Report of Geotechnical Engineering Services

Proposed Crossroads Interchange
Reconstruction
Bridge "J"
NEQ I-35/I-240 Interchange
Oklahoma City, Oklahoma

Prepared for

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

Prepared by

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

December 13, 2017

PSI Project 05471207-7

Information To Build On
Engineering • Consulting • Testing

December 13, 2017

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

Attention: Mr. Tim Purkeypile, P.E.

Re: Geotechnical Engineering Services Report
Crossroads Interchange Reconstruction
Bridge "J"
NEQ I-35/I-240 Interchange
Oklahoma City, Oklahoma
PSI File No. 5471207-7

Dear Mr. Purkeypile:

Professional Service Industries, Inc. (PSI) 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,

PROFESSIONAL SERVICE INDUSTRIES, INC.
CA No. 1111 Exp. 6/30/2019

REGISTERED PROFESSIONAL ENGINEER
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12-13-17	

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1 PROJECT INFORMATION

1.1 PROJECT AUTHORIZATION

Professional Service Industries, Inc. (PSI) has completed a geotechnical exploration for the proposed Crossroads Interchange Reconstruction Project Bridge "J" at the NEQ of the I-35/I-240 Interchange in Oklahoma County, Oklahoma. PSI's services were authorized via The Oklahoma 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 bridge "J".

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 Bridge "J"

From the information provided, PSI understands the following about Bridge "J":

Structure
- The bridge will be an adjoining ramp to the north end of Bridge "A"
- The bridge will be 3-span, 450'-0" total length and skewed 0 degrees
- Span configuration: 3 - 150' Cont. Plate Girder Spans, 29' clear roadway with 42" F-shape parapets
- Centerline station: 85+09.90 Baseline Ramp "J"
- Begin bridge station: 82+84.90
- Abutment end bridge station: 87+34.90
- Pier No. 1 station: 84+34.90
- Pier No. 2 station: 85+84.90
- Single pier support at abutting spans

Grading
- Existing ground lines will be essentially maintained
- An 8' x 4' RCB with flowline elevation of 1279.95 feet is proposed at Bridge Span No. 2, between Piers No. 1 and 2

Loading
- Load and Resistance Factor Design (LRFD) criteria

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, bridge 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 PURPOSE AND SCOPE

The purpose of this study was to explore the subsurface conditions at the proposed Bridge "J" site in order to prepare geotechnical recommendations for foundation design for the piers and abutment (at the end of Bridge "J") for the proposed ramp bridge construction. PSI's scope of services included drilling 4 soil test borings at the project site for the bridge into the bedrock, Pressuremeter testing, 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 for steel H-pile foundation system (though not anticipated), recommendations for drilled shaft foundation systems to transfer the bridge loads to the subsurface materials, and potential spread footing foundation to transfer the bridge's abutment load to the subsurface. 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 (as permitted for the site 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 Bridge "J" for the Crossroads (I-35/I-240) interchange reconstruction is located in Oklahoma City, Oklahoma. The proposed bridge alignment will be on a new grade in the northeast sector of the existing I-35/I-240 interchange. The surface around the project site was essentially grass covered at the time of drilling. Boring J-2 was performed through the existing I-35 Service Road asphalt surface.

2.2 SITE GEOLOGY

"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.

2.3 SUBSURFACE CONDITIONS

The site subsurface conditions were explored with 4 soil and bedrock test borings. Additionally, the borings were supplemented with Pressuremeter testing (PMT) in Boring J-03 to aid in evaluating lateral resistance and axial capacities. The borings were denoted in the field by Poe & Associates' survey crew with wooden stakes and paint. Elevations and boring locations were later provided to PSI by Poe and Associates, with the exception of Borings J-02 and J-03. The elevation at Borings J-02 and J-03 was determined by PSI using conventional optical techniques and available elevation of a nearby boring. The provided location and elevation and the PSI determined numbers for Borings J-02 and J-03 are reported on the boring logs and summarized in the table below.

Elevation and Location of Borings					
Location	Northing	Easting	Latitude	Longitude	Elev., ft.
Boring J-01	143203.30	2151409.47	35.39231	-97.49313	1290.6
Boring J-02	143317.15	2151311.34	35.39260	-97.49347	1287.4
Boring J-03	143419.54	2151202.60	35.39292	-97.49380	1285.4
Boring J-04	143510.42	2151088.29	35.39320	-97.49417	1285.9

The borings were advanced with solid stem auger and wet rotary drilling method. The borings were advanced from CME-55 Truck-mounted drill rigs equipped with automatic hammers using a 140-pound weight dropping 30 inches. Soil samples were routinely obtained during the sampling process. Drilling and sampling techniques were accomplished generally in accordance with AASHTO and ASTM procedures and under the supervision of a PSI geotechnical engineer.

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 in the appendix.

The boring samples within the overburden mainly consisted of lean clay, fat clay, and shaley clay with medium stiff to hard consistencies to depths of approximately 7.5 to 21 feet below the present surface. The overburden materials are underlain by siltstone, and shale that extended to the boring termination depths of approximately 38 to 50 feet below the drilling 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 5.3 inches of penetration to 100 blows for 0.6 inches of penetration.

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

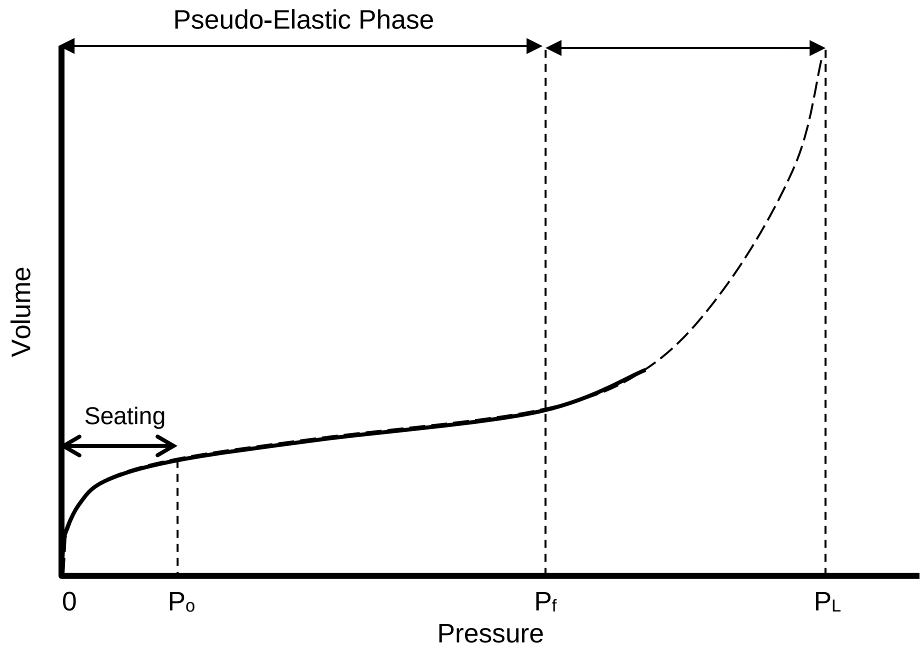
Boring Bedrock Depth Information				
Boring No.	Approx. Depth to Bedrock, Ft.	Approximate Bedrock Elevation, Ft.	Approximate Boring Depth, Ft.	Approx. Bottom of Boring Elevation, Ft.
J-01	7.5	1283.1	38.0	1252.6
J-02	15.0	1272.4	50.0	1237.4
J-03	21.0	1264.4	50.0	1235.4
J-04	11.0	1274.4	42.0	1243.4

2.3.1 PRESSUREMETER TESTING

In-situ Pressuremeter tests (PMT) were performed in the bedrock in Borings J-02 and J-03.

PSI typically utilizes a G-Am Pressuremeter using an N-sized probe for Pressuremeter testing of soil and geologic materials. The Pressuremeter test consists of placing a cylindrical probe in the ground and expanding the cylinder to pressurize the soil horizontally into the wall of the specially prepared borehole. The pressure on the soil and the relative increase in the cavity radius are thereby obtained to generate an insitu stress/strain measurement. The relationship between the pressure required to expand the cell into the surrounding bedrock and the corresponding increase in cell volume was analyzed to determine geotechnical parameters for the drilled shaft foundation axial capacity and to augment end bearing capacities, if required.

PMT is an in-situ load test measuring the stress-strain characteristics of the existing material with minimal disturbance. The process generally consists of drilling a hole to the desired test depth, near the circumference of the PMT probe, inserting the probe, and measuring the radial strain for various applied fluid pressures. During loading, a seating pressure is applied (0=>P_o) to mobilize relaxed soil from the drilling activities back to its initial state, which is near the in-situ stress. Once the seating load is applied the soil is further mobilized through the pseudo-elastic phase of the soil behavior (P_o=>P_f). In this zone, cyclic loading is commonly performed to evaluate the reloading soil response. Once the soil enters a plastic state, the strain rate rapidly increases with minor increases in applied stresses (P_f=>P_L). Failure is defined as the limit pressure (P_L) where the soil has to significantly strain to support the applied stress. Depending on the probe/soil capacity and size of the bore hole, it can be difficult to fully mobilize the soil enough to reach the limit pressure. The following diagram illustrates the stress-strain sequence for the PMT.



The results of the PMT tests are summarized in the table below. Complete reports of the PMT tests are included in Appendix F of this report.

Pressuremeter Test Results			
Boring	Approximate Test Probe Depth (ft)	PMT Modulus, E_m (psi)	Limit Pressure, P_L (psi)
J-2	22	9050	600
	33	1990	300
	37	68100	1200
J-3	27	9670	350
	37	40050	750
	47	7770	1000

It should be noted, PSI was unable to perform the tests to the PMT limit pressures provided. The limit pressures above were extrapolated based on the quadratic trends in the attained test data.

For the client's information, PSI was going to perform PMT testing in the overburden soil material. However, due to the relatively shallow depths to bedrock relative to fixity of the drilled shaft, information at such shallow depths may not be considered of much use for such a sophisticated test in analyzing the shaft response to lateral displacements. The fixity of the drilled shafts is expected to be near the top of the bedrock or approximately 21 feet or less. Hence, soil and bedrock lateral resistance p-y parameters were not developed.

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 borings during drilling, after completion, and up to 24-hour delay indicating the groundwater may be below the maximum depth of the borings or will require additional time to collect in the open holes. Boring J-02 performed in the roadway of the Service Road and backfilled prior to leaving the site is not expected to have significantly different water level than that measured during the fieldwork. 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.

Groundwater Depth Information			
Boring No.	Estimated Water Depth During Drilling, Ft.	Water Depth at Completion, Ft.	Delayed Water Depth
J-01	N/O	N/O	N/O
J-02	N/O	N/O	N/A
J-03	N/O	N/O	N/O
J-04	N/O	N/O	N/O

N/A – Not Applicable; holes performed in roadway and closed up after completion
N/O – None observed

3 VALUATION AND RECOMMENDATION

3.1 STEEL H-PILES

Steel H-piles supported spread footing may be considered for support of the abutment at the end of Span No. 3 and parts of the related retaining walls at the end of the bridge. Approximately ±10 feet of fill is expected at the end of the bridge and approach. Generally, moderately hard to very hard shale bedrock was encountered from approximate reference elevation of 1275 feet (approximate depth of 11 feet below grade) in Boring J-04. The bedrock material was evaluated using the Texas Cone Penetrometer (TCP) test for a depth of approximately 30 feet below the interpreted soil-rock contact. Results from the TCP tests are presented on the attached boring logs and in the tables included in the Appendix. Once the overburden soils and upper weathered bedrock materials are penetrated, the bearing capacity may be dependent on the structural capacity of the pile material.

Driven piles penetrating the bedrock can be used to support the abutment foundation system of the proposed bridge within the stratum provided adequate pile lengths are installed (15 feet or more recommended). However, construction of at least 15 feet or more length of steel pile may be problematic at this bridge site without the utilization of pre-drilling or pilot holes prior to driving. Low displacement piles such as steel H-piles can be used due to the capability to penetrate, to a limited extent, the bedrock strata as well as withstand high driving stresses.

Pre-drilling for adequate steel pile length installation, if necessary, can be performed in the shale bedrock to within approximately 1-foot of the planned pile tip elevation. The pile can then be driven the remainder of the way to capacity and anchorage. The hole can be backfilled to the original rock line with flowable fill such as controlled Low Strength Material (CLSM) or with lean Portland cement concrete such as Class "C" concrete. The backfill of the hole can then be completed with sand or excavated soil. It should be noted that ideally pilot holes can be performed with a slightly smaller diameter to allow for a snug fit during pile driving into softer or weakly cemented rocks. However, for client's information, such practice in very hard rocks can cause torsional stress/distortion of the steel pile during driving and slightly larger diameter pilot holes may be necessary for such rocks. Larger holes may also be necessary for integral abutment type (in applicable bridges) pile installation to allow for the flexibility of the piles along the weak-axis orientation. However, standard abutment type with vertical and battered piles is indicated for this bridge. Shale encountered at this site is expected to lend itself to pile driving into pilot holes that are performed with diameter that are slightly smaller than the longest diagonal cross-section measurement of the pile. The pile installation contractor should monitor the pile response to the driving to determine if the holes should be made larger. Since use of pier drilling equipment will be part of the overall bridge foundation construction needs, issue of mobilizing two types of construction equipment (pile driving and pier drilling) simultaneously, may not be of consideration for this project. Alternatively, if the proposed abutment configuration allows (i.e. not integral type) a cast-in-place concrete drilled shaft system may be considered for support of the bridge abutment loads.

AASHTO recommends maximum driving loads for top driven piles not exceeding $0.9\varphi F_y A_g$ 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 1.0 is recommended.

AASHTO recommends a resistance factor for the 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 based on the pile installation and pile capacity verification method. The resistance factors for the service limit state may be taken as 1.0.

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 or the ODOT modified Gates Formula to determine the bearing capacity and should be terminated after the required resistance has been attained or as provided for by ODOT in the contract documents. Information about the pile construction and anticipated factored pile load are not available to PSI. However, for estimation purposes, and due to the relatively hard but weathered and shallow conditions of the bedrock upper material at the interface with the overburden, penetration of the steel piles into the bedrock at the bottom of the required pilot hole is estimated to be on the order of approximately 2 to 3 feet to attain required bearing resistance at the abutment, and the anticipated resistance to driving without damage. This recommendation is based on the Texas Cone penetration (TCP) values shown on the boring logs. The actual penetration can be more accurately determined once the magnitude of the pile loads and pile cross section are available. The bedrock nominal unit end bearing capacity and nominal unit skin friction values provided for drilled shafts in the pier design information table in the appendix are applicable. PSI recommends a pile driving hammer with rated energy in excess of 25,000 ft-lbs. as a trial hammer in the analysis of drivability, but could vary based on pile design loads. Hammer efficiencies also may vary with manufacturers, cushion material, age, maintenance, etc. Backfiring or misfiring pile driving hammers should not be allowed for the pile installation.

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 driving 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 and is not expected to present a problem for this project.

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. Depending on the method of controlling installation of piles and verification of pile capacity during driving or after driving, values of the driving resistance factor, λ_v, can be as low as 0.80 for installation based on the ENR equation without stress wave measurement during driving. Using the wave equation analysis and pile load tests to estimate the driving loads and pile capacity may enable the use of a higher resistance factor (up to 1.0) in verifying the pile capacity. Table 10.5.5.2 of the AASHTO LRFD Bridge Design Specifications presents the applicable factors.

The pile tips may be protected with prefabricated commercially available driving shoes, preferably specialized for pile driving into very hard bedrock material, 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" can also be considered. Manufacturers recommend different shapes and configurations of pile shoes for various applications. For example, 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 engineer approved equivalent for nominal 10- 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 HP10X42 or HP12X53 steel piles or other steel 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. Settlement of H-piles driven into the bedrock at this site is expected to be negligible or within tolerable limits. Piles proposed for abutment wings and retaining walls need only be driven sufficiently into overburden soil or bedrock to provide anchorage to resist lateral loads and, as such, may not require pile driving shoes. Alternatively, support of abutment wings can consist of cast-in-place concrete drilled shafts.

3.2 DRILLED SHAFTS

Based on the results of the Texas Cone Penetrometer tests, drilled shafts for support of the piers (and if considered for the end of bridge abutment) founded in the siltstone, sandstone, and/or shale bedrock can be designed using the nominal unit end bearing pressure and nominal unit skin friction values presented in the table below and in the Appendix. Since the bridge will be a grade crossing, PSI recommends the drilled shaft sockets in the bedrock not be less than 5 feet and by PSI's estimation, may not necessarily exceed 15 feet deep from the top of the bedrock encountered, unless other circumstances such as the design loads including the lateral and torsional forces warrant deeper embedment. The recommended minimum embedment options have accounted for the weathered upper bedrock materials and softer zones encountered by the borings. The actual depth of embedment into the bedrock can be determined based on the drilled shaft diameter and the factored total service loads, P_u. Resistance factors of 0.7 and 0.45 are applicable to the nominal unit end bearing and nominal unit skin friction capacities, respectively, that are provided in the table for TCP test results. PSI recommends neglecting a section of the bedrock strata for friction capacity calculations. Discounting a minimum drilled shaft diameter, but not exceeding 5 feet, in the friction capacity calculations is recommended. PSI recommends considering the weaker stratum of the bedrock as the discounted section in the friction capacity calculations.

The total pier settlement (elastic shortening of shaft concrete plus shaft base settlement) is estimated to be less than 0.4 inch for a working load at the top of the socket, P_i, of about 1,400 kips indicating settlement will be minimal. Suggested drilled shaft design parameters and corresponding estimated penetrations are provided in the table below using the pertinent boring information. The embedment depths into the bedrock are estimated based on the TCP penetration and nominal capacities on the boring logs and in the table, "Foundation Design Information and Texas Cone Penetrometer Test Results," presented in the appendix. The drilled shaft design geotechnical parameters presented in the table below and in the appendix, were obtained from the Texas Highway Department chart titled, "Drilled Shaft Foundation Design," modified for LRFD design. The drilled piers should penetrate the bedrock strata to the depths shown on the final plans.

Drilled Pier Capacity Recommendations (Based on TCP Results)

Boring No.	Shaft Tip Embedment Below Top of Boring, Depth, ft.	Nominal Unit Bearing Resistance, $(R_n)_{Brg}$, tsf	Nominal Unit Friction Resistance, $(R_n)_{Fr}$, tsf	Comment
J-01	8.0 – 18.0	41	6.3	Neglect maximum 5' of top of bedrock in skin friction capacity calculation; as applicable
	18.0 – 43.0	60	9.0	
J-02	15.0 – 25.0	26	3.9	Neglect maximum 5' of top of bedrock in skin friction capacity calculation; as applicable
	25.0 – 55.0	60	9.0	
J-03	25.0 – 45.0	60	9.0	Neglect maximum 5' of top of bedrock in skin friction capacity calculation; as applicable
	45.0 – 55.0	44	6.9	
J-04	12.0 – 22.0	38	5.7	Neglect maximum 5' of bedrock in skin friction capacity calculation; as applicable
	35.0 – 55.0	60	9.0	

Note: Maximum 5 feet neglect of skin friction is PSI's recommendation; ODOT's practice
Is minimum 5 feet and may decide to exceed the maximum recommended above.

3.2.1 PRESSUREMETER CAPACITIES

PSI performed Pressuremeter Testing (PMT) in the bedrock at Borings J-02 and J-03. Based on AASHTO design criteria, Pressuremeter test results may predict much higher nominal bearing capacities than maximum limit allowed for TCP tests and can be used to enable higher design values where and if needed. Consequently, shorter and scaled down drilled pier sizes can be considered. The AASHTO design criteria does not incorporate skin friction resistance. The recommended nominal (ultimate) tip resistance R_u (LRFD R_n equivalent) from the AASHTO bearing capacity calculation is a function of the drilled shaft socket diameter (D_S) and the length of the socket (H_S). The ratio of the diameter/length gives an empirical non-dimensional coefficient (K_b). The equation for determining the ultimate capacity of a socketed shaft/pile in rock, R_u, is as follows:

$R_u = K_b (P_L - P_o) + \gamma_o$ where

P_L= limit pressure as determined from Pressuremeter tests in the zone extending two pile diameters above and below the pile toe

P_o= at rest horizontal stress in the rock at the elevation of the pile/shaft toe

γ_o= total overburden pressure at the toe of the pile/shaft

K_b = Bearing Capacity Coefficient (Derived from socket depth/Diameter of shaft)

The recommended Bearing Capacity Coefficient, K_b, as a function of normalized depth is cited in the table below:

PMT Bearing Capacity Coefficient						
Depth/Diameter (H_S/D_S)	0	1	2	3	5	7
K_b	0.8	2.8	3.6	4.2	4.9	5.2

* H_s = Rock socket length
D_s= Rock Socket diameter

Based on a minimum Pressuremeter limit pressure, P_L, of 300 psi, an at-rest horizontal stress, P_0, of 33 psi, a depth/diameter ratio of 1 (for a K_b value of 2.8), and neglecting the effect of overburden, the derived nominal bearing capacity for such a drilled shaft is more than 60 tsf for the bridge site. Actual end bearing nominal value is expected to exceed this assessment and resultant relative to the limited results from the TCP tests. This information is provided for the client's and ODOT's information and for their consideration in arriving at the engineering decision for the design of drilled shaft for Bridge "J".

