DGL Consulting Engineers, LLC
Maumee, Ohio

Geotechnical Subsurface Investigation
Proposed Connector Trail
Swan Creek Metropark
Toledo, Ohio

February 2019

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Environmental, Geotechnical Engineering & Testing

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February 6, 2019

TTL Project No. 1726801

Mr. Joshua J. O'Neil, P.E., CPESC
DGL Consulting Engineers, LLC
3455 Briarfield Boulevard, Suite E
Maumee, Ohio 43537

Geotechnical Subsurface Investigation
Proposed Connector Trail
Swan Creek Metropark
Toledo, Ohio

Dear Mr. O'Neil:

Following is the report of the geotechnical subsurface investigation performed by TTL Associates, Inc. (TTL) for the referenced project. This study was performed in accordance with TTL Proposal No. 1726801, dated August 20, 2018, and authorized via email from you on October 5, 2018.

"Draft" boring logs were provided to you via email on December 3, 2018. This final report contains the results of our study, our engineering interpretation of the results with respect to the project characteristics, and our recommendations for design and construction of foundations and pavements.

Soil samples collected during this investigation will be stored at our laboratory for 90 days from the date of this report. The samples will be discarded after this time unless you request that they be saved or delivered to you.

Should you have any questions regarding this report or require additional information, please contact our office.

Sincerely,

TTL Associates, Inc.

STATE OF OHIO
CHRISTOPHER P IOTT
REGISTERED PROFESSIONAL ENGINEER

Christopher P. Iott, P.E.
Senior Geotechnical Engineer

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GEOTECHNICAL SUBSURFACE INVESTIGATION
PROPOSED CONNECTOR TRAIL
SWAN CREEK METROPARK
TOLEDO, OHIO

FOR

DGL CONSULTING ENGINEERS, LLC
3455 BRIARFIELD BOULEVARD, SUITE E
MAUMEE, OHIO 43537

SUBMITTED

FEBRUARY 6, 2019
TTL PROJECT NO. 1726801

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1.0 INTRODUCTION

This geotechnical subsurface investigation report has been prepared for the proposed connector trail to be constructed at Swan Creek Metropark in Toledo, Ohio. The trail will extend from nearby the park entrance off of Airport Highway east to Byrne Road. The general area of the project is shown on the attached Site Location Map (Plate 1.0).

This report summarizes our understanding of the proposed construction, describes the investigative and testing procedures, presents the findings, discusses our evaluations and conclusions, and provides our design and construction recommendations for foundations and pavements.

This study was performed in accordance with TTL Proposal No. 1726801, dated August 20, 2018, and authorized via email from Mr. Josh O'Neil of DGL Consulting Engineers, LLC (DGL) on October 5, 2018.

The purpose of this investigation was to evaluate the subsurface conditions and laboratory data relative to the installation and support of culverts, as well as design and construction of bridge foundations and pavements at the referenced site. This investigation included ten test borings, field and laboratory soil testing, and a geotechnical engineering evaluation of the test results.

This report includes:

- A description of the subsurface soil and groundwater conditions encountered in the borings.
- Design recommendations for pavements, as well as culvert and bridge foundations, related to the proposed development.
- Recommendations concerning soil- and groundwater-related construction procedures such as site preparation, earthwork, culvert installation, foundation and pavement construction, as well as related field testing.

The scope of this study did not include an environmental assessment of the surface or subsurface materials.

2.0 INVESTIGATIVE PROCEDURES

This subsurface investigation included ten test borings drilled by TTL during the period from October 18 through 25, 2018. Borings B-1 through B-4, B-7, and B-10 were located in the field by TTL based on coordination with DGL and site reconnaissance by DGL and TTL. The remaining borings were staked in the field by DGL. The general locations of the borings are summarized in the following table. Additionally, the approximate locations of the borings and connector trail that was proposed at the time of our field services are shown on the Test Boring Location Plan (Plate 2.0). It is our understanding that the connector trail route has been changed somewhat from what was shown on Plate 2.0.

Table 2.0.A. Test Boring Locations

Boring Number	General Location	Proposed Development	Boring Relocation Information
B-1 through B-3	Existing crushed stone path extending northeast then east from parking lot at Airport Highway entrance.	Asphalt trail	-
B-4	Two track path south of Swan Park Apartments.	Asphalt trail	-
B-5 and B-6	West and east sides, respectively, of ravine crossing.	Culvert	B-5 was moved approximately 33 feet west of staked location due to slope.
B-7	West of triangular property area along north side of Swan Creek. Approximately 40 feet west of property stake and 7 feet north of Swan Creek bank edge.	Potential look-out	-
B-8 and B-9	Northwest and Southeast sides, respectively, of Swan Creek, south of dead end of Fries Avenue.	Bridge across Swan Creek	B-8 was moved to crest of slope, due to boring staked along steep slope. B-9 was moved approximately 10 feet west of staked location due to access constraints.
B-10	At toe of slope associated with the mound behind the guardrail where the proposed trail will exit to Byrne Road.	Asphalt trail	-

Boring B-9 encountered auger refusal at a depth of 36 feet below existing grade, at an elevation much higher than mapped bedrock in the area. To evaluate whether auger refusal was due to encountered bedrock or cobbles/boulders, an offset borehole was advanced at a location 8 feet west of the Boring B-9 location. Auger refusal was again encountered at a depth of 36 feet in the offset borehole, which is typically indicative of bedrock.

Borings B-5, B-6, B-8, and B-9 were surveyed by DGL. Survey data for these borings are summarized in the following table.

Table 2.0.B. Boring Survey Data				
Boring Number	**Location**	**Northing**	**Easting**	**Ground Surface Elevation (feet)**
B-5	Ravine Culvert	713478.5	1658644	618.81
B-6		713520.8	1658727	613.54
B-8	Swan Creek Bridge	713806.7	1659981	619.03
B-9		713673.8	1660207	586.37

The test borings were performed in general accordance with geotechnical investigative procedures outlined in ASTM Standards D 1452 and D 5434. The test borings performed during this investigation were drilled with an ATV-mounted drilling rig utilizing 3¼-inch inside diameter hollow-stem augers. Boring termination depths (and elevations for the surveyed borings) are summarized in the following table.

Table 2.0.C. Boring Termination Depths, Elevations, and Criteria			
Boring Number	**Boring Termination Depth (feet)**	**Approximate Boring Termination Elevation (feet)**	**Comments**
B-1	5	-	Planned depth.
B-2	5	-	Planned depth.
B-3	5	-	Planned depth.
B-4	5	-	Planned depth.
B-5	50	569	Planned depth.
B-6	50	563	Planned depth.
B-7	50	-	Planned depth.
B-8	70	549	Planned depth.
B-9	36 (AR)	550	Auger refusal on apparent bedrock or boulder zone since offset borehole also encountered auger refusal at a depth of 36 feet.
B-10	10	-	Planned depth.

AR = Auger Refusal.

During auger advancement, soil samples were generally collected at 2½-foot intervals to a depth of 30 feet and at 5-foot intervals thereafter. Split-spoon (SS) samples were obtained by the Standard Penetration Test (SPT) Method (ASTM D 1586), which consists of driving a 2-inch outside diameter split-barrel sampler into the soil with a 140-pound weight falling freely through a distance of 30 inches. The sampler was driven in three successive 6-inch increments with the number of blows per increment being recorded. The sum of the number of blows required to advance the sampler the second and third 6-inch increments is termed the Standard Penetration

Resistance (N-value) and is presented on the Logs of Test Borings attached to this report. The samples were sealed in jars and transported to our laboratory for further classification and testing.

Shelby tube (ST) samples were obtained at varying depths from Borings B-5 through B-9. The Shelby tube samples were obtained by hydraulically advancing a 3-inch diameter, thin-walled sampler approximately 24 inches beyond the hollow-stem auger into relatively undisturbed soil in accordance with ASTM D 1587. Each Shelby tube was then extracted from the subsoils, and the ends were capped and sealed. The samples were transported to our laboratory where selected samples were extruded, classified, and tested.

All of the recovered samples of the subsoils were tested in our laboratory for moisture content (ASTM D 2216), and were visually or manually classified in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 and D 2488). Dry density determinations and unconfined compressive strength tests by the constant rate of strain method (ASTM D 2166) were performed on selected intact cohesive samples. Unconfined compressive strength estimates were obtained for the remaining intact cohesive samples using a calibrated hand penetrometer. Particle size analyses (ASTM D 422) and Atterberg limits tests (ASTM D 4318) were performed on representative samples to determine soil classification and soil index properties. The test results are presented on the Logs of Test Borings, Tabulation of Test Data sheets, and Grain Size Distribution sheets attached to this report.

A one-point unconsolidated-undrained (UU) triaxial compressive strength test (ASTM D 2850) and a one-dimensional consolidation test (ASTM D 2435) were performed on the Shelby tube sample from Boring B-5. The UU test was performed using a confining pressure approximately equal to the overburden pressure at the midpoint of the sample interval. Results of these tests are attached to this report.

Soil conditions encountered in the test borings are presented in the Logs of Test Borings, along with information related to sample data, SPT results, water conditions observed in the borings, and laboratory test data. It should be noted that these logs have been prepared on the basis of laboratory classification and testing as well as field logs of the encountered soils.

Experience indicates that the actual subsoil conditions at a site could vary from those generalized on the basis of test borings made at specific locations. Therefore, it is essential that a geotechnical engineer be retained to provide soil engineering services during the site preparation, excavation, and foundation phases of the proposed project. This is to observe compliance with the design concepts, specifications, and recommendations, and to allow design changes in the event subsurface conditions differ from those anticipated prior to the start of construction.

3.0 PROPOSED CONSTRUCTION

It is our understanding that the project consists of construction of a new connector trail at Swan Creek Metropark in Toledo, Ohio. The trail will extend from the parking lot off of the Airport Highway entrance to the park, east to Byrne Road, generally along the northern portion of the park. The trail will be asphalt-paved, include an overlook of Swan Creek, a culvert at a ravine crossing and a bridge over Swan Creek, as well as a boardwalk extending out to Byrne Road.

Loads associated with the culvert and lookout structures were not available at the time of preparing this report. It is assumed that the culvert and lookout structure will be supported on shallow spread foundations.

It was indicated that the Swan Creek bridge will be a two-span structure. The span from the rear (northwest) abutment to the intermediate pier will be approximately 140 feet. The span from the intermediate pier to the forward (southeast) abutment will be approximately 90 feet. It was indicated that the forward abutment will actually be a pier supporting the bridge and the boardwalk extending to the southeast. Preliminary data regarding the location and bottom of pile cap elevation for the bridge substructures, as well as maximum total factored load, are summarized in the following table.

Table 3.0. Swan Creek Bridge Substructure Data

Substructure	Northing	Easting	Bottom of Pile Cap Elevation (feet)	Total Factored Load (kips)
Rear (Northwest) Abutment	713782.8	1659965.6	616.0	128
Intermediate Pier	713719.7	1660106.0	581.0	201
Forward (Southeast) Abutment	713675.3	1660204.2	586.5	133

It is assumed that each bridge substructure will be supported using 2 to 3 piles.

The boardwalk will be constructed using a box beams or double tees. The structure will be supported by 15 piers. Each pier is planned to include three piles. The bottom of pile cap elevation for each pier was not available at the time of preparing this report. We have assumed a bottom of pile cap elevation of Elev. 586.5 based on the forward abutment for the bridge, which will also be a pier for the boardwalk. Total factored load for the piers was indicated to be 120 kips for the box beam option or 90 kips for the double tees option.

We have assumed that piles will include 2 feet of embedment into the pile caps. Based on the provided total factored loads, 2 to 3 piles per substructure, and the encountered subsurface conditions, pile foundations are anticipated to be friction piles with capacity provided by side resistance and end-bearing. Ohio Department of Transportation (ODOT) prescribes cast-in-place (CIP) concrete piles with driven pipe shells for friction piles, with an option for use of H-piles. It is our recent experience in this area that H-piles have been favored over CIP piles. The smallest typical H-pile section recommended by ODOT is HP10x42, for which a maximum Ultimate Bearing Value (R_{ndr}) of 350 kips is prescribed. Based on the total factored loads for this project, the R_{ndr} values for individual piles supporting each substructure are expected to be less than 350 kips, such that HP10x42 piles are suitable for the vertical loads.

