PSI Project Number: 05471208-12
August 28, 2019

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

August 28, 2019

Mr. Scott Donwerth, PE
Structural Engineer
Poe & Associates, Inc.
Consulting Engineers
1601 Northwest Expressway, #400
Oklahoma City, Oklahoma 73118

Re: Geotechnical Engineering Services Report
Crossroads Interchange Reconstruction
Retaining Wall "P"
I-35/I-240 Interchange
Oklahoma City, Oklahoma
PSI File No. 05471208-12

Dear Mr. Donwerth:

Professional Service Industries, Inc. (PSI), an Intertek company, performed the geotechnical services that you requested for the referenced project.

PSI thanks you for choosing us as your geotechnical consultant for this project. Please contact us at 405.735.6052 if you have any questions or we may be of further service.

Respectfully Submitted,

For Professional Service Industries, Inc.
CA NO. 1111 Expires 06/30/21

Ming-Cheng "Peter" Shau, P.E., D.GE
Department Manager
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8/28/19

Report of Geotechnical Engineering Services

Proposed Crossroads Interchange Reconstruction
Retaining Wall "P"
I-35/I-240 Interchange
Oklahoma City, Oklahoma

Prepared for

Poe & Associates, Inc.
1601 Northwest Expressway, #400
Oklahoma City, Oklahoma 73118

Prepared by

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PSI Project 05471208-12

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1 PROJECT INFORMATION

1.1 PROJECT AUTHORIZATION

Professional Service Industries, Inc. (PSI), an Intertek company, has completed a geotechnical exploration for the proposed Crossroads Interchange Reconstruction Project Retaining Wall "P" at the northwest sector of the I-35/I-240 Interchange in Oklahoma City, Oklahoma County, Oklahoma. PSI's services were authorized via Geotechnical Service Contract with the Oklahoma Department of Transportation and Poe & Associates. This exploration was accomplished in general accordance with PSI's itemized scope for the proposed Retaining Wall "P".

1.2 PROJECT DESCRIPTION

Limited project information was provided to PSI. However, PSI has been furnished with the following plan prepared by Poe & Associates:

- A General Plan and Elevation for Retaining Wall "P", entitled "C.I.P. Retaining Wall "P" Plan & Elevation – Phase 2" and includes the required boring locations Northing & Easting
- A list providing the location and elevation of the staked borings
- Information about the actual location and bearing of the borings performed in the field

This report is also based on the following:

- The basal earth retention system for Wall "P" is Cast-in-Place (CIP) retaining wall
- The proposed CIP wall will have typical panel widths of 25' for panels 1 through 4 and 15.97' for panels 5 and 6 and a total wall length of 131.95'
- Begin Wall "P" Station: 0+00.00 and End Wall "P" Station: 1+31.95
- Top of Wall Elevations will range from 1283.57' to 1286.86'
- Top of Footing Elevations will range from 1275.56' to 1278.16'
- Wall heights will range from approximately 4.6 to 11.3'
- Amount of anticipated cut from existing ground surface to the proposed Ramp "B" at face of Wall "P" is approximately ±1' at the south end to ±7' at the north end; no fill to the bottom of the footing is anticipated
- H-Piles (straight and battered) are idealized for the support of the CIP wall; however, PSI does not anticipate deep foundation, including drilled shafts/piers, will be practical for the wall since all cuts will in the bedrock, based on the test boring information. Spread footings supported on the excavated bedrock formation may be considered and is anticipated to be more practical for this project site
- Solid TR4 concrete traffic rail is proposed at the top of the wall
- The back of the wall will contain an abutting 4' – 0" paved ditch and the N.W. Service Road beyond
- A Mechanically Stabilized Earth (MSE) wall system may be substituted for the basal CIP wall and may be supported directly on or in the excavated bedrock present at the site

PSI has not been provided detailed structural loading information at this time. The geotechnical recommendations presented in this report are based on the available project information, wall location, laboratory testing, and the subsurface materials described in this report. If the noted information is incorrect, please inform PSI in writing so

that the recommendations presented in this report can be amended if appropriate and desired by the client. PSI will not be responsible for the implementation of its recommendations when PSI has not been notified of changes in the project.

1.3 SCOPE OF SERVICES

The purpose of this study was to explore the subsurface conditions at the proposed Retaining Wall "P" site in order to provide soil/bedrock strength parameters for design of earth retention system foundations. PSI's scope of services included drilling 2 soil test borings at the project site into the bedrock, select laboratory testing, and preparation of this geotechnical report. This report briefly outlines the testing procedures, presents available project information, describes the site and subsurface materials, and presents recommendations regarding the following:

- Foundation types, depths, nominal bearing capacities, and potential settlement estimations
- General lateral resistance design parameters that may be considered
- Retaining wall stability comments
- Comments regarding factors that may impact construction and performance of the proposed construction

As required by ODOT for LRFD design projects and as indicated by ODOT on the "Geomorphic Provinces of Oklahoma" geologic map, the bedrock in the project area was sampled with the Texas Cone Penetrometer (TCP), as permitted for the site's geology.

The scope of services did not include an environmental assessment for determining the presence or absence of wetlands or hazardous or toxic materials in the soil, bedrock, surface water, groundwater, or air, on, or below, or around this site. Statements in this report or on the boring logs regarding odors, colors, and unusual or suspicious items or conditions are strictly for informational purposes.

2 SITE AND SUBSURFACE CONDITIONS

2.1 SITE LOCATION AND DESCRIPTION

The site for the proposed Retaining Wall "P" for the Crossroads (I-35/I-240) interchange reconstruction is located in Oklahoma City, Oklahoma. The wall is located in the northwest sector of the interchange, on the west side of the existing southbound I-35 and east of the associated off-ramp to I-240. Access to the boring locations was difficult and impractical for the drill rig due to a combination of traffic condition, steep slopes, and possible below ground utilities in the staked wall alignment area. The borings were relocated as closely as possible to the west of the requested test areas. The relocated borings are at higher elevations relative to the proposed wall alignment.

2.2 SITE GEOLOGY

"Division Four" of the "Engineering Classification of Geological Materials", published by the Oklahoma Department of Transportation (ODOT) indicates that below alluvium, the site is underlain by the Hennessey Unit (Phy) in Oklahoma County. The geologic unit is described below.

This unit consists of red platy to blocky clay shales and mudstone. The mudstones are hard and appear blocky. The red clay shale of the Hennessey Unit is characterized by numerous bands of streaks of white or light green color ranging from a few inches to four feet in thickness.

The total thickness of the unit is about 400 feet.

The Hennessey Unit outcrops in a 5 to 15 mile north-south band across Grant, Garfield, Kingfisher, Logan, Canadian, and Oklahoma Counties in Division Four. Topographically, the unit is near level to gently rolling and is generally grass covered or cultivated.

2.3 SUBSURFACE CONDITIONS

The site subsurface conditions were explored with 2 soil and bedrock test borings. The boring locations were reestablished and provided to PSI by Poe & Associates' survey crew upon completion of the borings at the new relocation points. Actual boring elevations and locations were provided to PSI by Poe and Associates. The elevations and locations are reported on the boring logs and summarized in the table below.

Elevation and Location of Borings					
Boring No.	Northing	Easting	Latitude (N)	Longitude (W)	Elev., ft.
WP-1	145195.6018	2150689.3892	35°23'52.16"N	97°29'43.79"W	1286.20
WP-2	145090.0844	2150690.7304	35°23'51.11"N	97°29'43.78"W	1285.89

Note: The Lat/Longs are based on the NAD 83 Datum
Assumed N/E coordinates are from ODOT coordinate system NAVD88 Elevation

The borings were advanced with solid flight augers. The exploration was performed from PSI's truck mounted CME-55 drill rig equipped with an automatic hammer using a 140-pound hammer dropping 30 inches. Select soil

samples were routinely obtained during the drilling process. Drilling and sampling techniques were accomplished generally in accordance with AASHTO and ASTM procedures. Select soil samples were tested in the laboratory to determine material properties for PSI's evaluation. Laboratory testing was accomplished generally in accordance with ASTM procedures. The results are included on the boring logs and presented in the **Appendix**.

The materials at the boring locations basically consisted of the roadway's surface 11- and 12-inch thick pavement over approximately 2-inch aggregate base material. The existing pavement is constructed on the underlying siltstone and sandstone bedrock materials that extended to the boring depths of approximately 15 feet below the surface. The field and laboratory test results are presented on the "Boring Logs" included in the **Appendix**.

The bedrock materials are generally moderately hard to very hard in shear strength, based on the TCP results, to the boring termination depths. Texas Cone Penetrometer (TCP) test results in the bedrock ranged from 100 blows for 2.9 inches of penetration to 100 blows for 0.6 inches of penetration.

The approximate depths to the interpreted bedrock geologic materials are summarized in the table below.

Summary of Depths to the Bedrock Formation Below Surface Pavement at Relocated Borings					
Boring No.	Boring Elevation, Ft.	Approximate Depth to Bedrock, Ft.	Approximate Bedrock Elevation, Ft.	Approximate Boring Depth, Ft.	Approx. Bottom of Boring Elevation, Ft.
WP-1	1286.20	1.2	1285.0	15.0	1271.2
WP-2	1285.89	1.1	1284.8	15.0	1270.9

The above subsurface description is of a generalized nature to highlight the major subsurface stratification features and material characteristics. The boring logs included in the appendix should be reviewed for specific information at individual boring locations. These boring logs include soil/rock descriptions, stratifications, penetration resistances, locations of the samples and laboratory test data. The stratifications shown on the boring logs represent the conditions only at the actual boring locations. Variations may occur and should be expected between boring locations. The stratifications represent the approximate boundary between subsurface materials and the actual transition may be gradual. Water level information obtained during field operations is also shown on the boring logs. The samples that are not altered by the laboratory testing will be retained for 60 days from the date of this report and discarded.

