/22*
Senior Engineer | Principal Consultant

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1.0 PROJECT INFORMATION

1.1 PROJECT AUTHORIZATION

Professional Service Industries, Inc. (PSI), an Intertek company, has completed a field exploration and geotechnical evaluation for the Kelley Elementary School Shelter Addition to be located at 1900 N. Janeway Avenue in Moore, Oklahoma. Mr. Michael L. Abla, AIA, representing AGP - the Abla Griffin Partnership LLC, authorized PSI's services on October 6, 2022 by signing PSI Proposal No. 0546-384428.

1.2 PROJECT DESCRIPTION

Based on information provided by the client, a summary of our understanding of the proposed project is provided below.

- A new single-story school shelter building with an approximate footprint of 3,900 square feet.

PSI was provided approximate structural loads by Mr. Brandon Birch, P.E., S.E., of KFC Engineering. PSI has prepared this report based on the anticipated loads detailed bellow:

- Pier foundations expect with pier reactions between 100 kips to 150 kips;
- Wall loads on the order of 8 kips per linear foot or less, however void forms are expected.

Site grading information was not provided at the time of this report. This report has been developed based on finished grades being within two (2) feet of existing site grade.

The geotechnical recommendations presented in this report are based on the available project information, structure locations, and the subsurface materials encountered during the field investigation. Should any of the above information or design basis made by PSI be inconsistent with the planned construction, it is requested that you contact us immediately to allow us to make any necessary modifications to this report. PSI will not be held responsible for changes to the project if not provided the opportunity to review the information and provide modifications to our recommendations.

2.0 SITE AND SUBSURFACE CONDITIONS

2.1 SITE GEOLOGY

As shown on the geologic map database for the State of Oklahoma, the site is located in an area where the Salt Plains Formation (Psp) is present at or near the ground surface. The Salt Plains Formation (Psp) consists of red to brown block shale and orange brown siltstone. The formation grades southward into Purcell Sandstone (Pp) in the Norman area. Thickness ranges about 200 feet.

2.2 SITE DESCRIPTION

The project site is located at 1900 N. Janeway Avenue in Moore, Oklahoma. The latitude and longitude of the proposed construction site are approximately at 35.3553° N and 97.5024° W. The project site currently consists of an open grass area with minor tree coverage. The site is bordered by the existing school building to the north, east and west, and the existing playground to the south. The truck-mounted drill rig did not experience significant mobility difficulty in accessing the boring at the time of the field work. The site is generally flat.

2.2.1 SEISMIC DESIGN PARAMETERS

We understand that the project is governed by the International Building Code (IBC), 2018 edition. As part of this code, the design of structures must consider dynamic forces resulting from seismic events. These forces are dependent upon the magnitude of the earthquake event as well as the properties of the soils that underlie the site.

As part of the procedure to evaluate seismic forces, the code requires the evaluation of the Seismic Site Class, which categorizes the site based upon the characteristics of the subsurface profile within the upper 100 feet of the ground surface. Our borings extended to depths of 30 feet bgs, but to define the Site Class for this project, we have interpreted the results of soil test borings drilled within the project site and estimated appropriate soil properties below the base of the borings to a depth of 100 feet as permitted by the code. The estimated soil properties were based upon the soils encountered at the site, data available in published geologic reports, and our experience with subsurface conditions in the general site area.

Based upon our evaluation, the subsurface conditions at the site are consistent with the characteristics of a **Site Class "C"** as defined in Chapter 20.3-1 of the ASCE 7-16. The associated probabilistic ground acceleration values and site coefficients for the general site area were obtained from the USGS geohazards web page (https://seismicmaps.org/) using the **ASCE 7-16** option and are presented in the table below.

TABLE 2.1: GROUND MOTION VALUES

Period (sec)	Mapped MCE Spectral Response Acceleration (g)		Site Coefficients		Adjusted MCE_R Spectral Response Acceleration (g)		Design Spectral Response Acceleration (g)	
0.2	S_s	0.33	F_a	1.3	S_{Ms}	0.429	S_{Ds}	0.286
1.0	S_1	0.083	F_v	1.5	S_{M1}	0.125	S_{D1}	0.083

2% Probability of Exceedance in 50 years for Latitude, Longitude: 35.35532° N, -97.50238° W
MCE_R = Maximum Considered Earthquake

2.3 FIELD EXPLORATION AND LABORATORY TESTING PROGRAM

2.3.1 FIELD EXPLORATION

Subsurface conditions at the site were explored by drilling two (2) borings at the approximate locations shown on the Boring Location Plan included in the Appendix. The boring locations were approximately located in the field by the drilling crew by estimating distances from known site reference points and with the aid of a handheld GPS device. The boring elevations on the boring logs are estimated from Google Earth and should be considered as accurate as the means and methods used to obtain them. The boring location information is included in the table below.

TABLE 2.2: BORING LOCATION INFORMATION

Boring Number	Boring Location	Depth(ft)
DB-1	School Shelter	28¾
DB-2	School Shelter	29

The borings were advanced utilizing solid flight auger drilling methods from an ATV CME-750 drill rig equipped with an automatic hammer using a weight of 140 pounds dropping 30 inches, and soil samples were routinely obtained during the drilling process. Drilling and sampling techniques were accomplished generally in accordance with ASTM procedures.

Groundwater level measurements were recorded at the boring locations during the field operations and were noted on the boring logs. The borings were backfilled with soil cuttings after the drilling operations were completed as per the local regulatory requirements.

Elevations of the ground surface at the boring locations were not provided and should be surveyed by others prior to construction. The references to elevations of various subsurface strata are based on depths below existing grade at the time of drilling. The approximate boring locations are depicted on the Boring Location Plan provided in the Figures.

During field activities, the encountered subsurface conditions were observed, logged, and visually classified (in general accordance with ASTM D 2488/D 2487). Field notes were maintained to summarize soil types and descriptions, water levels, changes in subsurface conditions, and drilling conditions. Samples were identified in the field, placed in sealed containers, and transported to the laboratory for further classification and testing.

PSI supplemented the field exploration with a laboratory testing program to determine additional engineering characteristics of the subsurface soils encountered. The laboratory testing program was conducted in general accordance with applicable ASTM Test Methods, and is included in Appendix A. Portions of samples not altered or consumed by laboratory testing will be discarded 30 days from the date shown on this report.

The geotechnical laboratory program included the following tests:

- Classification (ASTM D 2487 / 2488)
- Moisture Content (ASTM D 2216)
- Atterberg Limits (ASTM D 4318)
- Percent Soil Particles Finer than No. 200 Sieve (ASTM D 1140)

The samples not tested in the laboratory will be stored for a period of 30 days subsequent to submittal of this report and will be discarded after this period, unless other arrangements are made prior to the disposal period.

2.4 SUBSURFACE CONDITIONS

The results of the field and laboratory testing have been used to generalize a subsurface profile at the project site. The following subsurface descriptions provide a highlighted generalization of the major subsurface stratification features and material characteristics.