4.0 GENERAL SITE AND SUBSURFACE CONDITIONS

4.1 General Site Conditions

The western half of the trail alignment is currently a stone path or 2-track path. The eastern half of the trail alignment is currently wooded. Site grades are relatively level along the stone path and 2-track path. Rolling topography was present in the wooded area. Approximately 45 feet of grade change is present in the area of Swan Creek where the new bridge is planned to be constructed.

The surface materials encountered in Borings B-1 through B-4 consisted of crushed stone varying in thickness from approximately 2 to 5 inches. The surface materials encountered in Borings B-5 through B-10 consisted topsoil generally varying in thickness from approximately 8 to 15 inches, with approximately 4 inches of topsoil encountered at the location of Boring B-8.

4.2 General Soil Conditions

Based on the results of our field and laboratory tests, the subsoils encountered underlying the surface materials consisted of wind-deposited (eolian) beach ridge granular deposits and interbedded granular and cohesive alluvial deposits, underlain by cohesive lacustrine deposits, underlain by cohesive glacial till deposits.

Stratum I consisted of wind-deposited (eolian) beach ridge granular deposits and interbedded granular and cohesive alluvial deposits encountered underlying the surface materials to depths ranging from approximately ½ foot to 13 feet below existing grades. Borings B-2 and B-4 were terminated within Stratum I at a depth of 5 feet. Boring B-10 was terminated within Stratum I at a depth of 10 feet. The Stratum I granular soils consisted of poorly graded sand with varying amounts of silt (SP and SP/SM), silty sand (SM), and clayey sand (SC). The cohesive soils consisted of lean clay (CL) with varying amounts of sand, as well as sandy silt (ML). Trace organics and root hairs were observed in occasional recovered alluvial samples. A zone of **peat (PT)** with sand was encountered within Stratum I from 6 to 8 feet in Boring B-7. SPT N-values for the granular soils generally ranged from 3 to 8 blows per foot (bpf), indicating **very loose** to **loose** compactness. SPT N-values for the cohesive soils generally ranged from 4 to 15 bpf, indicating **soft** to stiff consistency. For the granular soils, moisture contents generally ranged from 4 to 15 percent, although higher moisture contents were determined for zones of granular soils encountered in Borings B-9 and B-10. For the cohesive soils, moisture contents ranged from 13 to 24 percent.

Stratum II consisted of predominantly stiff to very stiff cohesive lacustrine deposits encountered underlying Stratum I in Borings B-1, B-3, B-5, B-6, and B-8 to depths ranging from 6 to 13 feet. Borings B-1 and B-3 were terminated within this stratum at a depth of 5 feet. The Stratum II cohesive soils consisted of lean clay (CL) with varying amounts of sand. SPT N-values ranged from 9 to 18 bpf. Unconfined compressive strengths ranged from 3,000 pounds per square foot (psf) to greater than 9,000 psf (maximum reading obtainable using a hand penetrometer). Moisture contents ranged from 18 to 25 percent.

Stratum III consisted of cohesive glacial till deposits encountered underlying Stratum I in Borings B-7 and B-9, as well as Stratum II in Borings B-5, B-6, and B-8, to boring termination at depths generally ranging from 50 to 70 feet. Boring B-9 encountered auger refusal at a depth of 36 feet. The Stratum III cohesive soils consisted of lean clay (CL) with varying amounts of sand and trace gravel, as well as sandy silt (ML) with trace gravel. SPT N-values generally ranged from 5 to 15 bpf, indicating medium stiff to stiff consistency. Higher SPT N-values, generally ranging from 16 to 19 bpf, indicating very stiff consistency, were determined for approximately 15 percent of the Stratum III samples. Unconfined compressive strengths generally ranged from 1,000 to 7,000 psf. Unconfined compressive strengths ranging from 480 to 585 psf, indicating **very soft** to **soft** consistency, were determined for samples obtained in the upper portion of this layer in Boring B-5 and the deeper portion of the layer encountered in Boring B-8. Moisture contents generally ranged from 14 to 27 percent.

A one-point unconsolidated-undrained (UU) triaxial compressive strength test and a one-dimensional consolidation test were performed on a sample obtained from Stratum III in Boring B-5 (ST-1). Results of these tests are attached to this report and summarized in the following table.

Table 4.2. UU Triaxial and One-Dimensional Consolidation Test Results (Stratum III Sample)

Boring Number	Sample	Sample Depth (feet)	Sample Elevation	Undrained Shear Strength (psf)	Previous Consolidation Pressure (psf)	Virgin Compression Index, Cc	Recompression Index, Cr	Initial Void Ratio
B-5	ST-1	18-20	601-599	315	2,600	0.12	0.03	0.53

Auger refusal was encountered in Boring B-9 at a depth of 36 feet (Elev. 550±). To evaluate whether auger refusal was due to encountered bedrock or cobbles/boulders, an offset borehole was advanced at a location 8 feet west of the Boring B-9 location. Auger refusal was again encountered at a depth of 36 feet in the offset borehole, which is typically indicative of bedrock. It should be noted that bedrock is mapped in the area of Boring B-9 on the order of Elev. 530, and even deeper in the western portion of the project site. Therefore, there is concern that boulders, rather than bedrock, is present at the depth of auger refusal in Boring B-9 (and the offset location adjacent to Boring B-9). Such concern was considered with respect to pile foundation recommendations for the proposed bridge across Swan Creek and support of the boardwalk extending southeast of the bridge.

Additional descriptions of the stratigraphy encountered in the borings are presented on the Logs of Test Borings.

4.3 Groundwater Conditions

Groundwater was initially encountered during drilling in Borings B-7, B-9, and B-10 at depths of 24 feet, 8 feet (Elev. 578±), and 8 feet below existing grades, respectively. Groundwater was observed upon completion of drilling in Borings B-9 and B-10 at depths of 24½ feet (Elev. 562±) and 8 feet, respectively. Groundwater was not initially encountered during drilling nor observed upon completion of drilling in any of the remaining borings performed for this investigation. It should be noted that each of the borings was drilled and backfilled within the same day, and stabilized water levels may not have occurred over this limited period. Instrumentation was not installed to observe long-term groundwater levels.

Based on the limited data available, such as the soil characteristics and the moisture conditions encountered in the borings, it is our opinion that the "normal" groundwater level for the portions of the site at higher elevations may be generally encountered at Elevs. 605± to 600±, corresponding to depths on the order of 8 to 16 feet below existing grades. It is our opinion that the "normal" groundwater level for portions of the site at lower elevations may be generally encountered at Elevs. 580± to 575±, corresponding to approximately 5 to 10 feet below existing grades. However, this investigation did not include research of possible hydrological influences at the project site. It should be noted that groundwater elevations can fluctuate with seasonal and climatic influences. In particular, "perched" water may be encountered in granular soils that are underlain by relatively impermeable native cohesive soils. Additionally, groundwater levels may be affected by the water level in Swan Creek and other site drainageways. Therefore, the groundwater conditions may vary at different times of the year from those encountered during this investigation.

5.0 DESIGN RECOMMENDATIONS

The following conclusions and recommendations are based on our understanding of the proposed construction and on the data obtained during the field investigation. If the project information or location as outlined is incorrect or should change significantly, a review of these recommendations should be made by TTL. These recommendations are subject to the satisfactory completion of the recommended site and subgrade preparation and fill placement operations described in Section 6.0, “Construction Recommendations”.

5.1 Ravine Crossing Culvert (Borings B-5 and B-6)

Consideration is being given to using a pipe along the ravine crossing. If the pipe option is utilized, sediment in the ravine should be removed prior to placement of the pipe and bedding as prescribed by the manufacturer.

If a culvert with foundations is utilized, recommendations are provided below for evaluation of foundations, culvert walls, and headwalls associated with the culvert.

5.1.1 Culvert Foundations

Culvert foundations should bear at least 3½ feet below finished grades to provide protection from frost penetration. Deeper embedment may be required depending on scour considerations. Based on a provided survey drawing with topographic contours, the bottom of ravine was on the order of Elevs. 602 to 601. Therefore, culvert foundations are anticipated to bear at Elev. 598±.

Based on the conditions encountered in Borings B-5 and B-6, the soils at this bearing elevation are anticipated to consist of **very soft** to medium stiff cohesive Stratum III glacial till deposits. The borderline soft to medium stiff cohesive soils are considered generally suitable for support of spread footing foundations using a relatively low factored bearing resistance. If very soft cohesive soils are encountered, they will require removal and replacement with new granular engineered fill as described below. Additionally, if sediment is present at the foundation bearing elevation, it will require over-excavation and replacement with new granular engineered fill.

If sediment, very soft native cohesive soils, or other unsuitable foundation bearing soils are encountered, over-excavation should extend through these materials to suitable bearing soils, with widening of the footing over-excavation as described below. The base of the over-excavation should be widened one foot for every foot of depth below the planned bearing depth, with the over-excavation centered along the footing. The over-excavated areas should be backfilled with dense-graded aggregate, placed in controlled lifts, and compacted to not less than 100 percent of the maximum dry density as determined by ASTM D 698 (Standard Proctor). Alternatively, the over-excavated areas could be backfilled with lean concrete having a minimum compressive strength of 1,500 pounds per square inch (psi) or other flowable controlled-density fill having a minimum compressive strength of 300 psi. If foundations will be placed at the base of the over-excavation or the lean concrete fill option will be utilized, widening the footing over-excavation will not be required. If the controlled-density fill option is utilized, the footing over-excavation shall be widened as discussed above.

We understand that the culvert and headwall foundations will be designed using LRFD specifications. At the **service** limit state, we recommend a nominal (unfactored) bearing resistance (q_n) of 1 kip per square foot (ksf) for foundations bearing on soft to medium stiff native cohesive soils, or the engineered replacement fill described above. At the service limit state, the resistance factor (ϕ_b) is 1.0. Therefore, the factored bearing resistance (q_r) is 1 ksf. From a conventional allowable stress design comparison, this is roughly akin to using an allowable bearing pressure. At the **strength** limit state, we recommend a nominal bearing resistance (q_n) of 2.8 ksf for foundations bearing on soft to medium stiff native cohesive soils, or the engineered replacement fill described above. At the strength limit state, the resistance factor (ϕ_b) is 0.45. Therefore, the factored bearing resistance (q_r) is 1.2 ksf. From a conventional allowable stress design comparison, this is roughly akin to calculating an ultimate bearing capacity and applying a factor of safety. Our evaluations were based on the soft to medium stiff bearing stratum exhibiting a minimum unconfined compressive strength of 1,000 psf.

Settlement of the culvert foundation was calculated by conventional consolidation theory utilizing the recompression index, based on one-dimensional consolidation test results and empirical relations using moisture content. Based on the bearing pressure of 1 ksf at the service limit state, total settlement was calculated to be less than 1 inch. It should be noted that settlement is dependent on the foundation width. The calculated total settlement was based on an assumed foundation width of 6 feet. If the foundation width is increased for overturning resistance, it is anticipated that the average bearing pressure would be less than 1 ksf. Presumably, increases in footing size with corresponding reductions in average bearing pressure would result in total settlements no greater than 1 inch. When the foundation design is being finalized, footing width and bearing pressures should be reviewed to confirm tolerable settlement.

We strongly recommend that the bearing capacity at the bottom of all footing excavations be checked during construction by a TTL geotechnical engineer or qualified representative to verify that the exposed soil conditions at the bearing elevations are consistent with the subsurface conditions encountered in the test borings, and that unsuitable materials have been over-excavated and replaced with properly placed new engineered fill. Additionally, the presence of our engineer will help facilitate the timely remediation of unsuitable soils. If the results of hand penetrometer or other strength tests indicate the exposed soil conditions are less favorable than those indicated by the borings, it may be necessary to increase the footing size to accommodate the lower bearing strengths, or to over-excavate and backfill with new engineered fill.