2.4 GROUNDWATER INFORMATION

Groundwater was not observed to collect in the boreholes during the field work, indicating the stable groundwater may be below the maximum depth of the borings or additional time may be required for the groundwater to collect in the open holes. Groundwater can and should be expected to exist at varying depths during other times of the year depending upon climatic and rainfall conditions. Discontinuous zones of perched water can exist within the overburden materials and/or at the contact with bedrock. The groundwater information observed in the field is summarized in the table below.

3 EVALUATION AND RECOMMENDATIONS

3.1 GEOTECHNICAL DISCUSSION

The recommendations presented in this report are based on the available project information and the subsurface materials encountered in the borings. The primary proposed earth retention system in the available project plan is cast-in-place concrete cantilever type retaining wall supported by driven steel piles. However, due to the shallow depth of the bedrock at the site, PSI anticipates shallow spread footings or, if desired, cast-in-place concrete drilled shafts may be more practical for construction and may be considered. PSI also anticipates the primary cantilever type wall may be substituted with mechanically stabilized earth (MSE), gravity type wall system. PSI has presented in the following report sections design and construction recommendations for spread footings, steel H-piles, and drilled shafts as support systems for the proposed cantilever type soil retention structure. The recommendations, particularly the shallow foundation recommendations and design parameters, are also applicable for the possible MSE wall substitute.

In addition to the wall design considerations presented above, PSI is of the opinion that due to the natural retained subgrade materials generally been shale, siltstone, and/or sandstone bedrock, to minimize excavation or sloping of this material to form the heel portion of the wall and backfilling with much soil, consideration may be given to designing the wall as a front facing L-shaped wall. This will allow the excavation to be performed vertically. Adequate drainage behind the wall to control hydrostatic pressure is essential and can be possible by providing a thickness of drainable material or prefabricated drainage blankets behind the wall, between the retained bedrock and the wall and should include a perforated drainpipe at the footing level. The drainpipe should be sloped to provide positive gravity drainage to a sump where water can be collected and removed or to a site storm sewer/drainage. The drain line should be wrapped with filter fabric to prevent intrusion of fines, and the area around and above the drain line should be backfilled with free draining granular material consisting of compacted crushed limestone/gravel. Compacted limestone or gravel or clean sand should be used to complete the backfill for the minimal excavation. The compacted material may extend vertically above the drain line to within one (1) foot of final grade at the top of the wall. PSI recommends the granular section behind the wall be a minimum width of one (1) foot and be encapsulated in a suitable filter fabric to minimize intrusion of fines, or according to ODOT criteria. Drainage blankets, if considered, should be installed in accordance with the manufacturer's recommendations.

Mechanically Stabilized Earth (MSE) retaining walls are considered proprietary gravity type walls and internal soil parameters and associated internal stability should be verified and provided by the supplier. Bearing width for MSE type walls are generally dictated by the length of the lowest reinforcing for the wall. A leveling pad construction is recommended for support of the wall's segmental panel sections or modular facing materials, if used for this urban setting. Based on the fact that the retained and bearing natural materials and foundations will be the sandstone and/or siltstone bedrock, settlement, overturning, and/or global stability will not be expected to be of concern for this wall construction during the life of the wall.

Generally, for moderately hard to very hard bedrock geologic materials, pile driving, and drilled shaft excavations may encounter difficulties. Difficulty in driving of the piles for anchorage and bearing into the bedrock may be encountered. Additionally, it may not be possible to achieve adequate pile lengths without pre-drilling pilot holes, due to the relatively shallow depths to the harder IGM and/or bedrock. If pre-drilling is required, it should be performed in accordance with ODOT guidelines. Adequate pile driving and pier drilling equipment should be provided by the contractor. Casing of the drilled shaft excavations or utilization of slurry (preferably polymer), or a combination of casing and slurry may be required to advance the drilled shaft holes, especially when groundwater and/or sloughing soil is encountered.

Though PSI is providing specific groundwater information measured at the time of drilling activities, it is recommended the Contractor determine the actual groundwater level at the site at the time of the construction activities.

3.1.1 STABILITY AND WALL SETTLEMENT

Based on available information, the location of the wall, the subsurface conditions, the proposed retaining wall configurations, as previously mentioned and among other parameters and limitations, internal, external, and global stability is not expected to be of concern for the wall constructed at this site whether supported by shallow or deep foundation system. However, if required, global stability analysis can be adequately performed once the wall type and configurations are available. A limited global stability and settlement analysis of the retaining wall construction (MSE) using idealized wall configurations of approximately 70% of the 11¼ feet wall height for footing/MSE wall width estimations in conjunction with backfill bearing pressure influence was performed and is anticipated to be negligible for settlement (much less than 1 inch) and a factor of safety in excess of 5 for global stability. As can be expected, footings or walls at this site that are supported by deep foundations in the bedrock are not anticipated to be susceptible to settlements and external stability issues that include bearing, overturning, sliding and global stability.

3.1.2 COMPACTED SOILS SELF-WEIGHT SETTLEMENT

Based on the past experiences, it is anticipated that properly compacted new structural fill material will settle approximately 1 to 2 percent of the fill height. The higher the clay content the longer it takes the fill to settle. Granular material typically settles as it is compacted. Settlement of the fill material, especially the fill supported by the wall depending on the type, could be on the order of 2½ inches or more depending on the thickness of fill, composition, and compaction.

3.1.3 RIPPABILITY OF BEDROCK

For drilled pier installations consideration, generally moderately hard to very hard sandstone and siltstone bedrock over shale bedrock materials were encountered in the borings. The bedrock was encountered virtually from the surface of the natural ground at the boring and same is expected at the wall aliment which is at a lower elevation than drilled and extended to the boring termination depths of approximately 15 feet below the surface. Difficulties in excavating for footings and utilities, if applicable, may be encountered due to the hardness of some of the bedrock. Generally, overburden soils and sedimentary rock which have a penetration of 3 inches or more per 50 blows as determined by the Standard Penetration Test (SPT) drive hammer or 2 inches or more per 100 blows of the Texas Cone Penetrometer (TCP) test can typically be excavated with normal excavating equipment. Sedimentary rock which has a penetration of less than 3 inches per 50 blows of the SPT drive hammer or 2 inches or less per 100 blows of the Texas Cone Penetrometer (TCP) test will typically require heavy duty equipment for excavations. However, soft seams of non-isotropic strata within the sandstone may allow for the excavation of the sandstone without the use of pneumatic breakers. Blasting of harder strata may be required. However, PSI does not recommend the use of explosives in the excavation for this project. It should also be noted the "Engineering Classification of Geologic Materials" manual published by O.D.O.T. denotes the apparent rippability of the bedrock materials (as defined for the Hennessey unit in Oklahoma County) as rippable. The ODOT publication also defines rippability as the susceptibility of a rock to be broken by a ripping device as pulled by a Caterpillar D9 or its equivalent. This report should be provided in its entirety to bidding contractors so that analysis of the rock hardness can be performed, and determination of the best removal techniques made where necessary.

3.1 SITE PREPARATION

Soft or loose soils encountered in the footing excavations should be undercut, moisture conditioned and recompacted and replaced with properly compacted crushed stone or replaced with CLSM or lean concrete. The undercutting activities should be witnessed by a representative of the geotechnical engineer and should be performed during a period of dry weather. Compaction of the backfills should be performed according to ODOT's construction specifications and the lean concrete or CLSM may be vibrated in place.

PSI's recommendation is that fill material be placed in a relatively uniform horizontal lift and be adequately keyed into stripped and scarified subgrade surface. PSI also recommends fill materials be free of organic or other deleterious material.

PSI does not recommend lifts be more than 8 inches of loose material and recommends compaction within the range of 1 percentage point below to 2 percentage points above the optimum moisture content value. If water must be added, it should be uniformly applied and thoroughly mixed into the soil by disking or scarifying. Each lift of compacted material should be tested by a representative of the geotechnical engineer prior to placement of subsequent lifts. All work must be performed according to ODOT construction criteria.

3.2 SHALLOW FOUNDATION RECOMMENDATIONS

Based on the heights of the wall (±11 feet) and the subgrade foundation materials, spread footing foundation system may be the most practical support for the proposed retaining wall "P". Footings bearing in or on the bedrock can be designed for nominal unit bearing capacity, q_n or q_{ult}, of 10 tsf. A strength limit state resistance factor for geotechnical design, ϕ, of 0.60 is applicable for the bedrock. Resistance factors at the service limit states should be taken as 1.0. The table below presents the suggested general nominal unit bearing capacities at each boring location for the subgrade bedrock profile encountered in the borings. It is important that the footing excavations be observed and documented prior to placement of concrete. PSI can perform this service if requested. The top of the footing should be a minimum of 2 feet below the finish ground line for frost protection. The footings should be designed so that the pressure under the footing is generally consistent for the rock type. If overturning or sliding will be a problem, the reinforcing or footings may be founded at a deeper elevation and/or longer reinforcing, or a larger footing width should be considered.

Recommended Footing Design Parameters

BORING NO.	DEPTH, Ft.	MATERIAL TYPE	FRICTION ANGLE OR UNDRAINED SHEAR STRENGTH, Cu	NOMINAL UNIT BEARING RESISTANCE, q_n, tsf	RESISTANCE FACTOR, Φ
WP-1	0 – 15	Bedrock	40° or 7.5 tsf	10.0	0.60
WP-2	0 – 15	Bedrock	40° or 7.5 tsf	10.0	0.60

The nominal adhesion for the shale bedrock can generally be taken as 4 tsf and the nominal friction capacity for the siltstone and sandstone bedrock can be taken as 0.56. The nominal passive pressure for the shale bedrock and sandstone/siltstone bedrock may be generally taken as 15 tsf and 0.30 tsf/ft, respectively. The horizontal lateral displacement of the foundation should be investigated at the service limit state. The appropriate AASHTO resistance factors should be applied and may include for passive pressure, a ϕ_τ of 0.45 for sand and clay and 0.60 for the IGM and bedrock. For sliding, a ϕ_τ of 0.80 for cast-in-place concrete on soil and 1.0 for soil on soil is recommended.