Casing or slurry drilling or a combination of both is anticipated to be required to reduce difficulties associated with sloughing of the soils and/or groundwater related problems. Once 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. Alternatively, slurry displacement or a combination of slurry and casing can be used to stabilize the drilled shaft walls. 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 long may be difficult to extract without the use of a higher capacity crane or an adequate vibratory hammer. PSI recommends polymer or mineral slurry for the 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) will be required to stabilize the top of the drilled shaft hole and create an adequate work environment.

The pier excavations 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 excavations 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 bedrock formations.

Typically, soil and bedrock penetrated by augers or the rock drilling bit and wet rotary drilling method used for this site can be removed with standard drill pier augers. Generally, the weathering process of sandstone, siltstone, and shale bedrock is erratic and variations in the sandstone, siltstone, and shale bedrock profile can occur in small lateral distances. These sandstone, siltstone, and shale 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 place 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 placement, the tip of the tremie or concrete pipe should not be more than 4 inches from the bottom of the drilled shaft hole.

3.3 SHALLOW FOUNDATION

It is anticipated the tail end portions of the proposed retaining walls abutting the end of the bridge abutment wings may be supported on shallow continuous footing constructed in the existing subgrade or placed fill material over the natural subgrade. Additionally, if the conditions and requirements for deep foundation systems installation are not feasible or desired due to the relatively shallow depth to the underlying bedrock, especially at the bridge end abutment, a spread footing foundation embedded into the bedrock (if possible) may be a viable alternative foundation system for the support of the proposed bridge abutment and retaining walls. If additional settlements can be tolerated, the spread footings can also be constructed on the fill or natural overburden soils. For subgrade foundation materials prepared as recommended and with a presumptive minimum footing embedment of 2 feet into the recommended materials and below the finish grade, as applicable, the nominal unit end bearing capacity for spread footings (continuous at the abutment and continuous at walls) may generally be taken as 3.0 tsf and 10.0 tsf for overburden soil and bedrock, respectively. The nominal unit bearing capacities for the individual borings at the distinct soil bearing strata of the overburden and bedrock are tabulated below. A strength limit state resistance factor of 0.45 is applicable for sand and clay. Resistance factors at the service limit states should be taken as 1.0. Footings supported by the fill or overburden should be founded on the existing soil material that is properly compacted and prepared. It is important that the excavation be observed and documented prior to placement of concrete. The top of the footing should be a minimum of 2 feet below the finish ground line and should be a minimum width of 2 feet. The footings should be designed so that the material under the footing is generally consistent for the soil or rock type. If overturning will be a problem, the footings may be founded at a deeper elevation and/or a larger footing width should be considered.

For sliding, the nominal base adhesion for clays and the nominal base friction coefficient for granular materials can generally be taken as 0.30 tsf and 0.46, respectively. The nominal passive pressure for the clay and sand (if applicable) may be generally taken as 1.0 tsf and 0.33 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.

Swelling of the soils and uplift of the foundations are not expected to be a problem under the proposed structures.

The foundation excavations should be observed by a representative of the Geotechnical Engineer 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 or bedrock zones encountered at the bottom of the footing excavations should be removed to the level of acceptable residual soils or adequately compacted structural fill or to bedrock as directed by the Geotechnical Engineer. Cavities formed as a result of excavation of soft or loose soil zones or excavation of bedrock should be backfilled with properly compacted or recompacted soil or lean concrete or dense graded compacted crushed stone, respectively, and as applicable.

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.

The service loads are not known at this time, but based on the known subsurface conditions and site geology, laboratory testing and past experience, PSI anticipates properly designed and constructed footings supported on the soil materials will experience maximum total and differential settlements of less than 1 inch and ½ inch, respectively and footings supported directly on the bedrock will experience total and differential settlements less than ½ inch.

3.4 SEISMIC CONSIDERATION

If required in accordance with AASHTO Guide Specifications for LRFD Seismic Bridge Design, the following parameters are applicable for the site location and Site Class C considerations.

Seismic Design Parameters			
Period (seconds)	5% Probability of Event in 75 years (g)	Site Coefficient F_a	Site Coefficient F_v
PGA	0.095	NA	NA
S_s	0.178	1.20	NA
S_1	0.042	NA	1.70

A_s = 0.114g

S_{DS}= 0.213g

S_{D1} = 0.072g

3.4.1 SOIL LIQUEFACTION

Based on the field test data including the depths of the measured groundwater and bedrock, the calculated ground acceleration, estimated earthquake magnitude of 7.5, and the clayey nature of the materials encountered, liquefaction potential is not anticipated to be of concern and settlement due to liquefaction is estimated to be much less than 1 inch.

3.5 RIPPABILITY OF BEDROCK

Moderately hard to very hard bedrock material was encountered in the borings at depths of approximately 6 to 10 feet below the boring grades. Difficulties in excavating for the bridge construction may be encountered due to the hardness of some of the bedrock, especially at the deeper depths. 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 hydraulic excavating equipment with rock teeth. 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 bedrock may allow for the excavation of these soils without the use of pneumatic breakers or hydraulic ram type equipment. Blasting of harder strata may sometimes be required. However, PSI does not recommend the use of explosives in the excavation for this project, due to area structures and being in urban development areas. It should also be noted the "Engineering Classification of Geologic Materials" manual published by ODOT indicates that the apparent rippability of the bedrock materials (as defined for the Hennessey geologic unit in Oklahoma County) as "appears 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 they can analyze the hardness of the rock and determine the best removal techniques.

Other information available from the publication includes the following:

- Apparent material suitability: none
- Apparent seepage: some seepage in sandy shale
- Landslides or backslope failures: none noted

3.6 EXCAVATION STABILITY

Although not anticipated but if applicable, a vertical or near vertical cut should be possible in the bedrock for the construction period without lateral support. However, for the overburden soil, a maximum slope of 2:1 may be possible for short term construction period but should be made flatter or protected if slope will be in place for extended period.

The angle of the excavation side slopes should be decided based on the soil type and the shear strength of the excavated soil. The contractor should be aware that slope height, slope inclination, and excavation depth may not exceed the specified local, state, and federal regulations without the Engineer's approval.

3.7 LATERAL RESISTANCE DESIGN PARAMETERS

If required in the drilled shaft design, the information in this report section is general for the subsurface conditions encountered in the borings and is applicable for all pier designs.

For drilled shafts, the soils and bedrock, as well as the rigidity of the shaft will resist the lateral loads applied to the shaft. Lateral load analysis can be performed based on methods ranging from chart solutions to the 'p-y' approach utilizing computer programs such as LPILE or the public domain COM624.

The lateral design information regarding the 'p-y' data is provided in this section. The relationship between the soil resistance (p) and pile deflection (y) is commonly referred to as 'p-y'. Along the depth of the shaft, soil resistance (p) is expressed as a non-linear function of lateral shaft deflection (y). Various researchers developed 'p-y' criteria for different kinds of soils. The 'p-y' curves can be automatically generated utilizing the computer program LPILE or the public domain COM 624. The program LPILE was developed by Lymon Reese and Shin-Tower Wang, Ensoft, Inc. and based on the COM 624 developed for the FHWA by the authors and made available by the FHWA. The parameters for generation of 'p-y' criteria from LPILE as well as COM 624 are provided for the analyses of the shafts. If desired p-y criteria generated from the PMT tests can also be provided for use in the lateral resistance design considerations.

PARAMETERS TO BE USED IN THE LATERAL LOAD ANALYSES

Stratum	'p-y' Criteria	**Total Unit Weight, γ (pcf)	Su or Qu (psf), or ϕ (deg.)	Ks (pci) or Kc (pci) or E (psi)	ε50 or Krm or RQD
I*	Clay Criteria	120	Su = 1,000	Ks = 240 or Kc = 80	ε50 = 0.0082
II**	Sand Criteria	115	ϕ = 31º	$K_{unsat.}$ = 55 or $K_{sat.}$ = 35	-
III	Rock/IGM Criteria	130	Qu = 15,000	E = 50,000	krm = 0.0005 RQD = 30% (idealized)

Note: Su: Undrained Shear Strength (psf); Qu: Unconfined Compressive Strength (psf); ϕ, Angle of Internal friction; ks: modulus of subgrade reaction (pci) for static loading condition; kc: modulus of subgrade reaction (pci) for cyclic loading condition; kunsat.: modulus of subgrade reaction (pci) for unsaturated soil condition; ksat.: modulus of subgrade reaction (pci) for saturated soil condition; E: Initial modulus (psi); ε50: strain corresponding to one-half the principle stress. Krm: a constant for overall stiffness; RQD: Rock Quality Designation.

*Neglect the top 3' of soils (Stratum I or if applicable, II) for the lateral load analysis appropriately based on the location of the pile head

** For submerged portion of pier, use effective unit weight γ'

***Only encountered in Boring M-4 from depth of 7 to 10 feet

PSI can assist in performing the lateral response analysis under a separate work proposal.

4 CONSTRUCTION CONSIDERATIONS

4.1 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 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 Bridge "J", NEQ I-35/I-240 Interchange in Oklahoma City, Oklahoma.

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

Crossroads Interchange Reconstruction
Bridge “J” NEQ I-35/I-240 Interchange
Oklahoma County, Oklahoma
PSI File No. 05471207-7

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)
J-01	8.0	4.3	1290.6	29	4.5
	13.0	3.0	1277.6	41	6.3
	18.0	1.5	1272.6	60	9.0
	23.0	1.5	1267.6	60	9.0
	28.0	0.8	1262.6	60	9.0
	33.0	1.3	1257.6	60	9.0
	38.0	0.6	1252.6	60	9.0
J-02	15.5	5.0	1271.9	25	3.9
	20.0	4.8	1267.4	26	3.9
	25.0	1.3	1262.4	60	9.0
	30.0	1.3	1257.4	60	9.0
	35.0	1.0	1252.4	60	9.0
	40.0	0.9	1247.2	60	9.0
	45.0	1.0	1242.4	60	9.0
	50.0	0.8	1237.4	60	9.0

* initial 12 blows to seat the cone

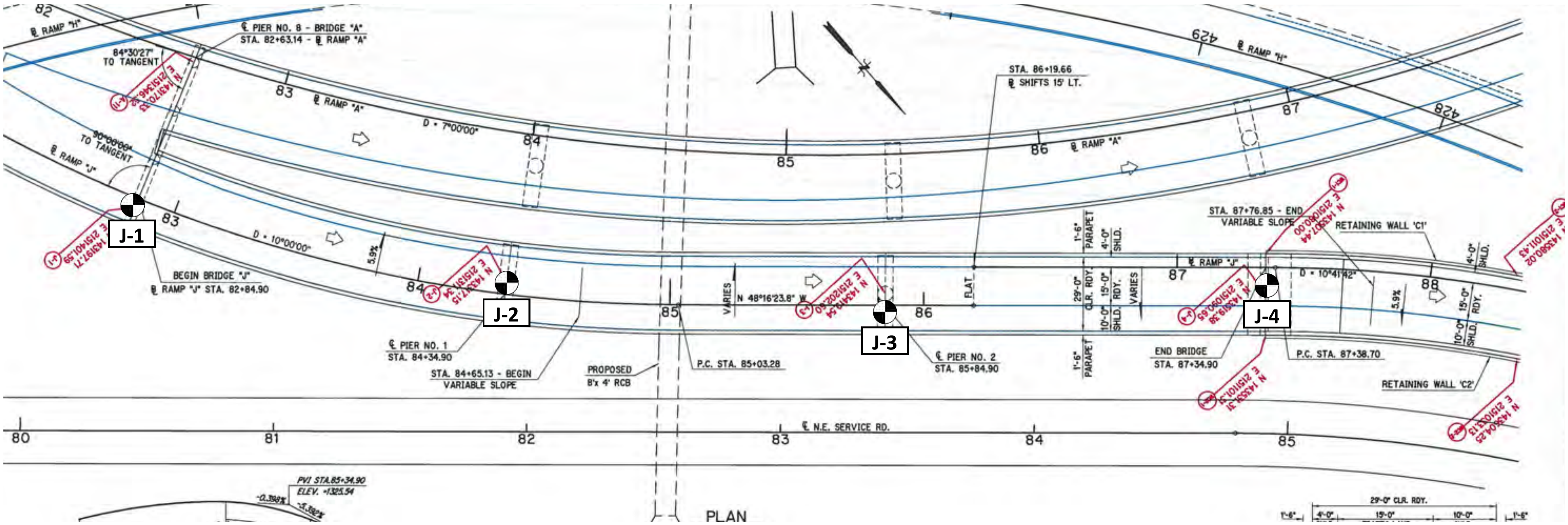
Crossroads Interchange Reconstruction
Bridge “J” NEQ I-35/I-240 Interchange
Oklahoma County, Oklahoma
PSI File No. 05471207-7

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)
J-03	25.0	5.0	1260.4	25	3.9
	30.0	2.0	1255.4	60	9.0
	35.0	1.5	1250.4	60	9.0
	40.0	0.6	1245.4	60	9.0
	45.0	0.8	1240.4	60	9.0
	50.0	2.8	1235.4	44	6.9
J-04	12.0	3.3	12.73.9	38	5.7
	17.0	5.3	1268.9	23	3.6
	22.0	1.0	1263.9	60	9.0
	27.0	1.5	1258.9	60	9.0
	32.0	0.6	1253.9	60	9.0
	37.0	1.3	1248.9	60	9.0
	42.0	0.8	1243.9	60	9.0

* initial 12 blows to seat the cone

APPENDIX B – BORING LOCATION DIAGRAM



DRAWN BY:
AAW

BRIDGE "J"
SWQ I-35/I-240 INTERCHANGE
OKLAHOMA CITY, OKLAHOMA

PSI REPORT NO.:
05471207-7

11825 S. PORTLAND AVENUE
OKLAHOMA CITY, OK 73170
(405) 735-6052

DATE:
DECEMBER 2017

APPROXIMATE BORING LOCATION

BORING LOCATION PLAN

APPENDIX C – BORING LOGS AND BORING PROFILE

BORING J-01

DATE STARTED:	5/9/17	**DRILL COMPANY:**	DSO
DATE COMPLETED:	5/9/17	**DRILLER:** B. Bettes	**LOGGED BY:** J. Sanders
COMPLETION DEPTH	38.0 ft	**DRILL RIG:**	CME-55
BENCHMARK:	N/A	**DRILLING METHOD:**	Solid Flight Auger
ELEVATION:	1290.631 ft	**SAMPLING METHOD:**	SS/TC
LATITUDE:	35.39231°	**HAMMER TYPE:**	Automatic
LONGITUDE:	-97.49313°	**EFFICIENCY**	N/A
STATION: N/A	**OFFSET:** N/A	**REVIEWED BY:**	

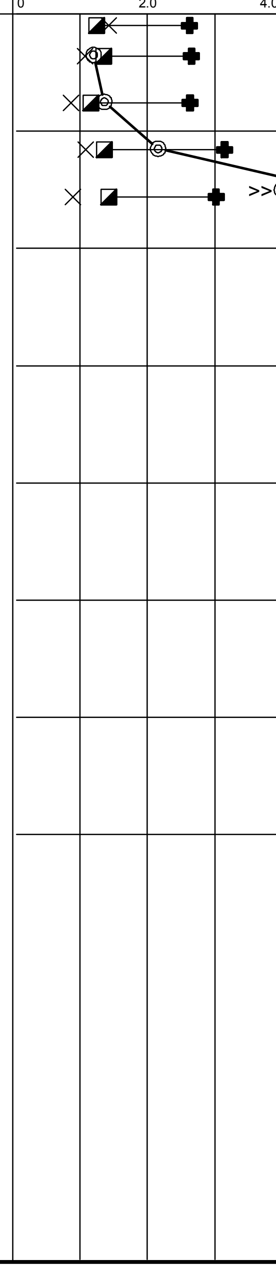
REMARKS: Grass at the surface

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

BORING LOCATION:
N: 143203.2957
E: 2151409.4673

Elevation (feet)	Depth, (feet)	Sample No.	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) Texas Cone (TC)	Moisture, %
1290	0	1	**Lean CLAY with Sand**, dark brown	CL		18
		2	**Lean CLAY with Sand**, very stiff, reddish-brown, calcareous nodule	CL	3/5/10 N=15	13
		3			7/8/9 N=17	11
1285	5	4	**Lean CLAY**, hard, reddish-brown, calcareous nodule	CL	7/9/18 N=27	14
		5			28	11
		6	**SHALE, Silty**, hard, red with gray		50 (3.00") 50 (2.75") 50 (1.50")	
1280	10					
		7	...siltstone interbedding		50 (2.00") 50 (1.00")	
1275	15					
		8	**SHALE, Silty**, hard to very hard, red with gray, siltstone interbedding		50 (0.75") 50 (0.75")	
1270	20					
		9			50 (1.00") 50 (0.50")	
1265	25					
		10			50 (0.50") 50 (0.25")	
1260	30					
		11			50 (0.75") 50 (0.50")	
1255	35					
		12	End of boring		50 (0.50") 50 (0.13")	

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



Additional Remarks

LL = 33
PL = 16
AASHTO: A-6(11)
OSI: 13.3
Fines=74.6%
LL = 33
PL = 17
AASHTO: A-6(12)
OSI: 13.0
Fines=81.2%
LL = 33
PL = 14
AASHTO: A-6(12)
OSI: 14.1
Fines=74.6%
LL = 39
PL = 17
AASHTO: A-6(19)
OSI: 16.6
Fines=89.2%
LL = 38
PL = 18
AASHTO: A-6(17)
OSI: 15.6
Fines=88.2%

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

PROJECT NO.: 05471207-7
PROJECT: Bridge "J"
LOCATION: I-35/I-240 Interchange
Oklahoma County, OK

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

BORING J-02

DATE STARTED:	10/25/17	**DRILL COMPANY:**	DSO
DATE COMPLETED:	10/25/17	**DRILLER:** J. Sanders	**LOGGED BY:** Dylan
COMPLETION DEPTH	50.0 ft	**DRILL RIG:**	CME-55
BENCHMARK:	N/A	**DRILLING METHOD:**	Solid Flight Auger
ELEVATION:	1287.4 ft	**SAMPLING METHOD:**	SS/TC
LATITUDE:	35.3926°	**HAMMER TYPE:**	Automatic
LONGITUDE:	-97.49347°	**EFFICIENCY**	N/A
STATION: N/A	**OFFSET:** N/A	**REVIEWED BY:**	

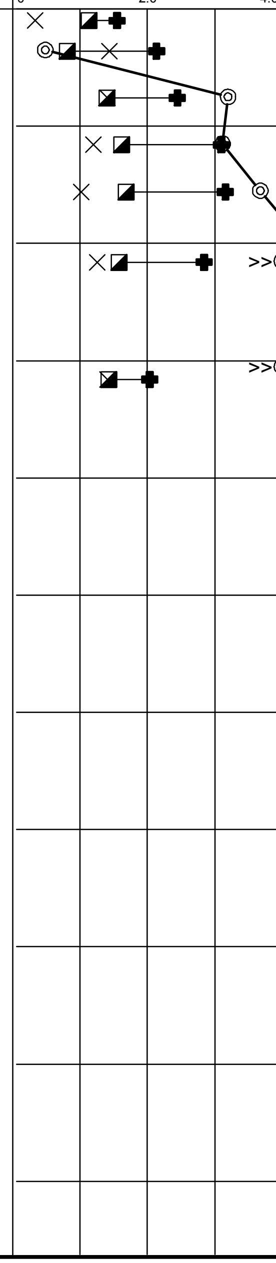
REMARKS: Asphalt at the surface

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

BORING LOCATION:
N: 143317.15
E: 2151311.34

Elevation (feet)	Depth, (feet)	Sample No.	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) Texas Cone (TC)	Moisture, %
	0	1	5" **ASPHALT**	SC		4
1285		2	**Silty, Clayey SAND with Gravel**, brown / **Lean CLAY with Sand**, medium stiff, brown	CL	3/3/3 N=6	18
		3	**Lean CLAY, Shaley**, hard, brown	CL	4/18/22 N=40	18
	5	4			17/19/20 N=39	15
1280		5			15/22/24 N=46	13
	10	6			20/25/32 N=57	16
1275						
	15	7	**Silty SHALE**, moderately hard to hard, red		50 (5.25")	
		8			50 (2.50") 50 (2.50")	18
1270						
	20	9			50 (3.00") 50 (1.75")	
1265		10				
			SILTSTONE, Shaley, hard to very hard, red with gray			
	25	11			50 (0.75") 50 (0.50")	
1260						
	30	12			50 (0.50") 50 (0.75")	
1255		13				
	35	14			50 (0.50") 50 (0.50")	
1250		15				
	40	16			50 (0.38") 50 (0.50")	
1245						
	45	17			50 (0.50") 50 (0.50")	
1240						
	50	18	End of boring		50 (0.50") 50 (0.25")	