If foundation excavations will not be concreted the same day as excavation occurs, we recommend that a thin mat of lean concrete be placed over the cohesive bearing soils to protect the bearing surface from groundwater seepage and/or construction.

Culvert and headwall footings should be at least 24 inches wide. For shallow foundations, overturning and sliding stability due to wall backfill should also be considered for the headwalls, and wider footings may be needed to satisfy stability for these loading conditions. Additionally, scour/erosion protection of the shallow foundations should be considered.

5.1.2 Culvert Walls and Headwalls

Headwall foundations should be designed in accordance with the recommendations presented in Section 5.1.1.

For culvert walls and headwalls that are restrained from rotation and are considered rigid and non-yielding, lateral earth pressure should be assumed for at-rest conditions. It is anticipated that excavated on-site cohesive soils will be utilized for the majority of the backfill behind the new walls. For these soils, an at-rest earth pressure coefficient (k_o) of 0.50 should be used in determining the lateral pressure acting on the walls, along with a total (moist) soil unit weight of 130 pounds per cubic foot (pcf). Alternatively, an equivalent fluid weight of 65 pcf may be used for the at-rest case design. If lower at-rest earth pressures are preferred for structural reasons or to improve overturning/sliding stability, we recommend that a select, free-draining granular fill (such as No. 57 or 67 stone) be utilized for the entire wall backfill zone. For these granular fill types, k_o may be taken as 0.40, and the soil unit weight may be assumed as 120 pcf. Alternatively, an equivalent fluid weight of 50 pcf may be used for these granular fills.

Headwalls that are not restrained at the top of the wall may be designed for active lateral earth pressure condition. If the on-site cohesive soils are utilized for the backfill behind the headwalls, a k_a value of 0.33 may be used for design along with a soil unit weight of 130 pcf or alternatively, an equivalent fluid weight of 45 pcf may be used. If a free-draining granular fill is utilized, a k_a value of 0.25 may be used for design along with a soil unit weight of 120 pcf, or alternatively, an equivalent fluid weight of 30 pcf may be used.

It should also be noted that these earth pressures do not include hydrostatic pressures that may result from elevated groundwater conditions above the normal waterway level. We recommend that consideration be given to headwall drainage to prevent build-up of unbalanced hydrostatic pressures behind the walls. We recommend that headwalls greater than 4 feet in height include a minimum 2-foot granular drainage zone behind the wall, in combination with wall weep holes and/or longitudinal foundation drain pipe at the base of the wall footing that is free to drain by gravity discharge. Otherwise, the wall design should consider an appropriate resistance factor based on flood elevations or other seasonal groundwater conditions.

Additionally, the earth pressures indicated above are based on a level backfill condition behind the headwall. However, if there are areas where appreciable sloping backfill is required near the top of the wall, surcharge loading or equivalent higher earth pressure coefficients should be evaluated, based on backfill material, backfill slope, and proximity to the wall. In general, 50 percent of the vertical surcharge load may be assumed for lateral loading in the design of the wall.

Culverts should be backfilled concurrently with nearly equal fill heights on both sides to avoid inducement of overturning or sliding instability. Headwall footings should also be checked for sliding stability. We recommend that passive pressure be considered negligible at the toe of the headwall due to the potential for erosion and/or freeze-thaw behavior that would significantly reduce reliance on passive earth pressure. Without passive pressure considerations, the LRFD factored sliding resistance (R_R) is determined by $\phi_T R_T$, where R_T is the nominal sliding resistance on the base of the footing. For cohesive soil beneath the wall foundation, the nominal sliding resistance may be taken as the cohesion of the clay. Based on the encountered conditions, we recommend a cohesion of the soft to medium stiff bearing soils of 500 pounds per square foot (psf). For cast-in-place or precast concrete bearing on cohesive soils, the resistance factor is 0.85. Therefore, the factored sliding resistance (R_R) is 425 psf.

5.2 Lookout Foundations (Boring B-7)

It is anticipated that the lookout foundations will consist of continuous (strip) and/or isolated (square) shallow spread foundations. Lookout foundations should bear at least 3½ feet below finished grades to provide protection from frost penetration. Based on the conditions encountered in Boring B-7, the soils encountered in the upper 8 feet consisted of **soft** cohesive alluvial deposits, underlain by **loose** granular alluvial deposits, underlain by a two feet zone of **peat**. These soils are not considered suitable for support of foundations. Depending on lookout foundation proximity to Swan Creek, unsuitable soils may extend shallower or deeper than 8 feet.

The soils encountered in Boring B-7 at a depth of 8 feet consisted of stiff to very stiff cohesive alluvial deposits extending to a depth of 13 feet, which were underlain by stiff to very stiff cohesive glacial till. These soils are considered generally suitable for support of the proposed structure.

Where soft native cohesive soils, loose native granular soils, peat, or other unsuitable foundation bearing soils are encountered, over-excavation should extend through these materials to suitable bearing soils, with widening of the footing over-excavation as described below. The base of the over-excavation should be widened one foot for every foot of depth below the planned bearing depth, with the overexcavation centered along the footing. The over-excavated areas should be backfilled with dense-graded aggregate, placed in controlled lifts, and compacted to not less than 100 percent of the maximum dry density as determined by ASTM D 698 (Standard Proctor). Alternatively, the over-excavated areas could be backfilled with lean concrete having a minimum compressive strength of 1,500 pounds per square inch (psi) or other flowable controlled-density fill having a minimum compressive strength of 300 psi. If foundations will be placed at the base of the over-excavation or the lean concrete fill option will be utilized, widening the footing over-excavation will not be required. If the controlled-density fill option is utilized, the footing over-excavation shall be widened as discussed above.

We understand that the structure will be designed using LRFD specifications. At the **service** limit state, a nominal (unfactored) bearing resistance (q_n) of 3.5 kips per square foot (ksf) was determined for foundations bearing on stiff to very stiff native cohesive soils, or the engineered replacement fill described above. At the service limit state, the resistance factor (ϕ_b) is 1.0. Therefore, the factored bearing resistance (q_r) is 3.5 ksf. From a conventional allowable stress design comparison, this is roughly akin to using an allowable bearing pressure. **However, a reduced factored bearing resistance may be required if total settlement must be limited to the typical limit of 1 inch, as discussed below.**

At the **strength** limit state, we recommend a nominal bearing resistance (q_n) of 9.2 ksf for strip foundations and 11.0 ksf for column foundations bearing on stiff to very stiff native cohesive soils, or the engineered replacement fill described above. At the strength limit state, the resistance factor (ϕ_b) is 0.45. Therefore, the factored bearing resistance (q_r) is 4.1 ksf for strip foundations and 4.9 ksf for column foundations. From a conventional allowable stress design comparison, this is roughly akin to calculating an ultimate bearing capacity and applying a factor of safety. Our evaluations were based on the stiff to very stiff bearing stratum exhibiting a minimum unconfined compressive strength of 3,500 psf.

Settlement of the lookout foundation was calculated by conventional consolidation theory utilizing the recompression index, based on one-dimensional consolidation test results and empirical relations using moisture content. Additionally, the FHWA C' method was considered for the cohesive alluvial deposits. It should be noted that settlement is dependent on the foundation width. Our settlement evaluations were based on an assumed foundation width (both strip and column) of 4 feet. Based on the bearing pressure of 3.5 ksf at the service limit state, total settlement was calculated to be on the order of 1 inch to 1¼ inches. **To limit total calculated settlement to approximately 1 inch or less, the bearing pressure was required to be reduced to 2.5 ksf. When the foundation design is being finalized, footing width and bearing pressures should be reviewed to confirm tolerable settlement.**

We strongly recommend that the bearing capacity at the bottom of all footing excavations be checked during construction by a TTL geotechnical engineer or qualified representative to verify that the exposed soil conditions at the bearing elevations are consistent with the subsurface conditions encountered in the test borings, and that unsuitable materials have been over-excavated and replaced with properly placed new engineered fill. Additionally, the presence of our engineer will help facilitate the timely remediation of unsuitable soils. If the results of hand penetrometer or other strength tests indicate the exposed soil conditions are less favorable than those indicated by the borings, it may be necessary to increase the footing size to accommodate the lower bearing strengths, or to over-excavate and backfill with new engineered fill.

If foundation excavations will not be concreted the same day as excavation occurs, we recommend that a thin mat of lean concrete be placed over the cohesive bearing soils to protect the bearing surface from groundwater seepage and/or construction.

Strip and square footings should be at least 24 inches wide and 30 inches wide, respectively. Scour/erosion protection of the shallow foundations should be considered due to proximity to Swan Creek.

5.3 Bridge and Boardwalk Foundations (Borings B-8 and B-9)

We understand that the pedestrian bridge across Swan Creek and the boardwalk extending southeast from the bridge will be designed using LRFD specifications. For piles not driven to refusal on bedrock, the ODOT Bridge Design Manual (BDM) provides options for cast-in-place (CIP) concrete piles with driven pipe shells and driven H-piles. It is our recent experience in this area that H-piles are the preferred option.

Bottom of pile cap elevations are summarized in Section 3.0. It was assumed that there will be 2 feet of pile stick-up embedded in the pile caps. As mentioned in Section 3.0, HP 10x42 piles with a maximum ultimate bearing value (R_{ndr}) of 350 kips will be suitable based on the provided total factored loads. ODOT Bridge Design Manual (BDM) Section 202.2.3.2.b indicates that, for piles not driven to refusal on bedrock, a dynamic resistance factor of 0.70 is to be utilized for piles installed in accordance with ODOT Construction and Materials Specifications (CMS) 507 and CMS 523. The following table contains a summary for each substructure of the total factored load and associated R_{ndr}, as well as the R_{ndr} associated with a singular pile assuming 3 piles or 2 piles will be utilized for support of the substructure.

Table 5.3.A. Total Factored Loads and Ultimate Bearing Values (R_{ndr})

Substructure	Total Factored Load (kips)	Substructure R_{ndr} (kips)	Individual Pile R_{ndr} (kips)	
			3 Piles per Substructure	2 Piles per Substructure
Bridge Rear (Northwest) Abutment	128	183	61	92
Bridge Intermediate Pier	210	287	96	144
Bridge Forward (Southeast) Abutment	133	190	64	95
Boardwalk Pier	120	172	58	86

We have assumed that the rear abutment will be constructed with a "spill-through" section with rock slope protection. Therefore, scour and associated loss of pile side resistance will not be a concern for these foundations. However, for the bridge intermediate pier and forward abutment, scour may be a concern for the foundations associated with these substructures. Scour depths were not available at the time of preparing this report. Therefore, we have not included scour in pile foundation evaluations. **If it is found that design should incorporate scour, the recommendations contained herein should be reviewed by TTL to evaluate whether the minimum required pile embedment should be extended deeper.**

Pile resistance analyses were performed using FHWA pile analysis software DRIVEN. In the DRIVEN analyses, adhesion for cohesive soils was modeled using the Tomlinson method (1979), and resistance in the "cohesionless" soils were determined by the Peck, Hanson, and Thornburn method (1974), using SPT N-values.

Results of the DRIVEN analyses are attached to this report, and are summarized in the following table. The summary table below includes the estimated pile length and order length. The estimated pile length includes the calculated length from anticipated pile cut-off elevation to pile tip elevation, rounded to the nearest 5 feet. The order length is the estimated length plus 5 feet.

Table 5.3.B. HP 10x42 Pile Foundation Recommendations

Location (Boring Number)	Bottom of Pile Cap Elevation (feet)	Cut-Off Pile Elevation (feet)	Number of Piles at Substructure	R_{ndr} (kips)	Recommended (Minimum) Pile Tip Elevation (feet)	Estimated Pile Length (feet)	Order Pile Length (feet)
Rear (Northwest) Abutment (B-8)	616	618	3 piles	61	600	20	25
			2 piles	92	588	35	40
Intermediate Pier (B-9)	581	583	3 piles	96	553	35	40
			2 piles	144	544(2)	40	45
Forward (Southeast) Abutment (B-9)	586.5	588.5	3 piles	64	563	30	35
			2 piles	95	554	35	40
Boardwalk Pier (B-9)	586.5(1)	588.5	3 piles	58	564	30	35
			2 piles	86	557	35	40

(1)Assumed based on forward abutment bottom of pile cap elevation.