Swelling of the soils is not expected to be a problem under the proposed structure.

The foundation excavations should be observed by a representative of PSI prior to steel or concrete placement to assess that the foundation materials are capable of supporting the design loads and are consistent with the materials discussed in this report. Soft or loose soil encountered at the bottom of the footing excavations should be removed to the level of acceptable residual soils or adequately compacted structural fill as directed by the geotechnical engineer. Cavities formed as a result of excavation of soft or loose soil zones should be backfilled with properly compacted or recompacted soil or lean concrete.

After opening, footing excavations should be observed, and concrete placed as quickly as possible to avoid exposure of the footing excavations to wetting and drying. Surface run-off water should be drained away from the excavations and not be allowed to pond. If possible, the foundation concrete should be placed during the same day the excavation is made. If it is required that footing excavations be left open for more than one day, they should be protected to reduce evaporation or entry of moisture.

Although it may not be applicable for the wall at this site based on the available information, it should be noted that, depending on the bottom of wall elevations, some portions of the wall may bear on dissimilar material (such as a portion may bear on soil while another portion may bear on IGM or bedrock). This condition has a potential to create excessive amount of differential settlement. If the wall designer deems the such may be the case and differential settlement is excessive for a cantilever wall or an MSE wall, it may be necessary to extend the foundations to the harder material or to undercut into the harder materials, as applicable. If undercut is required, it should be performed to a minimum depth of 2 feet below the planned wall bottom. The undercut should be backfilled with properly compacted and moisture conditioned site soil or structural fill.

3.3 DEEP FOUNDATION RECOMMENDATIONS

If the movements associated with a shallow foundation system are not acceptable, or if overturning or settlement of the wall (including potential for excessive differential settlement of foundation bearing on dissimilar materials) will be a problem, then a drilled shaft or steel pile foundation system can be considered for support of the proposed retaining wall, especially cast-in-place wall types. As discussed earlier, the use of a deep foundation system may not be practical, or feasible, or economically viable for the construction of the wall proposed, due to the bedrock been relatively at the surface at this site.

3.3.1 DRILLED SHAFTS

A straight drilled shaft foundation system can be used to support earth retention walls, especially conventional cast-in-place cantilever concrete wall system and when overturning and/or sliding may be of concern. The drilled shaft embedment into the subsurface strata should be sufficient to provide adequate vertical and lateral load capacities and acceptable displacement. The following table summarizes the soil and bedrock design parameters that may be used for the design of drilled shafts. Alternatively, the nominal capacities provided in the table of "Recommended Drilled Shaft Design Values" included at the end of this section may be used for individual drilled shafts or sections of the wall. The resistance factors presented in the general soil parameter table are applicable. A lateral load analysis can be performed using computer programs such as L-Pile or COM-624.

General design parameters for the drilled shafts for the proposed walls are as follows in the table below and presented in greater detail by soil strata at the end of this report section:

General Drilled Shaft Design Parameters							
Soil Type	End Bearing		Side Resistance		Lateral Modulus, (k) pci		Strain Factor, $^{(1)}E_{50}$ or $^{(2)}k_{rm}$
	Nominal Capacity (tsf)	Resistance Factor	Nominal Capacity (tsf)	Resistance Factor	Static or Unsaturated	Cyclic or Saturated	
Bedrock (Borings WP-1 & WP-2)	43.0	0.70	6.60	0.45	-	-	0.0003

Note: Total unit weight 135 pcf for shale, siltstone, & sandstone
Total unit weight 120 pcf for clay
Total unit weight 110 pcf for sand
Static/Cyclic designation is for clay criteria & Unsaturated/Saturated designation is for sand criteria

(1) for clay
(2) for rock

It is recommended that skin friction resistance in the rock socket be neglected for a depth equal to a drilled shaft diameter and overburden capacities be neglected for drilled shaft founded in bedrock.

The design loads are not known at this time but based on the known subsurface conditions and site geology, laboratory testing, and past experience, PSI anticipates that a properly designed and constructed single drilled shaft foundation supported on the recommended materials should experience maximum total settlements of less than ½ inch for the bedrock.

The resistance factors tabulated above are applicable to the soil parameters presented in the table at the end of this section.

Although not anticipated, casing will generally be required to reduce difficulties associated with sloughing of the soils and/or groundwater related problems. Once the casing is advanced and sealed into the bedrock the remaining soil can be removed, and the excavation can be pumped to remove groundwater remaining in the drilled shaft hole. Casing may be vibrated into the overburden material with a vibratory hammer. Alternatively, slurry displacement or a combination of slurry and casing can be used to stabilize the drilled shaft walls. Slurry displacement is anticipated for drilled shafts not extended into the bedrock. It will be advantageous to install casing when groundwater or wet soil is encountered. Fluid in the drilled shaft hole should be maintained at a higher elevation (typically 5 feet) above the groundwater outside the shaft. It should be noted that installed casing in excess of 60 feet may be difficult to extract without the use of a higher capacity crane or adequate vibratory hammer. PSI recommends polymer slurry for the granular overburden material encountered by the borings. If slurry is used, it should be designed, and the properties monitored during the excavation. Surface casing (typically 5 feet long) may be required to stabilize the top of the drilled shaft hole and to create an adequate work environment, especially the sandy subgrade profile.

The pier bottoms and sockets should be observed for continuity and to determine that the material is acceptable for support of structural loads and consistent with the material identified in the borings. Loose material should be removed from the pier bottoms and the sockets should be cleaned.

Drilled pier construction and inspection should be accomplished in accordance with the Oklahoma Department of Transportation (ODOT) Standard Specifications. The pier diameter should be sized to distribute the load to the soil or bedrock formations.

Typically, soil and bedrock penetrated by augers or the mud rotary drilling method used for this site can be removed with standard drill pier augers. Generally, the weathering process of shale, siltstone, and sandstone bedrock is erratic and variations in the bedrock profile can occur in small lateral distances. These bedrock depths should not be used for lump sum bid purposes. If a lump sum construction bid is necessary, the bedrock profile should be more completely defined prior to requesting bids. The bedrock depths should be confirmed by the engineer at the time of construction.

When the drilling operations and inspections are complete, concrete should be placed inside the casing or drilled shaft excavations immediately. During simultaneous concrete placing and casing removal operations, sufficient concrete head should be maintained inside the casing to offset the hydrostatic head of the groundwater outside the casing, and to prevent the intrusion of soil and groundwater and/or slurry into the pier concrete.

If slurry is used, it may be possible to pour the concrete under the wet slurry displacement condition. The concrete pipe or tremie should be maintained a minimum of 5 feet below the surface of the concrete in the excavation during the pour. Care should be exercised to avoid contamination of the drilled shaft concrete with the slurry. At the start of the concrete pour, the tip of the tremie or concrete pipe should not be more than 4 inches from the bottom of the drilled shaft hole.

The following table presents drilled shaft design parameters based on the individual soil borings performed for the wall design.

Recommended Drilled Shaft Nominal Design Values

Boring No.	Depth, Ft.	Undrained Shear Strength or Friction Angle	Nominal Unit Bearing Resistance, $(R_n)_{Brg}$, tsf	Nominal Unit Friction Resistance, $(R_n)_{Fr}$,tsf	Static Lateral Modulus, k, pci	Strain Factor, E_{50}	Strain Factor, k_{rm}	Material Type
WP-1	0 – 1	-	-	-	-	-	-	Pavement
	1 – 15	7.5 tsf	43.0	6.60	-	-	0.0003	Bedrock
WP-2	0 – 1	-	-	-	-	-	-	Pavement
	1 – 15	7.5 tsf	50.0	7.80	-	-	0.0005	Bedrock

3.3.2 STEEL PILES

As previously discussed, pilot holes/predrilling will be required to install driven piles for this site as a foundation system, due to the bedrock been practically at the surface. The bedrock subsurface material was evaluated using the Texas Cone Penetrometer. Results from these tests are presented on the attached boring logs and in the tables included in the Appendix. If driven steel pile foundation is desired, once the upper soils and upper weathered bedrock materials are penetrated, the bearing capacity may be dependent on the structural capacity of the pile. PSI recommends steel H-piles driven into the bedrock, once the anticipated predrilling is completed.

Low displacement piles such as steel H-piles could be used due to their capability to penetrate, to a limited extent, the shale strata as well as withstand high driving stresses. The Federal Highway Administration recommends the

following "allowable" stresses for H-piles. If applicable, PSI recommends respective adjustments for LRFD design criteria.

AASHTO recommends maximum driving loads for top driven piles not exceeding $0.9\phi F_y A_g$ for steel piles in compression, according to section 10.7 of the AASHTO LRFD design manual. A resistance factor, ϕ, for strength limit state during driving of 1.00 is recommended.

AASHTO recommends a resistance factor for geotechnical strength limit state in axially loaded piles of $0.5\lambda_v$. Table 10.5.5.2 of the AASHTO LRFD Bridge Design Specifications presents values of λ_v that should be considered. The resistance factors for the service limit state may be taken as 1.00.

Piles should be driven to the minimum tip elevations and required ultimate capacity shown on the contract documents using, as a minimum, the appropriate form of the Engineering News Record (ENR) formula to determine the bearing capacity and should be terminated after the required resistance has been attained or as provided by ODOT or in the contract documents. For estimation purposes, and due to the hardness of the bedrock material, penetration of the steel piles into the bedrock to attain the adequate resistance (specified on plan) is summarized in the following table for each boring performed.

Estimated Steel Pile Penetration

Boring No.	Pile Penetration, inches	
	Nominal 10-inch Size	Nominal 12-inch Size
WP-1	2	2
WP-2	2	2

Note: Depending on the depth of the retaining wall footing, pilot holes will possibly be required to install the pile into the bedrock in order to attain proper steel pile length (minimum 15 ft. recommended).