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



Additional Remarks

LL = 19
PL = 14
AASHTO: A-1-b
OSI: 0.33
Fines=21.6%
LL = 27
PL = 10
AASHTO: A-6(10)
OSI: 12.2
Fines=78.6%
LL = 31
PL = 17
AASHTO: A-6(13)
OSI: 11.8
Fines=96.0%
LL = 39
PL = 20
AASHTO: A-6(18)
OSI: 15.4
Fines=91.6%
LL = 39
PL = 21
AASHTO: A-6(16)
OSI: 15.0
Fines=86.1%
LL = 35
PL = 20
AASHTO: A-6(14)
OSI: 13.0
Fines=93.5%
LL = 25
PL = 18
AASHTO: A-4(5)
OS: 7.8
Fines=91.0%

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PROJECT NO.: 05471207-7
PROJECT: Bridge "J"
LOCATION: I-35/I-240 Interchange
Oklahoma County, OK

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

BORING J-03

DATE STARTED:	10/27/17	**DRILL COMPANY:**	DSO
DATE COMPLETED:	10/27/17	**DRILLER:** J. Sanders	**LOGGED BY:** Dylan
COMPLETION DEPTH	50.0 ft	**DRILL RIG:**	CME-55
BENCHMARK:	N/A	**DRILLING METHOD:**	Solid Flight Auger
ELEVATION:	1285.4 ft	**SAMPLING METHOD:**	SS/TC
LATITUDE:	35.39292°	**HAMMER TYPE:**	Automatic
LONGITUDE:	-97.4938°	**EFFICIENCY**	N/A
STATION: N/A	**OFFSET:** N/A	**REVIEWED BY:**	

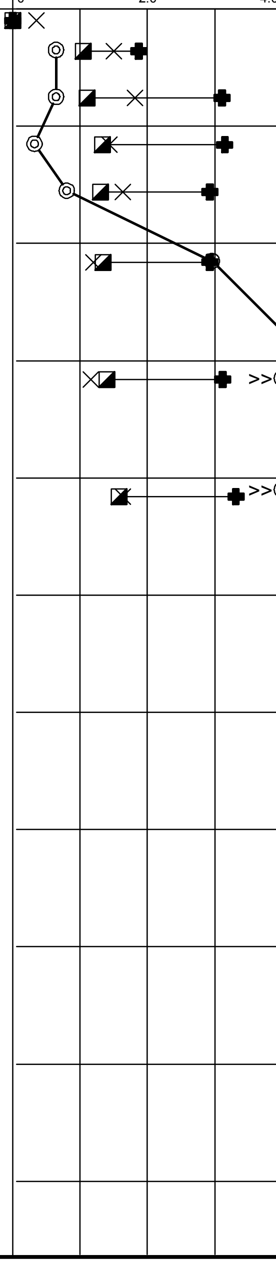
REMARKS: Grass at the surface

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

BORING LOCATION:
N: 143419.54
E: 2151202.60

Elevation (feet)	Depth, (feet)	Sample No.	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) Texas Cone (TC)	Moisture, %
1285	0		**Silty SAND**, brown	SM		4
		1	**Lean CLAY with Sand**, medium stiff to stiff, reddish-brown	CL	3/3/5 N=8	19
		2			2/3/5 N=8	23
1280	5	3			2/2/2 N=4	18
		4			2/3/7 N=10	20
1275	10	5	**Lean CLAY, Shaley**, hard, reddish-brown	CL	9/15/22 N=37	15
1270	15	6			15/22/37 N=59	14
1265	20	7	**SHALE**, moderately hard to hard, reddish-brown	CL	23 50 (5.00")	20
1260	25	8			50 (3.25") 50 (1.75")	
		9	**SANDSTONE**, hard to very hard, red			
1255	30	10			50 (1.00") 50 (1.00")	
1250	35	11			50 (0.75") 50 (0.75")	
		12				
1245	40	13			50 (0.50") 50 (0.13")	
1240	45	14			50 (0.50") 50 (0.25")	
		15				
	50	16	End of boring		50 (1.50") 50 (1.25")	

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



Additional Remarks

Non-Plastic
AASHTO: A-1-b
OSI: 0
Fines=18.2%
LL = 23
PL = 13
AASHTO: A-4(5)
OSI: 8.6
Fines=80.8%
LL = 39
PL = 14
AASHTO: A-6(20)
OSI: 17.8
Fines=84.0%
LL = 39
PL = 17
AASHTO: A-6(16)
OSI: 16.6
Fines=76.7%
LL = 36
PL = 16
AASHTO: A-6(18)
OSI: 15.2
Fines=92.7%
LL = 37
PL = 17
AASHTO: A-6(19)
OSI: 15.4
Fines=94.1%
LL = 39
PL = 17
AASHTO: A-6(21)
OSI: 16.6
Fines=92.7%
LL = 41
PL = 20
AASHTO: A-7-6(20)
OSI: 16.6
Fines=93.0%

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PROJECT NO.: 05471207-7
PROJECT: Bridge "J"
LOCATION: I-35/I-240 Interchange
Oklahoma County, OK

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

BORING J-04

DATE STARTED:	5/9/17	**DRILL COMPANY:**	DSO
DATE COMPLETED:	5/9/17	**DRILLER:** B. Bettes	**LOGGED BY:** J. Sanders
COMPLETION DEPTH	42.0 ft	**DRILL RIG:**	CME-55
BENCHMARK:	N/A	**DRILLING METHOD:**	Solid Flight Auger
ELEVATION:	1285.938 ft	**SAMPLING METHOD:**	SS/TC
LATITUDE:	35.3932°	**HAMMER TYPE:**	Automatic
LONGITUDE:	-97.49417°	**EFFICIENCY**	N/A
STATION: N/A	**OFFSET:** N/A	**REVIEWED BY:**	

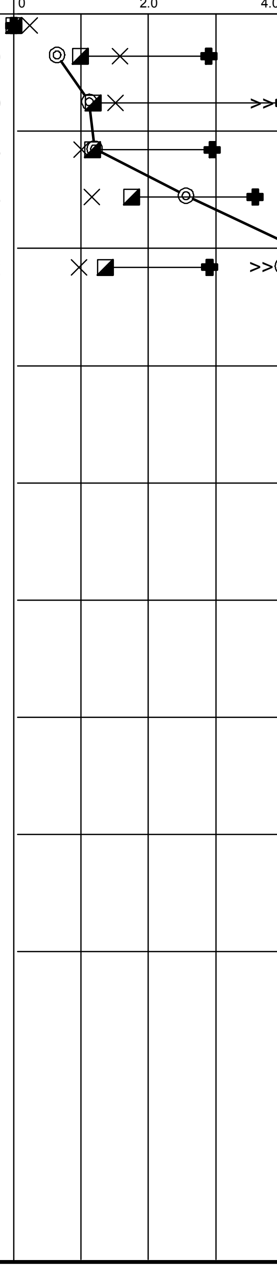
REMARKS: Grass at the surface

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

BORING LOCATION:
N: 143510.4236
E: 2151088.2877

Elevation (feet)	Depth, (feet)	Sample No.	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) Texas Cone (TC)	Moisture, %
1285	0	1	**Silty SAND with Gravel**, brown	SM		3
		2	**Lean CLAY**, stiff, brown to reddish-brown	CL	2/3/5 N=8	20
		3	**Fat CLAY**, stiff, reddish-brown, mottled with dark gray, calcareous inclusion from 5'	CH	3/5/9 N=14	19
1280	5	4	**Lean CLAY**, stiff to hard, reddish-brown	CL	4/6/9 N=15	13
		5			9/15//17 N=32	14
1275	10	6	**Lean CLAY, Shaley**, hard, reddish-brown	CL	28/40	12
		7	**SHALE, Sandy**, moderately hard to hard, red with gray, siltstone interbedding		50 (6.00") 50 (2.00") 50 (1.25")	
1270	15	8			50 (3.25") 50 (2.00")	
1265	20	9	**SHALE, Sandy**, hard to very hard, red with gray, siltstone interbedding		50 (0.50") 50 (0.50")	
1260	25	10			50 (1.00") 50 (0.50")	
1255	30	11			50 (0.38") 50 (0.25")	
1250	35	12			50 (0.75") 50 (0.50")	
1245	40	13	End of boring		50 (0.50") 50 (0.25")	

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



Additional Remarks

Non-Plastic
AASHTO: A-1-a
OSI: 0
Fines=12.7%
LL = 36
PL = 12
AASHTO: A-6(20)
OSI: 16.8
Fines=87.6%
LL = 66
PL = 15
AASHTO: A-7-6(51)
OSI: 33.6
Fines=92.8%
LL = 37
PL = 15
AASHTO: A-6(18)
OSI: 16.2
Fines=85.4%
LL = 45
PL = 22
AASHTO: A-7-6(22)
OSI: 18.2
Fines=90.5%
LL = 36
PL = 17
AASHTO: A-6(16)
OSI: 14.8
Fines=86.1%

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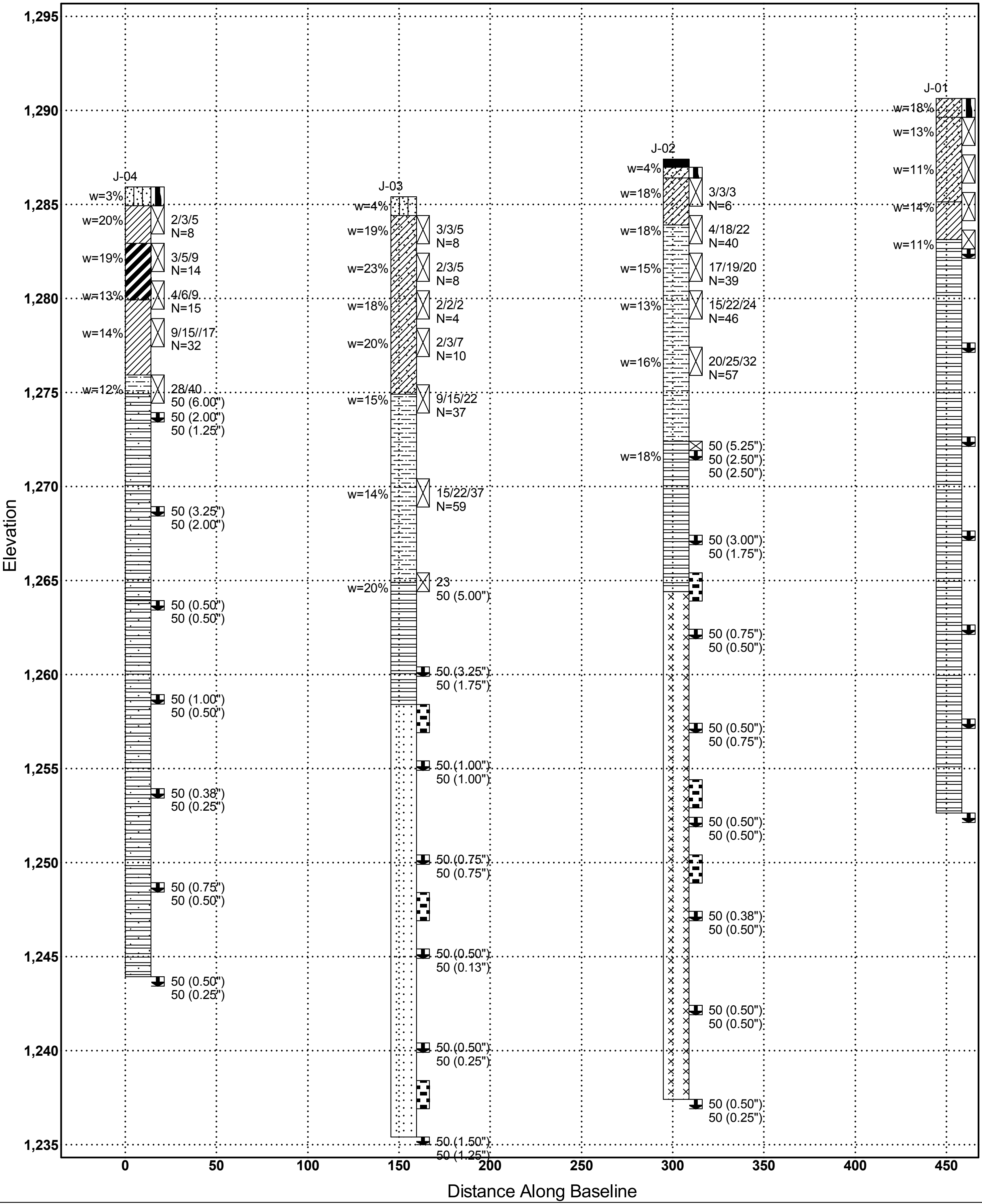
PROJECT NO.: 05471207-7
PROJECT: Bridge "J"
LOCATION: I-35/I-240 Interchange, Oklahoma County, OK

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

Profile



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Bridge "J"
PSI Project Number: 05471207-7

APPENDIX D – SUMMARY OF TESTS IN THE OVERBURDEN & BEDROCK

SUMMARY OF TESTS IN THE OVERBURDEN
Crossroads Interchange Reconstruction
Bridge “J” – NEQ I-35/I-240 Interchange
Oklahoma County, Oklahoma
PSI File No. 05471207-7

Boring No.	Sample Depth, Ft.	Soil Description	N Blows/ft.	TCP inch /100 Blows	Moisture Content, %	Atterberg Limits, %			Grain Analysis, %						AASHTO	OSI
						LL	PL	PI	3”	¾”	#4	#10	#40	#200		
J-01	0	Lean CLAY, Lean CLAY with Sand, SHALE-Silty	-	-	18	33	16	17	100	100	99	98	90	74.6	A-6(11)	13.3
	1.0		15	-	13	33	17	16	100	100	98	95	91	81.2	A-6(12)	13.0
	3.0		17	-	11	33	14	19	100	100	99	96	91	74.6	A-6(12)	14.1
	5.0		27	-	14	39	17	22	100	100	100	99	96	89.2	A-6(19)	16.6
	7.0		PR	-	11	38	18	20	100	100	100	99	96	88.2	A-6(17)	15.6
	8.0		-	4.3	-	-	-	-	-	-	-	-	-	-	-	-
	13.0		-	3.0	-	-	-	-	-	-	-	-	-	-	-	-
	18.0		-	1.5	-	-	-	-	-	-	-	-	-	-	-	-
	23.0		-	1.5	-	-	-	-	-	-	-	-	-	-	-	-
	28.0			0.8	-	-	-	-	-	-	-	-	-	-	-	-
	33.0			1.3	-	-	-	-	-	-	-	-	-	-	-	-
	38.0		-	0.6	-	-	-	-	-	-	-	-	-	-	-	-
J-02	0	Silty, Clayey SAND, Lean CLAY with Sand, Lean CLAY-Silty, SHALE-Silty, SILITSTONE-Shaley	-	-	4	19	14	5	100	100	77	64	45	21.6	A-1-b	0.3
	1.0		6	-	18	27	10	17	100	100	100	99	96	78.6	A-6(10)	12.2
	3.0		40	-	18	31	17	14	100	100	100	100	98	96.0	A-6(13)	11.8
	5.0		39	-	15	39	20	19	100	100	99	98	95	91.6	A-6(18)	15.4
	7.0		46	-	13	39	21	18	100	100	97	93	89	86.1	A-6(16)	15.0
	10.0		57	-	16	35	20	15	100	100	99	98	96	93.5	A-6(14)	13.0
	15.0		PR	-	18	25	18	7	100	100	98	97	95	91.0	A-4(5)	7.8
	15.5		-	5.0	-	-	-	-	-	-	-	-	-	-	-	-
	20.0		-	4.8	-	-	-	-	-	-	-	-	-	-	-	-
	25.0		-	1.3	-	-	-	-	-	-	-	-	-	-	-	-
	30.0		-	1.3	-	-	-	-	-	-	-	-	-	-	-	-
	35.0		-	1.0	-	-	-	-	-	-	-	-	-	-	-	-
	40.0		-	0.9	-	-	-	-	-	-	-	-	-	-	-	-
	45.0		-	1.0	-	-	-	-	-	-	-	-	-	-	-	-
	50.0		-	0.8	-	-	-	-	-	-	-	-	-	-	-	-

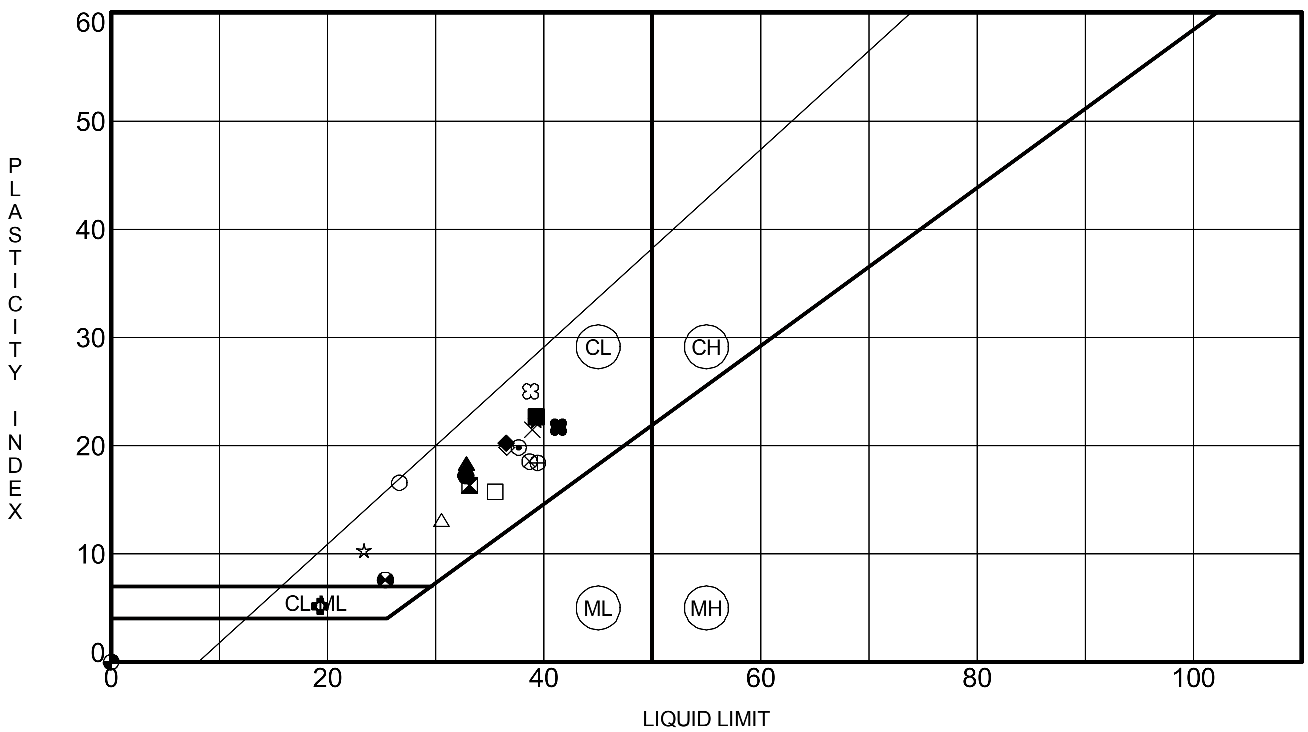
LL - Liquid Limit; PL - Plastic Limit; PI - Plasticity Index; PR - Practical Refusal

SUMMARY OF TESTS IN THE OVERBURDEN
Crossroads Interchange Reconstruction
Bridge “J” – NEQ I-35/I-240 Interchange
Oklahoma County, Oklahoma
PSI File No. 05471207-7