(2)Auger refusal at Elev. 550± in B-9 not considered reliable due to mapped bedrock at Elev. 530±. For evaluation of pile embedment, assumed material present just prior to auger refusal extended deeper. However, if piles installed for this substructure encounter refusal, we recommend a static pile load test be performed to confirm suitable resistance is provided by the end-bearing material.

ODOT specifications indicate that the maximum center-to-center spacing of driven piles should be 8 feet in capped pile abutments. The maximum center-to-center spacing of driven piles should be 7 feet for the front row of wall-type abutments. Although close pile spacing is not anticipated, we recommend that the minimum center-to-center spacing for piles be 3 pile diameters to avoid superposition of stresses and possible reduction in group resistance due to close spacing.

A static pile load test (ASTM D 1143) is required only if the total pile order length for an individual structure exceeds 10,000 feet for piling of the same size and R_{ndr}. As such, a static pile load test is not expected to be required for this project. As mentioned previously, pile design is based on piles installed in accordance with ODOT CMS Item 523 "Dynamic Load Test." ODOT requires dynamic load testing to establish the driving criteria (i.e., blow count) for all piling not driven to refusal on bedrock. For an individual structure, the designer shall specify one dynamic load testing item for each pile size. If multiple pile capacities are required for a given pile size, the designer shall specify one testing item for each R_{ndr}. Although not anticipated, if static load tests are required, additional provisions include two dynamic load testing items **and** two restrike items for each static load test item. One dynamic load testing item consists of testing a minimum of two piles and performing a Case Pile Wave Analysis Program (CAPWAP) analysis on one of the two piles. One restrike item consists of performing dynamic testing on two piles and performing CAPWAP analysis on one of the two piles. Driven piles should be installed under adequate specifications and monitored by a qualified geotechnical engineer.

Although Boring B-9 encountered auger refusal at a depth of 36 feet (Elev. 550±), and an offset boring 8 feet from Boring B-9 also encountered auger refusal at the same depth/elevation, bedrock is mapped in the area at Elev. 530±. It is not apparent whether Boring B-9 and the offset boring encountered auger refusal on bedrock that was shallower than typical, or on boulders. It should be noted that cobbles and/or boulders are not uncommon in glacial till soils. If cobbles or boulders are encountered, these conditions could hamper pile-driving operations and possibly damage some piles. If piles are observed to meet refusal at a depth/elevation less than that indicated above, cobble or boulder obstruction may be indicated. For an isolated occurrence, one or more replacement piles could be driven with relatively little additional cost on pile cap re-design. If persistent boulder conditions are indicated, a static pile load test should be performed in accordance with the standard referenced above to evaluate the bearing resistance of the pile(s).

5.4 Trail Subgrades

5.4.1 Existing Subgrade

The subgrades that would result upon the satisfactory completion of the site preparation as described in Section 6.0 of this report are considered generally acceptable for support of the proposed trail pavements. Based on field and laboratory data developed during this investigation, the subgrade soils consist of granular and cohesive alluvial deposits, as well as cohesive lacustrine deposits. Laboratory analyses for Borings B-2 (SS-1) and B-4 (SS-1), as well as visual descriptions of the upper profile indicate that the granular subgrade soils may be generally classified as Group A-3a and A-4a in accordance with the Ohio Department of Transportation (ODOT) system of soil classification. Laboratory analyses for Borings B-3 (SS-1) and B-10

(SS-2), as well as visual descriptions of the upper profile indicate that the cohesive subgrade soils may be generally classified as Group A-4a and A-6a in accordance with the ODOT system of soil classification. The granular soils are considered good to fair as subgrade materials. The cohesive soils are considered fair to poor as subgrade materials because they have relatively low permeabilities and a high percentage of silt and clay particles, which makes them susceptible to moisture, frost penetration, and frost heave. Therefore, the cohesive soils will dictate trail pavement design.

At the time of this investigation, the moisture contents in the upper 5 feet of the cohesive subgrade soils ranged from approximately 18 to 25 percent. These moisture contents are estimated to vary from near to significantly above the expected optimum moisture content for these soils. Moisture contents in the upper 5 feet of the granular subgrade soils ranged from approximately 8 to 14 percent. These moisture contents are estimated to vary from near to slightly above the expected optimum moisture content for these soils. Therefore, remedial action may be needed to adjust the moisture contents of the existing materials and achieve proper compaction of the subgrade. Remedial action should also be anticipated based on the **loose** compactness of the upper profile granular subgrade soils which were encountered in Borings B-2 and B-4, as well as the **soft** consistency of the upper profile cohesive soils encountered in Boring B-10.

5.4.2 Modified Subgrade

Although not anticipated to be prevalent, if soils are dry of optimum, water should be uniformly mixed into the subgrade. More likely to be encountered at this site are soils that are wet of optimum. Where soils wet of optimum are encountered, lowering the moisture content by scarification and aeration (discing and exposure to sun and wind) may be required. However, this may not be feasible if construction occurs during wet seasonal conditions. Very moist to wet soils will "pump" under the operation of heavy equipment, resulting in deep rutting and perhaps rendering the operation of grading and paving equipment difficult or impossible.

Therefore, other methods of subgrade modification may be required in areas of high moisture content. Modification may be achieved by undercutting and replacement with granular subbase (possibly in combination with a geotextile separation layer or geogrid reinforcement), mixing stone into the subgrade, or treating the subgrade with cement or lime. The method of subgrade modification should be determined at the time of construction (See Section 6.2, "Construction Recommendations - Site and Subgrade Preparation").

5.5 Flexible (Asphalt) Pavement

Based on the results of the plasticity and gradation testing for the subgrade soils, we recommend a subgrade CBR value of 6 percent for the Group A-6a or better soils. This CBR value is based on subgrade compacted to at least 100 percent of the maximum dry density as determined by ASTM D 698 (Standard Proctor) or verified as stable through proof rolling.

It should be noted that we are not privy to the design traffic loads or intended design life. The subgrade support recommendations indicated herein should be reviewed by the site engineer in conjunction with the design traffic criteria to determine the required pavement sections. In any case, we recommend the light-duty pavement cross-section consist of at least 3 inches of asphalt underlain by 6 inches of aggregate base for even the lightest-duty pavements based on our experience regarding environmental exposure and reasonable serviceability.

All paving operations should conform to the Ohio Department of Transportation (ODOT) specifications. The pavement and subgrade preparation procedures outlined in this report should result in a reasonably workable and satisfactory pavement. It should be recognized, however, that all flexible pavements need repairs or overlays from time to time as a result of progressive yielding under repeated traffic loads for a prolonged period of time, as well as exposure to freeze-thaw conditions.

5.6 Pavement Drainage

Based on the poorly-drained nature of the cohesive subgrade soils and silty/clayey granular soils, it is anticipated that surface water infiltration may collect in the aggregate base course. Without adequate drainage, water will remain in the base for extended periods of time, creating localized wet, soft pockets. The presence of these pockets will increase the likelihood that pavement distress (cracking, potholes, etc.) will develop. Drainage features may include grading the subgrade surface to slope downward to the outside edge of pavements and/or providing longitudinal edge drains connected to storm sewers or other outlets.

5.7 Groundwater Control and Drainage

Groundwater conditions encountered in the borings performed for this investigation are summarized in Section 4.3. Based on the limited data available, such as the soil characteristics and the moisture conditions encountered in the borings, it is our opinion that the "normal" groundwater level for the portions of the site at higher elevations may be generally encountered at Elevs. 605± to 600±, corresponding to depths on the order of 8 to 16 feet below existing grades. It is our opinion that the "normal" groundwater level for portions of the site at lower elevations may be generally encountered at Elevs. 580± to 575±, corresponding to approximately 5 to 10 feet below existing grades. However, groundwater elevations can fluctuate with seasonal and climatic influences. In particular, "perched" water may be encountered in granular soils that are underlain by relatively impermeable native cohesive soils. Additionally, groundwater levels may be affected by the water level in Swan Creek and other site drainageways. Therefore, the groundwater conditions may vary at different times of the year from those encountered during this investigation.

It is our experience that adequate control of groundwater seepage, perched water, or surface water run-off into shallow excavations should be achievable by minor dewatering systems, such as pumping from prepared sumps. If excavations extend below the groundwater table in granular soils, installation of multiple well points may be required in addition to pumping from prepared sumps.

If construction will be performed in Swan Creek, temporary sheet-pile cutoff walls or cofferdams to direct streamflow may be required to manage groundwater in addition to pumping from prepared sumps. In the event excessive seepage is encountered during construction, TTL should be notified to evaluate whether other dewatering methods are required.

5.8 Excavations and Slopes

The sides of temporary excavations for foundations, utility installations, and other construction should be adequately sloped to provide stable sides and safe working conditions. Otherwise, the excavation must be properly braced against lateral movements. In any case, applicable Occupational Safety and Health Administration (OSHA) safety standards must be followed.

The soils encountered during this investigation, within the anticipated depths of excavation, consist of the following OSHA Type soils:

- OSHA Type A soils (cohesive soils with unconfined compressive strengths of 3,000 pounds per square foot (psf) or greater),
- OSHA Type B soils (cohesive soils with unconfined compressive strengths greater than 1,000 psf but less than 3,000 psf), and
- OSHA Type C soils (granular soils and cohesive soils with unconfined compressive strengths less than 1,000 psf).

For temporary excavations in Type A, B, and C soils, side slopes must be no steeper than ¾ horizontal to 1 vertical (¾H:1V), 1H:1V, and 1½H:1V, respectively. For situations where a higher strength soil is underlain by a lower strength soil and the excavation extends into the lower strength soil, the slope of the entire excavation is governed by that required by the lower strength soil. In all cases, flatter slopes may be required if lower strength soils or adverse seepage conditions are encountered during construction.

For permanent excavations and slopes, we recommend that grades generally be no steeper than 3H:1V. It should be noted that ODOT routinely uses 2H:1V slopes for roadway embankments and spill-through sections. While these steeper slopes may be used, it is our experience that the embankment faces on these slopes are more prone to erosion and sloughing.

6.0 CONSTRUCTION RECOMMENDATIONS

6.1 Sedimentation and Erosion Control

In planning the implementation of earthwork operations, special consideration should be given to provide measures to prevent or reduce soil erosion and the subsequent sedimentation into nearby waterways. These measures may include some or all of the following:

1. Scheduling of earthwork operations such that erodible areas are kept as small as possible and are exposed for the shortest possible time.
2. Using special grading practices, along with diversion or interceptor structures, to reduce the amount of run-off water from an erodible area.
3. Providing vegetative buffer zones, filter berms, or sedimentation basins to trap sediment from surface run-off water.

A specific and detailed soil erosion and sedimentation control program and permits may be required by local, state, or federal regulatory agencies.

6.2 Site and Subgrade Preparation

Prior to proceeding with construction operations, all topsoil, root mat, vegetation, and other deleterious non-soil materials should be removed from the proposed construction areas. Suitable topsoil may be stockpiled for later use in landscape areas. Topsoil thicknesses may vary across the site. Due to the wooded site, areas may be present with topsoil thicker than what was encountered in the borings. Dark soils having the appearance of topsoil, but exhibiting only root "hairs" or trace organics less than approximately five percent, may not require stripping for the full depth of the darkly colored zone, provided the subgrade can be satisfactorily proof rolled as described below. Conversely, the site may contain areas where additional excavation will be required beyond the darkly colored zone due to organics or high moisture in order to provide a stable subgrade for construction. The actual amount of required stripping should be determined in the field by a geotechnical engineer or qualified representative.