This recommendation is based on Texas Cone values shown on the boring logs. The actual penetration can be more accurately determined once the pile section and the magnitude of the pile loads are available. The bedrock nominal unit end bearing capacity and nominal unit skin friction values provided for drilled shafts are applicable. PSI recommends a pile driving hammer with rated energy of 25,000 to 35,000 ft-lbs as a trial hammer in the pile "drivability analysis". However, hammer efficiencies may vary with manufacturers.

The pile driving hammer should have sufficient energy and efficiency to drive the piles to the required capacity and the minimum required penetration rate at the end of driving. The pile should not be overstressed during driving and driving should be discontinued if it is causing damage to the pile. Piles should not be spaced closer than three times the maximum pile dimension measured center to center. However, this requirement may be irrelevant for point bearing piles, as is generally the case for piles embedded into bedrock.

A more accurate pile capacity can be determined by performing more detailed analysis using the Wave Equation Method, although results for piles in bedrock have certain limitations. If required, PSI should be contacted to provide this service. Using the wave equation analysis and pile load tests to estimate the driving loads and pile capacity may enable the use of a higher factor of safety in the pile capacity analysis.

The pile tips should be protected with prefabricated commercially available driving shoes, preferably specialized for pile driving into very hard rock, thereby resulting in a lower risk of damage. The tips should be welded to the H-piles with fillet welds along the outside of each flange. Welded reinforcement as provided for by ODOT in the "Standard Piling Details for Steel Piling" may be considered. Manufacturers recommend different shapes and

configurations of pile shoes for various applications. PSI recommends tips such as VS.310 and VS.312 by Versa Steel, or PAR-10T and PAR-12T by Piling Accessories, or HPH-10-RB and HPH-12-RB by International Construction Equipment, or equivalent for nominal 10-inch and 12-inch steel HP piles, respectively. If pile splicing is required, the number of splices should be kept at a minimum, as required by Section 514.04 of the ODOT Standard Specifications for Highway Construction, to protect the integrity of the piles. The axis of the spliced pile section should be straight. The estimated pile penetrations for HP piles can be provided by PSI once the design pile capacities are known. It should be noted that actual penetrations during construction may vary from the estimated values depending on changes in site conditions, hammer types and hammer operating efficiency.

3.4 RETAINING WALL LATERAL PRESSURE RECOMMENDATIONS

The actual earth pressure on the wall will vary according to material retained, backfill materials used, and how the backfill is compacted. Based on available information, the retaining wall will support the placed ramp embankment fill. PSI has provided in the following report section, design parameters for the in-situ bedrock and potential granular fill supported by the wall.

The values for lateral resistance design provided in this report section is based on a horizontal ground section behind the wall does not include hydrostatic pressure and also assume that positive drainage will be provided to prevent buildup of hydrostatic pressure.

PSI anticipates MSE walls may also be considered at this site. It should be noted that MSE walls are proprietary and the manufacturer's design recommendations and specifications may be different from those presented in this report.

For the in-situ bedrock and possible granular backfill, the following design parameters may be considered:

<u>In-Situ Bedrock</u>:					
γ (estimated)	=	135 pcf			
ϕ (assumed)	=	40°			
Active, K_a	=	0.22	Equivalent fluid pressure	=	28 psf/ft
Passive, K_p	=	4.60	Equivalent fluid pressure	=	621 psf/ft
At Rest, K_o	=	0.36	Equivalent fluid pressure	=	49 psf/ft
<u>Granular Backfill</u>:					
γ (estimated)	=	110 pcf			
ϕ (assumed)	=	32°			
Active, K_a	=	0.31	Equivalent fluid pressure	=	34 psf/ft
Passive, K_p	=	3.25	Equivalent fluid pressure	=	358 psf/ft
At Rest, K_o	=	0.47	Equivalent fluid pressure	=	52 psf/ft

The at-rest condition is applicable if walls cannot yield at the top during back-filling and service conditions. Passive pressure value is only applicable where a structure will push into the soil.

The walls should be designed to resist additional uniform lateral load based on the active coefficient to account for the imposed live loads. Care should be exercised during the backfilling of the walls to prevent overstressing and damage to the walls. Exterior subdrains should be installed to avoid the buildup of hydrostatic pressure behind the retaining walls.

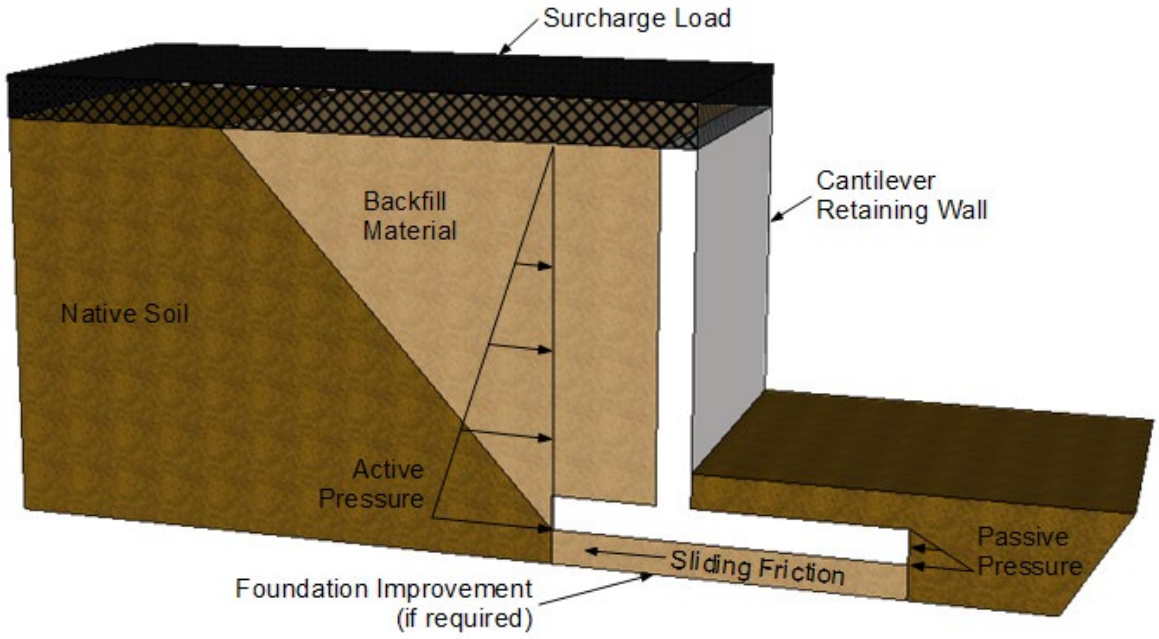
Information about the wall's backfill or retained material is not available from this investigation. However, the material may potentially consist of clays, silts, sands, or combinations. For compacted lean clay and silt, if permissible by ODOT, the following lateral pressure coefficient values behind the wall can be used.

Compacted Soil Lateral Pressure Coefficients				
Soil Type	**Active**		**At-Rest**	
	K_a	γ_{eq}, psf/ft	**K_o**	γ_{eq}, psf/ft
Clay	0.40	45	0.60	70
Silt	0.35	40	0.50	60

$\gamma_{eq,}$ - equivalent fluid pressure

Typically, only half of passive pressure, if applicable in the design, may be used to resist lateral loads due to the amount of strain required to fully mobilize the passive pressure. The applicable AASHTO design factors should be applied to the wall design.

The following illustration provides general components for the design and installation of cantilever retaining walls and is generally adaptable to MSE type wall, and with the exception that "native soil" will be representative of the embankment soil retained.



Be advised that as a part of the foundation design selection process, there is always a cost benefit evaluation. Although PSI is recommending a specific foundation type, PSI has not performed a cost/benefit evaluation.

4 CONSTRUCTION CONSIDERATIONS

4.1 MOISTURE SENSITIVE SOILS/WEATHER RELATED CONCERNS

Some of the upper fine-grained soils encountered at this site will be sensitive to disturbances caused by construction traffic and to 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 which become wet may be slow to dry and thus significantly retard the progress of grading and compaction activities. It will, therefore, be advantageous to perform earthwork and foundation construction activities during dry weather.

4.2 DRAINAGE AND GROUNDWATER CONSIDERATIONS

Water should not be allowed to collect in the foundation excavations 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, groundwater, or surface runoff. Positive site drainage should be provided.

Groundwater elevations could not be accurately measured after the drilling operations. However, measurements of groundwater observed to collect in the boreholes after a period of delay as well as borehole cave-in depths are provided in the "Groundwater Information" section of this report. It is possible that seasonal variations will cause fluctuations or a water table to be present in the upper soils. Additionally, perched water may be encountered in discontinuous zones within the overburden or near the contact with bedrock. Any water accumulation should be removed from excavations by pumping. Should excessive and uncontrolled amounts of seepage occur, the Geotechnical engineer should be consulted.

4.3 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". 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 PSI's understanding that these regulations are being strictly enforced and if not closely followed, the owner and the contractor could be liable for substantial penalties.

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. 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.

PSI is providing this information solely as a service to the client. PSI does not assume responsibility for construction site safety or the contractor's compliance with local, state, and Federal safety or other regulations.

5 REPORT LIMITATIONS

The recommendations submitted are based on the available subsurface information obtained by PSI and design details furnished by Poe & Associates, Inc. for the proposed project. If there are 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 recommendations are required. If PSI is not retained to perform these functions, PSI will not be responsible for the impact of those conditions on the project.

The Geotechnical Engineer warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed.

After the plans and specifications are more complete, the Geotechnical Engineer should be retained and provided the opportunity to review the final design plans and specifications to check that PSI's engineering recommendations have been properly incorporated into the design documents. At this time, it may be necessary to submit supplementary recommendations. This report has been prepared for the exclusive use of Poe & Associates and the Oklahoma Department of Transportation for the specific application to the proposed Crossroads Interchange Reconstruction Retaining Wall "P", I-35/I-240 Interchange in Oklahoma City, Oklahoma.