Boring No.	Sample Depth, Ft.	Soil Description	N Blows/ft.	TCP inch /100 Blows	Moisture Content, %	Atterberg Limits, %			Grain Analysis, %						AASHTO	OSI
						LL	PL	PI	3”	¾”	#4	#10	#40	#200		
J-03	0	Silty SAND with Gravel, Lean CLAY with Sand, Lean CLAY with Gravel, Lean CLAY, SHALE-Silty, SILTSTONE	-	-	4	-	-	NP	100	100	69	58	40	18.2	A-1-b	0
	1.0		8	-	19	23	13	10	100	100	94	92	89	80.8	A-4(5)	8.6
	3.0		8	-	23	39	14	25	100	100	97	94	90	84.0	A-6(20)	17.8
	5.0		4	-	18	39	17	22	100	100	88	87	84	76.7	A-6(16)	16.6
	7.0		10	-	20	36	16	20	100	100	99	98	97	92.7	A-6(18)	15.2
	10.0		37	-	15	37	17	20	100	100	99	98	96	94.1	A-6(19)	15.4
	15.0		59	-	14	39	17	22	100	100	100	98	97	92.7	A-6(21)	16.6
	20.0		PR	-	20	41	20	21	100	100	99	98	97	93.0	A-7-6(20)	16.6
	21.0		-	5.5	-	-	-	-	-	-	-	-	-	-	-	-
	25.0		-	4.0	-	-	-	-	-	-	-	-	-	-	-	-
	30.0		-	2.0	-	-	-	-	-	-	-	-	-	-	-	-
	35.0		-	1.5	-	-	-	-	-	-	-	-	-	-	-	-
	40.0		-	0.6	-	-	-	-	-	-	-	-	-	-	-	-
	45.0		-	0.8	-	-	-	-	-	-	-	-	-	-	-	-
	50.0		-	2.8	-	-	-	-	-	-	-	-	-	-	-	-
J-04	0	Silty SAND with Gravel, Lean CLAY, Fat CLAY, Lean CLAY-Shaley, SHALE-Sandy	-	-	3	-	-	NP	100	100	53	45	29	12.7	A-1-b	0
	1.0		8	-	20	36	12	24	100	100	99	98	97	87.6	A-6(20)	16.8
	3.0		14	-	19	66	15	51	100	100	99	98	97	92.8	A-7-6(51)	33.6
	5.0		15	-	13	37	15	22	100	100	96	94	91	85.4	A-6(18)	16.2
	7.0		32	-	14	45	22	23	100	100	100	98	96	90.5	A-7-6(22)	18.2
	10.0		PR	-	12	36	17	19	100	100	99	98	96	86.1	A-6(16)	14.8
	12.0		-	3.3	-	-	-	-	-	-	-	-	-	-	-	-
	17.0		-	5.3	-	-	-	-	-	-	-	-	-	-	-	-
	22.0		-	1.0	-	-	-	-	-	-	-	-	-	-	-	-
	27.0		-	1.5	-	-	-	-	-	-	-	-	-	-	-	-
	32.0		-	0.6	-	-	-	-	-	-	-	-	-	-	-	-
	37.0		-	1.3	-											
	42.0		-	0.8	-	-	-	-	-	-	-	-	-	-	-	-

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

APPENDIX E – LABORATORY SUMMARIES



	Boring	Depth (ft)	LL	PL	PI	Fines	Classification (*Visual)
●	**J-01**	**0.5**	**33**	**16**	**17**	**74.6**	**Lean CLAY with Sand**
⧗	**J-01**	**1.8**	**33**	**17**	**16**	**81.2**	**Lean CLAY with Sand**
▲	**J-01**	**3.8**	**33**	**14**	**19**	**74.6**	**Lean CLAY with Sand**
★	**J-01**	**5.8**	**39**	**17**	**22**	**89.2**	**Lean CLAY**
⊙	**J-01**	**7.8**	**38**	**18**	**20**	**88.2**	**Lean CLAY**
✿	**J-02**	**0.5**	**19**	**14**	**5**	**21.6**	**Silty, Clayey SAND**
○	**J-02**	**1.8**	**27**	**10**	**17**	**78.6**	**Lean CLAY with Sand**
△	**J-02**	**3.8**	**31**	**17**	**14**	**96.0**	**Lean CLAY**
⊗	**J-02**	**5.8**	**39**	**20**	**19**	**91.6**	**Lean CLAY**
⊕	**J-02**	**7.8**	**39**	**21**	**18**	**86.1**	**Lean CLAY**
□	**J-02**	**10.8**	**35**	**20**	**15**	**93.5**	**Lean CLAY**
⊗	**J-02**	**15.8**	**25**	**18**	**7**	**91.0**	**Silty CLAY**
◕	**J-03**	**0.5**	**NP**	**NP**	**NP**	**18.2**	**Silty SAND**
☆	**J-03**	**1.8**	**23**	**13**	**10**	**80.8**	**Lean CLAY with Sand**
✿	**J-03**	**3.8**	**39**	**14**	**25**	**84.0**	**Lean CLAY with Sand**
■	**J-03**	**5.8**	**39**	**17**	**22**	**76.7**	**Lean CLAY with Sand**
◆	**J-03**	**7.8**	**36**	**16**	**20**	**92.7**	**Lean CLAY**
◇	**J-03**	**10.8**	**37**	**17**	**20**	**94.1**	**Lean CLAY**
✕	**J-03**	**15.8**	**39**	**17**	**22**	**92.7**	**Lean CLAY**

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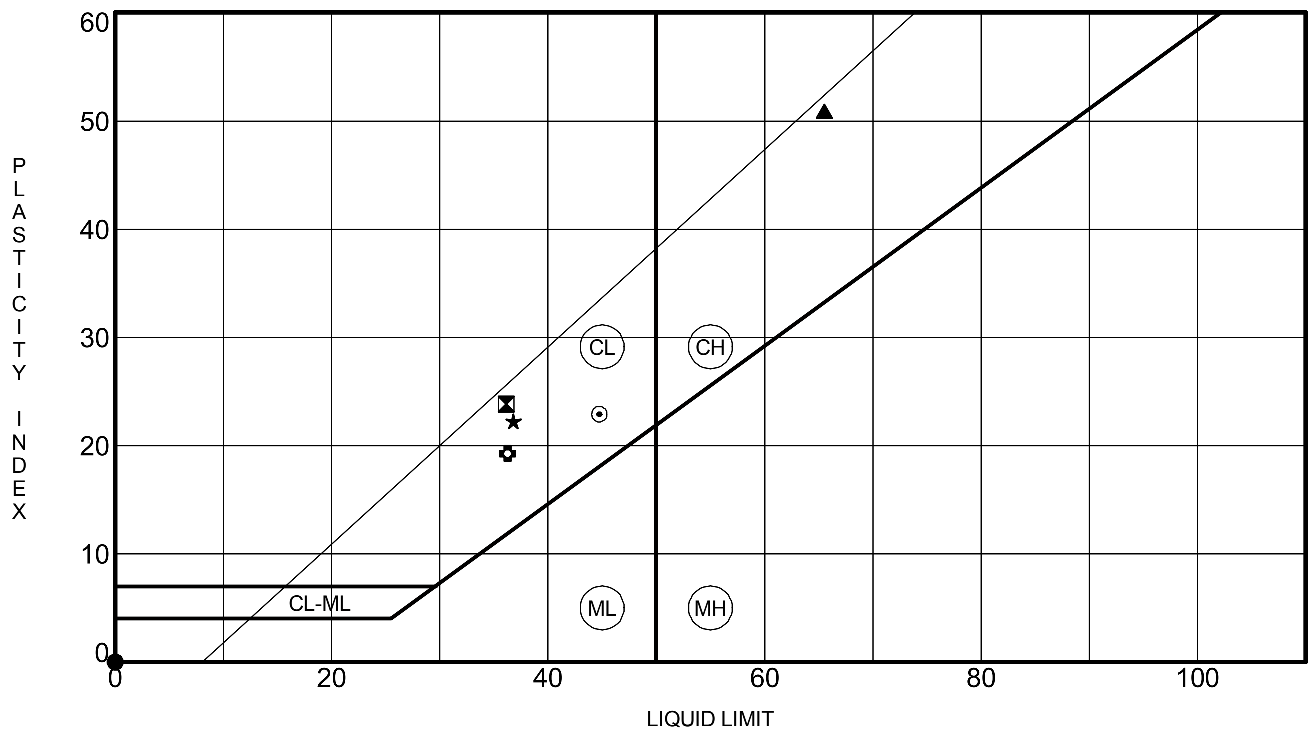
ATTERBERG LIMIT RESULTS

PSI Job No.: 05471207-7
Project: Bridge "J"
Location: I-35/I-240 Interchange
Oklahoma County, OK



	Boring	Depth (ft)	LL	PL	PI	Fines	Classification (*Visual)
●	**J-04**	**0.5**	**NP**	**NP**	**NP**	**12.7**	**Silty SAND**
⧗	**J-04**	**1.8**	**36**	**12**	**24**	**87.6**	**Lean CLAY**
▲	**J-04**	**3.8**	**66**	**15**	**51**	**92.8**	**Fat CLAY**
★	**J-04**	**5.8**	**37**	**15**	**22**	**85.4**	**Lean CLAY**
⊙	**J-04**	**7.8**	**45**	**22**	**23**	**90.5**	**Lean CLAY**
✿	**J-04**	**10.8**	**36**	**17**	**19**	**86.1**	**Lean CLAY**

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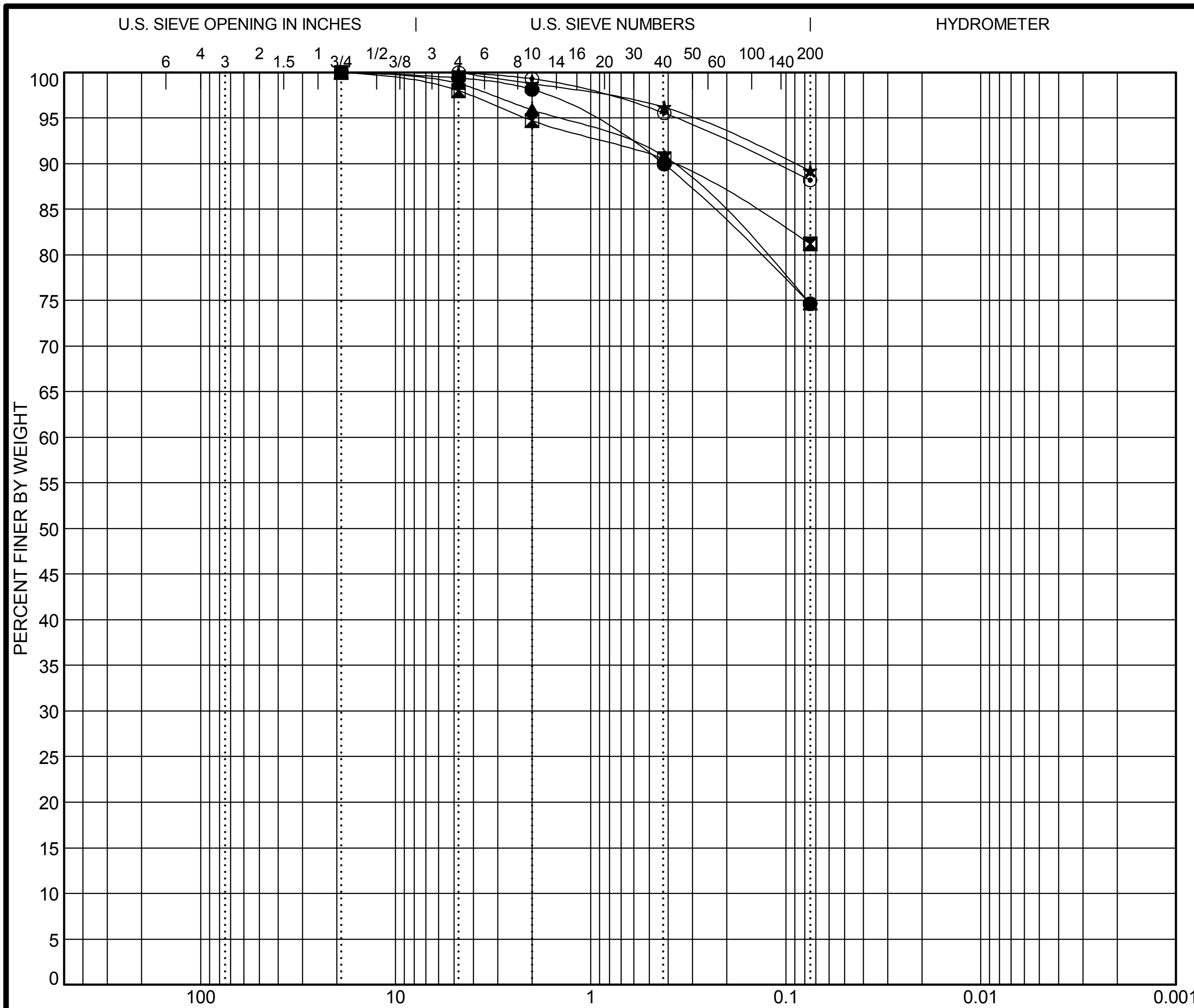
ATTERBERG LIMIT RESULTS

PSI Job No.: 05471207-7
Project: Bridge "J"
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
● J-01	0.5	**Lean CLAY with Sand**	33	16	17		
⧗ J-01	1.8	**Lean CLAY with Sand**	33	17	16		
▲ J-01	3.8	**Lean CLAY with Sand**	33	14	19		
★ J-01	5.8	**Lean CLAY**	39	17	22		
⊙ J-01	7.8	**Lean CLAY**	38	18	20		

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● J-01	0.5	19				0.6	24.8	74.6	
⧗ J-01	1.8	19				2.0	16.8	81.2	
▲ J-01	3.8	19				1.2	24.2	74.6	
★ J-01	5.8	4.75				0.0	10.8	89.2	
⊙ J-01	7.8	4.75				0.0	11.8	88.2	

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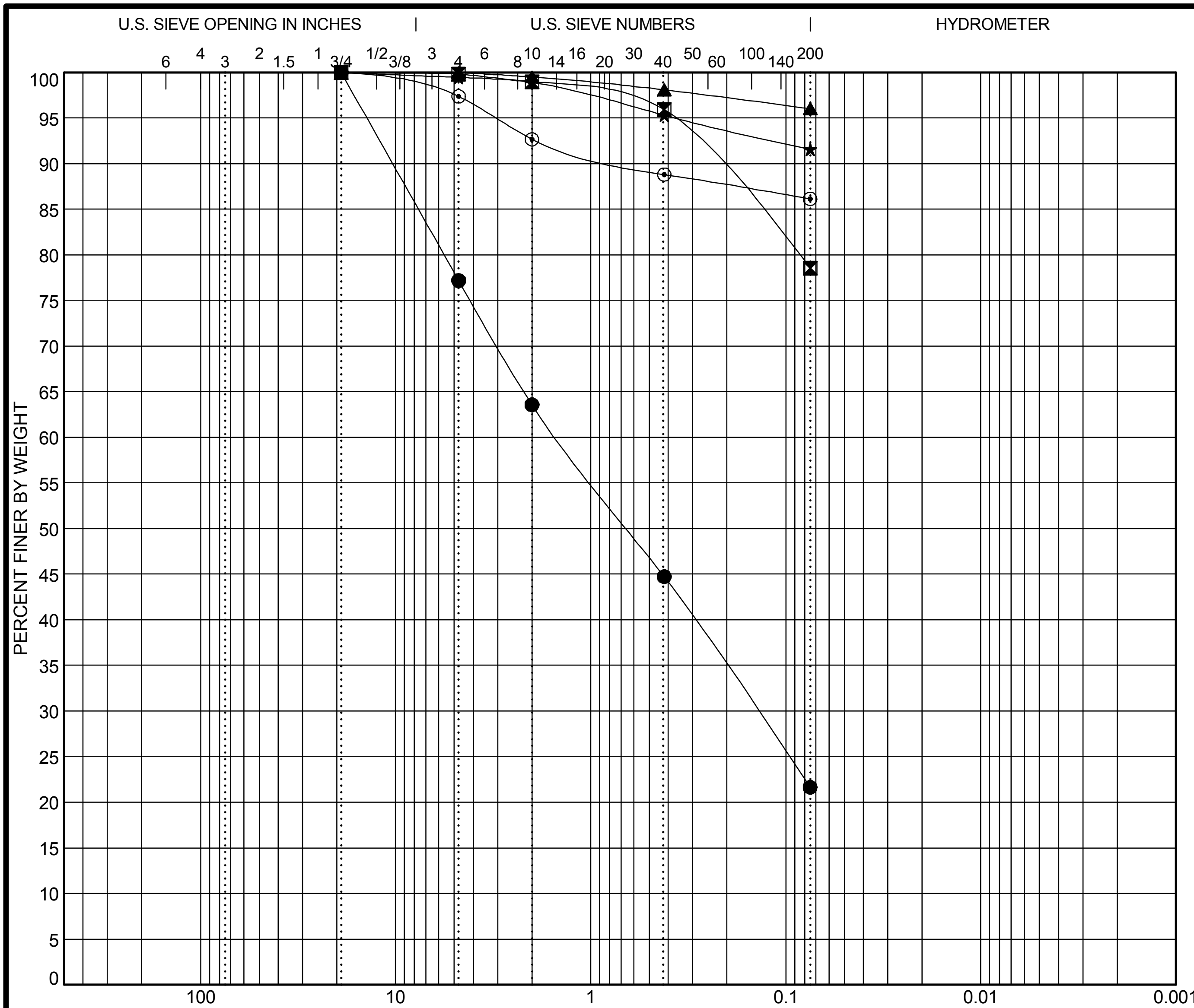
GRAIN SIZE DISTRIBUTION

Project: Bridge "J"
PSI Job No.: 05471207-7
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
●	J-02	0.5	Silty, Clayey SAND	19	14	5		
⧗	J-02	1.8	Lean CLAY with Sand	27	10	17		
▲	J-02	3.8	Lean CLAY	31	17	14		
★	J-02	5.8	Lean CLAY	39	20	19		
⊙	J-02	7.8	Lean CLAY	39	21	18		

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	J-02	0.5	19	1.489	0.14		22.8	55.6	21.6	
⧗	J-02	1.8	19				0.2	21.3	78.6	
▲	J-02	3.8	4.75				0.0	4.0	96.0	
★	J-02	5.8	19				0.5	7.9	91.6	
⊙	J-02	7.8	19				2.6	11.2	86.1	

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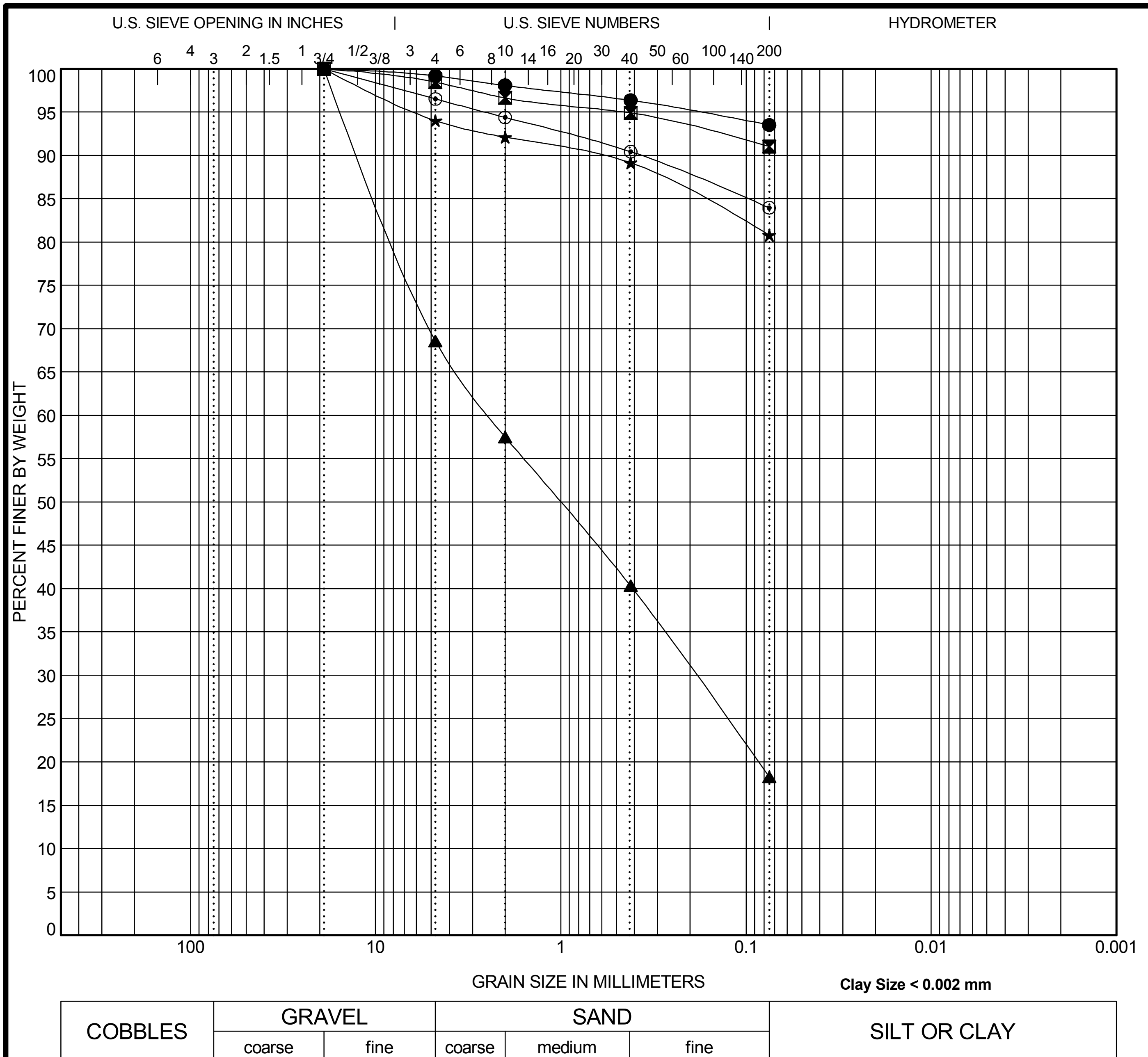
GRAIN SIZE DISTRIBUTION