Upon completion of stripping and clearing, the areas intended to support new fill and pavements should be carefully inspected by a geotechnical engineer. At that time, the engineer may require proof rolling of the cohesive soil subgrades and silty/clayey sand subgrades utilizing a 20- to 30-ton loaded truck or other pneumatic-tired vehicle of similar size and weight. For granular subgrades containing little silt or clay fraction, proof rolling/compaction of these soils should be performed utilizing a vibratory smooth drum roller. The truck or roller should make a minimum of two passes in each of two perpendicular directions covering the proposed development area, with additional passes as necessary to achieve required compaction and/or subgrade stabilization.

The purpose of proof rolling the cohesive soil subgrades and silty/clayey sand subgrades is to locate any weak, soft, loose, or excessively wet materials that may be present at the time of construction. The purpose of vibratory compaction for the less silty/clayey granular soils is to densify zones of loose materials that are encountered in the upper portion of the soil profile, thereby providing more uniform subgrade support. We recommend a roller with a minimum dead weight on the drums of 8 tons, vibrating at 30 Hz or greater, and traveling at speeds not exceeding approximately 4 feet per second (about 3 miles per hour). These operational criteria should provide sufficient dynamic compaction energy to alleviate loose soil conditions within the zone of influence for subgrade support.

Any unsuitable materials observed during the inspection and proof-rolling operations should be undercut and replaced with compacted fill or stabilized in place utilizing conventional remedial measures such as discing, aeration, and recompaction. Remedial action should be anticipated based on the encountered **loose** native granular soils and **soft** cohesive soils.

Once the site has been proof rolled, inspected, and stabilized, the proof-rolled or inspected subgrades should not be exposed to wet conditions. It should be recognized that during periods of wet weather, the clayey soils that will be exposed at design subgrades will tend to pond water for short periods of time, with the potential to deteriorate the prepared subgrade.

The results of the inspection and proof-rolling operations will be partially dependent on construction operations, the moisture content of the soil, and the weather conditions prevalent at the time. If pumping or rutting is encountered and difficulty is experienced in the operation of construction equipment, TTL should be notified in order to determine which method of subgrade modification may be best suited for the conditions encountered. Should such conditions be experienced, we may recommend that a small test area be used to determine the necessary depth of undercutting and stone replacement or other remedial action necessary to achieve a stable subgrade condition.

6.3 Fill

Material for engineered fill or backfill required to achieve design grades may consist of any non-organic soils having a maximum dry density as determined by the Standard Proctor (ASTM D 698) of 90 pounds per cubic foot (pcf) or greater. On-site soils may be used as engineered fill materials provided that they are free of organic matter, debris, excessive moisture, and rock or stone fragments larger than 3 inches in diameter. Depending on seasonal conditions, the on-site soils may be wet of optimum and may require scarification and aeration to achieve satisfactory compaction. If the construction schedule does not allow for scarification and aeration activities, it may be more practical or economical to utilize imported granular fill.

Fill should be placed in uniform layers no more than 8 inches thick (loose measure) and adequately keyed into stripped and scarified soils. All fill within the structure areas and pavement subgrades should be compacted to not less than 100 percent of the maximum dry density as determined by ASTM D 698 (Standard Proctor).

Based on the borings, the subgrade soils are anticipated to consist of native cohesive and granular soils. The contractor should be prepared to use a sheepsfoot roller to provide effective compaction of the cohesive subgrade soils. For granular engineered fill and encountered granular soils, a vibratory, smooth-drum roller would provide effective compaction of these materials. In narrow utility or footing excavations, the on-site cohesive soils may be difficult to compact; therefore, a clean granular material may be required in these areas.

Scarified subgrade soils and all fill material should be within 3 percent of the optimum moisture content to facilitate compaction. Furthermore, fill material should not be frozen or placed on a frozen base. It is recommended that all earthwork and site preparation activities be conducted under adequate specifications and properly monitored in the field by a qualified geotechnical testing firm.

6.4 Foundation Excavations

As mentioned in Sections 5.1 and 5.2, foundations used to support the structures should have a detailed footing inspection performed for each foundation. A geotechnical engineer or qualified representative should perform these inspections to verify that the exposed materials are similar to those encountered in the borings, unsuitable soils are over-excavated, and that engineered fill has been properly placed and compacted such that it is capable of supporting the design bearing pressure.

We recommend that the foundation excavations be concreted as soon as practical after they are excavated and that water not be allowed to pond in any excavation. If it is necessary to leave the bearing surface open for any extended period of time, we recommend that a thin mat of lean concrete be placed over the bottom of the excavation to reduce damage to the surface from weather or construction. Foundation concrete should not be placed on frozen or saturated subgrade.

Additional foundation subgrade inspection and modification recommendations are provided in Sections 5.1 and 5.2.

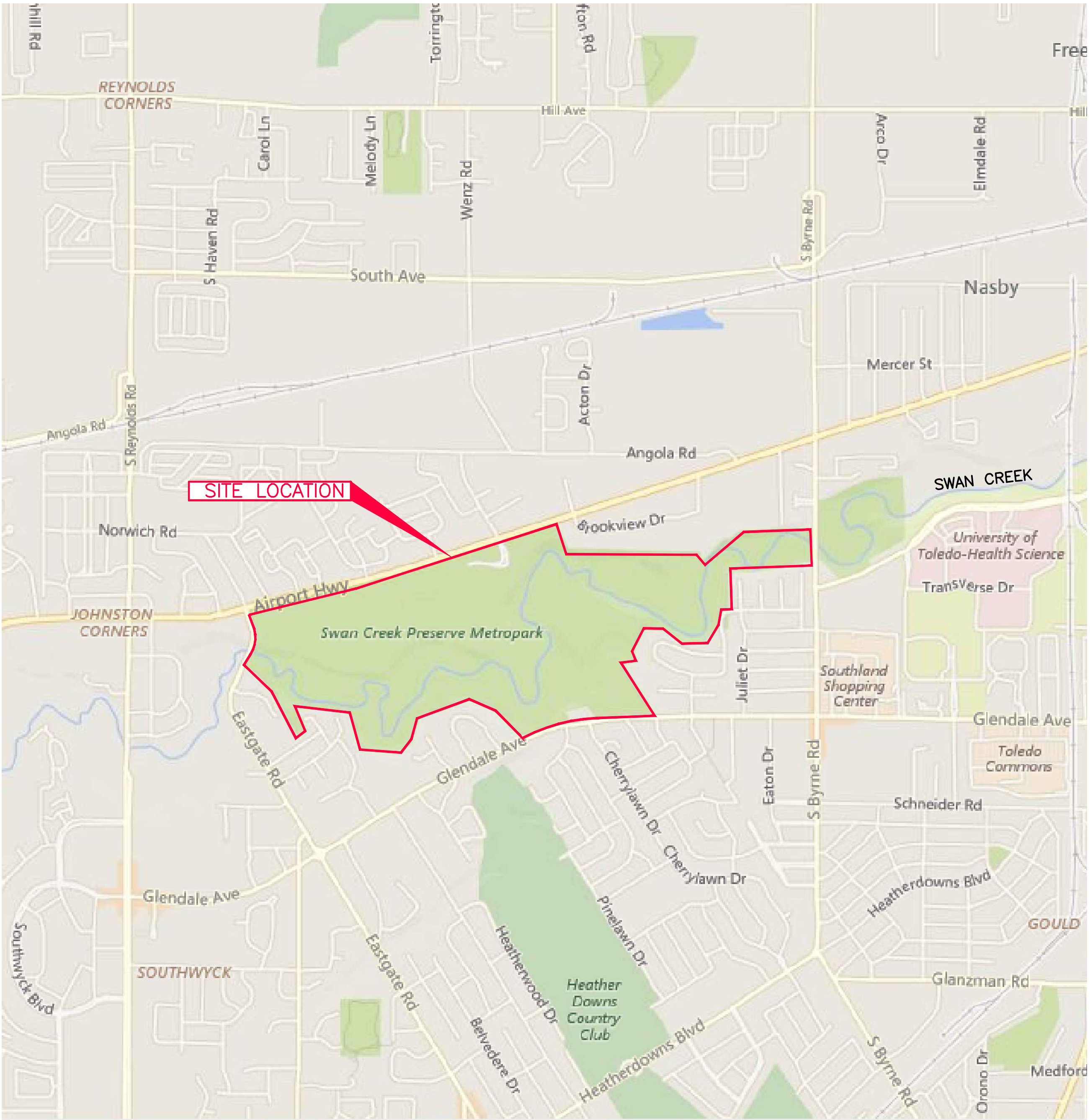
7.0 QUALIFICATION OF RECOMMENDATIONS

Our evaluation of foundation and pavement design and construction conditions has been based on our understanding of the site and project information and the data obtained during our field investigation. The general subsurface conditions were based on interpretation of the subsurface data at specific boring locations. Regardless of the thoroughness of a subsurface investigation, there is the possibility that conditions between borings will differ from those at the boring locations, that conditions are not as anticipated by the designers, or that the construction process has altered the soil conditions. Therefore, experienced geotechnical engineers should observe earthwork and foundation construction to confirm that the conditions anticipated in design are noted. Otherwise, TTL assumes no responsibility for construction compliance with the design concepts, specifications, or recommendations.

The design recommendations in this report have been developed on the basis of the previously described project characteristics and subsurface conditions. If project criteria or locations change, a qualified geotechnical engineer should be permitted to determine whether the recommendations must be modified. The findings of such a review will be presented in a supplemental report.

The nature and extent of variations between the borings may not become evident until the course of construction. If such variations are encountered, it will be necessary to reevaluate the recommendations of this report after on-site observations of the conditions.

Our professional services have been performed, our findings derived, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices. This warranty is in lieu of all other warranties either expressed or implied. TTL is not responsible for the conclusions, opinions, or recommendations of others based on this data.