APPENDIX

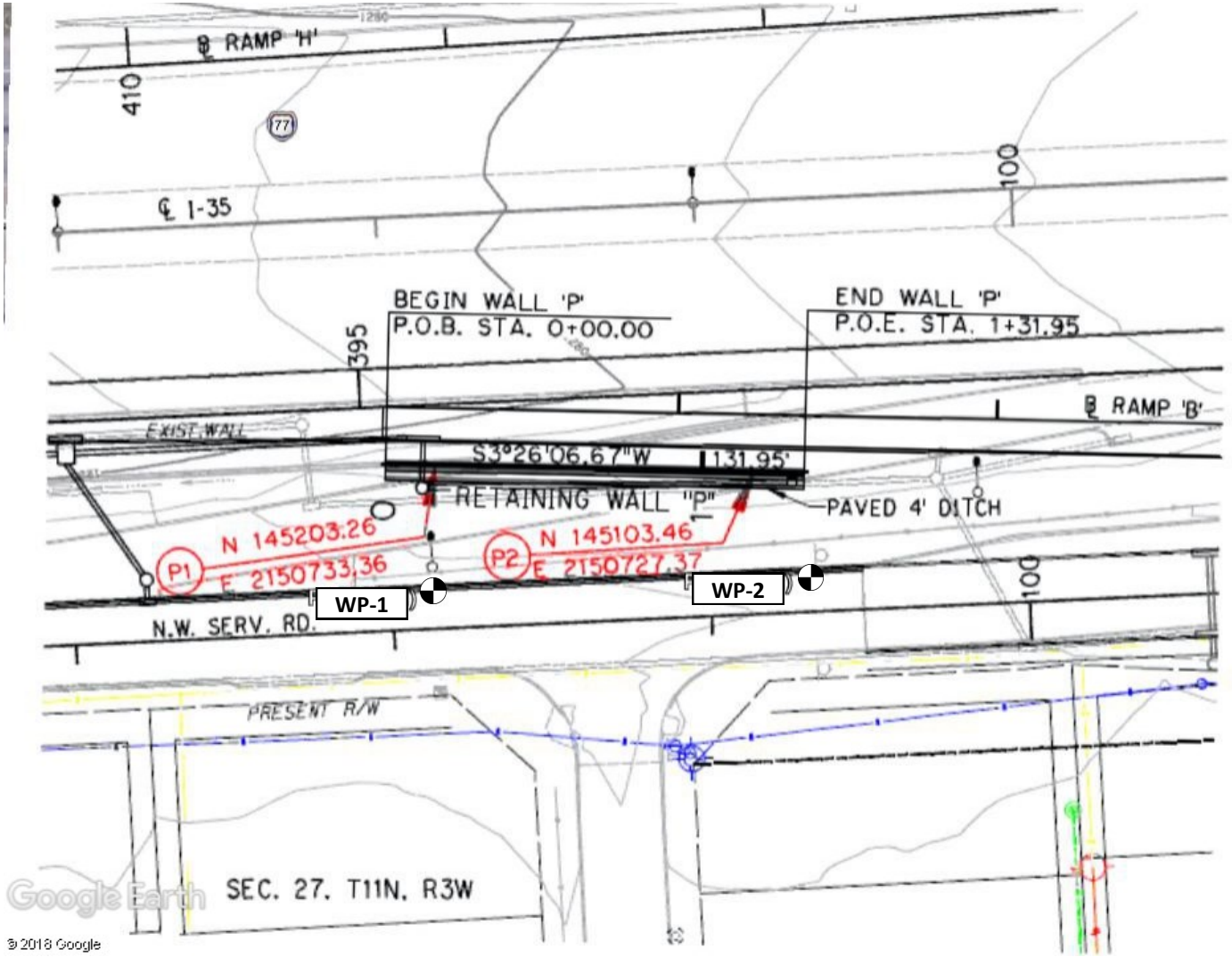
APPENDIX A – PIER DESIGN INFORMATION & TEXAS CONE PENETROMETER TEST RESULTS

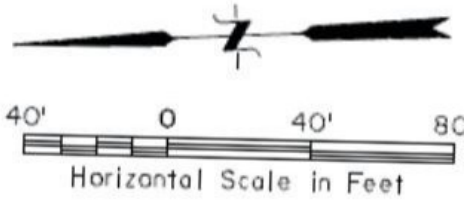
Crossroads Interchange
Reconstruction
RW "P" I-35/I-240 Interchange
Oklahoma County, Oklahoma
PSI File No. 05471208-12

LRFD PIER DESIGN INFORMATION & TEXAS CONE PENETROMETER TEST RESULTS

Boring #	Test Depth (feet)	*Actual Penetration (in./100 blows)	Elevation of Test (feet)	Nominal End Bearing Capacity (tsf)	Nominal Side Friction (tsf)
WP-1	3.0	1.3	1283.2	60	9.0
	5.0	0.6	1281.2	60	9.0
	7.0	1.6	1279.2	60	9.0
	10.0	2.9	1276.2	44	6.9
	15.0	2.0	1271.2	60	9.0
WP-2	3.0	1.6	1282.9	60	9.0
	5.0	1.0	1280.9	60	9.0
	7.0	1.5	1278.9	60	9.0
	10.0	2.5	1275.9	50	7.8
	15.0	1.6	1270.9	60	9.0

APPENDIX B – BORING LOCATION DIAGRAM





Earth Point

intertek psi	*Project* RETAINING WALL "P", I-35/I-240 INTERCHANGE—OKLAHOMA CITY, OK	
	Drawing BORING LAYOUT PLAN	*Project No.* 05471208-12
	Drawn By Y. Zhang	*Figure* **FIGURE 1**
	Date August 2019	

APPENDIX C – BORING LOGS AND BORING PROFILE

BORING WP-1

DATE STARTED:	4/11/19	**DRILL COMPANY:**	PSI, Inc.
DATE COMPLETED:	4/11/19	**DRILLER:** B. Rodgers	**LOGGED BY:** C. Bevers
COMPLETION DEPTH	15.0 ft	**DRILL RIG:**	CME-55
BENCHMARK:	N/A	**DRILLING METHOD:**	Solid Flight Auger
ELEVATION:	1286.20 ft	**SAMPLING METHOD:**	SS/TC
LATITUDE:	35.39782°	**HAMMER TYPE:**	Automatic
LONGITUDE:	-97.4955°	**EFFICIENCY**	N/A
STATION: N/A	**OFFSET:** N/A	**REVIEWED BY:**	A. Oyesanya

REMARKS:

Water		
	While Drilling	None observed
	Upon Completion	None observed
	Delay	N/A

BORING LOCATION:
145195.6018
2150689.3892

Elevation (feet)	Depth, (feet)	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) Texas Cone (TC)	Moisture, %	Additional Remarks
	0	1		12" **CONCRETE**			4	
1285		2	6	2" **BASE**, sandy, brown (Rock) **Sandy SILTSTONE**, moderately hard to very hard, red		50 (6.00")	11	LL = 20 PL = 18 Fines=68.5%
		3			ML	50 (0.88") 50 (0.50")	12	Non-Plastic Fines=56.1%
	5	4		(Rock) **Silty SANDSTONE**, very hard, red	SM	50 (0.50") 50 (0.13")	11	Non-Plastic Fines=36.0%
1280		5		(Rock) **Sandy SHALE**, hard, red		50 (1.00") 50 (0.63")	11	
	10	6				50 (2.00") 50 (0.88")	14	
1275								
	15	7		End of boring		50 (1.25") 50 (0.75")	12	

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

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

PROJECT NO.: 05471208-12
PROJECT: Retaining Wall "P"
LOCATION: I-35/I-240 Interchange
Oklahoma County, OK

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

BORING WP-2

DATE STARTED:	4/11/19	**DRILL COMPANY:**	PSI, Inc.
DATE COMPLETED:	4/11/19	**DRILLER:** B. Rodgers	**LOGGED BY:** C. Bevers
COMPLETION DEPTH	15.0 ft	**DRILL RIG:**	CME-55
BENCHMARK:	N/A	**DRILLING METHOD:**	Solid Flight Auger
ELEVATION:	1285.89 ft	**SAMPLING METHOD:**	SS/TC
LATITUDE:	35.39753°	**HAMMER TYPE:**	Automatic
LONGITUDE:	-97.49549°	**EFFICIENCY**	N/A
STATION: N/A	**OFFSET:** N/A	**REVIEWED BY:**	A. Oyesanya

REMARKS:

Water		
	While Drilling	None observed
	Upon Completion	None observed
	Delay	N/A

BORING LOCATION:
145090.0844
2150690.7304

Elevation (feet)	Depth, (feet)	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) Texas Cone (TC)	Moisture, %	Additional Remarks
1285	0	1		11" **CONCRETE**			4	
		2	5	2" **BASE**, sandy, brown (Rock) **Sandy SILTSTONE**, moderately hard to very hard, red		50 (5.00")	12	>> LL = 20 PL = 19 Fines=69.1%
		3			ML	50 (1.00") 50 (0.63")	12	>> Non-Plastic Fines=57.7%
1280	5	4		(Rock) **Silty SANDSTONE**, very hard, red	SM	50 (0.75") 50 (0.25")	11	>> Non-Plastic Fines=33.0%
		5		(Rock) **Sandy SHALE**, hard, red		50 (0.88") 50 (0.63")	12	>>
1275	10	6				50 (1.50") 50 (1.00")	12	>>
	15	7		End of boring		50 (1.00") 50 (0.63")	12	>>