Project: Bridge "J"
PSI Job No.: 05471207-7
Location: I-35/I-240 Interchange
Oklahoma County, OK



	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	**J-02**	**10.8**	**Lean CLAY**	**35**	**20**	**15**		
⧗	**J-02**	**15.8**	**Silty CLAY**	**25**	**18**	**7**		
▲	**J-03**	**0.5**	**Silty SAND**	**NP**	**NP**	**NP**		
★	**J-03**	**1.8**	**Lean CLAY with Sand**	**23**	**13**	**10**		
⊙	**J-03**	**3.8**	**Lean CLAY with Sand**	**39**	**14**	**25**		

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**J-02**	**10.8**	**19**				**0.8**	**5.7**	**93.5**	
⧗	**J-02**	**15.8**	**19**				**1.5**	**7.4**	**91.0**	
▲	**J-03**	**0.5**	**19**	**2.43**	**0.188**		**31.4**	**50.4**	**18.2**	
★	**J-03**	**1.8**	**19**				**6.0**	**13.2**	**80.8**	
⊙	**J-03**	**3.8**	**19**				**3.5**	**12.6**	**84.0**	

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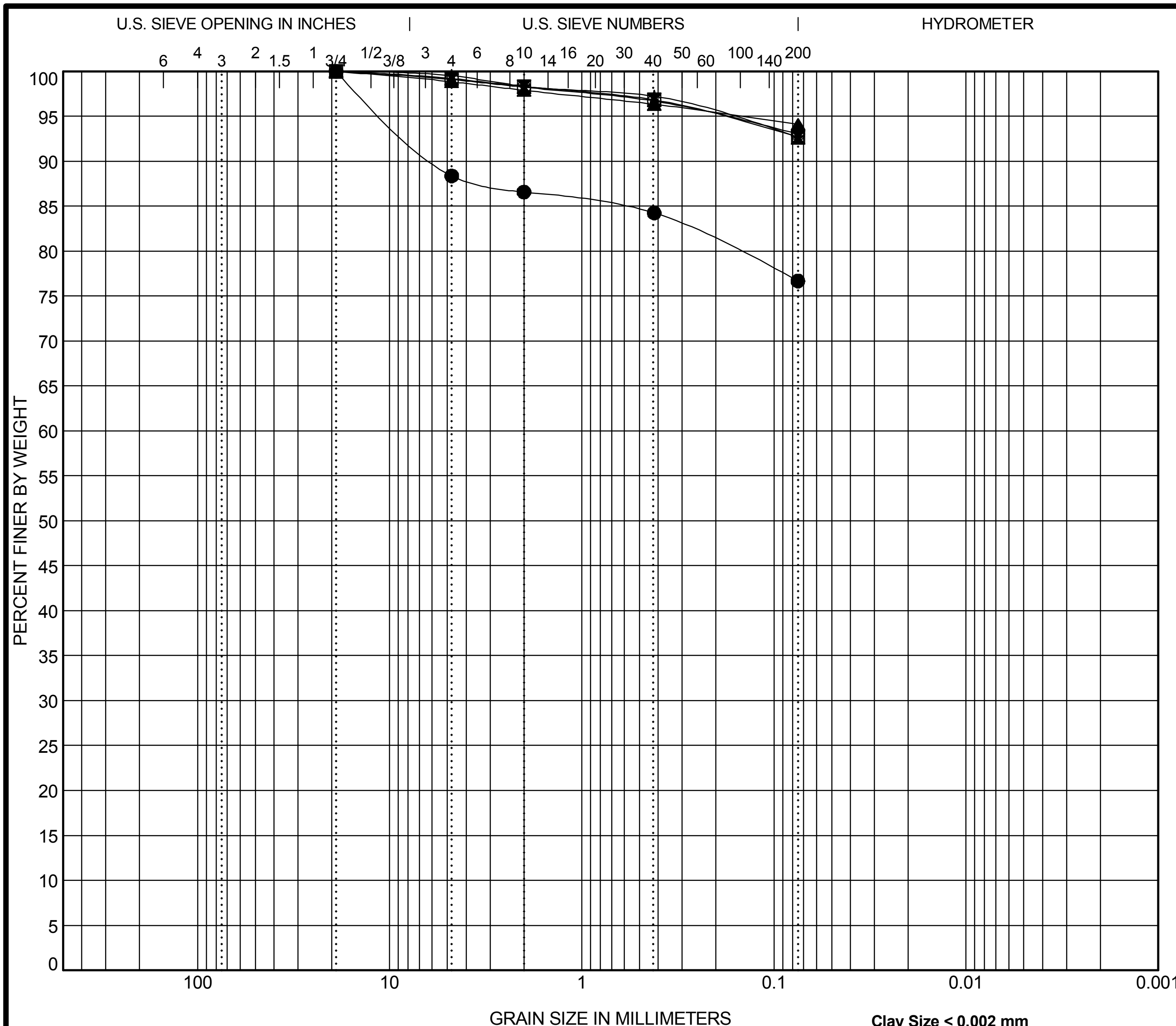
GRAIN SIZE DISTRIBUTION

Project: Bridge "J"
PSI Job No.: 05471207-7
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
● J-03	5.8	**Lean CLAY with Sand**	39	17	22		
⧗ J-03	7.8	**Lean CLAY**	36	16	20		
▲ J-03	10.8	**Lean CLAY**	37	17	20		
★ J-03	15.8	**Lean CLAY**	39	17	22		
⊙ J-03	20.8	**Lean CLAY**	41	20	21		

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● J-03	5.8	19				11.6	11.7	76.7	
⧗ J-03	7.8	19				0.8	6.5	92.7	
▲ J-03	10.8	19				1.2	4.7	94.1	
★ J-03	15.8	19				0.5	6.9	92.7	
⊙ J-03	20.8	19				0.9	6.1	93.0	

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GRAIN SIZE DISTRIBUTION

Project: Bridge "J"
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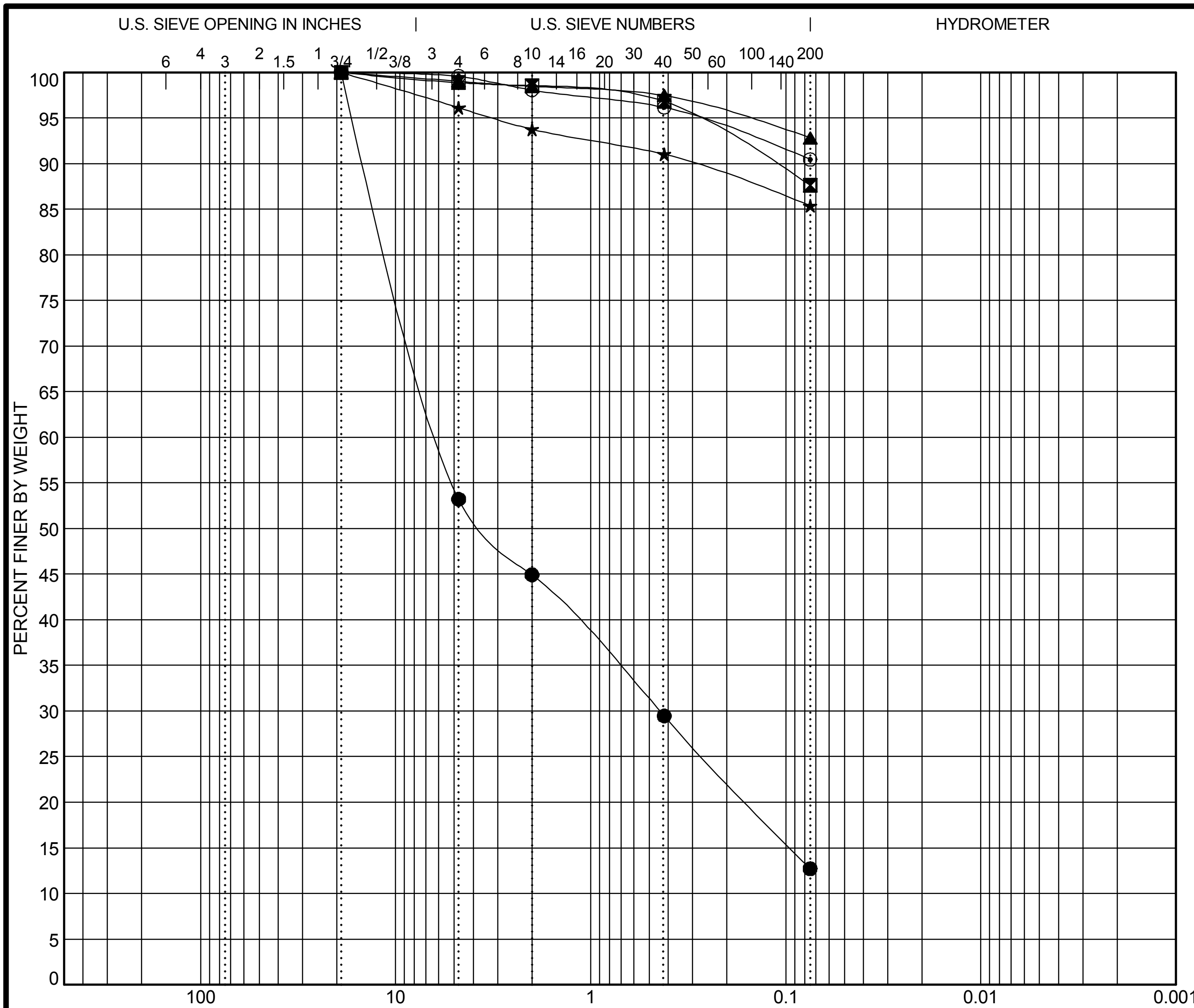

Clay Size < 0.002 mm

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification		Classification	LL	PL	PI	Cc	Cu
● J-04	0.5	**Silty SAND**	**NP**	**NP**	**NP**		
⧗ J-04	1.8	**Lean CLAY**	**36**	**12**	**24**		
▲ J-04	3.8	**Fat CLAY**	**66**	**15**	**51**		
★ J-04	5.8	**Lean CLAY**	**37**	**15**	**22**		
⊙ J-04	7.8	**Lean CLAY**	**45**	**22**	**23**		

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● J-04	0.5	**19**	**5.808**	**0.444**		**46.8**	**40.5**	**12.7**	
⧗ J-04	1.8	**19**				**1.1**	**11.3**	**87.6**	
▲ J-04	3.8	**19**				**1.0**	**6.2**	**92.8**	
★ J-04	5.8	**19**				**3.9**	**10.7**	**85.4**	
⊙ J-04	7.8	**19**				**0.4**	**9.2**	**90.5**	

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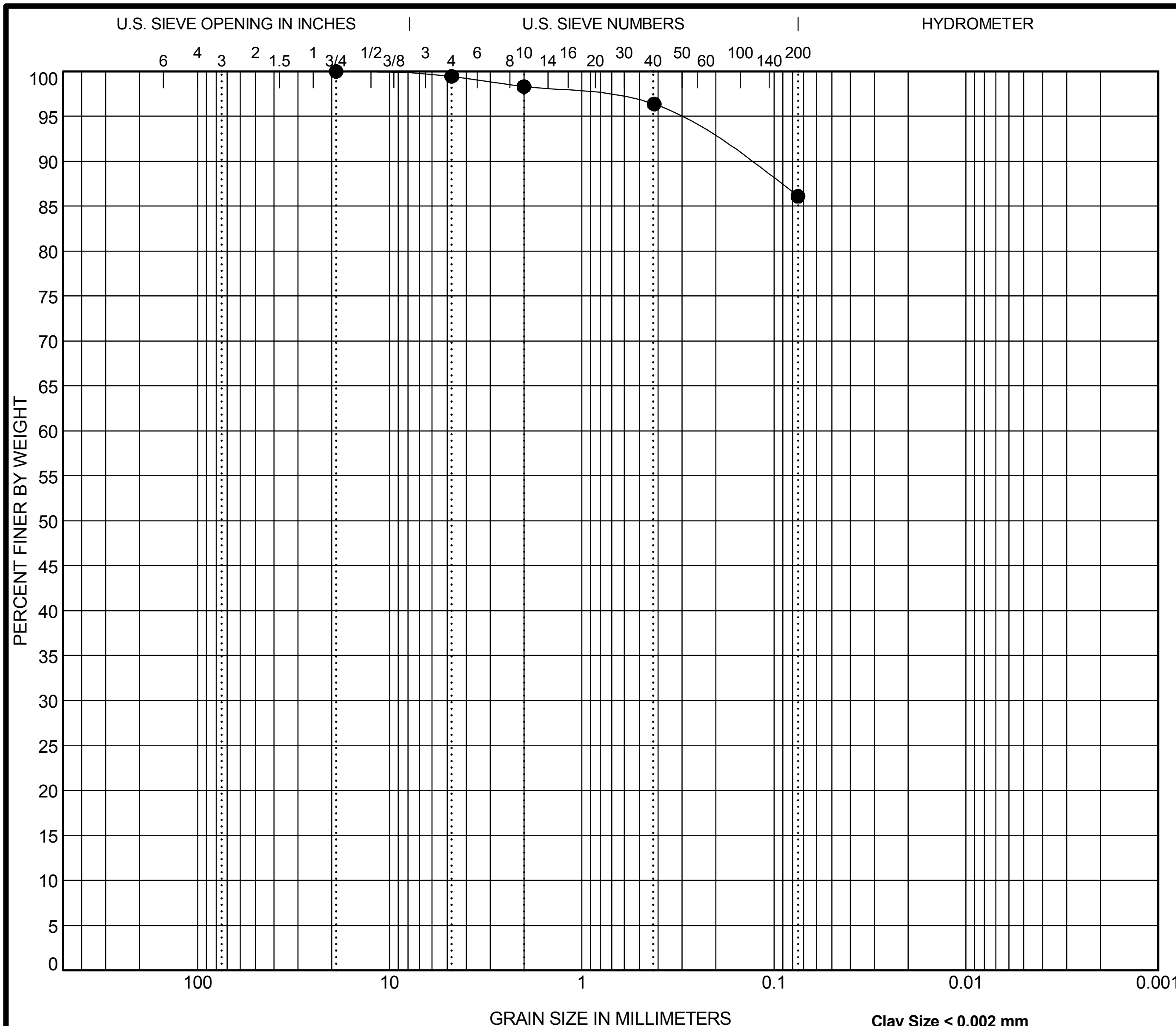
Telephone: (405) 735-6052
Fax: (405) 735-6086

GRAIN SIZE DISTRIBUTION

Project: Bridge "J"
PSI Job No.: 05471207-7
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
●	J-04	10.8	Lean CLAY	36	17	19		

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	J-04	10.8	19				0.5	13.3	86.1	

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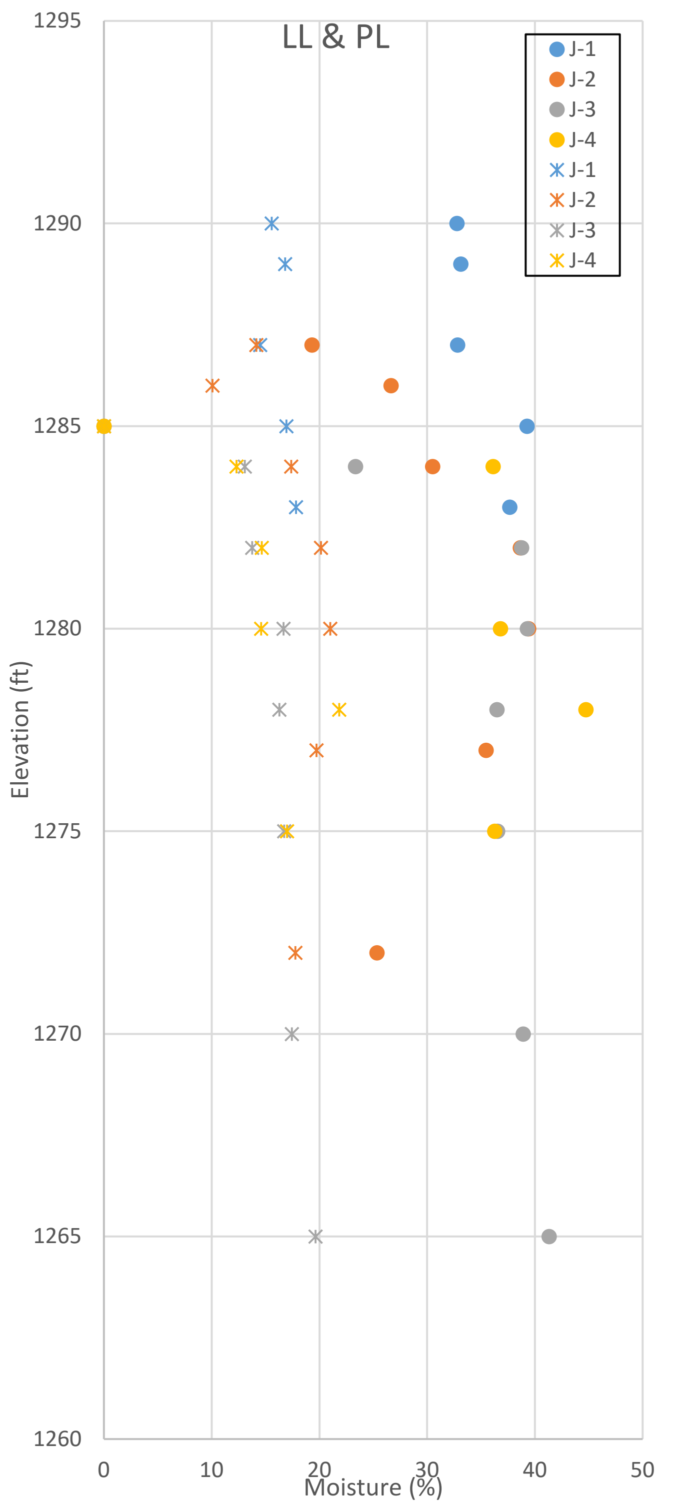
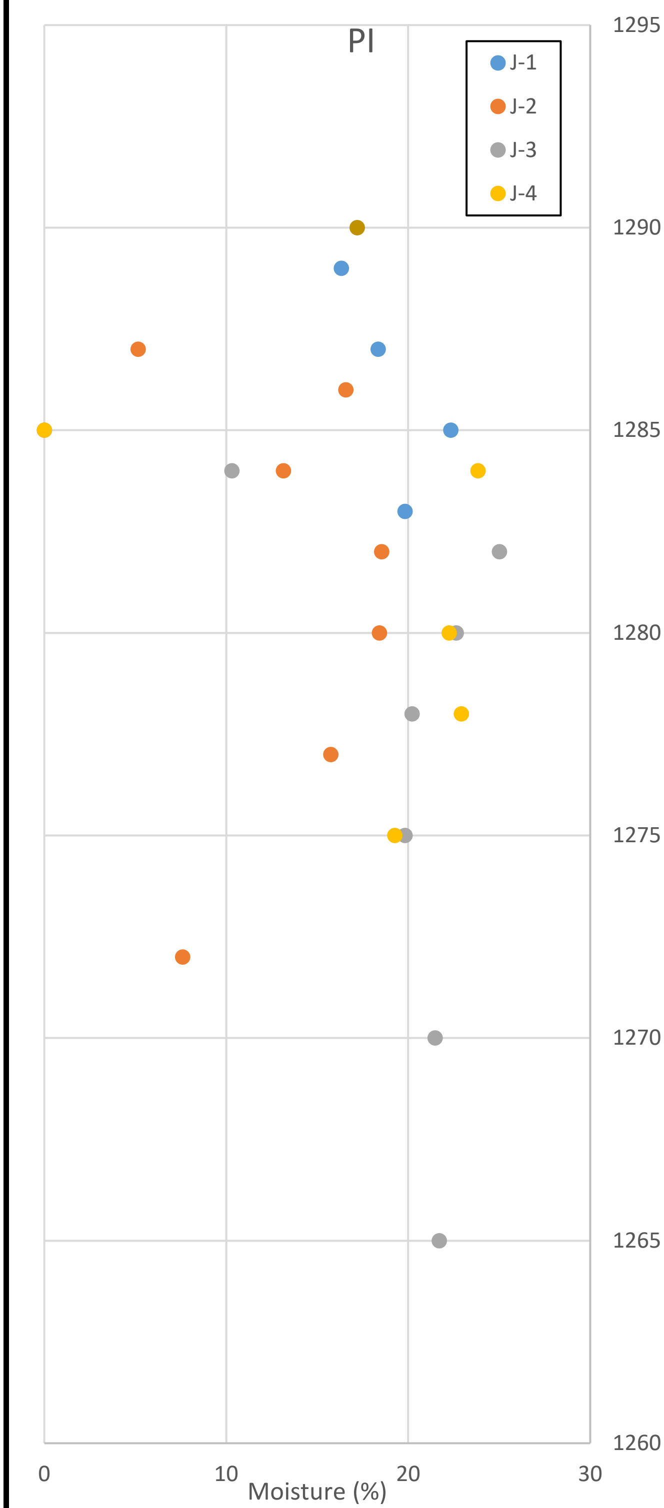
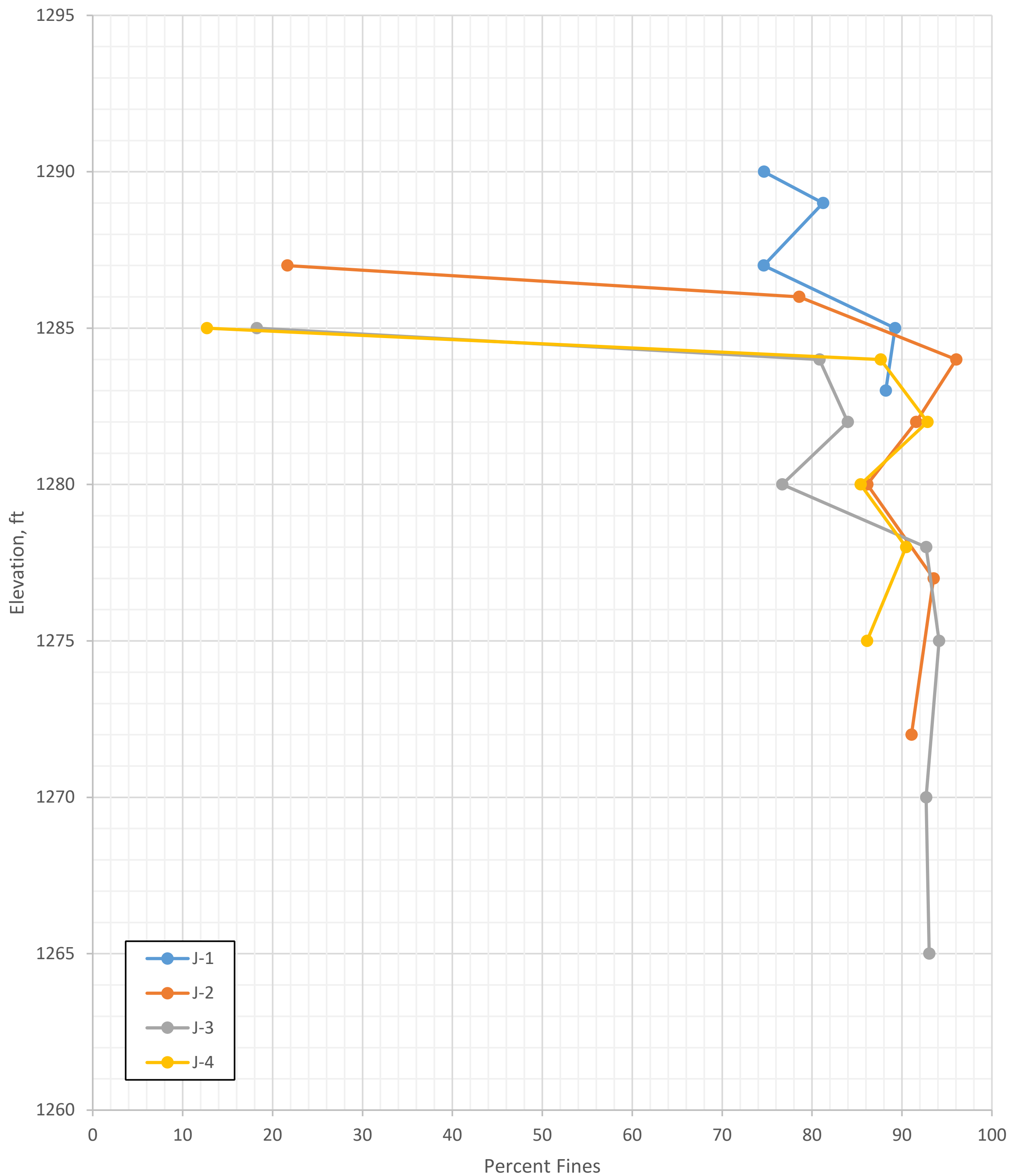
GRAIN SIZE DISTRIBUTION