TABLE 2.3: GENERALIZED SOIL PROFILE

Stratum	Top (ft)	Bottom (ft)	Description
I	0	6	Lean CLAY (CL), medium stiff to very stiff, dark reddish brown, light gray inclusions, trace sand
II	6	13½ to 14½	Shaley Lean CLAY (CL), hard, reddish brown
III	13½ to 14½	30	Clayey SHALE, hard, light reddish brown, with silt seams and sand

The boring logs included in Appendix A should be reviewed for specific information at individual boring locations. The boring logs include soil descriptions, stratifications, locations of the samples, and field and laboratory test data. The descriptions provided on the logs only represent the conditions at that actual boring location; the stratifications represent the approximate boundaries between subsurface materials. The actual transitions between strata may be more gradual and less distinct. Variations will occur and should be expected across the site.

2.5 GROUNDWATER INFORMATION

The initial water levels were monitored in the open boreholes during drilling and attempts were made to measure final water levels. Groundwater information is summarized below in Table 2.4

TABLE 2.4: SUMMARY OF GROUNDWATER CONDITIONS

Boring Number	Boring Depth	Measured Groundwater Levels		
		While Drilling	Upon Completion	24 Hour Delay
DB-1	28¾ feet	22 feet	15 feet	7 feet
DB-2	29 feet	7 feet	25 feet	7.5 feet

Groundwater levels fluctuate seasonally as a function of rainfall, proximity to creeks, rivers and lakes, the infiltration rate of the soil, seasonal and climatic variations, and land usage. In relatively pervious soils, such as sandy soils, the indicated depths are a relatively reliable indicator of groundwater levels. In relatively impervious soils, water levels observed in the borings may not provide a reliable indication of groundwater elevations, even after several days. If a detailed water level evaluation is required, observation wells or piezometers can be installed at the site to monitor water levels.

The groundwater levels presented in this report were measured at the time of PSI field activities. The contractor should determine the actual groundwater levels at the site before construction activities.

3.0 GEOTECHNICAL CONCLUSIONS AND RECOMMENDATIONS

3.1 GEOTECHNICAL DISCUSSION

The primary geotechnical consideration at this site is the presence of low to medium plasticity soils. These conditions will govern the site preparation and earthwork activities and the project foundation designs. Should changes in the project criteria occur, a review must be made by PSI to determine if modifications to our recommendations will be required.

The following geotechnical design recommendations have been developed based on the previously described project characteristics and subsurface conditions encountered. The proposed construction should be performed in accordance with these recommendations and the applicable building code, and local governmental standards which have jurisdiction over this project. If there are changes in the project criteria, PSI should be retained to determine if modifications in the recommendations will be required. The findings of such a review would be presented in a supplemental report. Once final design plans and specifications are available, a general review by PSI is recommended to confirm that the conditions anticipated in preparing this geotechnical report are consistent with the earthwork and foundation recommendations contained within the construction documents.

3.1.1 SOIL SHRINK-SWELL POTENTIAL

The results of laboratory plasticity tests indicate that low to medium plasticity clay soils are present on site, particularly at the vicinity of boring DB-2. The materials generally have a low to moderate potential for shrinking and swelling. The amount of potential soil movement due to shrinking and swelling with soil moisture variations for this site could be on the order of 1-inch or less for these soils, assuming that the subgrade materials are allowed to increase in moisture content from a relatively dry condition to a relatively wet condition over a depth of approximately 8 feet. The relatively dry condition can occur with severe dry weather situations, thereby resulting in a significant degree of shrinkage and eventual potential swell in the foundation material. Differential movements are expected to be about ½ of the PVR. However, it should be noted that for extreme conditions (i.e., soils dry and shrink in one area with soils in another area being exposed to water and swelling) differential movement can be equal to or even double the PVR.

Poor drainage and water infiltration into the foundation soils may result in reduction of soil strength, thereby causing adverse and damaging movements. It is recommended that the moisture-related problems be corrected immediately as they can be detrimental to the ground supported structures.

It is important to control the possibility of moisture changes by following the precautions shown below:

- Direct surface runoff away from structures by sloping the subgrade away from the structure.
- Extend impervious coverings, such as sidewalks, to the structure's edge.
- Extend roof drain downspouts so that the discharge is at least 5 feet from the structure.
- Avoid placing trees or shrubs adjacent to structure.
- Avoid excessive drying of soil around the structure.

3.2 SITE PREPARATION

The proposed building pad areas should be stripped and grubbed of any construction debris, trash, vegetation, organic laden materials, and other structures in conflict with the proposed construction a minimum 5 feet outside the structural limits. Depressions or low areas resulting from stripping and grubbing should be backfilled with approved soil and compacted in accordance with *EARTHWORK* section herein.

3.2.1 BUILDING PAD PREPARATION

A ground supported slab can be constructed provided the movements associated with shrinking and swelling soils are reduced to a tolerable level and the owner understands the risk associated with such movements. To provide uniform support to the shallow foundation system discussed in the *FLOOR SLAB* section of this report and the floor slab-on-grade, the following options for subgrade preparation are provided to reduce the soil movements. Depending on the amount of risk that the owner is willing to take, the owner must select the following preparation option for this project.

3.2.1.1 SELECT FILL/STRUCTURAL FILL

In order to provide uniform support to the floor slab-on-grade and to reduce the potential movements associated with shrink/swell soils that may be encountered at the vicinity of boring DB-2, the recommended soil layers to be placed below the floor slab are given in the table below.

TABLE 3.1: PVR REDUCTION FOR BUILDING PAD PREPARATION

PVR	Material Type	Layer Thickness (feet)	Elevation [a] (feet)
1-inch	Select fill	1	+0 to -1
	Remove and Replace with Clean Compacted Imported Common fill	As Needed [b]	-1 to -X (Bottom of fill materials or To the Top of Natural soils if Site Grade Raised)

Note(s):

a) Existing grade is anticipated to be finished grade at +0 feet.
b) Fill should be as per *FILL MATERIAL* section of this report.

The select/structural fill should be placed within the plan area of the structure and to a distance of at least 5 feet beyond the perimeter of the structure and include building entrances and flatwork sensitive to movements. Plasticity and compaction requirements for the select fill are provided later in this section. On-site materials are mostly marginal to unsuitable for use as structural fill without modification (lime or cement, or fly ash or CKD treated) as described later in this report, but the material may be stockpiled for use in non-load bearing areas such as landscaping.

3.3 EARTHWORK

Following site preparation and any excavation to proposed grades, the newly exposed subgrades in site improvement areas intended for structures must be approved by the Geotechnical Engineer prior to fill placement. These exposed subgrades should be proof rolled with a loaded tandem axle dump truck or similar piece of rubber-tired equipment (20 tons or greater) in the presence of the Geotechnical Engineer's representative. The purpose of the proof rolling is to detect the existence of marginal or loose near-surface materials or unsuitable soils that may require undercutting. Areas which deflect, rut or pump excessively during proof rolling, and which cannot de densified in-place, should be undercut to suitable soils and backfilled and/or as directed by the geotechnical engineer. Proof rolling should not be performed on saturated, frozen or during wet weather conditions. Once approved, the soils exposed at the base of all excavations should be scarified to a depth of at least 12 inches, moisture conditioned and then compacted as described below prior to placing Engineered Fill above.