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

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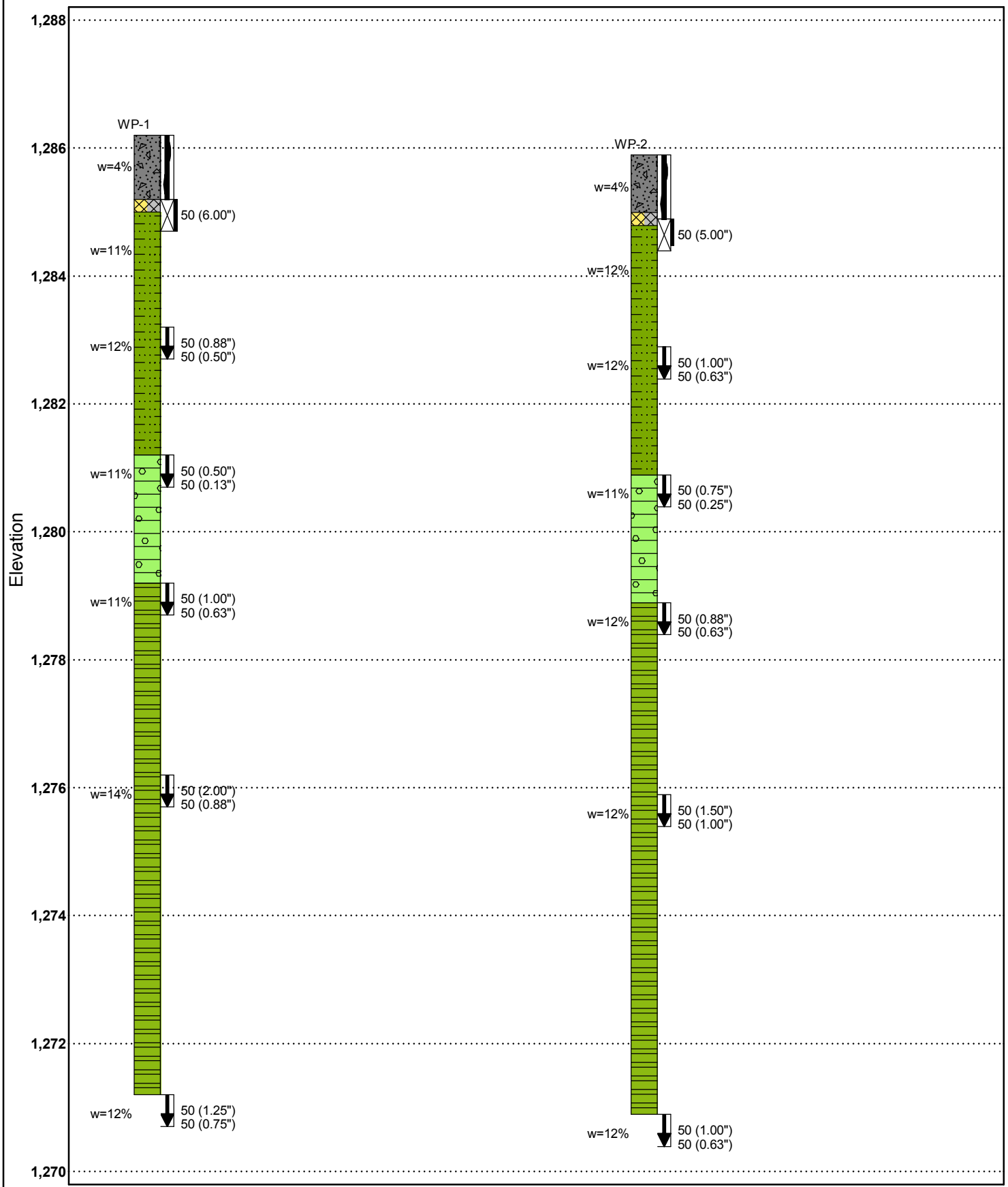
PROJECT NO.: 05471208-12
PROJECT: Retaining Wall "P"
LOCATION: I-35/I-240 Interchange
Oklahoma County, OK

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

Profile



Distance Along Baseline

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Retaining Wall "P"
PSI Project Number: 05471208-12

APPENDIX D – SUMARY OF TESTS IN THE OVERBURDEN & BEDROCK

I-35 I-240 Interchange
Oklahoma County

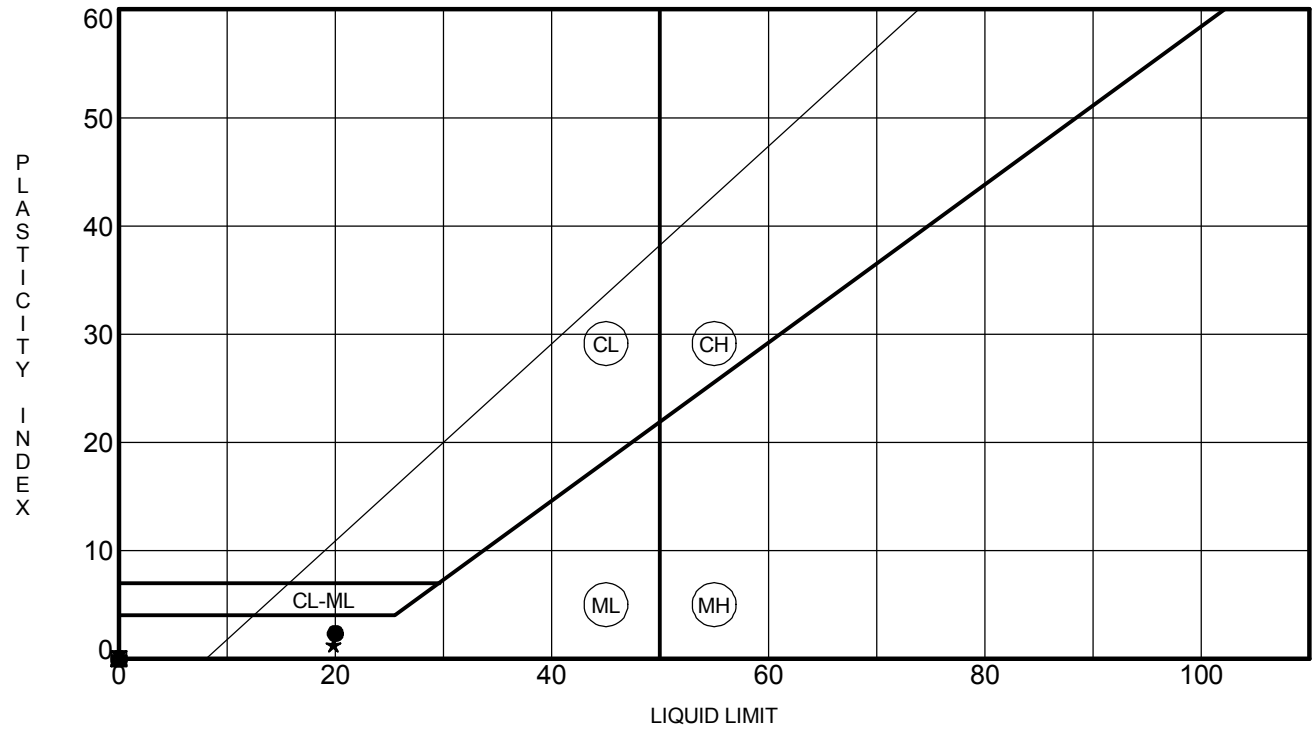
SUMMARY OF TESTS IN THE OVERBURDEN

PSI Project No.: 05471208-12

Boring No.	Sample Depth, ft.	Soil Description	N-Value, blows/ft	TCP, in/100	Moisture Content,	Atterberg Limits,			Grain Analysis, %						AASHTO	OSI
						LL	PL	PI	3"	¾"	#4	#10	#40	#200		
WP-1	0	Sandy SILTSTONE; Silty SANDSTONE; Sandy SHALE	-	-	4	-	-	-	-	-	-	-	-	-	-	-
	1		PR	-	11	NP	NP	NP	100	100	100	100	100	68.5	A-4(0)	0.0
	3		-	1.4	12	NP	NP	NP	100	100	100	98	95	56.1	A-4(0)	0.0
	5		-	0.6	11	NP	NP	NP	100	100	100	90	84	36.0	A-4(0)	0.0
	7		-	1.6	11	-	-	-	-	-	-	-	-	-	-	-
	10		-	2.9	14	-	-	-	-	-	-	-	-	-	-	-
	15		-	2.0	12	-	-	-	-	-	-	-	-	-	-	-
WP-2	0	Sandy SILTSTONE; Silty SANDSTONE; Sandy SHALE	-	-	4	-	-	-	-	-	-	-	-	-	-	-
	1		PR	-	12	NP	NP	NP	100	100	100	100	100	69.1	A-4(0)	0.0
	3		-	1.6	12	NP	NP	NP	100	100	100	98	96	57.7	A-4(0)	0.0
	5		-	1.0	11	NP	NP	NP	100	100	100	87	82	33.0	A-2-4	0.0
	7		-	1.5	12	-	-	-	-	-	-	-	-	-	-	-
	10		-	2.5	12	-	-	-	-	-	-	-	-	-	-	-
	15		-	1.6	12	-	-	-	-	-	-	-	-	-	-	-

LL - Liquid Limit; PL - Plastic Limit; PI - Plasticity Index;
NP - Non-Plastic; PR - Practical Refusal

APPENDIX E – LAB SUMMARIES



	Boring	Depth (ft)	LL	PL	PI	Fines	Classification (*Visual)
●	**WP-1**	**1.8**	**20**	**18**	**2**	**68.5**	**(Rock) Sandy SILT**
⊠	**WP-1**	**3.3**	**NP**	**NP**	**NP**	**56.1**	**(Rock) Sandy SILT**
▲	**WP-1**	**5.3**	**NP**	**NP**	**NP**	**36.0**	**(Rock) Silty SAND**
★	**WP-2**	**1.8**	**20**	**19**	**1**	**69.1**	**(Rock) Sandy SILT**
⊙	**WP-2**	**3.3**	**NP**	**NP**	**NP**	**57.7**	**(Rock) Sandy SILT**
✿	**WP-2**	**5.3**	**NP**	**NP**	**NP**	**33.0**	**(Rock) Silty SAND**