Project: Bridge "J"
PSI Job No.: 05471207-7
Location: I-35/I-240 Interchange
Oklahoma County, OK





psi Information To Build On — Engineering • Consulting • Testing	PREPARED BY: AAW	BRIDGE "J" I-35/I-240 INTERCHANGE OKLAHOMA CITY, OK	PSI REPORT NO 05471207-7
11825 S PORTLAND AVE OKLAHOMA CITY, OK (405) 735-6052	DATE: DECEMBER 2017	ATTERBERG LIMITS	



psi Information To Built On
Engineering • Consulting • Testing

PREPARED BY:
AAW

BRIDGE "J"
I-35/I-240
INTERCHANGE
OKLAHOMA CITY, OK

PSI REPORT NO
05471207-7

11825 S PORTLAND AVE
OKLAHOMA CITY, OK
(405) 735-6052

DATE:
DECEMBER 2017

PERCENT FINES PASSING THE
#200 SIEVE

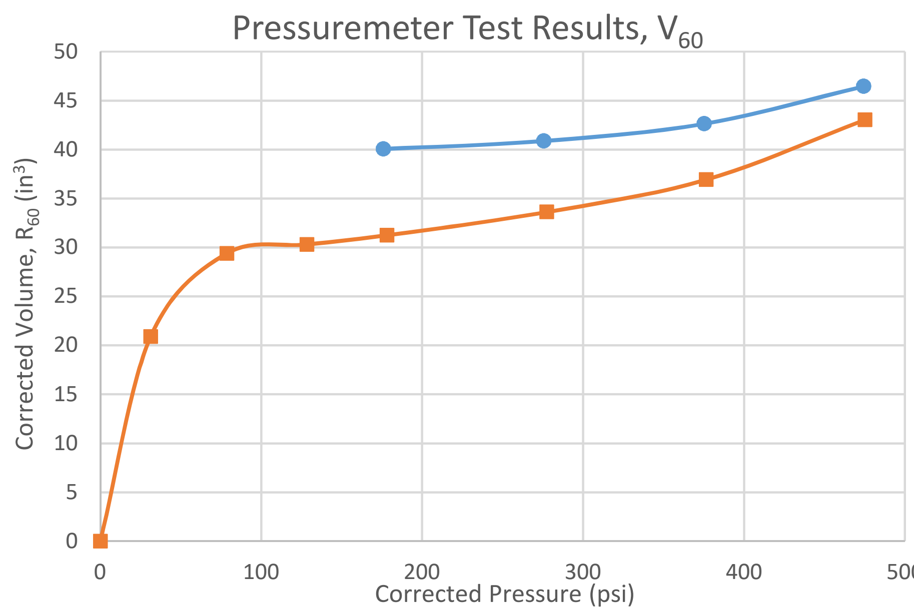
APPENDIX F – PRESSUREMETER RESULTS

Depth of Probe: 22 ft
Initial Height of Fluid: 55 in
System Compressibility: 0.00221 in^3/psi

V_o: 120 in^3
a: 14.27 (Pressure Correction)
b: 0.2476 (Pressure Correction)
γ: 0.33

Test Data

Pressure (psi)	R_{30} (in)	R_{60} (in)	HydrostaticP res. (psi)	Correct. Pressure (psi)	Correct. Vol., R_{30} (in^3)	Correct. Vol., R_{60} (in^3)
0	0	0	0	0	0	0
50	5	10.1	62	31	10.3	20.9
100	14.25	14.25	112	79	29.4	29.4
150	14.75	14.75	162	128	30.3	30.3
200	15.1	15.25	212	178	30.9	31.2
300	16.35	16.5	312	277	33.3	33.6
400	17.75	18.2	412	376	36.0	36.9
500	20.5	21.25	512	475	41.5	43.0
200	19.5	19.5	212	176	40.1	40.1
300	20	20	312	276	40.9	40.9
400	20.85	20.95	412	375	42.4	42.6
500	22.6	22.9	512	474	45.8	46.4



V_m: 38 in^3
V_i: 25 in^3

E_m: 9046 psi
P_L: 600 psi

P_{OH}: 31 psi
P_{OV}: 2640 psf

psi Information To Build On
Engineering • Consulting • Testing

DRAWN BY:
AAW

BRIDGE "J"
I-35 & I-240 INTERCHANGE
OKLAHOMA CITY, OK

PROJECT NO.:
05471207-7

11825 S. PORTLAND AVENUE
OKLAHOMA CITY, OK 73170
(405) 735-6052

DATE:
DECEMBER 2017

BORING J-2
@ 22 FEET

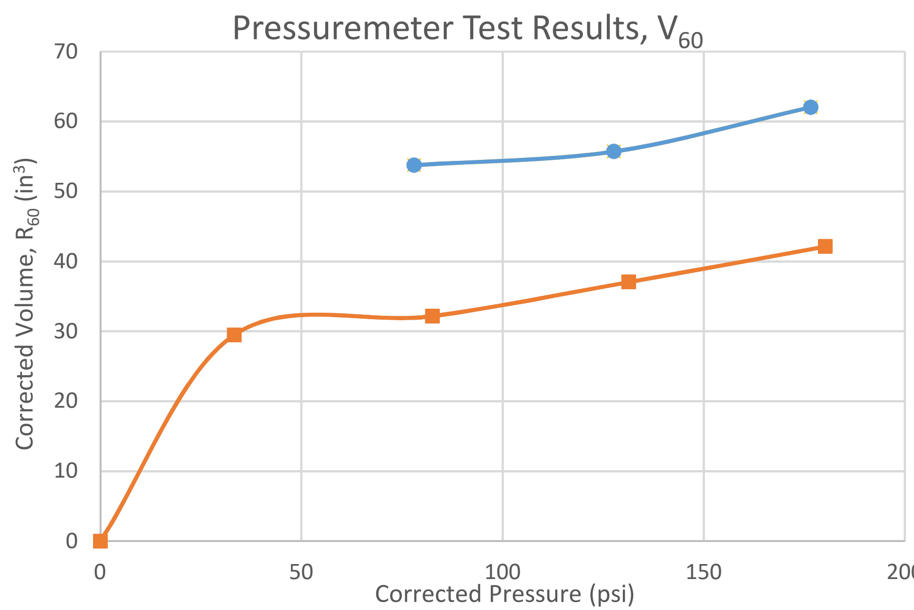
PRESSUREMETER TEST RESULTS

Depth of Probe:	33	ft
Initial Height of Fluid:	55	in
System Compressibility:	0.00221	in^3/psi

V_o:	120	in^3
a:	14.27	(Pressure Correction)
b:	0.2476	(Pressure Correction)
γ:	0.33	

Test Data

Pressure (psi)	R_{30} (in)	R_{60} (in)	HydrostaticP res. (psi)	Correct. Pressure (psi)	Correct. Vol., R_{30} (in^3)	Correct. Vol., R_{60} (in^3)
0	0	0	0	0	0	0
50	9.5	14.25	66	33	19.6	29.5
100	15.4	15.6	116	83	31.8	32.2
150	17.4	18	166	131	35.8	37.0
200	19.85	20.5	216	180	40.8	42.1
100	26	26	116	78	53.8	53.8
150	26.95	27	166	128	55.6	55.7
200	29.1	30.1	216	177	60.0	62.0



V_m:	46	in^3	**E_m:**	**1988**	**psi**	P_{OH}:	33	psi
V_i:	51	in^3	**P_L:**	**300**	**psi**	P_{OV}:	3960	psf

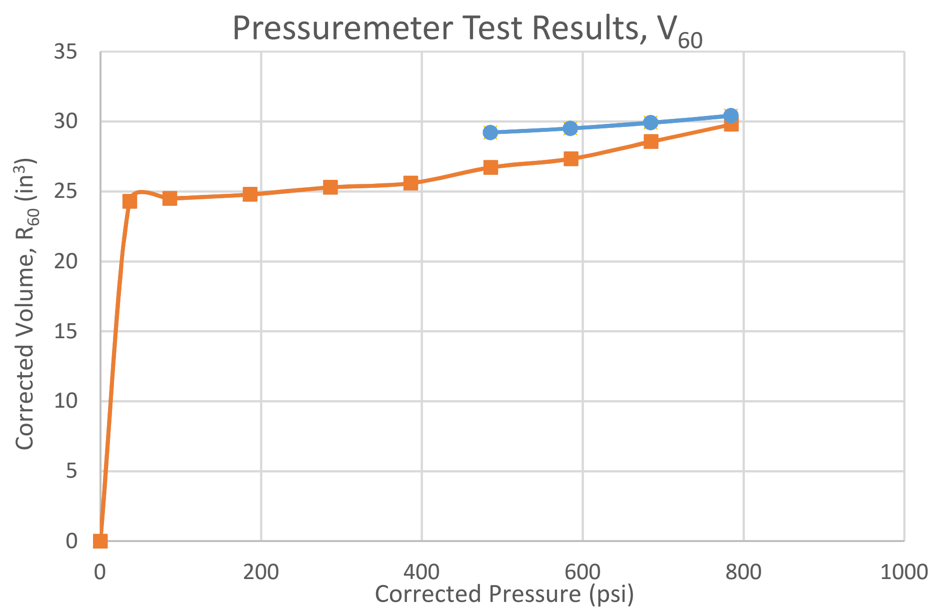
psi Information To Build On — Engineering • Consulting • Testing	DRAWN BY: AAW	BRIDGE "J" I-35 & I-240 INTERCHANGE OKLAHOMA CITY, OK	PROJECT NO.: 05471207-7
11825 S. PORTLAND AVENUE OKLAHOMA CITY, OK 73170 (405) 735-6052	DATE: DECEMBER 2017	BORING J-2 @ 33 FEET	PRESSUREMETER TEST RESULTS

Depth of Probe: 37 ft
Initial Height of Fluid: 55 in
System Compressibility: 0.00221 in^3/psi

V_o: 120 in^3
a: 14.27 (Pressure Correction)
b: 0.2476 (Pressure Correction)
γ: 0.33

Test Data

Pressure (psi)	R_{30} (in)	R_{60} (in)	HydrostaticPres. (psi)	Correct. Pressure (psi)	Correct. Vol., R_{30} (in^3)	Correct. Vol., R_{60} (in^3)
0	0	0	0	0	0	0
50	11.25	11.75	68	37	23.3	24.3
100	11.9	11.9	118	86	24.5	24.5
200	12.15	12.15	218	186	24.8	24.8
300	12.5	12.5	318	286	25.3	25.3
400	12.75	12.75	418	386	25.6	25.6
500	13.25	13.4	518	486	26.4	26.7
600	13.75	13.8	618	585	27.2	27.3
700	14.4	14.5	718	685	28.4	28.6
800	15.1	15.2	818	784	29.6	29.8
500	14.6	14.6	518	485	29.2	29.2
600	14.85	14.85	618	585	29.5	29.5
700	15.15	15.15	718	685	29.9	29.9
800	15.5	15.5	818	784	30.4	30.4



V_m: 26 in^3
V_i: 24 in^3

E_m: **68097** **psi**
P_L: **1200** **psi**

P_{OH}: 37 psi
P_{OV}: 4440 psf

psi Information To Build On
Engineering • Consulting • Testing

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BRIDGE "J"
I-35 & I-240 INTERCHANGE
OKLAHOMA CITY, OK

PROJECT NO.:
05471207-7

11825 S. PORTLAND AVENUE
OKLAHOMA CITY, OK 73170
(405) 735-6052

DATE:
DECEMBER 2017

BORING J-2
@ 37 FEET

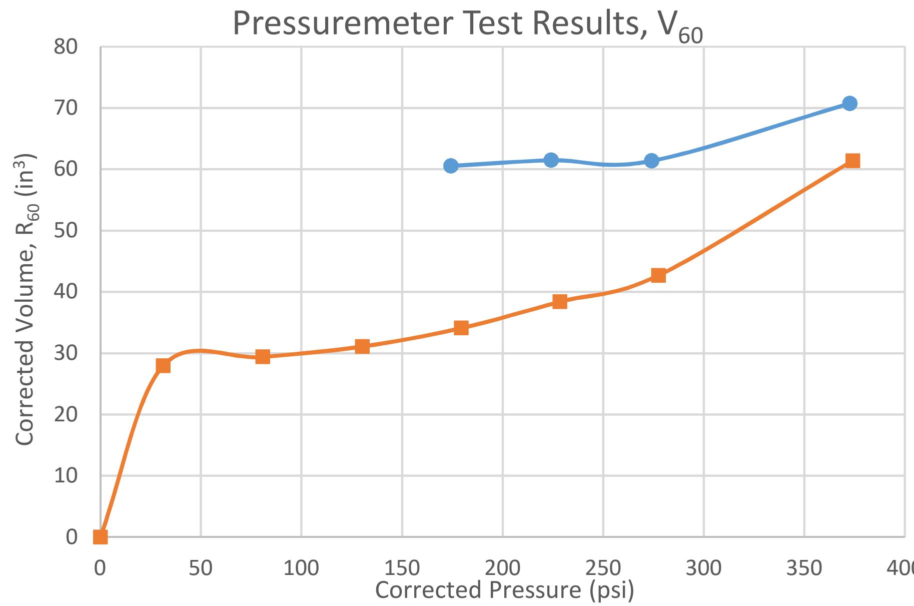
PRESSUREMETER TEST RESULTS

Depth of Probe: 27 ft
Initial Height of Fluid: 55 in
System Compressibility: 0.00221 in^3/psi

V_o: 120 in^3
a: 14.27 (Pressure Correction)
b: 0.2476 (Pressure Correction)
γ: 0.33

Test Data

Pressure (psi)	R_{30} (in)	R_{60} (in)	HydrostaticPres. (psi)	Correct. Pressure (psi)	Correct. Vol., R_{30} (in^3)	Correct. Vol., R_{60} (in^3)
0	0	0	0	0	0	0
50	13.25	13.5	64	31	27.4	27.9
100	14.25	14.25	114	81	29.4	29.4
150	15.125	15.125	164	130	31.1	31.1
200	16.5	16.625	214	179	33.8	34.1
250	18.5	18.75	264	228	37.9	38.4
300	20.625	20.875	314	277	42.2	42.7
400	29	30	414	374	59.3	61.4
200	29.375	29.375	214	174	60.5	60.5
250	29.875	29.875	264	224	61.5	61.5
300	29.875	29.875	314	274	61.4	61.4
400	34	34.5	414	373	69.7	70.7



V_m: 31 in^3
V_i: 23 in^3

E_m: 9669 psi
P_L: 350 psi

P_{OH}: 31 psi
P_{OV}: 3240 psf

psi Information To Build On
Engineering • Consulting • Testing

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BRIDGE "J"
I-35 & I-240 INTERCHANGE
OKLAHOMA CITY, OK

PROJECT NO.:
05471207-7

11825 S. PORTLAND AVENUE
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(405) 735-6052

DATE:
DECEMBER 2017

BORING J-3 @ 27 FEET

PRESSUREMETER TEST RESULTS

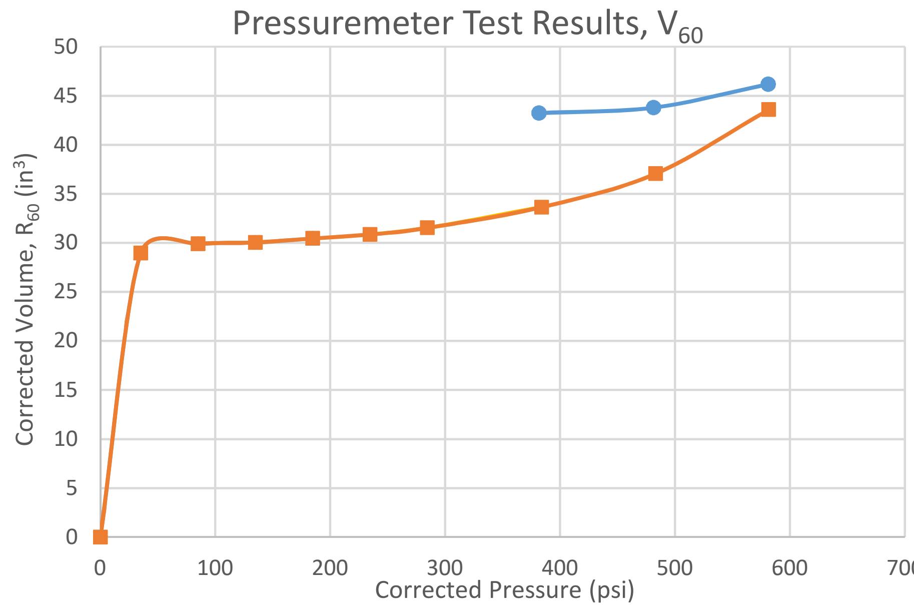
Depth of Probe:	37	ft
Initial Height of Fluid:	55	in
System Compressibility:	0.00221	in^3/psi

V_o:	120	in^3
a:	14.27	(Pressure Correction)
b:	0.2476	(Pressure Correction)