LEGEND

APPROXIMATE SITE LOCATION

N

APPROXIMATE SCALE – FEET

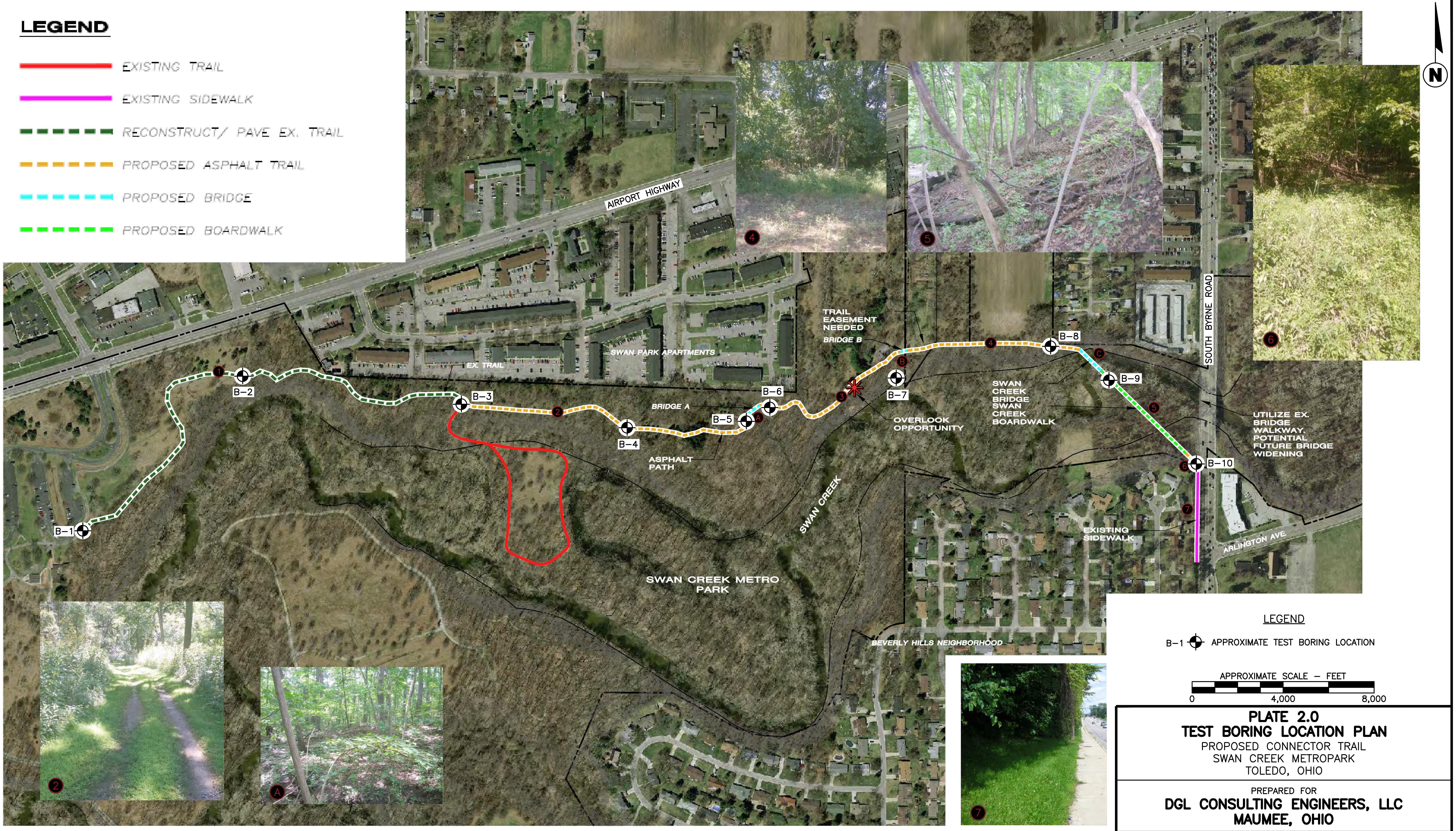
0 2,500 5,000

PLATE 1.0
SITE LOCATION MAP
PROPOSED CONNECTOR TRAIL
SWAN CREEK METROPARK
TOLEDO, OHIO

PREPARED FOR
DGL CONSULTING ENGINEERS, LLC
MAUMEE, OHIO

DRAWN	TRR/11–19–18	CHECKED	CPI/12–3–18
REVISED		APPROVED	
JOB NO.	1726801		
DRAWING NUMBER	**1726801–01G**		

TTL associates inc
Environmental, Geotechnical Engineering & Testing



LEGEND

B-1 APPROXIMATE TEST BORING LOCATION

APPROXIMATE SCALE – FEET

0 4,000 8,000

SWAN CREEK TRAIL

CONCEPTUAL TRAIL PLAN

BASE PLAN "CONCEPTUAL TRAIL PLAN" PROVIDED VIA EMAIL BY DGL CONSULTING ENGINEERS, LLC ON AUGUST 13, 2018.

PLATE 2.0
TEST BORING LOCATION PLAN
PROPOSED CONNECTOR TRAIL
SWAN CREEK METROPARK
TOLEDO, OHIO

PREPARED FOR
DGL CONSULTING ENGINEERS, LLC
MAUMEE, OHIO

DRAWN	TRR/11-19-18	CHECKED	CPI/12-3-18
REVISED		APPROVED	
JOB NO.	1726801		
DRAWING NUMBER	1726801-02G		

TTL associates inc
Environmental, Geotechnical Engineering & Testing

TTL Associates, Inc.
1915 N 12th Street
Toledo, Ohio 43624
Telephone: 419-324-2222
Fax: 419-241-1808

BORING NUMBER B-1

CLIENT DGL Consulting Engineers
PROJECT NAME Proposed Connector Trail
PROJECT NUMBER 1726801
PROJECT LOCATION Toledo, OH
DRILLING CONTRACTOR TTL Associates TB MB
RIG NO. 550
GROUND ELEVATION
DRILLING METHOD 3 in. SSA
GROUND WATER LEVELS:
DATE STARTED 10/18/18 **COMPLETED** 10/18/18
AT TIME OF DRILLING None
LOGGED BY KKC **CHECKED BY** CPI
AT END OF DRILLING None
NOTES
0hrs AFTER DRILLING Backfilled w/Cuttings and Bentonite Chips

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	UNCONF. COMP. STR. (tsf)	DRY UNIT WT. (pcf)	PL / MC / LL (20 40 60 80); ▲ SPT N VALUE ▲ (20 40 60 80)
	0		CRUSHED STONE - 4 Inches (0.3')						
			Moist Medium Dense Brown SILTY SAND (SM) (3.0')	SS 1	100	6-10-9 (19)	NP		9
			Moist Medium Dense Brown POORLY GRADED SAND (SP) (4.5')	SS 2	100	6-5-2 (7)	1.50		23
	5		Moist Medium Stiff to Stiff Gray/Brown SANDY LEAN CLAY (CL) (5.0')						

Bottom of hole at 5.0 feet.

TTL Associates, Inc.
1915 N 12th Street
Toledo, Ohio 43624
Telephone: 419-324-2222
Fax: 419-241-1808

BORING NUMBER B-2

CLIENT DGL Consulting Engineers
PROJECT NAME Proposed Connector Trail
PROJECT NUMBER 1726801
PROJECT LOCATION Toledo, OH
DRILLING CONTRACTOR TTL Associates TB MB
RIG NO. 550
GROUND ELEVATION
DRILLING METHOD 3 in. SSA
GROUND WATER LEVELS:
DATE STARTED 10/18/18 **COMPLETED** 10/18/18
AT TIME OF DRILLING None
LOGGED BY KKC **CHECKED BY** CPI
AT END OF DRILLING None
NOTES
0hrs AFTER DRILLING Backfilled w/Cuttings and Bentonite Chips

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	UNCONF. COMP. STR. (tsf)	DRY UNIT WT. (pcf)	PL / MC / LL (20 40 60 80); ▲ SPT N VALUE ▲ (20 40 60 80)
	0		CRUSHED STONE - 3 Inches 0.3'						
			Moist Loose Brown SILTY SAND (SM)	SS 1	78	3-4-3 (7)	NP		8
	5		5.0'	SS 2	100	4-4-4 (8)	NP		10
			Bottom of hole at 5.0 feet.						

TTL_GEOTECH_STANDARD 1726801.GPJ GINT US LAB.GDT 1/25/19

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BORING NUMBER B-3

CLIENT DGL Consulting Engineers
PROJECT NAME Proposed Connector Trail
PROJECT NUMBER 1726801
PROJECT LOCATION Toledo, OH
DRILLING CONTRACTOR TTL Associates TB MB
RIG NO. 550
GROUND ELEVATION
DRILLING METHOD 3 in. SSA
GROUND WATER LEVELS:
DATE STARTED 10/18/18 **COMPLETED** 10/18/18
AT TIME OF DRILLING None
LOGGED BY KKC **CHECKED BY** CPI
AT END OF DRILLING None
NOTES
0hrs AFTER DRILLING Backfilled w/Cuttings and Bentonite Chips

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	UNCONF. COMP. STR. (tsf)	DRY UNIT WT. (pcf)	PL / MC / LL (20 40 60 80); ▲ SPT N VALUE ▲ (20 40 60 80)
	0		CRUSHED STONE - 2 Inches (0.2')						
			Brown SILTY SAND (SM) (0.6')						
			Moist Stiff to Very Stiff Brown LEAN CLAY w/Trace Sand (CL)	SS 1	100	2-3-6 (9)	2.50		25
	5		(5.0')	SS 2	100	3-5-5 (10)	2.25		25

Bottom of hole at 5.0 feet.

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BORING NUMBER B-4

CLIENT DGL Consulting Engineers
PROJECT NAME Proposed Connector Trail
PROJECT NUMBER 1726801
PROJECT LOCATION Toledo, OH
DRILLING CONTRACTOR TTL Associates TB MB
RIG NO. 550
GROUND ELEVATION
DRILLING METHOD 3 in. SSA
GROUND WATER LEVELS:
DATE STARTED 10/18/18 **COMPLETED** 10/18/18
AT TIME OF DRILLING None
LOGGED BY KKC **CHECKED BY** CPI
AT END OF DRILLING None
NOTES
0hrs AFTER DRILLING Backfilled w/Cuttings and Bentonite Chips

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION		SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	UNCONF. COMP. STR. (tsf)	DRY UNIT WT. (pcf)	PL / MC / LL (20 40 60 80); ▲ SPT N VALUE ▲ (20 40 60 80)
	0		CRUSHED STONE - 5 Inches	0.4'						
			Moist Loose Brown SILTY SAND (SM)		SS 1	89	3-2-3 (5)	NP		13
	5			5.0'	SS 2	100	2-3-4 (7)	NP		14

Bottom of hole at 5.0 feet.

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BORING NUMBER B-5

CLIENT DGL Consulting Engineers
PROJECT NAME Proposed Connector Trail
PROJECT NUMBER 1726801
PROJECT LOCATION Toledo, OH
DRILLING CONTRACTOR TTL Associates TB MB
RIG NO. 550
GROUND ELEVATION 618.81 ft
DRILLING METHOD 3-1/4 in. HSA
GROUND WATER LEVELS:
DATE STARTED 10/18/18 **COMPLETED** 10/18/18
AT TIME OF DRILLING None
LOGGED BY KKC **CHECKED BY** CPI
AT END OF DRILLING None
NOTES
0hrs AFTER DRILLING Backfilled w/Cuttings and Bentonite Chips

ELEVATION (ft)	DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	UNCONF. COMP. STR. (tsf)	DRY UNIT WT. (pcf)	MC (%)
	0	TOPSOIL - 15 Inches (1.3')						
		Moist Very Loose Brown POORLY GRADED SAND w/Silt (SP/SM)	SS 1	78	2-1-2 (3)	NP		4
615		(4.3')	SS 2	100	3-5-6 (11)	3.75		13
	5	Moist Stiff to Very Stiff Brown SANDY LEAN CLAY w/Trace Roots and Root Hairs (CL) (6.0')						
		Moist Stiff to Very Stiff Brown LEAN CLAY w/Sand and Trace Root Hairs (CL) (8.5')	SS 3	100	5-7-8 (15)	>4.5		21
610		Moist Stiff to Very Stiff Brown LEAN CLAY w/Sand (CL)	SS 4	100	5-8-7 (15)	2.50		18
	10	@11': Very Stiff (13.0')	SS 5	100	6-7-9 (16)	3.00		22
605		Moist Very Soft to Medium Stiff Brown/Gray LEAN CLAY w/Sand and Trace Gravel (CL)	SS 6	100	4-3-3 (6)	0.24	93	31
	15	@16': Gray	SS 7	100	3-3-4 (7)	0.29	102	19
600		@18': Soft UU: c = 2.2 psi= 315 psf (21.0')	ST 1	46		UU	113	19
	20	Moist Soft to Medium Stiff Gray LEAN CLAY w/Sand and Trace Gravel (CL)	SS 8	100	2-2-2 (4)	0.50		19
595		@23.5': Medium Stiff	SS 9	100	1-2-3 (5)	0.50		19
	25							

(Continued Next Page)

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BORING NUMBER B-5

CLIENT DGL Consulting Engineers

PROJECT NAME Proposed Connector Trail

PROJECT NUMBER 1726801

PROJECT LOCATION Toledo, OH

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	UNCONF. COMP. STR. (tsf)	DRY UNIT WT. (pcf)	PL / MC / LL; SPT N VALUE (MC)
				SS 10	100	2-2-3 (5)	0.50		19
590	30			SS 11	100	1-2-3 (5)	1.00		18
585	35		33.5' Moist Medium Stiff to Stiff Gray LEAN CLAY w/Sand and Trace Gravel (CL)	SS 12	100	2-3-4 (7)	2.00		20
580	40		37.0' Moist Stiff to Very Stiff Gray LEAN CLAY w/Trace Sand and Gravel (CL)	SS 13	100	2-4-6 (10)	2.50		23
575	45			SS 14	100	4-7-8 (15)	2.91	109	18
570	50		50.0'	SS 15	100	4-6-9 (15)	3.75		20

Bottom of hole at 50.0 feet.