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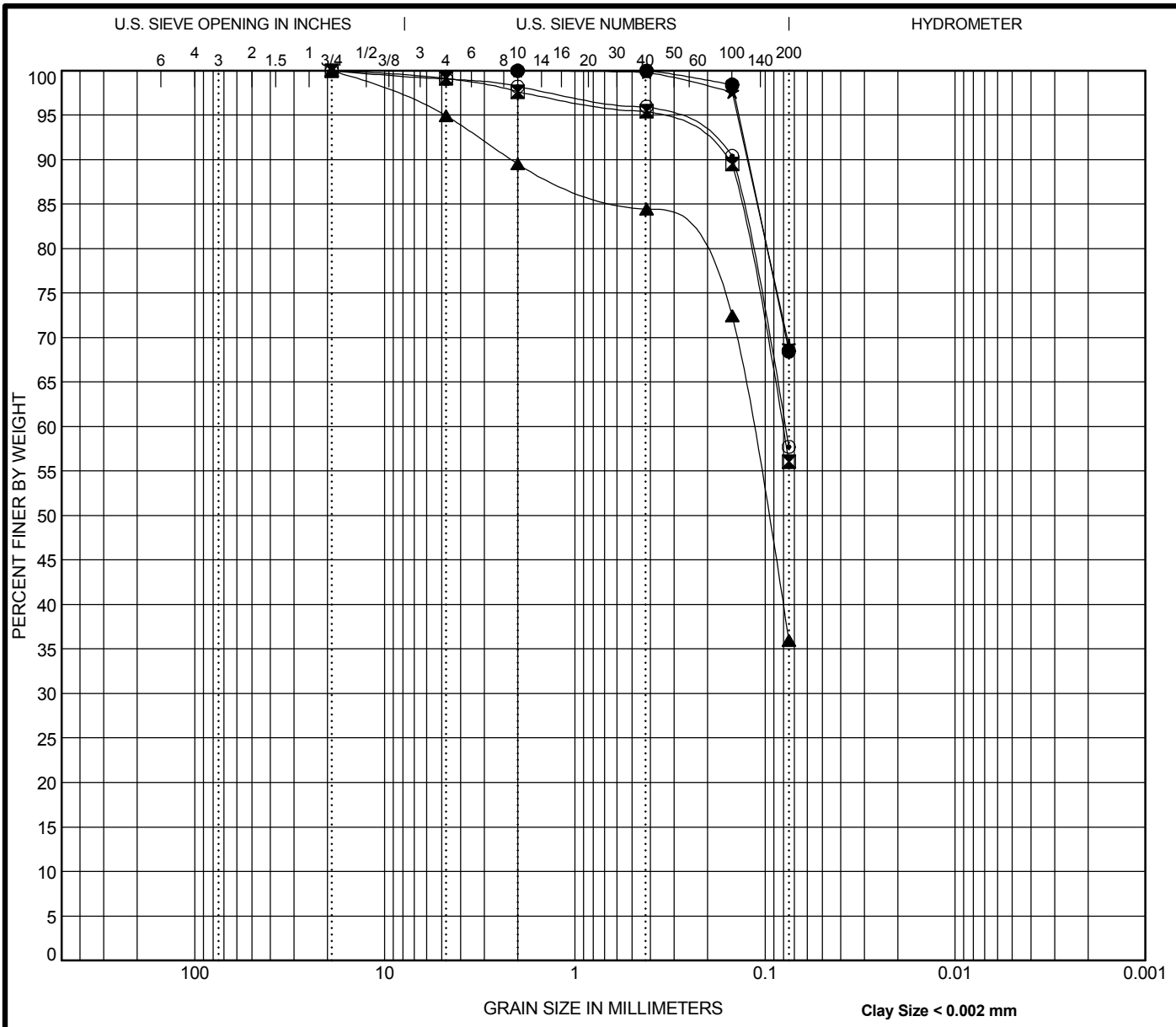
ATTERBERG LIMIT RESULTS

PSI Job No.: 05471208-12
Project: Retaining Wall "P"
Location: I-35/I-240 Interchange
Oklahoma County, OK



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	WP-1	1.8	(Rock) Sandy SILT	20	18	2		
⊠	WP-1	3.3	(Rock) Sandy SILT	NP	NP	NP		
▲	WP-1	5.3	(Rock) Silty SAND	NP	NP	NP		
★	WP-2	1.8	(Rock) Sandy SILT	20	19	1		
⊙	WP-2	3.3	(Rock) Sandy SILT	NP	NP	NP		

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	WP-1	1.8	2				0.0	31.5	68.5	
⊠	WP-1	3.3	19	0.081			0.9	43.1	56.1	
▲	WP-1	5.3	19	0.118			5.1	59.0	36.0	
★	WP-2	1.8	2				0.0	30.9	69.1	
⊙	WP-2	3.3	19	0.079			0.9	41.3	57.7	

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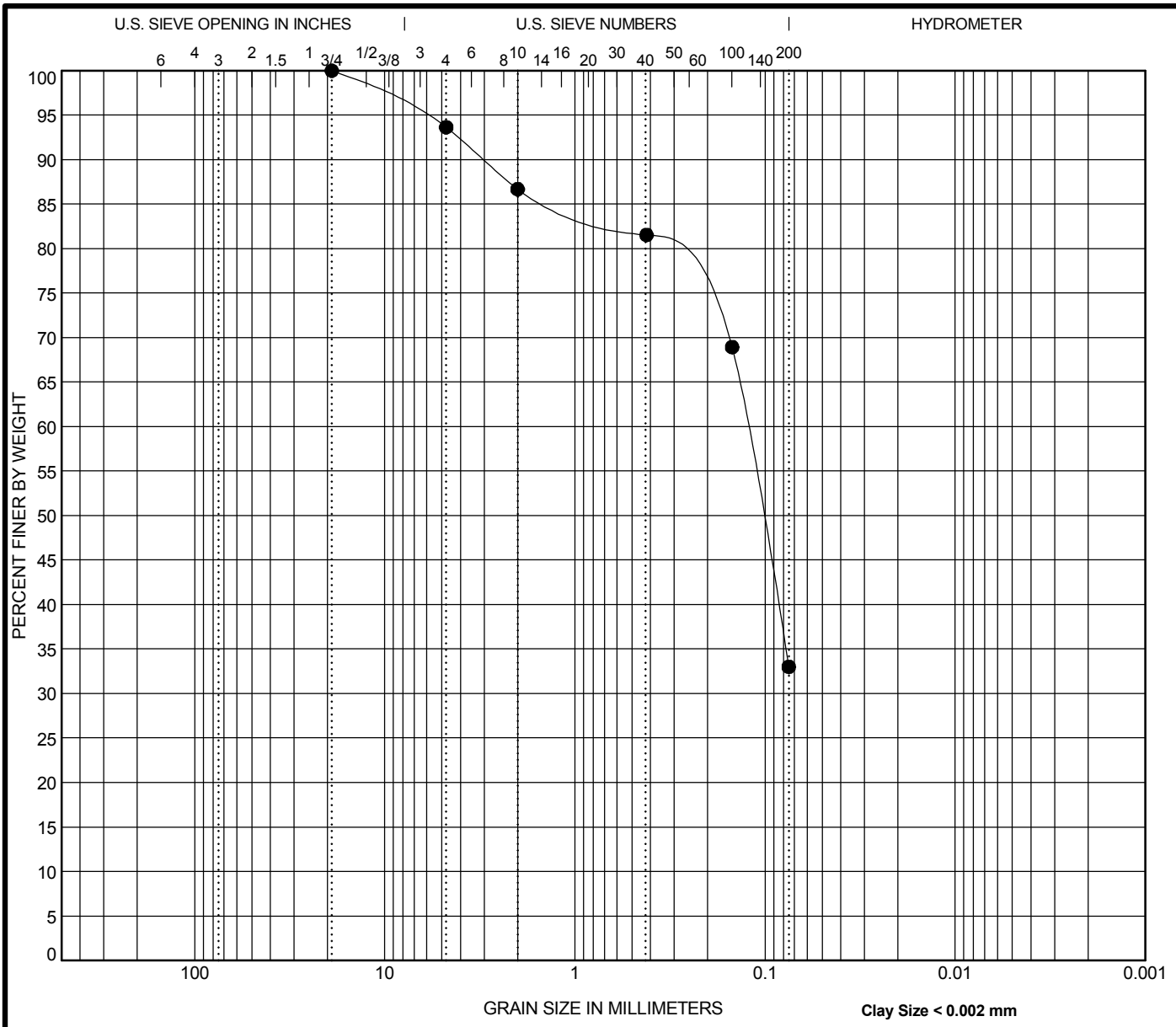
GRAIN SIZE DISTRIBUTION

Project: Retaining Wall "P"
PSI Job No.: 05471208-12
Location: I-35/I-240 Interchange
Oklahoma County, OK



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	**WP-2**	**5.3**	**(Rock) Silty SAND**	**NP**	**NP**	**NP**		

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**WP-2**	**5.3**	**19**	**0.126**			**6.4**	**60.6**	**33.0**	