3.3.1 Fill Materials

The resulting engineered fill must produce a stable, uniform, and consistent compacted fill body. Fill materials should be free of organic or other deleterious materials and should be placed in maximum lifts of 8 inches of loose material and should be compacted in accordance with the below table. Compaction of the fill material should be performed with appropriate types of power, pneumatic or tamping equipment. Monitoring of the backfilling should include sufficient compaction testing by the Geotechnical Engineering representative to document that each lift of fill has been compacted to the required density. Each lift of compacted engineered fill should be tested by a representative of the Geotechnical Engineer prior to placement of subsequent lifts. If any lift or portion of a lift does not conform to the density requirements, the lift should be thoroughly scarified and re-compacted until the required density is obtained. If water must be added, it should be uniformly applied and thoroughly mixed into the soil by disking or scarifying. Care should be taken to apply compactive effort throughout the fill and fill slope areas. The moisture content and the degree of compaction of the engineered fill soils should be maintained until the construction of the structures within the area.

All proposed fill materials should be tested and approved by the geotechnical engineer prior to placement in the field, in accordance the *Compaction Criteria and Testing Frequency* table below. The following types of fill can be used as recommended in this report.

Common Fill: Common fill may consist of on-site or imported materials. Common fill should be free of organics or other deleterious materials and have a particle size of 3 inches or less. Common fill materials shall be classified in accordance with ASTM D2487. Satisfactory soils will be Clayey SAND (SC), Sandy lean CLAY or lean clay (CL) soils. Common fill shall have a plasticity index of less than 30.

Select Fill: Select fill should be free of organic or other deleterious materials, should have a maximum particle size less than 3 inches, and at least 35% fines (passing #200 sieve). Select fill materials shall be classified in accordance with ASTM D 2487. Satisfactory soils will be Sandy lean CLAY or lean CLAY (CL) soils. Select fill shall have a liquid limit not greater than 35 and a plasticity index between 8 and 20.

On-site soils: The onsite soils can be stockpiled and tested in bulk for compliance with the fill specifications prior to their reuse.

Lime, Cement, Fly Ash, and CKD Treated Soils: The lime and Portland cement treated soils are soils that are treated with 4 to 6% of lime or cement expressed as percent of the dry weight of the soil to be treated. The fly ash and cement kiln dust (CKD) treated soils are soils that are treated with 12 to 14% of fly ash or CKD expressed as percent of the dry weight of the soil to be treated. Lime, cement, fly ash, and CKD treated soils can also be used as Select Fill materials and also to strengthen the subgrade. These treated/modified soils, if used, should be placed and compacted to the specifications as shown in the below table.

- **Lime Treatment** should be performed when the wet subgrade soils are classified as CL or CH or SC in accordance with ASTM D 2487.
- **Lime-Fly Ash Treatment** should be performed when the wet subgrade soils are classified as SC, SC-SM, CL-ML, ML, SM in accordance with ASTM D 2487.
- **Cement Treatment** should be performed when the wet subgrade soils are classified as SM with less than 20% fines, SP-SC, SP-SM, SW-SC, SW-SM, SP, SW in accordance with ASTM D 2487.

Flexible Base: Flexible base materials should meet ODOT Type A Aggregate Base. Recycled concrete can be used. Flexible base should be placed and compacted to the specifications as mentioned in table below.

TABLE 3.2: COMPACTION CRITERIA AND TESTING FREQUENCY

Material Type (Location)	Per Standard Proctor Test (ASTM D 698)			
	Minimum Compaction (%)	Moisture Content Range (ref. to optimum moisture content)		Testing Frequency (min. 3 per lift)
		Minimum	Maximum	
Common Fill or On-Site Soils	95	-2%	+3%	1 per 2,500 sf
Select Fill	95	-1%	+3%	1 per 2,500 sf
Modified/Treated Soil	95	0%	+4%	1 per 2,500 sf
Flexible Base	95	-2%	+2%	1 per 5,000 sf
Utility Backfill	95	-1%	+4%	1 per 1,000 lf

3.3.2 EXCAVATIONS

In Federal Register, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, part 1926, Subpart P". This document was issued to better ensure the safety of workmen entering trenches or excavations. It is mandated by this federal regulation that excavations, whether they be utility trenches, basement excavation or footing excavations, be constructed in accordance with the new OSHA guidelines. It is our understanding that these regulations are being strictly enforced and if they are not closely followed the owner and the contractor could be liable for substantial penalties.

The contractor is solely responsible for designing and constructing stable excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability. The contractor's "responsible person", as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

We are providing this information solely as a service to our client. PSI does not assume responsibility for construction site safety or other parties' compliance with local, state, and federal safety or other regulations.

3.3.3 SLOPES

Any permanent cut or fill slopes should not exceed 2 Horizontal to 1 Vertical (2H:1V). Excavations extending below a 1H:1V plane extending down from any adjacent footings should be shored for safety. All excavations should be inspected by a representative of the geotechnical engineer during construction to allow any modifications to be made due to variation in the soil types. All work should be performed in accordance with Department of Labor Occupational Safety and Health Administration (OSHA) guidelines as described in the previous section.

3.3.4 UTILITIES

Utility trenches may be backfilled with suitable onsite native soils or imported soil above the pipe zone. If rocks larger than 3-inches in maximum size are encountered, they should be removed from the backfill material prior to placement in the utility trenches. Pipe zone backfill requirements should be in conformance with the requirements of the local agencies having jurisdiction. Jetting or flooding of utility backfill is not recommended. If smaller compaction equipment such as jumping jacks or plate compactors are used, thinner lifts will be required to achieve compaction. Where utilities cross building perimeters, concrete or concrete slurry should be used for backfilling around the utility to prevent moisture from migrating along the utility trench and entering the building envelope.

3.3.5 FLATWORK

For sidewalks or other flatwork located adjacent to grade-supported foundations, the undercutting and select fill placement operations for the building should extend beyond the perimeter of the building to at least the width of the adjacent sidewalk or flatwork.

Any other sidewalks or flatwork not adjacent to buildings should be placed on an improved subgrade meeting or exceeding the subgrade improvement methods previously recommended. If the sidewalk subgrade consists of material with a plasticity index of 20 or greater, a 12-inch-thick layer of material satisfying the requirements of select fill provided in the *FILL MATERIALS* section must be placed below the sidewalk.

Proper drainage around grade-supported sidewalks and flatwork is also very important to reduce potential movements. Elevating the sidewalks where possible and providing rapid, positive drainage away from them will reduce moisture variations within the underlying soils and will therefore provide valuable benefit in reducing the full magnitude of potential movements from being realized.

3.4 FOUNDATIONS

In our opinion, the structural loads of the proposed development can be supported on deep foundations constructed in accordance with the following design criteria. Alternatively, the structurally supported foundation can be used in lieu of the building pad preparation of removal and replacement activity. Additionally, PSI recommends that foundation type and bearing strata be consistent throughout a structure.

3.5 DRILLED PIER RECOMMENDATIONS

The subgrade conditions encountered in the borings appear suitable for use of a drilled shaft (aka drilled pier) foundation system bearing in the underlying hard shaley clay or clayey shale bedrock materials. The following considerations should be given to a drilled shaft foundation system at the project site.

The shafts should be a minimum total length of 5 feet or a ratio of length to diameter (L/D) not less than 3, whichever is longer, below the grade beam, when applicable, and should bear a minimum of 2 feet or one pier diameter, whichever is greater, below the bearing layer. In addition, a minimum of 2 shaft-diameter of the bearing layer materials should be below the tip of the shaft to use the allowable end bearing capacities provided in the below table.