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CLIENT DGL Consulting Engineers
PROJECT NAME Proposed Connector Trail
PROJECT NUMBER 1726801
PROJECT LOCATION Toledo, OH
DRILLING CONTRACTOR TTL Associates TB MB
RIG NO. 550
GROUND ELEVATION 613.54 ft
DRILLING METHOD 3-1/4 in. HSA
GROUND WATER LEVELS:
DATE STARTED 10/19/18 **COMPLETED** 10/19/18
AT TIME OF DRILLING None
LOGGED BY KKC **CHECKED BY** CPI
AT END OF DRILLING None
NOTES
0hrs AFTER DRILLING Backfilled w/Cuttings and Bentonite Chips

MATERIAL DESCRIPTION	Bottom Depth (ft)
TOPSOIL - 8 Inches	0.7'
Moist Medium Dense Brown SILTY SAND (SM)	1.5'
Moist Stiff to Very Stiff Brown LEAN CLAY w/Sand, Trace Calcite Stain Seam and Root Hairs (CL)	3.0'
Moist Very Stiff to Hard Brown LEAN CLAY w/Sand (CL)	6.0'
Moist Very Stiff to Hard Brown LEAN CLAY w/Sand and Trace Gravel (CL)	8.0'
Moist Stiff Gray LEAN CLAY w/Sand and Trace Gravel (CL) @11': Stiff to Very Stiff	13.5'
Moist Medium Stiff Gray LEAN CLAY w/Sand and Trace Gravel (CL)	21.0'
Moist Stiff Gray LEAN CLAY w/Sand and Trace Gravel (CL)	26.0'

SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	UNCONF. COMP. STR. (tsf)	DRY UNIT WT. (pcf)	MC
SS 1	89	5-6-8 (14)	>4.5		18
SS 2	100	9-7-10 (17)	>4.5		20
SS 3	100	10-13-16 (29)	>4.5		17
SS 4	100	5-5-6 (11)	1.72	112	16
SS 5	100	5-7-9 (16)	1.50		16
SS 6	100	2-2-3 (5)	1.00		18
SS 7	100	2-3-3 (6)	0.51	104	18
SS 8	100	1-3-2 (5)	1.25		18
SS 9	100	3-4-5 (9)	1.50		18
ST 1	100				

PL MC LL: 20 40 60 80
▲ SPT N VALUE ▲: 20 40 60 80

Elevation (ft): 610, 605, 600, 595, 590
Depth (ft): 0, 5, 10, 15, 20, 25

(Continued Next Page)

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BORING NUMBER B-6

CLIENT DGL Consulting Engineers
PROJECT NAME Proposed Connector Trail
PROJECT NUMBER 1726801
PROJECT LOCATION Toledo, OH

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	UNCONF. COMP. STR. (tsf)	DRY UNIT WT. (pcf)	PL / MC / LL; SPT N VALUE (20 40 60 80)
			Moist Stiff to Very Stiff Gray LEAN CLAY w/Sand and Trace Gravel (CL)	SS 10	100	3-4-5 (9)	2.00		17
585	30			SS 11	100	2-5-8 (13)	2.50		17
580	35			SS 12	100	4-5-8 (13)	3.09	106	18
575	40			SS 13	100	5-5-8 (13)	3.50		19
570			43.5'						
	45		Moist Very Stiff Gray LEAN CLAY w/Sand and Trace Gravel (CL)	SS 14	100	5-7-9 (16)	4.00		20
			47.0'						
565			Moist Medium Stiff Gray LEAN CLAY w/Sand (CL)						
	50		50.0'	SS 15	100	4-4-4 (8)	0.50		29
			Bottom of hole at 50.0 feet.						

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BORING NUMBER B-7

CLIENT DGL Consulting Engineers
PROJECT NAME Proposed Connector Trail
PROJECT NUMBER 1726801
PROJECT LOCATION Toledo, OH
DRILLING CONTRACTOR TTL Associates TB MB
RIG NO. 550
GROUND ELEVATION
DRILLING METHOD 3-1/4 in. HSA
GROUND WATER LEVELS:
DATE STARTED 10/22/18 **COMPLETED** 10/22/18
AT TIME OF DRILLING 24.0 ft
LOGGED BY KKC **CHECKED BY** CPI
AT END OF DRILLING None
NOTES
0hrs AFTER DRILLING Backfilled w/Cuttings and Bentonite Chips

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	UNCONF. COMP. STR. (tsf)	DRY UNIT WT. (pcf)	PL / MC / LL (20 40 60 80); SPT N VALUE (20 40 60 80)
0 – 0.7'	TOPSOIL - 8 Inches						
0.7' – 3.0'	Moist Soft Brown/Gray LEAN CLAY w/Sand and Trace Organics (CL)	SS 1	89	2-1-2 (3)	3.25		21
3.0' – 6.0'	Moist Loose Brown SILTY SAND w/Trace Organics (SM)	SS 2	33	2-2-3 (5)	NP		8
6.0' – 8.0'	Moist Loose Brown PEAT w/Sand (PT)	SS 3	6	4-4-4 (8)	NP		>>102
8.0' – 13.0'	Moist Stiff to Very Stiff Brown LEAN CLAY w/Sand (CL)	SS 4	100	5-6-8 (14)	>4.5		16
	@11': Very Stiff	SS 5	100	7-9-11 (20)	>4.5		16
13.0' – 24.0'	Moist Stiff to Very Stiff Brown LEAN CLAY w/Sand and Trace Gravel (CL)	SS 6	100	5-5-6 (11)	2.39	101	17
		SS 7	100	6-7-8 (15)	2.57	105	18
	@18': Gray	SS 8	100	4-4-7 (11)	2.00		17
	@21': Stiff	SS 9	100	4-6-7 (13)	1.50		18
24.0' –	Moist Very Stiff Gray SANDY LEAN CLAY w/Trace Gravel (CL)	SS 10	100	6-10-9 (19)	2.00		15

(Continued Next Page)

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BORING NUMBER B-7

CLIENT DGL Consulting Engineers

PROJECT NAME Proposed Connector Trail

PROJECT NUMBER 1726801

PROJECT LOCATION Toledo, OH

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	UNCONF. COMP. STR. (tsf)	DRY UNIT WT. (pcf)	MC (PL–LL; SPT N VALUE 20 40 60 80)
			27.0'	SS 11	100	6-9-13 (22)	2.33	120	14
			Moist Very Stiff Gray LEAN CLAY w/Sand and Trace Gravel (CL)						
	30			SS 12	100	7-10-10 (20)	>4.5		14
	35		@33': Stiff to Very Stiff	SS 13	100	6-6-8 (14)	3.25		20
			38.5'						
	40		Moist Stiff Gray LEAN CLAY w/Sand and Trace Gravel (CL)	SS 14	100	4-6-6 (12)	1.01	104	27
				ST 1	100		1.79	98	26
	45			SS 15	100	3-4-6 (10)	1.75		22
	50		50.0'	SS 16	100	4-4-6 (10)	1.50		30
			Bottom of hole at 50.0 feet.						

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BORING NUMBER B-8

CLIENT DGL Consulting Engineers
PROJECT NAME Proposed Connector Trail
PROJECT NUMBER 1726801
PROJECT LOCATION Toledo, OH
DRILLING CONTRACTOR TTL Associates TB MB
RIG NO. 550
GROUND ELEVATION 619.03 ft
DRILLING METHOD 3-1/4 in. HSA
GROUND WATER LEVELS:
DATE STARTED 10/25/18 **COMPLETED** 10/25/18
AT TIME OF DRILLING None
LOGGED BY KKC **CHECKED BY** CPI
AT END OF DRILLING None
NOTES
0hrs AFTER DRILLING Backfilled w/Cuttings and Bentonite Chips

ELEVATION (ft)	DEPTH (ft)	MATERIAL DESCRIPTION
619.03	0	TOPSOIL - 4 Inches (0.3')
	0.3	Moist Very Stiff Brown LEAN CLAY w/Sand and Trace Calcite Stain Seam (CL) (4.5')
615	4.5	Moist Medium Dense Brown CLAYEY SAND w/Trace Root Hairs (SC) (6.0')
	6.0	Moist Very Stiff Brown LEAN CLAY w/Sand and Trace Calcite Stain Seam (CL)
610		@8.5': Stiff to Very Stiff
	10	
		@11': Very Stiff (13.0')
605	13.0	Moist Stiff Brown/Gray LEAN CLAY w/Sand and Trace Gravel (CL)
	15	
		@16': Gray
600	20	(21.0')
	21.0	Moist Medium Stiff Gray LEAN CLAY w/Sand and Trace Gravel (CL)
595	25	

SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	UNCONF. COMP. STR. (tsf)	DRY UNIT WT. (pcf)	MC (%)
SS 1	89	6-8-8 (16)	>4.5		15
SS 2	100	8-8-8 (16)	NP		8
SS 3	89	8-8-10 (18)	>4.5		18
SS 4	100	4-5-7 (12)	>4.5		22
SS 5	100	5-7-9 (16)	>4.5		21
SS 6	100	3-4-5 (9)	1.50		17
SS 7	100	4-4-5 (9)	1.50		17
ST 1	96		1.12	111	18
SS 8	100	2-3-3 (6)	0.75		18
SS 9	100	2-3-4 (7)	0.65	108	17

PL MC LL: 20 40 60 80
▲ SPT N VALUE ▲: 20 40 60 80

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BORING NUMBER B-8

CLIENT DGL Consulting Engineers

PROJECT NAME Proposed Connector Trail

PROJECT NUMBER 1726801

PROJECT LOCATION Toledo, OH

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	UNCONF. COMP. STR. (tsf)	DRY UNIT WT. (pcf)	PL MC LL (20 40 60 80) / ▲ SPT N VALUE ▲ (20 40 60 80)
				SS 10	100	3-4-4 (8)	0.75		18
590	30			SS 11	100	2-3-3 (6)	1.00		18
585	35			SS 12	100	3-3-5 (8)	1.00		18
			38.5'						
580	40		Moist Stiff Gray LEAN CLAY w/Sand and Trace Gravel (CL)	SS 13	100	2-4-5 (9)	1.50		18
575	45		@43.5': Stiff to Very Stiff	SS 14	100	3-7-8 (15)	2.18	112	18
			@47': Very Stiff						
570	50			SS 15	100	9-9-9 (18)	3.25		19
565	55			SS 16	100	4-7-9 (16)	3.00		24

(Continued Next Page)

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BORING NUMBER B-8

CLIENT DGL Consulting Engineers
PROJECT NAME Proposed Connector Trail
PROJECT NUMBER 1726801
PROJECT LOCATION Toledo, OH

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	UNCONF. COMP. STR. (tsf)	DRY UNIT WT. (pcf)	PL / MC / LL (20 40 60 80); ▲ SPT N VALUE ▲ (20 40 60 80)
			58.5'						
560	60		Moist Stiff Gray LEAN CLAY w/Sand and Trace Gravel (CL)	SS 17	100	3-5-5 (10)	1.25		27
			62.0'						
555	65		Moist Medium Stiff Gray LEAN CLAY w/Sand and Trace Gravel (CL)	SS 18	100	1-2-4 (6)	0.29	110	25
550	70		70.0'	SS 19	100	1-3-4 (7)	1.00		20

Bottom of hole at 70.0 feet.