γ: 0.33

Test Data

Pressure (psi)	R_{30} (in)	R_{60} (in)	HydrostaticPres. (psi)	Correct. Pressure (psi)	Correct. Vol., R_{30} (in^3)	Correct. Vol., R_{60} (in^3)
0	0	0	0	0	0	0
50	12.5	14	68	35	25.8	29.0
100	14.375	14.5	118	85	29.6	29.9
150	14.625	14.625	168	135	30.0	30.0
200	14.875	14.875	218	185	30.4	30.4
250	15.125	15.125	268	235	30.9	30.9
300	15.375	15.5	318	284	31.3	31.5
400	16.625	16.625	418	384	33.6	33.6
500	18.25	18.375	518	483	36.8	37.0
600	21.25	21.625	618	581	42.8	43.6
400	21.625	21.25	418	382	44.0	43.2
500	22.875	21.625	518	481	46.4	43.8
600	26.375	22.875	618	581	53.4	46.2



V_m:	30	in^3	**E_m:**	**40052**	**psi**	P_{OH}:	35	psi
V_i:	29	in^3	**P_L:**	**750**	**psi**	P_{OV}:	4440	psf

Information To Build On
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AAW

BRIDGE "J"
I-35 & I-240 INTERCHANGE
OKLAHOMA CITY, OK

PROJECT NO.:
05471207-7

11825 S. PORTLAND AVENUE
OKLAHOMA CITY, OK 73170
(405) 735-6052

DATE:
DECEMBER 2017

BORING J-3 @ 37 FEET

PRESSUREMETER TEST RESULTS

Depth of Probe:	47	ft
Initial Height of Fluid:	55	in
System Compressibility:	0.00221	in^3/psi

V_o: 120 in^3

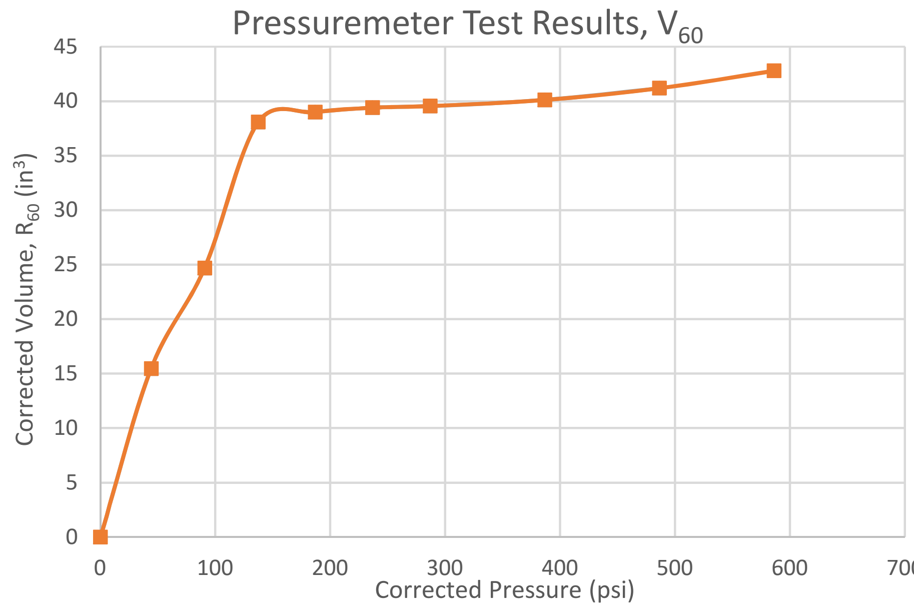
a: 14.27 (Pressure Correction)

b: 0.2476 (Pressure Correction)

γ: 0.33

Test Data

Pressure (psi)	R_{30} (in)	R_{60} (in)	HydrostaticP res. (psi)	Correct. Pressure (psi)	Correct. Vol., R_{30} (in^3)	Correct. Vol., R_{60} (in^3)
0	0	0	0	0	0	0
50	6	7.5	72	44	12.3	15.5
100	9.875	12	122	91	20.3	24.7
150	16	18.5	172	137	32.9	38.1
200	19	19	222	187	39.0	39.0
250	19.25	19.25	272	237	39.4	39.4
300	19.375	19.375	322	287	39.6	39.6
400	19.75	19.75	422	387	40.1	40.1
500	20.25	20.375	522	486	40.9	41.2
600	21.125	21.25	622	586	42.5	42.8
700	22.5	22.75	722	685	45.2	45.7



V_m:	32	in^3	**E_m:**	**7770**	**psi**	P_{OH}:	44	psi
V_i:	28	in^3	**P_L:**	**1000**	**psi**	P_{OV}:	5640	psf

psi Information To Build On
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DRAWN BY:
AAW

BRIDGE "J"
I-35 & I-240 INTERCHANGE
OKLAHOMA CITY, OK

PROJECT NO.:
05471207-7

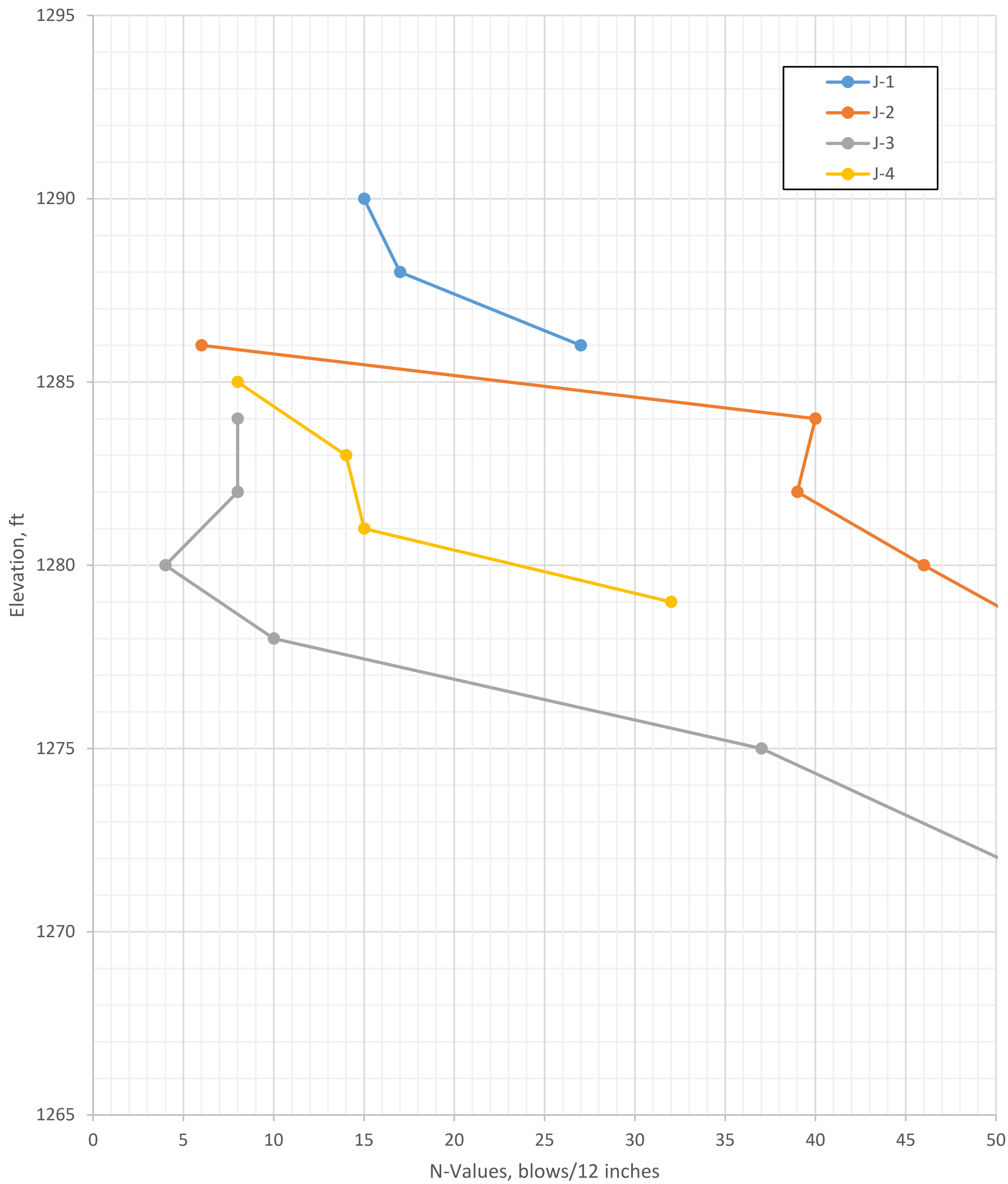
11825 S. PORTLAND AVENUE
OKLAHOMA CITY, OK 73170
(405) 735-6052

DATE:
DECEMBER 2017

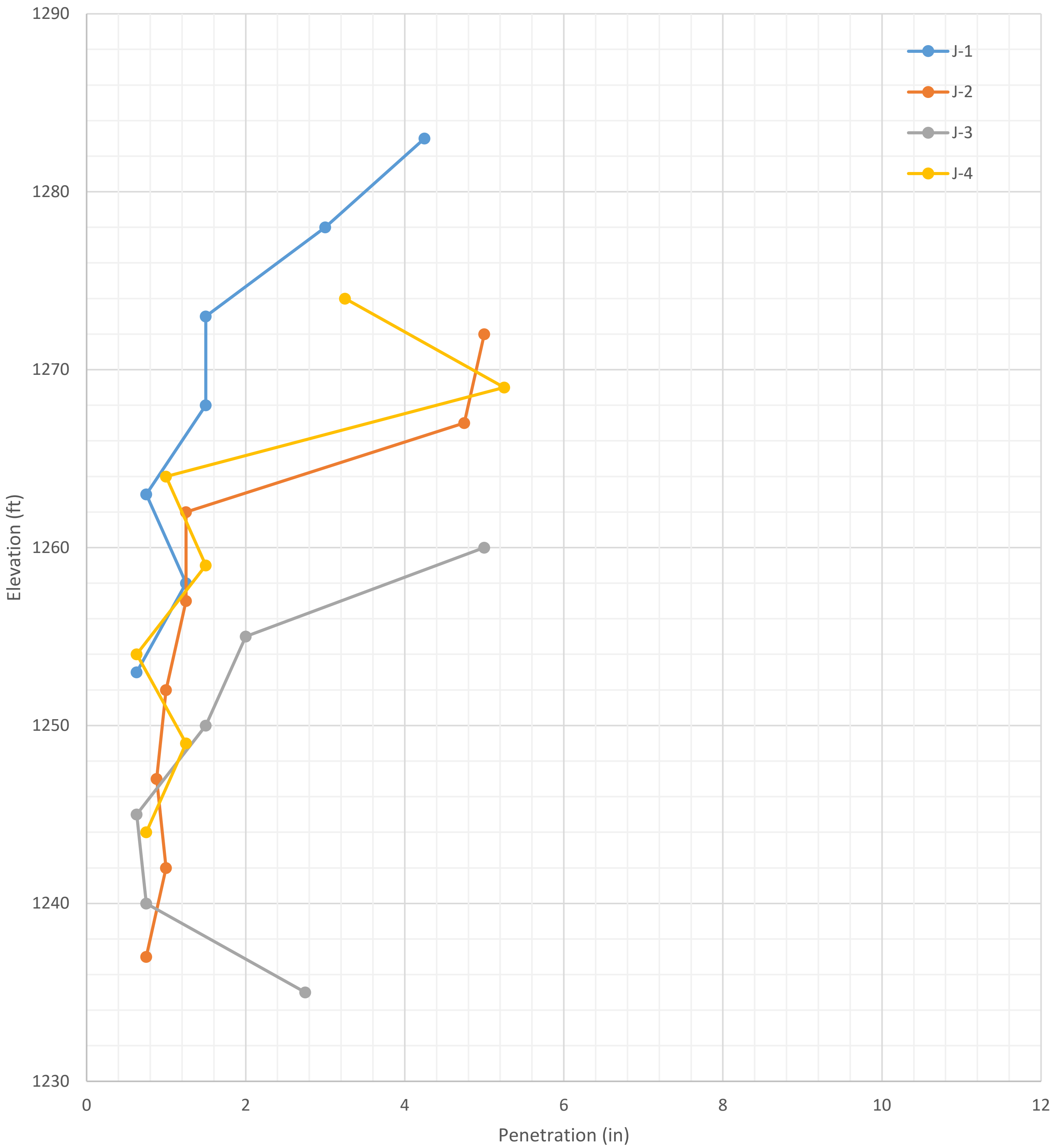
BORING J-3
@ 47 FEET

PRESSUREMETER TEST RESULTS

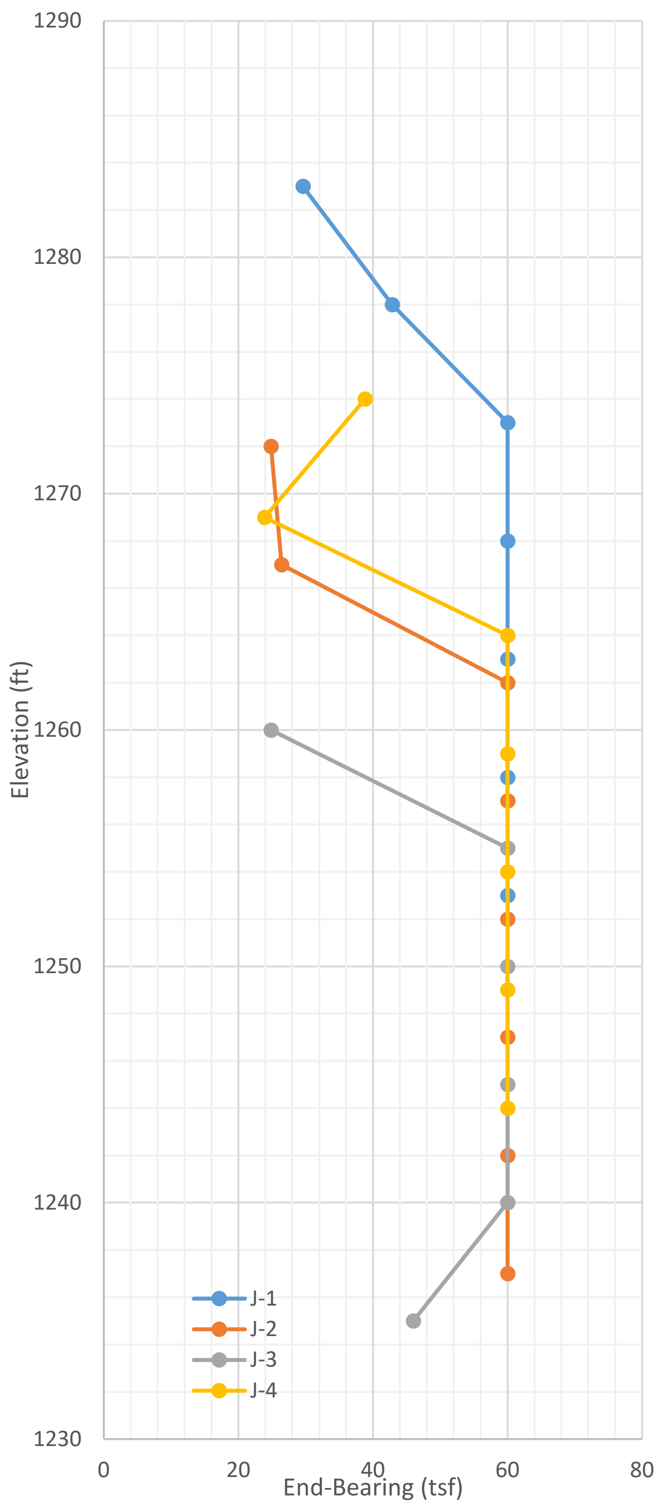
APPENDIX G – PLOTS OF TCP TESTS & CAPACITIES

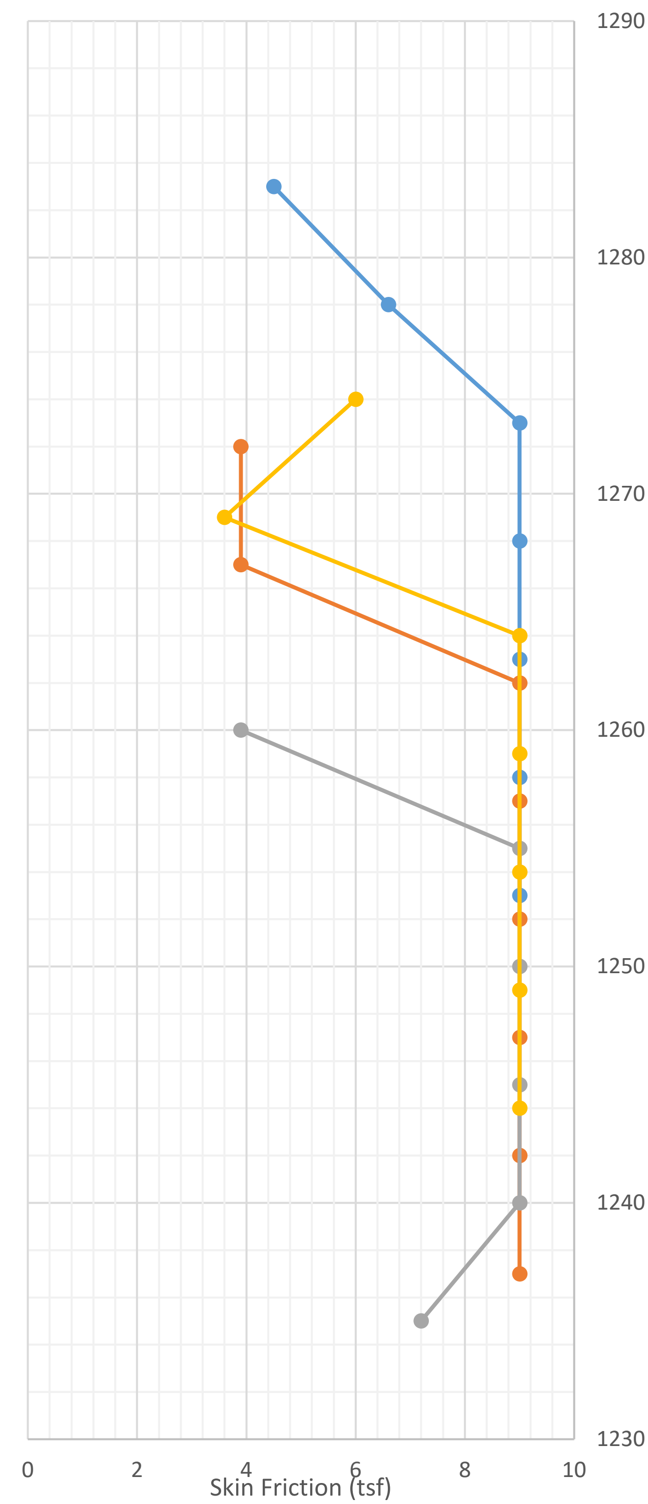


psi Information To Build On — Engineering • Consulting • Testing	PREPARED BY: AAW	BRIDGE "J" I-35/I-240 INTERCHANGE OKLAHOMA CITY, OK	PSI REPORT NO 05471207-7
11825 S PORTLAND AVE OKLAHOMA CITY, OK (405) 735-6052	DATE: DECEMBER 2017	STANDARD PENETRATION TEST RESULTS	



psi Information To Build On Engineering • Consulting • Testing	PREPARED BY: AAW	BRIDGE "J" I-35/I-240 INTERCHANGE OKLAHOMA CITY, OK	PSI REPORT NO 05471207-7
11825 S PORTLAND AVE OKLAHOMA CITY, OK (405) 735-6052	DATE: DECEMBER 2017	TEXAS CONE PENETROMETER TEST RESULTS	





psi Information To Build On
Engineering • Consulting • Testing

PREPARED BY:
AAW

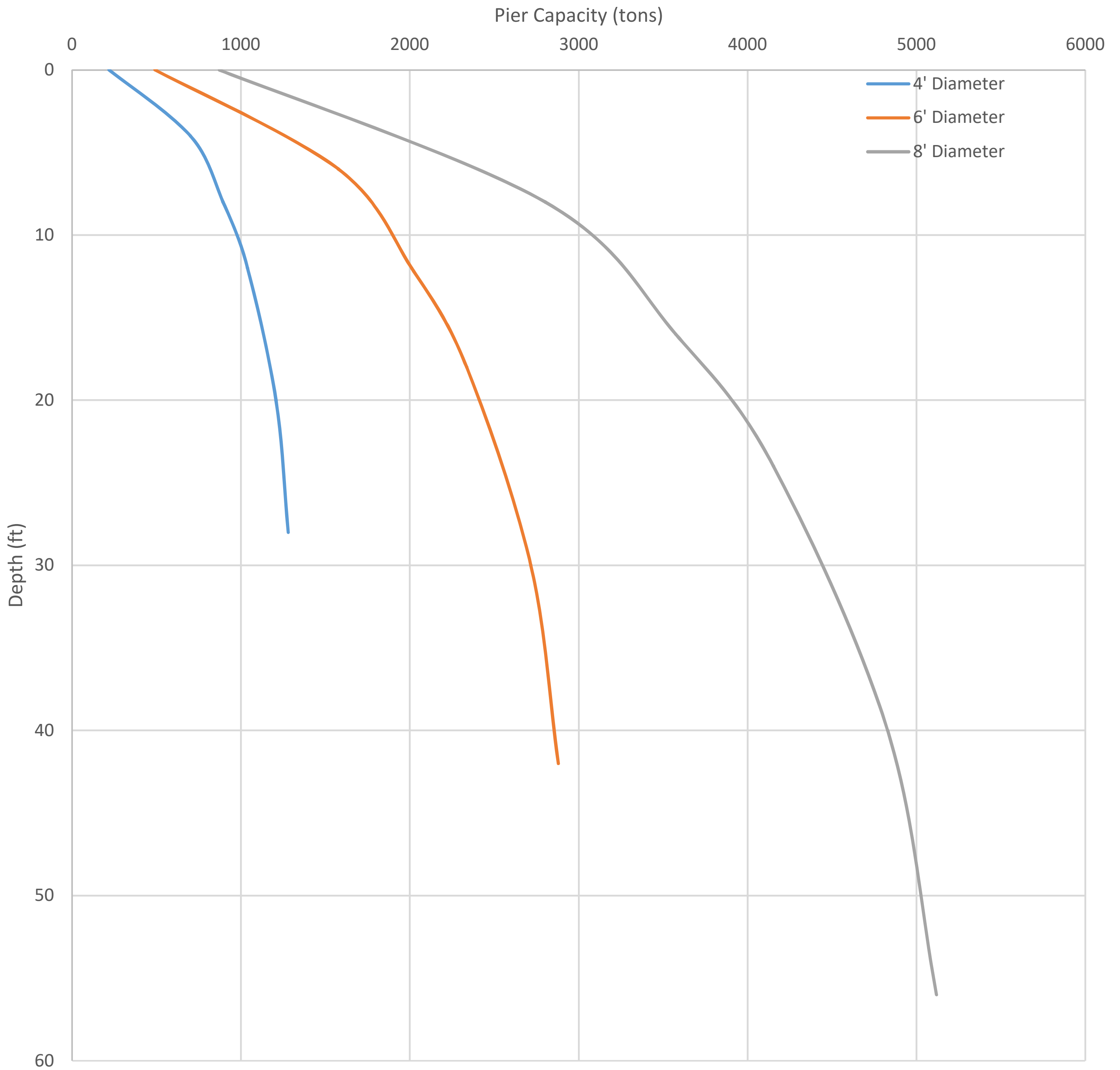
BRIDGE "J"
I-35/I-240
INTERCHANGE
OKLAHOMA CITY, OK