TABLE 3.3: DRILLED PIER ALLOWABLE SKIN FRICTION & ALLOWABLE END BEARING CAPACITY

Drilled Pier Allowable Skin Friction & Allowable End Bearing Capacity				
Stratum	**Material**	**Approximate Depth Range (ft)***	**Allowable Skin Friction (psf)**	**Allowable End Bearing Capacity (psf)**
I	Clay (Ave. N-value of 15)	3** to 6	500	---
II	Shaley Clay (N-value 34 to 50 blows for 5.5 inches penetration)	6 to 14.5	700	7,000
III	Clayey Shale (N-value of 50 blows for 3 to 5 inches penetration)	14.5 to BTD***	1,750	17,800

Note:
* Approximate depth below existing subgrade surface at the time for our field exploration.
** First three feet ignored due to frost depth, effects of moisture variations, and possible separation of subsurface clays from the shafts.
*** BTD: Bore Termination Depth

The resistance against external uplift forces is equal to sum of dead loads and the soil resistance. The soil resistance could be considered equal to two-thirds of the allowable skin friction provided in the above table.

The shafts should be reinforced for the full depth to resist uplift forces due to the expansive clays. Reinforcement quantity should be adequate to resist tensile uplift forces generated by the clay soils equal to 30xd kips over the upper 8 feet of the pier shaft, which “d” is the diameter of shaft in foot.

Piers should be designed with a shaft diameter of at least 18 inches. Properly constructed piers bearing in the recommended materials are expected to experience total maximum settlement on the order of 1 inch or less.

The pier construction should also be observed by a representative of the Geotechnical Engineer to assess that the foundation materials have adequate strength to support the design loads and are consistent with the materials recommended in this report. Particular attention should be given to observation at locations where soil sloughing or groundwater inflow problems may occur.

Soft or loose soil zones encountered at the bearing level should be removed from the drilled shafts. If the exposed bearing materials become significantly wet or dry, they should be removed, and the pier deepened until more uniform moisture conditions are achieved. Concrete should be placed in the piers the same day they are excavated to prevent weakening of the shaft wall and bottom.

Groundwater was encountered during our field exploration. Casing and/or slurry may be required to advance the drilled piers, especially if sloughing soil and/or groundwater is encountered. Concrete placed in the piers should have a slump in the range of 5 to 7 inches. This range of slump will help to reduce the potential for formation of voids, especially as casing is extracted. The concrete mix should be designed to attain the required strength when placed at such a slump. The drilled shafts should be filled with concrete as soon as practical to reduce the potential of groundwater related problems and weathering of the excavation wall. During simultaneous concrete placement and casing removal operations, sufficient concrete head should be maintained inside the casing to offset hydrostatic head outside the casing, and to prevent the intrusion of soil and possible groundwater into the pier concrete, if present.

3.5.1.1 LPILE DESIGN CRITERIA

Piers having lateral loads should be designed utilizing the following LPILE input parameters for this project.

TABLE 3.4: PARAMETERS FOR LATERAL DESIGN USING LPILE

Soil Type	Depth (ft)	γ_e, pcf	c, psf	ϕ, Deg.	k_s or k, pci	k_c, pci	ε_{50} or k_{rm}
Stiff Clay w/o Free Water	0 to 6	125	1,500	—	500	—	0.007
Stiff Clay w/ Free Water	6 to 14½	130	3,000	—	1,000	400	0.005
[Clayey Shale] Stiff Clay w/ free water	14½ to BTD	135	5,000	—	2,000	800	0.004

Note:
- γ_e: Effective Soil Unit Weight
- c: Undrained Cohesion for Clay
- ϕ: Friction Angle for Sand ($\phi = (12N)^{0.5} + 15 \leq 32$; Dunham (1954))
- k_s: Clay Static Loading Modulus of Subgrade Reaction (LPILE Manual Table 3-3)
- RQD: Rock Quality Designation
- ε_{50}: Axial Strain Factor for Soil (LPILE Manual Table 3-2 and 3-4)
- BTD: Bore Termination Depth

3.5.1.2 GENERAL PIER CONSTRUCTION RECOMMENDATIONS

The performance of the foundation system is highly dependent on the quality of the installation. PSI recommends the installation procedure in accordance with FHWA-NHI-10-016, May 2010.

PSI recommends that the drilling contractor review the field exploration logs of this report before starting excavations for the drilled piers. If used, temporary casing must be removed during concrete placement, keeping a concrete head of at least 2 feet above the bottom of the casing as it is being removed. A representative of the Geotechnical Engineer should be on site to observe and document the entire drilling and installation of the deep foundation system, if used.

When the drilling processes are completed for the pier, the reinforcing steel and the concrete should be placed immediately after the final cleanout pass is conducted on the base. The tremie method of concrete placement should be adopted when placing concrete below the groundwater table (if present) to prevent segregation of the concrete materials. If concrete is placed by the free-fall method into a dry excavation, it should be placed to avoid contact with the excavation sidewalls to prevent segregation and be limited to a drop of less than 4 feet.

Concrete placed in the pier excavations should have a slump in the range of 7 to 9 inches to reduce the potential for the formation of voids as the temporary pier casing is extracted. The concrete mix should be designed to attain the required 28-day design strength when placed at this slump. PSI should be retained to observe and document the drilled pier construction and to evaluate whether the subsurface and pier bearing conditions are as anticipated in this report. The contractor should submit their procedures for drilled pier installation to the Geotechnical Engineer for approval prior to the start of the drilled pier construction.

3.6 FLOOR SLABS

The grade supported floor slab used in conjunction with a conventional shallow foundation or drilled pier and grade beam foundation system should be grade supported on a properly compacted and moisture conditioned select fill material as per the *BUILDING PAD PREPARATION* section. Proof-rolling should be accomplished to identify any soft or unstable soils which should be removed from the floor slab area prior to fill placement and/or floor slab construction.

An allowable net bearing pressure of 600 psf can be used for slab-on-grade bearing on compacted fill. If the site is prepared as recommended below, total settlement of the ground supported slab should not exceed one inch. A vapor retarder such as polyethylene sheeting should be provided beneath the ground supported slab in accordance with ACI procedures. Adequate construction joints and reinforcement should be provided to reduce the potential for cracking of the floor slab due to differential movement.

For the properly constructed grade supported floor slab, a modulus of subgrade, k, value of 120 pci is applicable in the grade supported floor slab design based on a typical 1 ft. x 1 ft. plate load test. However, depending on how the slab load is applied, the value will have to be geometrically modified. The value should be adjusted for larger areas using the following expression for cohesive and cohesionless soils:

Modulus of Subgrade Reaction:

$k_s = \frac{k}{B}$ for cohesive soil, and

$k_s = k\left(\frac{B+1}{2B}\right)^2$ for cohesionless soil (not recommended in undercut)

where:

- k_s = coefficient of vertical subgrade reaction for loaded area,
- k = coefficient of vertical subgrade reaction for 1x1 square foot area,
- B = width of area loaded, in feet (or effective width, B', for grade beam, continuous footing, or mat/raft foundation)

PSI recommends that a minimum four-inch-thick free draining granular mat be placed beneath the building floor slabs to enhance drainage. Prior to placing drainage layer, the subgrade should be graded to drain and not provide pockets to trap water. In moisture sensitive areas for equipment and flooring, vapor retarder should be installed with the grade supported slab construction according to ACI criteria. The floor slabs should have an adequate number of joints to reduce cracking resulting from differential movement and shrinkage.