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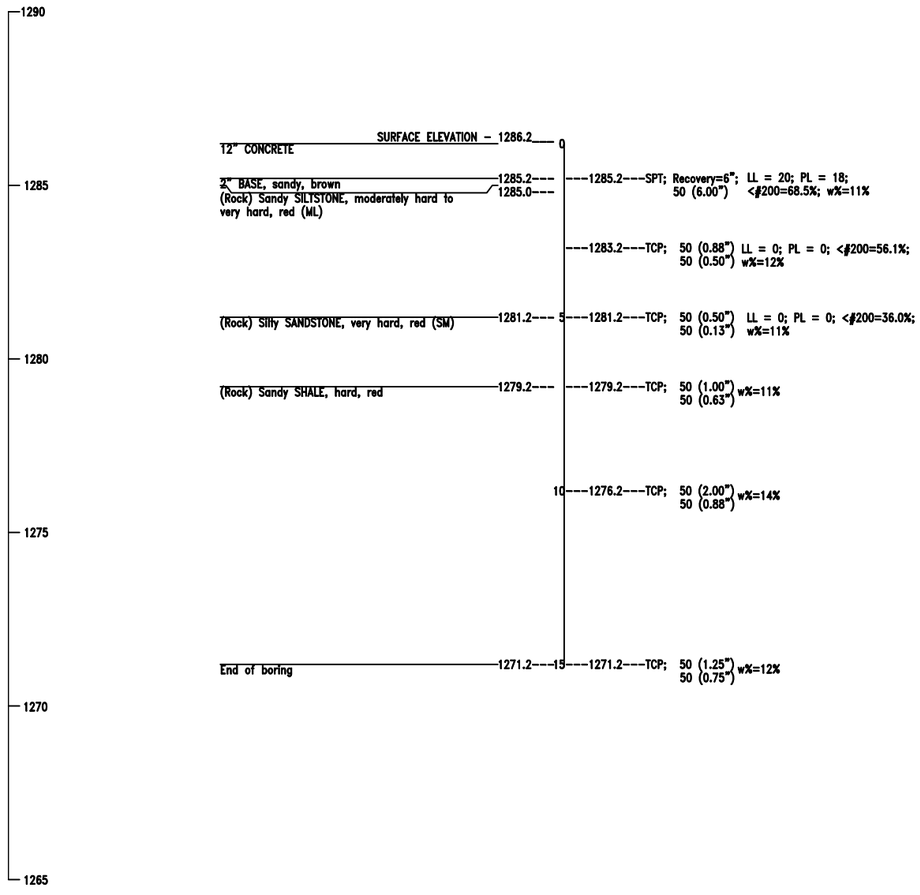
GRAIN SIZE DISTRIBUTION

Project: Retaining Wall "P"
PSI Job No.: 05471208-12
Location: I-35/I-240 Interchange
Oklahoma County, OK

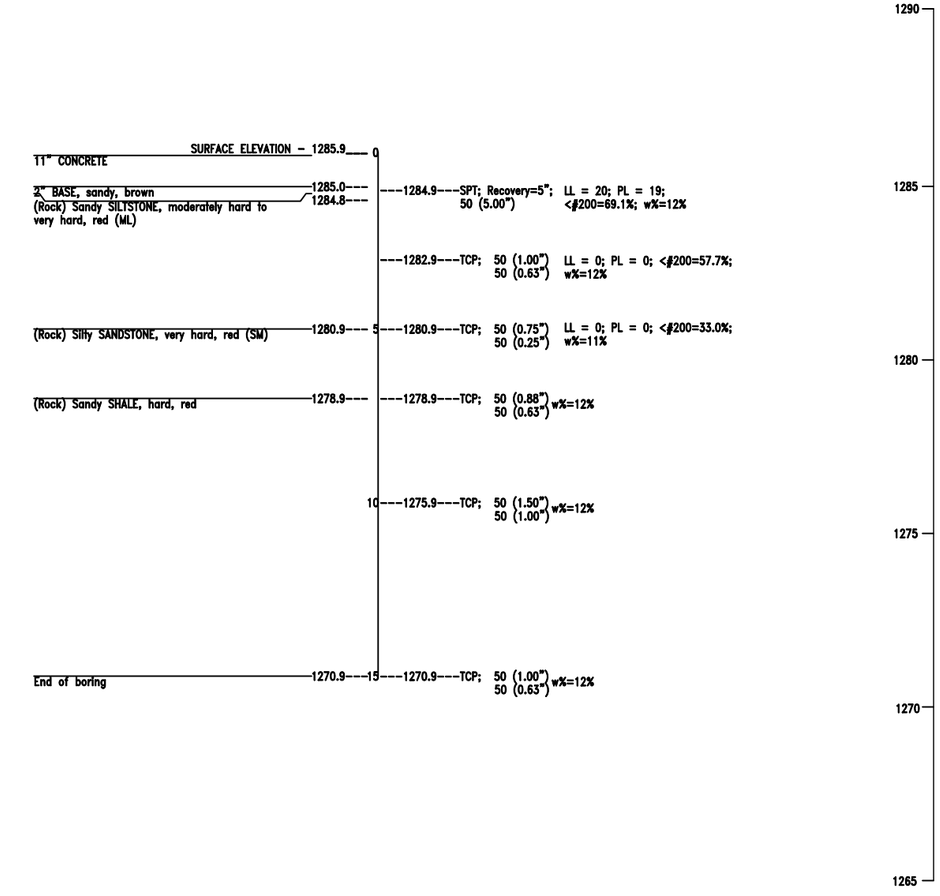
APPENDIX F – FOUNDATION PLAN

DESCRIPTION	REVISIONS	DATE

Boring Number WP-1
Northing: 145195.6018
Easting: 2150689.3892
Date Drilled: 04/11/19



Boring Number WP-2
Northing: 145090.0844
Easting: 2150690.7304
Date Drilled: 04/11/19



GEOLOGIC STATEMENT

"Division Four" of the "Engineering Classification of Geological Materials", published by the Oklahoma Department of Transportation (ODOT) indicates that the site is underlain by the Hennessey Unit (Phy). The geologic unit is described below.

This unit consists of red platy to blocky clay shales and mudstone. The mudstones are hard and appear blocky. The red clay shale of the Hennessey Unit is characterized by numerous bands of streaks of white or light green color ranging from a few inches to four feet in thickness.

The total thickness of the unit is about 400 feet.

The Hennessey Unit outcrops in a 5- to 15-mile north-south band across Grant, Garfield, Kingfisher, Logan, Canadian, and Oklahoma Counties in Division Four. Topographically, the unit is near level to gently rolling and is generally grass covered or cultivated.

GEOTECHNICAL REPORT

All geotechnical information contained on this sheet is covered by the engineering seal affized to an original geotechnical engineering report that has been stamped and sealed by a professional engineer licensed in Oklahoma. To obtain a copy of the complete report, contact the ODOT office engineer at (405) 521-2625. The contractor should be fully aware of the site conditions prior to beginning work. Any additional geotechnical information which may be desired is the responsibility of the contractor.

Note: Water level elevations shown were obtained at the time the borings were drilled and may fluctuate throughout the year.

NOTE: SPT – Split Spoon Sampling
TCP – Texas Cone Penetrator Test
AUG – Augur Cutting Sample
LL – Liquid Limit
PL – Plastic Limit

<#200 – Percent passing #200 sieve
w% – Moisture content
N – Number of blows per 12 inches
▪ Denotes depth to observed groundwater

DESIGN		
DRAWN	YZ	
CHECKED		
APPROVED		
SQUAD	PSI	

OKLAHOMA COUNTY
RETAINING WALL "P" AT I-35 & I-240 INTERCHANGE

PHASE – IV
FOUNDATION REPORT
(SHEET 1 OF 1)

STATE JOB NO. 09032(07) SHEET NO. BB

$TIME$
$DATE$
$FILE$

APPENDIX G – 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
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

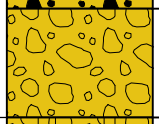
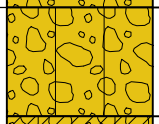
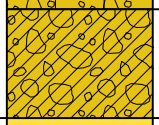
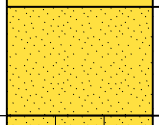
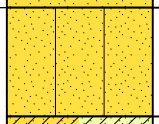
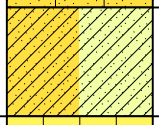
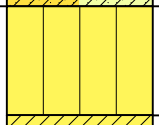
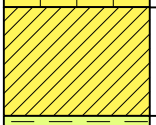
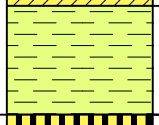
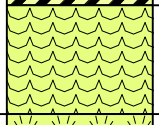
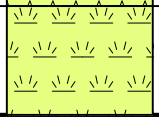
Rock Mass Description	RQD Value
Excellent	90 -100
Good	75 - 90
Fair	50 - 75
Poor	25 -50
Very Poor	Less than 25

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 GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	(LITTLE OR NO FINES)		GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	CLEAN SANDS		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	(LITTLE OR NO FINES)		SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES		SM	SILTY SANDS, SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS	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
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	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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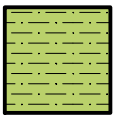
Graphic Symbols for Materials and Rock Deposits

CONCRETE
Portland Cement Concrete

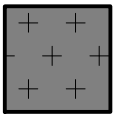
BITUMINOUS CONCRETE



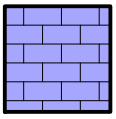
CLAYSTONE

COAL
Coal, Anthracite Coal

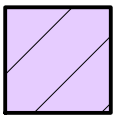
CONGLOMERATE/BRECCIA
Conglomerate, Breccia



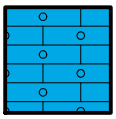
IGNEOUS ROCK
Anorthsite, Basalt, Metabasalt, Diabase (Gabbro), Gabbro, Granite/Granodionite, Homfels, Pegmatite, Rhyolite/Metarhyolite



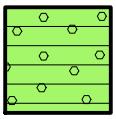
LIMESTONE
Limestone, Dolomite



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

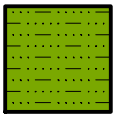


CHERT

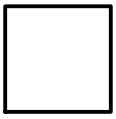


SANDSTONE
Sandstone, Orthoquarzite (Sandstone)

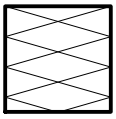
SHALE



SILTSTONE



NO RECOVERY



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