PSI REPORT NO
05471207-7

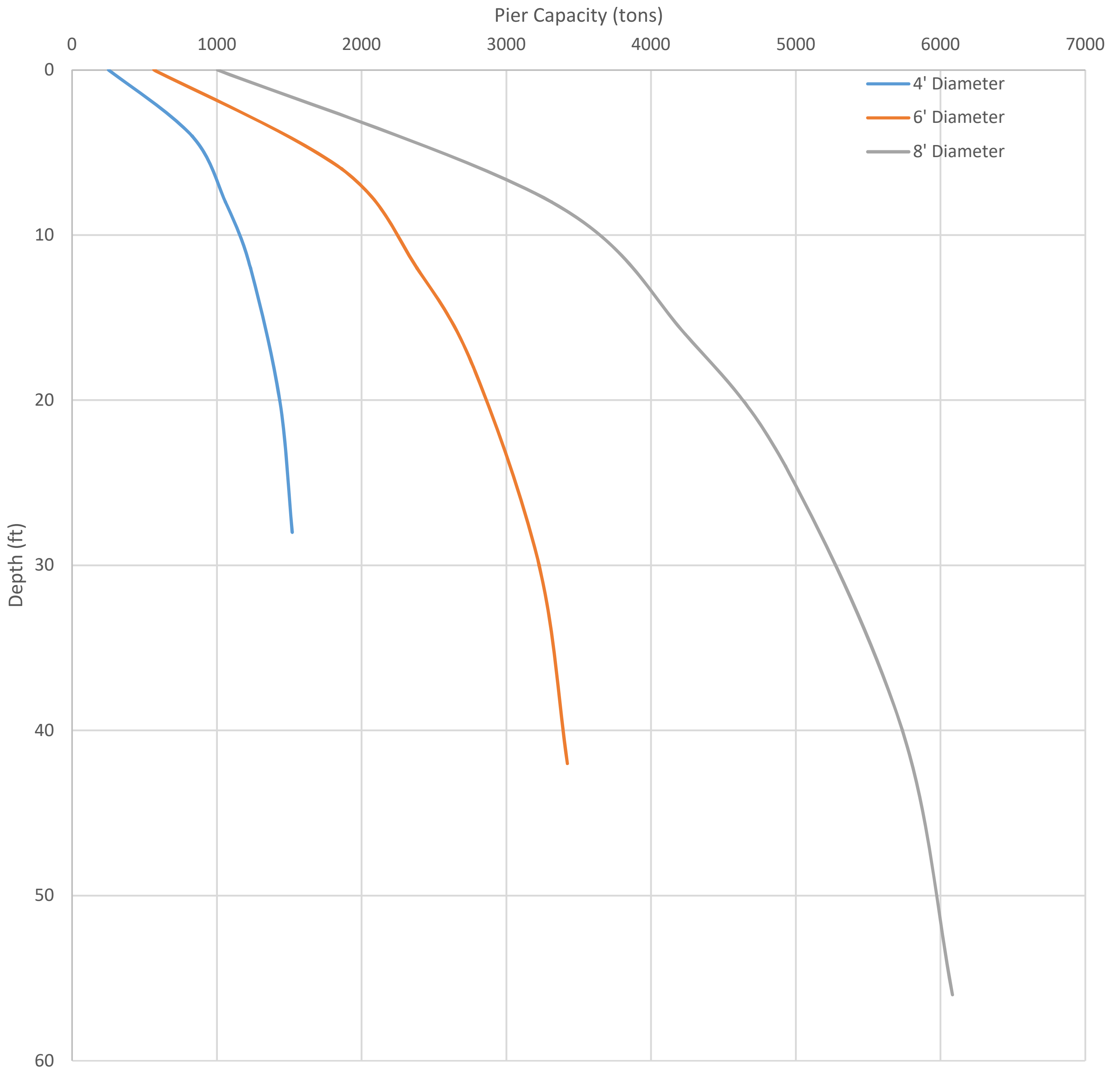
11825 S PORTLAND AVE
OKLAHOMA CITY, OK
(405) 735-6052

DATE:
DECEMBER 2017

NOMINAL BEARING CAPACITY



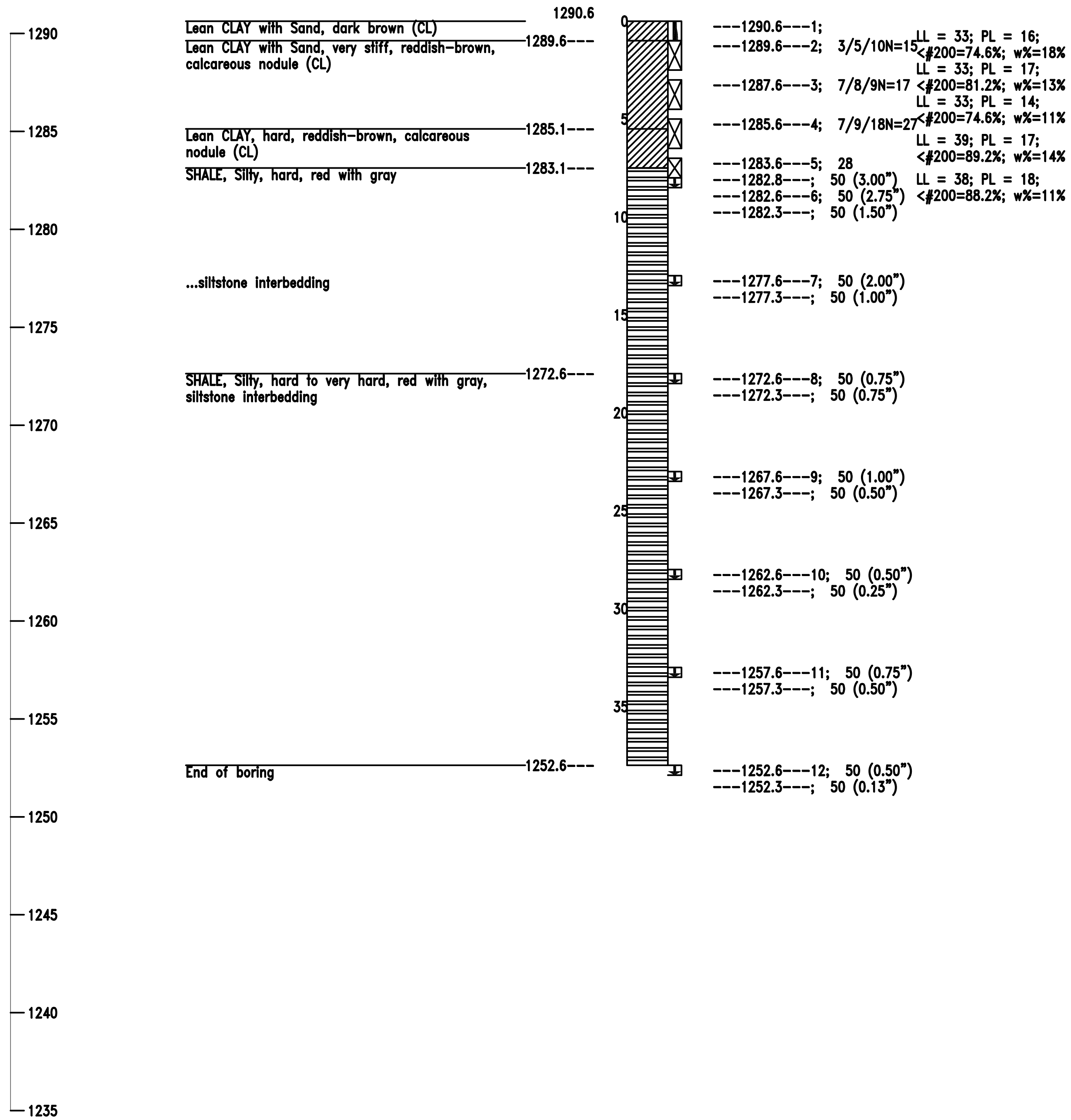
psi Information To Build On Engineering • Consulting • Testing	DRAWN BY: AAW	BRIDGE "J" I-35 & I-240 INTERCHANGE OKLAHOMA CITY, OK	PROJECT NO.: 05471207-7
11825 S. PORTLAND AVENUE OKLAHOMA CITY, OK 73170 (405) 735-6052	DATE: DECEMBER 2017	BORING J-2	PRESSUREMETER BEARING DIAGRAM



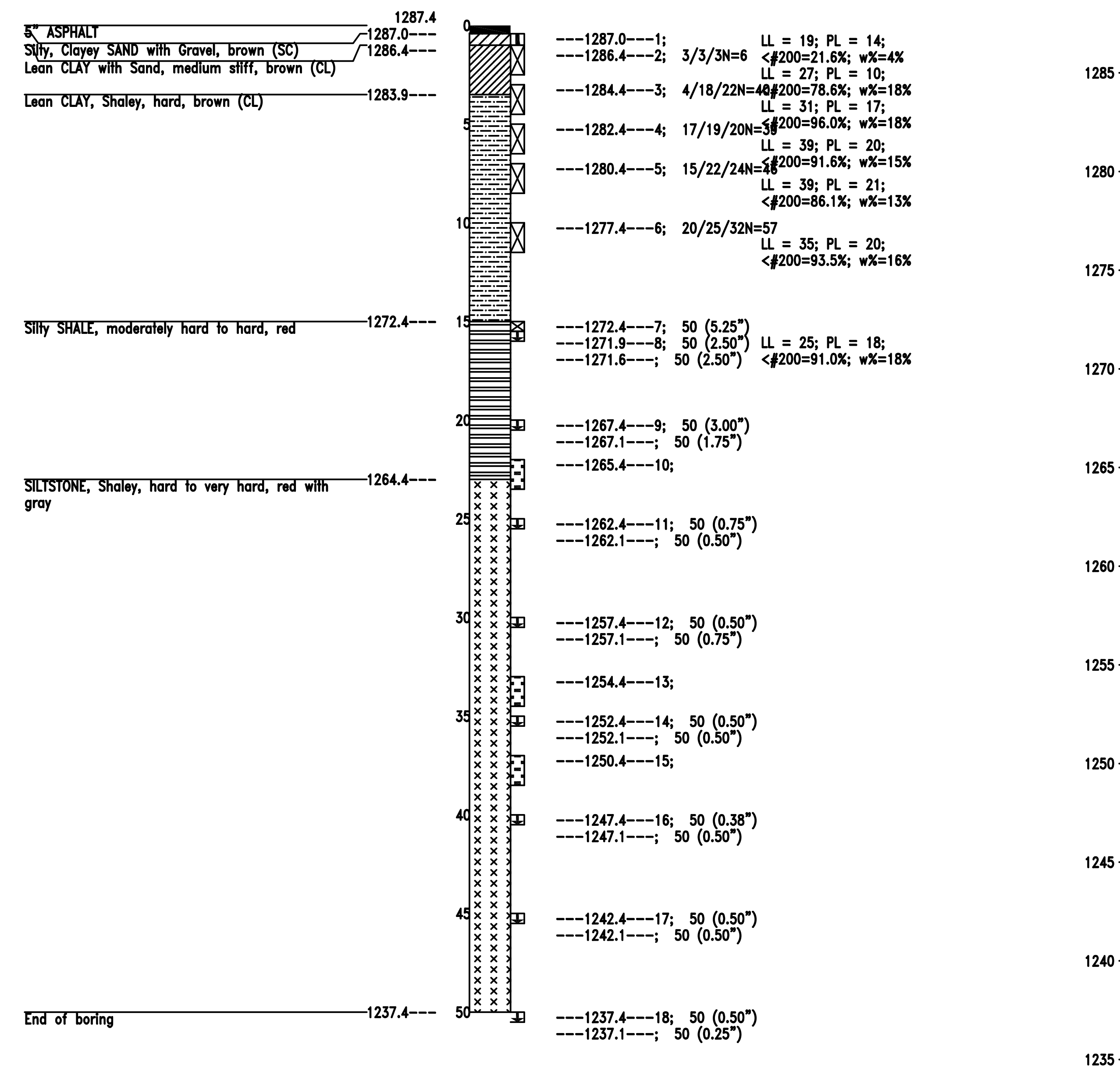
psi Information To Build On — Engineering • Consulting • Testing	DRAWN BY: AAW	BRIDGE "J" I-35 & I-240 INTERCHANGE OKLAHOMA CITY, OK	PROJECT NO.: 05471207-7
11825 S. PORTLAND AVENUE OKLAHOMA CITY, OK 73170 (405) 735-6052	DATE: DECEMBER 2017	BORING J-3	PRESSUREMETER BEARING DIAGRAM

APPENDIX H – FOUNDATION PLAN

Boring Number J-1
Northing: 143203.30
Easting: 2151409.47
Date Drilled: 5/9/2017



Boring Number J-2
Northing: 143317.15
Easting: 2151311.34
Date Drilled: 10/25/2017



REV. NO.	DESCRIPTION	DATE
1	Plan Format 9-9-16	

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 affixed 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:
- Denotes Split Spoon Sampling
- Denotes Texas Cone Penetrator Test
- 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

PROFESSIONAL SERVICE INDUSTRIES, INC.

psi Information To Build On
Engineering • Consulting • Testing

Bridge "J" OKLAHOMA COUNTY
I-35 & I-240 Interchange
SUBSURFACE PROFILE
(SHEET 1 OF 2)

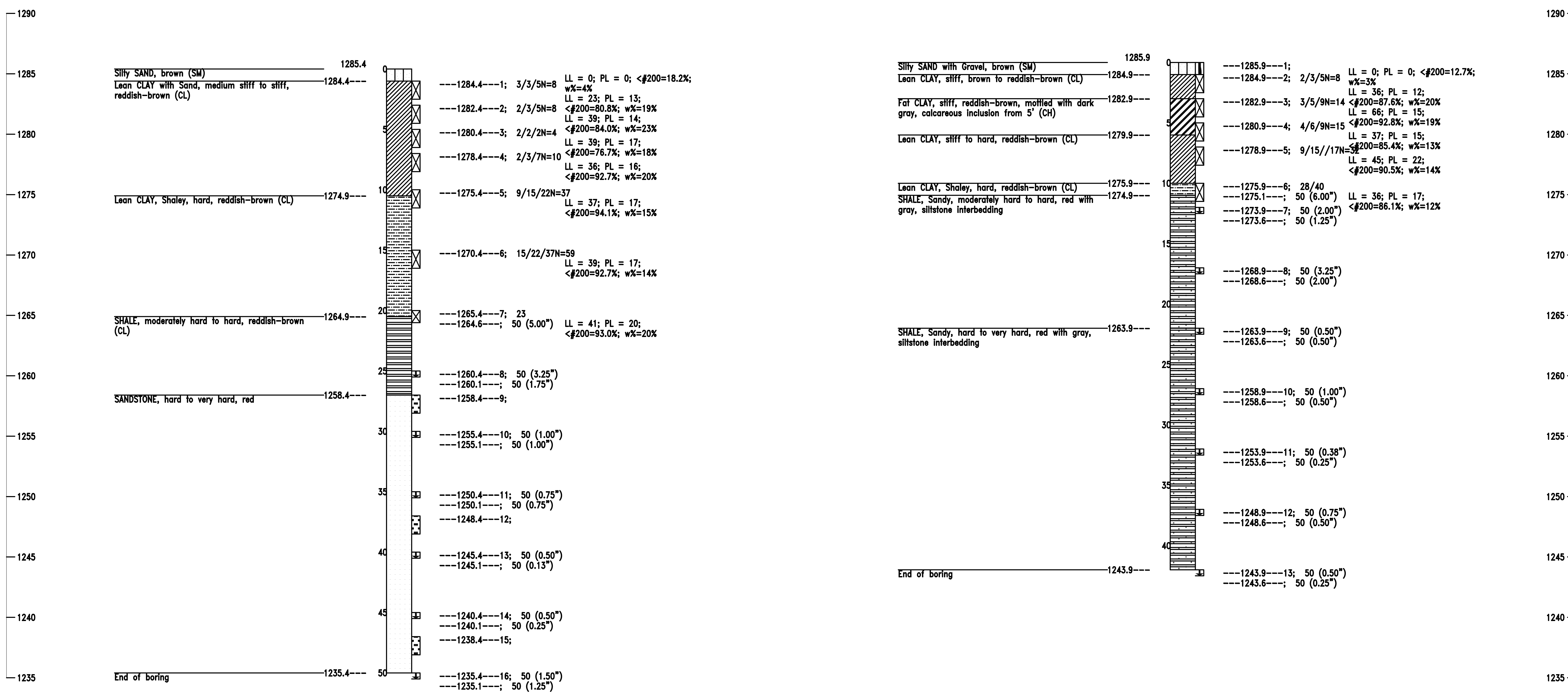
Design	XX	XX
Detail	AAW	12/17
Check	XX	XX
Squad	XXXXXX	
Engr.	XXXXXX	

STATE OF OKLAHOMA
ODOT
JOB PIECE NO.
SHEET NO. B000

REV. NO.	DESCRIPTION	DATE
1	Plan Format 9-9-16	

Boring Number J-3
Northing: 143419.54
Easting: 2151202360
Date Drilled: 10/27/2017

Boring Number J-4
Northing: 143510.42
Easting: 2151088.29
Date Drilled: 5/9/2017



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All geotechnical information contained on this sheet is covered by the engineering seal affixed 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: ☒ Denotes Split Spoon Sampling
⊞ Denotes Texas Cone Penetrator Test
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

PROFESSIONAL SERVICE INDUSTRIES, INC.
psi Information To Build On
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Bridge "J" OKLAHOMA COUNTY
I-35 & I-240 Interchange
SUBSURFACE PROFILE (SHEET 2 OF 2)

Design	XX	XX
Detail	AAW	12/17
Check	XX	XX

Squad: XXXXXX
Engr.: XXXXXX

STATE OF OKLAHOMA
ODOT
JOB PIECE NO.
SHEET NO. B000

APPENDIX I – 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 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	50 - 80
Extremely Dense	80+

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 - 2	Very Soft
0.25 - 0.50	2 - 4	Soft
0.50 - 1.00	4 - 8	Firm (Medium Stiff)
1.00 - 2.00	8 - 15	Stiff
2.00 - 4.00	15 - 30	Very Stiff
4.00 - 8.00	30 - 50	Hard
8.00+	50+	Very 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 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
		(LITTLE OR NO FINES)		GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	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
		(LITTLE OR NO FINES)		SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	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