3.7 PLAN REVIEW AND CONSTRUCTION OBSERVATION

After final plans and specifications are complete, PSI should review the final design and specifications so that the earthwork and foundation recommendations are properly interpreted and implemented. It is considered imperative that the Geotechnical Engineer and/or their representative be present during earthwork operations and foundation installations to observe the field conditions with respect to the design documents and specifications. PSI will not be responsible for changes in the project design or project information it was not provided, or interpretations and field quality control observations made by others. PSI would be pleased to provide these services for this project.

4.0 CONSTRUCTION CONSIDERATIONS

4.1 SECONDARY DESIGN CONSIDERATIONS

The following information has been developed after review of numerous problems concerning foundations throughout the area. It is presented here for your convenience. If these features are incorporated in the overall design and specifications for the project, performance of the project will be improved.

Prior to construction, the area to be covered by building should be prepared so that water will not pond beneath or around the building after periods of rainfall.

Roof drainage should be collected and transmitted by pipe to a storm drainage system or to an area where the water can drain away from buildings and pavements without entering the soils supporting buildings and pavements.

Sidewalks should not be structurally connected to buildings. They should be sloped away from buildings so that water will be drained away from structures.

Paved areas and the general ground surface should be sloped away from buildings on all sides so that water will always drain away from the structures. Water should not be allowed to pond near buildings after the floor slabs and foundations have been constructed.

Backfill for utility lines that are located in sidewalk or building areas should consist of on-site fill. The backfill should be compacted as described in the *EARTHWORK* sections of this report. Lesser lift thicknesses may be required to obtain adequate compaction.

Care should be exercised to make sure that ditches for utility lines do not serve as conduits that transmit water beneath structures or pavements. The top of the ditch should be sealed to inhibit the inflow of surface water during periods of rainfall.

Flower beds and planting areas should not be constructed along building perimeters. Constructing sidewalks adjacent to buildings would be preferable. If required, flower beds and planting areas could be constructed beyond the sidewalks away from the buildings. If it is desired to have flower beds and planting areas adjacent to a building, the use of above grade concrete box planters, or other methods that reduce the likelihood of large changes in moisture content of soils adjacent to or below structures should be considered.

Water sprinkling systems should not be located where water will be sprayed onto building walls and subsequently drain downward and flow into the soils beneath foundations.

Trees in general should not be planted closer to a structure than the mature height of the tree. A tree planted closer to a structure than the recommended distance may extend its roots beneath the structure, allowing removal of subgrade moisture and/or causing structural distress.

Utilities that project through the floor slab should be designed with some degree of flexibility and/or with a sleeve to reduce the potential for damage to the utilities should movement occur.

Soil supported floor slabs are subject to vertical movements. This often causes distress to interior wall partitions supported on soil supported floor slabs. This should be considered in the design of soil supported floor slabs.

4.2 Construction Materials Testing

It is recommended that PSI be retained to provide observation and testing of construction activities involved in the foundations, earthwork, and related activities of this project. PSI cannot accept any responsibility for any conditions that deviates from those described in this report, nor for the performance of the foundations if not engaged to also provide construction observation and testing for this project.

Observation of all foundation bearing materials, pier construction activities, structural steel and subgrade treatment operations should be performed by a representative of PSI.

4.3 Moisture Sensitive Soil / Weather Related Concerns

The upper fine-grained soils discovered at this site could be sensitive to disturbances caused by construction traffic and changes in moisture content. During wet weather periods, increases in the moisture content of the soil can cause significant reduction in the soil strength and support capabilities. In addition, soils that become wet may be slow to dry and thus significantly retard the progress of grading and compaction activities. Construction schedules should account for these conditions during wetter times of the year.

4.4 Drainage and Groundwater Concerns

Water should not be allowed to collect in the foundation excavation, on floor slab areas, or on prepared subgrades of the construction area either during or after construction. Undercut or excavated areas should be sloped toward one corner to facilitate removal of any collected rainwater, ground water, or surface runoff. Positive site surface drainage should be provided to reduce infiltration of surface water around the perimeter of the building and beneath the floor slabs. The grades should be sloped away from the building and surface drainage should be collected and discharged such that water is not permitted to infiltrate the backfill and floor slab areas of the building.

PSI recommends that the contractor determine the actual ground water levels at the site at the time of the construction activities. It may be expedient to drill auger holes or excavate test pits adjacent to the building area immediately prior to construction to determine the prevailing water level elevation. Any water accumulation should be removed from excavations by pumping. Should excessive and uncontrolled amounts of seepage occur, the geotechnical engineer should be consulted.

5.0 GEOTECHNICAL RISK AND REPORT LIMITATIONS

The concept of risk is an important aspect of the geotechnical evaluation. The primary reason for this is that the analytical methods used to develop geotechnical recommendations do not comprise an exact science. The analytical tools which geotechnical engineers use are generally empirical and must be used in conjunction with engineering judgment and experience. Therefore, the solutions and recommendations presented in the geotechnical evaluation should not be considered risk-free and, more importantly, are not a guarantee that the interaction between the soils and the proposed structure will perform as planned. The engineering recommendations presented in the preceding sections constitute PSI's professional estimate of those measures that are necessary for the proposed structure to perform according to the proposed design based on the information generated and referenced during this evaluation, and PSI's experience in working with these conditions.

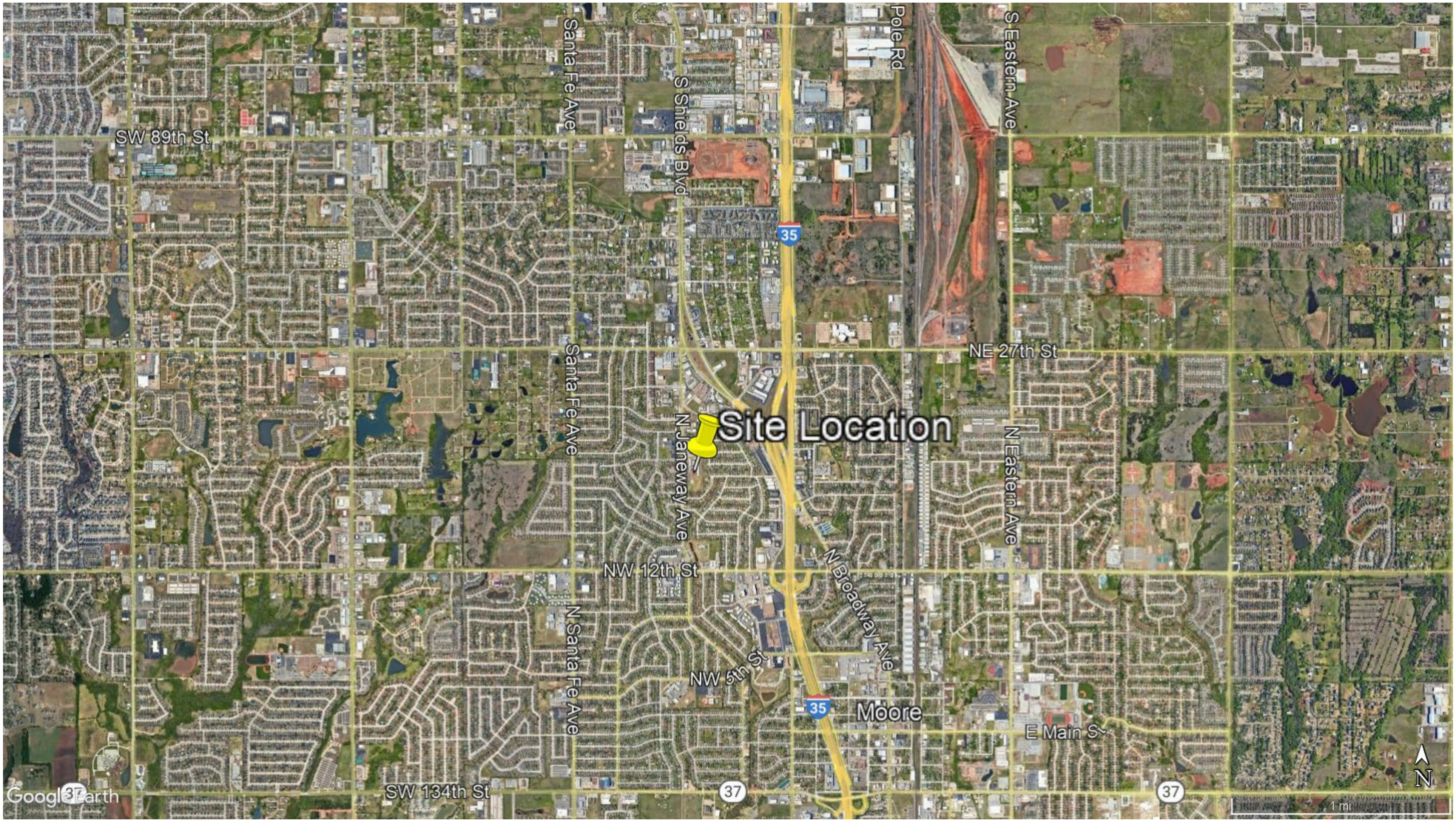
Services performed by PSI for this project have been conducted with that level of care and skill ordinarily exercised by members of the profession currently practicing in this area. No warranty, expressed or implied, is made.

The recommendations submitted are based on the available subsurface information obtained by PSI, and information provided by the client, client's representative and client's design consultants. If there are any revisions to the plans for this project or if deviations from the subsurface conditions noted in this report are encountered during construction, PSI should be notified immediately to determine if changes in the foundation and/or other recommendations are required. If PSI is not retained to perform these functions, PSI cannot be responsible for the impact of those conditions on the performance of the project.

The Geotechnical Engineer should be retained and provided the opportunity to review the final design plans and specifications to check that our engineering recommendations have been properly incorporated into the design documents. At that time, it may be necessary to submit supplementary recommendations.

This report has been prepared for the exclusive use of Client and their design consultants, for the aforementioned project parameters.

FIGURES



Professional Service Industries, Inc.
11825 S. Portland Avenue
Oklahoma City, OK 73107
Phone: (405) 735–6052

Site Vicinity Map

PSI Project No. 05462609

Kelley Elementary School Shelter Addition
1900 N. Janeway Avenue
Moore, Oklahoma

Professional Service Industries, Inc.
11825 S. Portland Avenue
Oklahoma City, OK 73107
Phone: (405) 735–6052

BORING LOCATION PLAN

PSI Project No. 05462609

Kelley Elementary School Shelter Addition
1900 N. Janeway Avenue
Moore, Oklahoma

APPENDIX A

Field Exploration & Laboratory Testing Program

BORING DB-1

DATE STARTED:	10/14/22	DRILL COMPANY:	DSO
DATE COMPLETED:	10/14/22	DRILLER: TS	LOGGED BY: JS
COMPLETION DEPTH	28.8 ft	DRILL RIG:	CME 750
BENCHMARK:	N/A	DRILLING METHOD:	Auger
ELEVATION:	1268 ft	SAMPLING METHOD:	SS
LATITUDE:	35.35527°	HAMMER TYPE:	Automatic
LONGITUDE:	-97.50248°	EFFICIENCY	N/A
STATION: N/A	OFFSET: N/A	REVIEWED BY:	SLS
REMARKS:			

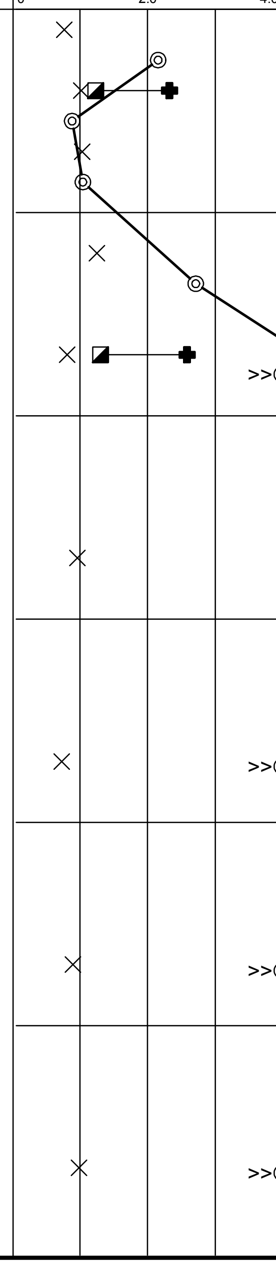
Water

▽ While Drilling	22	feet
▼ Upon Completion	15	feet
▽ Delay	7	feet

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	Additional Remarks
1268	0			**Lean CLAY**, very stiff to stiff, dark reddish brown	CL			
		1	8	... 0.5 to 3.5 feet, with trace roots and gravel		6-12-15 N=27	10	
		2	5			8-6-5 N=11	13	LL = 29 PL = 15 Fines=72.1%
1265		3	12	... 3.5 to 5 feet, with silt seams		5-6-7 N=13	13	
	6	4	15	**Shaley Lean CLAY**, hard, reddish brown	CL	8-15-19 N=34	16	
1260		5	12			16-50/5.5"	10	LL = 32 PL = 16 Fines=90.1%
1255		6	16			10-35-50/3.5"	12	
	14.5			**Clayey SHALE**, hard, light reddish brown				
1250		7	3	... 18.5 to 23.5 feet, with silt seams and sand		50/3"	9	
1245		8	4			50/3.5"	11	
1240		9	3			50/3"	12	
	28.8			End of boring				

STANDARD PENETRATION TEST DATA
N in blows/ft ◎
× Moisture ◪ PL ✚ LL
0 25 50
STRENGTH, tsf
▲ Qu ✳ Qp
0 2.0 4.0



Professional Service Industries, Inc.
11825 S. Portland Avenue
Oklahoma City, OK 73170
Telephone: (405) 735-6052

PROJECT NO.:	05462609
PROJECT:	Kelley Elementary School Shelter
LOCATION:	1900 N. Janeway Ave. Moore, OK

The stratification lines represent approximate boundaries. The transition may be gradual.

DATE STARTED: 10/14/22	**DRILL COMPANY:** DSO		
DATE COMPLETED: 10/14/22	**DRILLER:** TS	**LOGGED BY:** JS	
COMPLETION DEPTH 28.8 ft	**DRILL RIG:** CME 750		
BENCHMARK: N/A	**DRILLING METHOD:** Auger		
ELEVATION: 1268 ft	**SAMPLING METHOD:** SS		
LATITUDE: 35.35538°	**HAMMER TYPE:** Automatic		
LONGITUDE: -97.50229°	**EFFICIENCY:** N/A		
STATION: N/A **OFFSET:** N/A	**REVIEWED BY:** SLS		
REMARKS:			

BORING DB-2

Water

▽ While Drilling	7	feet
▼ Upon Completion	25	feet
▽ Delay	7.5	feet

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	N (blows/ft)	Additional Remarks
	0				**Lean CLAY**, very stiff to medium stiff, dark reddish brown	CL				
		SS	1	6	... 0.5 to 2 feet, with trace roots and gravel		8-8-8 N=16	13		
1265		SS	2	12	... 2 to 3.5 feet, with ferrous nodules		5-7-9 N=16	13		
		SS	3	18	... 3.5 to 5 feet, reddish brown and gray, trace sand		1-2-4 N=6	24		LL = 40 PL = 17 Fines=84.9%
	5									
		SS	4	17	**Shaley Lean CLAY**, hard, reddish brown	CL	6-38-50/4.5"	11		
1260		SS	5	18			12-17-38 N=55	14	>>	
	10									
1255										
		SS	6	12	**Clayey SHALE**, hard, light reddish brown, with silt seams		10-50/6"	12	>>	LL = 34 PL = 18 Fines=85.9%
	15									
1250		SS	7	5	... 18.5 to 23.5 feet, reddish and light gray, with sand		50/5"	11	>>	
	20									
1245		SS	8	5			50/5"	9	>>	
	25									
1240		SS	9	4			50/4"	13	>>	
					End of boring					

STANDARD PENETRATION TEST DATA — N in blows/ft ◎; × Moisture; PL; LL (0, 25, 50); STRENGTH, tsf — ▲ Qu, ✳ Qp (0, 2.0, 4.0)

Graphic log: Lean CLAY 0–6 ft; Shaley Lean CLAY 6–13.5 ft; Clayey SHALE 13.5–28.8 ft.

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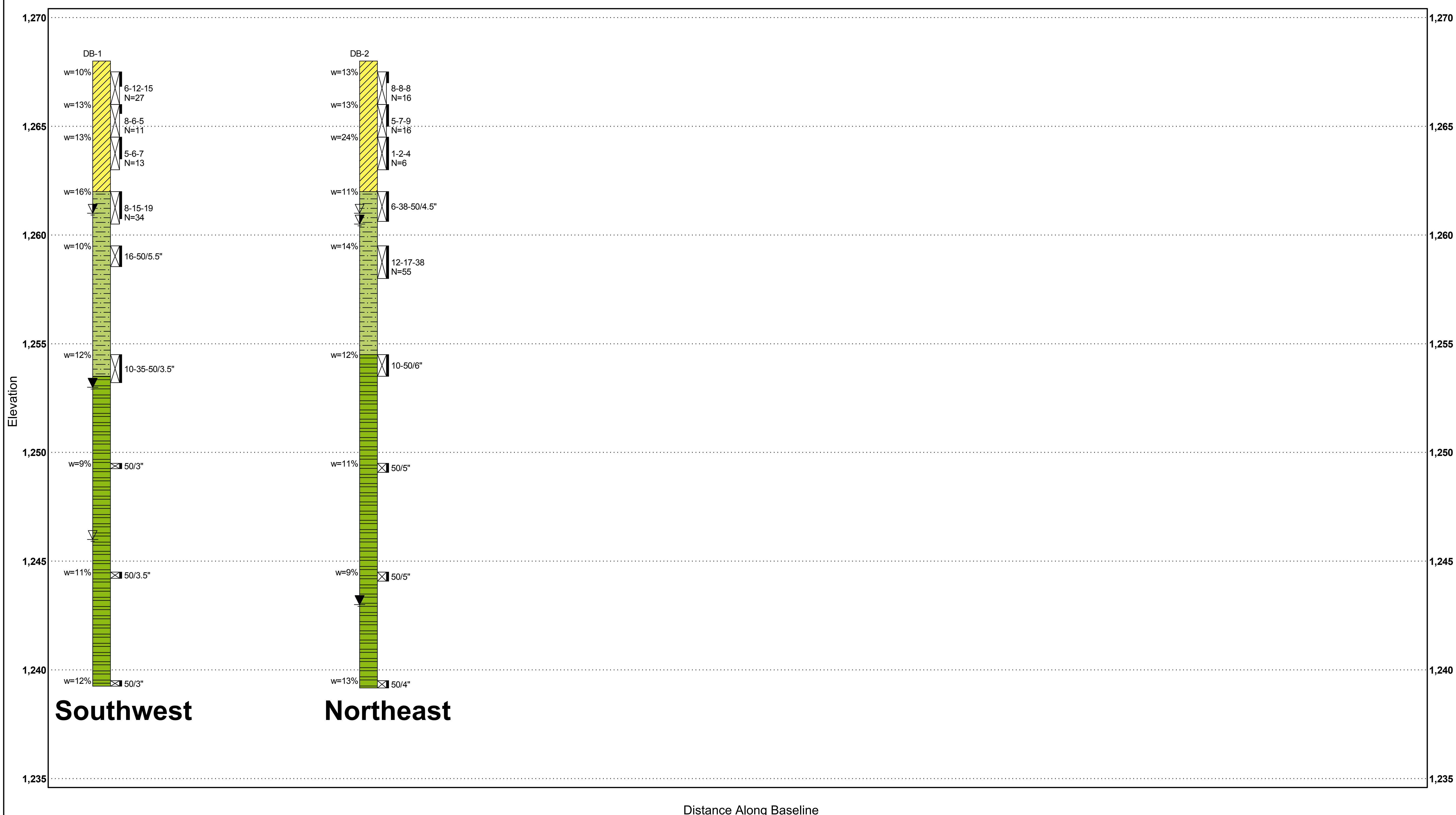
Professional Service Industries, Inc.
11825 S. Portland Avenue
Oklahoma City, OK 73170
Telephone: (405) 735-6052

PROJECT NO.: 05462609
PROJECT: Kelley Elementary School Shelter
LOCATION: 1900 N. Janeway Ave. Moore, OK

The stratification lines represent approximate boundaries. The transition may be gradual.

APPENDIX B

Subsurface Profiles



Professional Service Industries, Inc.
11825 S. Portland Avenue
Oklahoma City, OK 73170

Profile

Kelley Elementary School Shelter
PSI Project Number: 05462609

1900 N. Janeway Ave.
Moore, OK

APPENDIX C

General Notes

GENERAL NOTES

SAMPLE IDENTIFICATION

The Unified Soil Classification System (USCS), AASHTO 1988 and ASTM designations D2487 and D-2488 are used to identify the encountered materials unless otherwise noted. Coarse-grained soils are defined as having more than 50% of their dry weight retained on a #200 sieve (0.075mm); they are described as: boulders, cobbles, gravel or sand. Fine-grained soils have less than 50% of their dry weight retained on a #200 sieve; they are defined as silts or clay depending on their Atterberg Limit attributes. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size.

DRILLING AND SAMPLING SYMBOLS

SFA: Solid Flight Auger - typically 4" diameter flights, except where noted.
HSA: Hollow Stem Auger - typically 3¼" or 4¼ I.D. openings, except where noted.
M.R.: Mud Rotary - Uses a rotary head with Bentonite or Polymer Slurry
R.C.: Diamond Bit Core Sampler
H.A.: Hand Auger
P.A.: Power Auger - Handheld motorized auger

SS: Split-Spoon - 1 3/8" I.D., 2" O.D., except where noted.
ST: Shelby Tube - 3" O.D., except where noted.
RC: Rock Core
TC: Texas Cone
BS: Bulk Sample
PM: Pressuremeter
CPT-U: Cone Penetrometer Testing with Pore-Pressure Readings

SOIL PROPERTY SYMBOLS

N: Standard "N" penetration: Blows per foot of a 140 pound hammer falling 30 inches on a 2-inch O.D. Split-Spoon.
N_{60}: A "N" penetration value corrected to an equivalent 60% hammer energy transfer efficiency (ETR)
Q_u: Unconfined compressive strength, TSF
Q_p: Pocket penetrometer value, unconfined compressive strength, TSF
w%: Moisture/water content, %
LL: Liquid Limit, %
PL: Plastic Limit, %
PI: Plasticity Index = (LL-PL),%
DD: Dry unit weight, pcf
▼,▽,▼: Apparent groundwater level at time noted

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Relative Density	N - Blows/foot
Very Loose	0 - 3
Loose	4 - 9
Medium Dense	10 - 29
Dense	30 - 49
Very Dense	50+

ANGULARITY OF COARSE-GRAINED PARTICLES

Description	Criteria
Angular:	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular:	Particles are similar to angular description, but have rounded edges
Subrounded:	Particles have nearly plane sides, but have well-rounded corners and edges
Rounded:	Particles have smoothly curved sides and no edges

GRAIN-SIZE TERMINOLOGY

Component	Size Range
Boulders:	Over 300 mm (>12 in.)
Cobbles:	75 mm to 300 mm (3 in. to 12 in.)
Coarse-Grained Gravel:	19 mm to 75 mm (¾ in. to 3 in.)
Fine-Grained Gravel:	4.75 mm to 19 mm (No.4 to ¾ in.)
Coarse-Grained Sand:	2 mm to 4.75 mm (No.10 to No.4)
Medium-Grained Sand:	0.42 mm to 2 mm (No.40 to No.10)
Fine-Grained Sand:	0.075 mm to 0.42 mm (No. 200 to No.40)
Silt:	0.005 mm to 0.075 mm
Clay:	<0.005 mm

PARTICLE SHAPE

Description	Criteria
Flat:	Particles with width/thickness ratio > 3
Elongated:	Particles with length/width ratio > 3
Flat & Elongated:	Particles meet criteria for both flat and elongated

RELATIVE PROPORTIONS OF FINES

Descriptive Term	% Dry Weight
Trace:	< 5%
With:	5% to 12%
Modifier:	>12%

GENERAL NOTES

(Continued)

CONSISTENCY OF FINE-GRAINED SOILS

Q_U - TSF	N - Blows/foot	Consistency
0 - 0.25	0 - 1	Very Soft
0.25 - 0.50	2 - 3	Soft
0.50 - 1.00	4 - 6	Medium Stiff
1.00 - 2.00	7 - 12	Stiff
2.00 - 4.00	13 - 26	Very Stiff
4.00 +	26+	Hard

MOISTURE CONDITION DESCRIPTION

Description	Criteria
Dry:	Absence of moisture, dusty, dry to the touch
Moist:	Damp but no visible water
Wet:	Visible free water, usually soil is below water table

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term	% Dry Weight
Trace:	< 15%
With:	15% to 30%
Modifier:	>30%

STRUCTURE DESCRIPTION

Description	Criteria
Stratified:	Alternating layers of varying material or color with layers at least ¼-inch (6 mm) thick
Laminated:	Alternating layers of varying material or color with layers less than ¼-inch (6 mm) thick
Fissured:	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided:	Fracture planes appear polished or glossy, sometimes striated

Description	Criteria
Blocky:	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed:	Inclusion of small pockets of different soils
Layer:	Inclusion greater than 3 inches thick (75 mm)
Seam:	Inclusion 1/8-inch to 3 inches (3 to 75 mm) thick extending through the sample
Parting:	Inclusion less than 1/8-inch (3 mm) thick

SCALE OF RELATIVE ROCK HARDNESS

Q_U - TSF	Consistency
2.5 - 10	Extremely Soft
10 - 50	Very Soft
50 - 250	Soft
250 - 525	Medium Hard
525 - 1,050	Moderately Hard
1,050 - 2,600	Hard
>2,600	Very Hard

ROCK BEDDING THICKNESSES

Description	Criteria
Very Thick Bedded	Greater than 3-foot (>1.0 m)
Thick Bedded	1-foot to 3-foot (0.3 m to 1.0 m)
Medium Bedded	4-inch to 1-foot (0.1 m to 0.3 m)
Thin Bedded	1¼-inch to 4-inch (30 mm to 100 mm)
Very Thin Bedded	½-inch to 1¼-inch (10 mm to 30 mm)
Thickly Laminated	1/8-inch to ½-inch (3 mm to 10 mm)
Thinly Laminated	1/8-inch or less "paper thin" (<3 mm)

ROCK VOIDS

Voids	Void Diameter
Pit	<6 mm (<0.25 in)
Vug	6 mm to 50 mm (0.25 in to 2 in)
Cavity	50 mm to 600 mm (2 in to 24 in)
Cave	>600 mm (>24 in)

GRAIN-SIZED TERMINOLOGY

(Typically Sedimentary Rock)

Component	Size Range
Very Coarse Grained	>4.76 mm
Coarse Grained	2.0 mm - 4.76 mm
Medium Grained	0.42 mm - 2.0 mm
Fine Grained	0.075 mm - 0.42 mm
Very Fine Grained	<0.075 mm

ROCK QUALITY DESCRIPTION

Rock Mass Description	RQD Value
Excellent	90 -100
Good	75 - 90
Fair	50 - 75
Poor	25 -50
Very Poor	Less than 25

DEGREE OF WEATHERING

Slightly Weathered:	Rock generally fresh, joints stained and discoloration extends into rock up to 25 mm (1 in), open joints may contain clay, core rings under hammer impact.
Weathered:	Rock mass is decomposed 50% or less, significant portions of the 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.

SOIL CLASSIFICATION CHART

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

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Graphic Symbols for Materials and Rock Deposits

CONCRETE
Portland Cement Concrete

BITUMINOUS CONCRETE

CLAYSTONE

COAL
Coal, Anthracite Coal

CONGLOMERATE/BRECCIA
Conglomerate, Breccia

IGNEOUS ROCK
Anorthsite, Basalt, Metabasalt, Diabase (Gabbro), Gabbro, Granite/Granodionite, Homfels, Pegmatite, Rhyolite/Metarhyolite

LIMESTONE
Limestone, Dolomite

METAMORPHIC ROCK
Amphibolite, Gneiss, Marble, Phyllite, Quartzite, Schist, Serpentinite, Slate

CHERT

SANDSTONE
Sandstone, Orthoquarzite (Sandstone)

SHALE

SILTSTONE

NO RECOVERY

VOID

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