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BORING NUMBER B-9

CLIENT DGL Consulting Engineers
PROJECT NAME Proposed Connector Trail
PROJECT NUMBER 1726801
PROJECT LOCATION Toledo, OH
DRILLING CONTRACTOR TTL Associates TB MB
RIG NO. 550
GROUND ELEVATION 586.37 ft
DRILLING METHOD 3-1/4 in. HSA
GROUND WATER LEVELS:
DATE STARTED 10/24/18 **COMPLETED** 10/24/18
▽ **AT TIME OF DRILLING** 8.0 ft / Elev 578.4 ft
LOGGED BY KKC **CHECKED BY** CPI
▼ **AT END OF DRILLING** 24.5 ft / Elev 561.9 ft
NOTES Auger refusal (AR) @ 36'. Offset hole AR @36'.
0hrs AFTER DRILLING Backfilled w/Cuttings and Bentonite Chips

ELEVATION (ft)	DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	UNCONF. COMP. STR. (tsf)	DRY UNIT WT. (pcf)	MC (%)
	0	TOPSOIL - 15 Inches (1.3')						
585		Moist Loose Brown SILTY SAND w/Trace Organics (SM) (3.5')	SS 1	78	2-2-3 (5)	NP		15
	5	Moist Loose Brown POORLY GRADED SAND w/Trace Silt (SP) (6.5')	SS 2	100	2-3-3 (6)	NP		10
580		Moist Very Loose Brown SILTY SAND (SM) (8.0')	SS 3	100	3-1-3 (4)	NP		22
	10	Moist Soft Gray SANDY LEAN CLAY w/Trace Gravel and Root Hairs (CL) (11.0')	SS 4	100	3-2-2 (4)	0.25		18
575		Wet Stiff Gray LEAN CLAY w/Sand and Trace Gravel (CL) (Free Water Noted in Jar) (13.0')	SS 5	100	4-5-5 (10)	NI		22
	15	Moist Stiff Gray LEAN CLAY w/Sand and Trace Gravel (CL)	SS 6	100	3-5-7 (12)	1.70	118	19
570		@16': Very Stiff	SS 7	100	5-10-15 (25)	2.75		23
	20	@18.5': Stiff	SS 8	100	4-5-6 (11)	1.13	112	19
565		@21': w/Trace Sand	SS 9	100	5-7-7 (14)	1.00		24
	25		SS 10	100	4-5-6 (11)	2.00		24

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BORING NUMBER B-9

CLIENT DGL Consulting Engineers **PROJECT NAME** Proposed Connector Trail

PROJECT NUMBER 1726801 **PROJECT LOCATION** Toledo, OH

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	UNCONF. COMP. STR. (tsf)	DRY UNIT WT. (pcf)	PL / MC / LL (20 40 60 80); ▲ SPT N VALUE ▲ (20 40 60 80)
560			28.5'	ST 1	100		1.01	95	28
	30		Moist Medium Stiff to Stiff Gray LEAN CLAY w/Sand (CL)	SS 11	100	3-4-4 (8)	1.25		28
555			34.0'						
	35		Moist Very Stiff Gray SANDY SILT w/Trace Gravel (ML) 36.0'	SS 12	89	4-9-10 (19)	1.50		11

Bottom of hole at 36.0 feet.

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BORING NUMBER B-10

CLIENT DGL Consulting Engineers
PROJECT NAME Proposed Connector Trail
PROJECT NUMBER 1726801
PROJECT LOCATION Toledo, OH
DRILLING CONTRACTOR TTL Associates TB MB
RIG NO. 550
GROUND ELEVATION
DRILLING METHOD 3-1/4 in. HSA
GROUND WATER LEVELS:
DATE STARTED 10/24/18 **COMPLETED** 10/24/18
AT TIME OF DRILLING 8.0 ft
LOGGED BY KKC **CHECKED BY** CPI
AT END OF DRILLING 8.0 ft
NOTES
0hrs AFTER DRILLING Backfilled w/Cuttings and Bentonite Chips

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	UNCONF. COMP. STR. (tsf)	DRY UNIT WT. (pcf)	PL / MC / LL (20 40 60 80); ▲ SPT N VALUE ▲ (20 40 60 80)
	0		TOPSOIL - 13 Inches (1.1')						
			Moist Medium Stiff Brown LEAN CLAY w/Sand and Trace Organics (CL) (3.0')	SS 1	78	3-2-3 (5)	3.00		18
	5		Moist Soft Brown SANDY SILT (ML)	SS 2	100	2-2-2 (4)	0.25		23
			(8.0')	SS 3	100	1-2-2 (4)	0.25		24
	10		Wet Very Loose Gray POORLY GRADED SAND w/Trace Silt (SP) (Free Water Noted in Jar) (10.0')	SS 4	72	1-2-2 (4)	NP		34

Bottom of hole at 10.0 feet.

LEGEND KEY

Unified Soil Classification System Soil Symbols

GW - WELL GRADED GRAVEL Includes Gravel-Sand mixtures, little or no fines.

GP - POORLY GRADED GRAVEL Includes Gravel-Sand mixtures, little or no fines.

GM - SILTY GRAVEL Includes Gravel-Sand-Silt mixtures.

GC - CLAYEY GRAVEL Includes Gravel-Sand-Clay mixtures.

SW - WELL GRADED SAND Includes Gravelly Sands, little or no fines.

SP - POORLY GRADED SAND Includes Gravelly Sands, little or no fines.

SM - SILTY SAND Includes Sand-Silt mixtures.

SC - CLAYEY SAND Includes Sand-Clay mixtures.

ML - SILT Includes Silt with Sand and Sandy Silt.

CL - LEAN CLAY Includes Sandy Lean Clay and Lean Clay with Sand and Gravel.

MH - ELASTIC SILT Includes Sandy Elastic Silt and Elastic Silt with Sand.

CH - FAT CLAY Includes Sandy Fat Clay and Fat Clay with Sand.

CL-ML - SILTY CLAY Includes Clayey Silt of low plasticity.

OL - ORGANIC SILT and ORGANIC CLAY of low plasticity.

OH - ORGANIC SILT and ORGANIC CLAY of medium to high plasticity.

Pt - PEAT Includes humus, swamp and other soils with high organic content.

FILL MATERIAL - Includes controlled and non-controlled soil and non-soil materials.

TOPSOIL

ASPHALT - Bituminous Asphalt

CONCRETE - Includes broken concrete rubble.

Shale

Weathered Shale

Sandstone

Weathered Sandstone

Sample Symbols

SS - Split Spoon

ST - Shelby Tube

RC - Rock Core

GS - Geoprobe Sleeve

AU - Auger Cuttings

GB - Grab

Notes:

1. Exploratory borings were drilled during the period from October 18 through 25, 2018, using 3¼-inch inside diameter hollow-stem augers.

2. These logs are subject to the limitations, conclusions, and recommendations in the report and should not be interpreted separate from the report.

3. Borings B-1 through B-4, B-7, and B-10 were located in the field by TTL Associates, Inc. (TTL) based on coordination with DGL Consulting Engineers, LLC (DGL) and site reconnaissance by DGL and TTL. The remaining borings were staked in the field by DGL. Borings B-5, B-6, B-8, and B-9 were surveyed by DGL.

4. Unconfined Compressive Strength (tsf):
   NI = Not Intact
   NP = Non-Plastic
   UU = Unconsolidated-Undrained Triaxial Compressive Strength Test per ASTM D 2850

PROJECT: Proposed Connector Trail, Toledo, Ohio	TTL Associates, Inc.	PROJECT NO: 1726801

TABULATION OF TEST DATA

Boring Number	Sample Number	Sample Interval Depth (Feet)		Standard Penetration (Blows per Foot)	Natural Moisture Content (% of Dry Weight)	In-Place Dry Density (Pounds per Cubic Foot)	Unconfined Compressive Strength (Pounds per Square Foot)		Particle Size Distribution (%)						Atterberg Limits (%)			Unified Soil Classification
									Gravel	Coarse Sand	Medium Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index	
B-1	SS-1	1.0-2.5		19	8.7													
	SS-2	3.5-5.0		7	22.5		*3,000											
B-2	SS-1	1.0-2.5		7	8.0				0	0	8	74	16	2	NON-PLASTIC			SM
	SS-2	3.5-5.0		8	9.7													
B-3	SS-1	1.0-2.5		9	25.4		*5,000		0	0	3	7	22	68	36	22	14	CL
	SS-2	3.5-5.0		10	25.3		*4,500											
B-4	SS-1	1.0-2.5		5	12.6				0	0	3	56	37	4	19	16	3	SM
	SS-2	3.5-5.0		7	13.6													

*Unconfined compressive strength derived from a calibrated hand penetrometer

PROJECT: Proposed Connector Trail, Toledo, Ohio | **TTL Associates, Inc.** | PROJECT NO: 1726801

TABULATION OF TEST DATA

Boring Number	Sample Number	Sample Interval Depth (Feet)		Standard Penetration (Blows per Foot)	Natural Moisture Content (% of Dry Weight)	In-Place Dry Density (Pounds per Cubic Foot)	Unconfined Compressive Strength (Pounds per Square Foot)		Particle Size Distribution (%)						Atterberg Limits (%)			Unified Soil Classification
									Gravel	Coarse Sand	Medium Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index	
B-5	SS-1	1.0-2.5		3	3.9													
	SS-2	3.5-5.0		11	13.4		*7,500											
	SS-3	6.0-7.5		15	21.1		*9,000+											
	SS-4	8.5-10.0		15	18.3		*5,000											
	SS-5	11.0-12.5		16	22.1		*6,000											
	SS-6	13.5-15.0		6	31.1	92.6	480											
	SS-7	16.0-17.5		7	18.6	102.4	585											
	ST-1	18.0-20.0			19.1	112.9	UU		1	5	6	14	22	52	27	17	10	CL
	SS-8	21.0-22.5		4	19.2		*1,000											
	SS-9	23.5-25.0		5	18.6		*1,000											
	SS-10	26.0-27.5		5	18.6		*1,000											
	SS-11	28.5-30.0		5	17.9		*2,000											
	SS-12	33.5-35.0		7	19.8		*4,000											
	SS-13	38.5-40.0		10	23.1		*5,000		1	1	2	5	24	67	30	19	11	CL
	SS-14	43.5-45.0		15	18.0	108.7	5,825											
	SS-15	48.5-50.0		15	20.1		*7,500											

*Unconfined compressive strength derived from a calibrated hand penetrometer

PROJECT: Proposed Connector Trail, Toledo, Ohio | **TTL Associates, Inc.** | PROJECT NO: 1726801

TABULATION OF TEST DATA

Boring Number	Sample Number	Sample Interval Depth (Feet)		Standard Penetration (Blows per Foot)	Natural Moisture Content (% of Dry Weight)	In-Place Dry Density (Pounds per Cubic Foot)	Unconfined Compressive Strength (Pounds per Square Foot)		Particle Size Distribution (%)						Atterberg Limits (%)			Unified Soil Classification
									Gravel	Coarse Sand	Medium Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index	
B-6	SS-1	1.0-2.5		14	18.0		*9,000+											
	SS-2	3.5-5.0		17	20.3		*9,000+											
	SS-3	6.0-7.5		29	16.8		*9,000+											
	SS-4	8.5-10.0		11	15.8	111.7	3,445											
	SS-5	11.0-12.5		16	16.3		*3,000											
	SS-6	13.5-15.0		5	17.5		*2,000											
	SS-7	16.0-17.5		6	17.7	104.1	1,025											
	SS-8	18.5-20.0		5	17.7		*2,500											
	SS-9	21.0-22.5		9	17.8		*3,000											
	ST-1	23.0-25.0																
	SS-10	26.0-27.5		9	17.4		*4,000											
	SS-11	28.5-30.0		13	17.1		*5,000											
	SS-12	33.5-35.0		13	18.5	106.4	6,180											
	SS-13	38.5-40.0		13	18.9		*7,000											
	SS-14	43.5-45.0		16	20.4		*8,000											
	SS-15	48.5-50.0		8	28.5		*1,000											

*Unconfined compressive strength derived from a calibrated hand penetrometer

PROJECT: Proposed Connector Trail, Toledo, Ohio | **TTL Associates, Inc.** | PROJECT NO: 1726801

TABULATION OF TEST DATA

Boring Number	Sample Number	Sample Interval Depth (Feet)		Standard Penetration (Blows per Foot)	Natural Moisture Content (% of Dry Weight)	In-Place Dry Density (Pounds per Cubic Foot)	Unconfined Compressive Strength (Pounds per Square Foot)		Particle Size Distribution (%)						Atterberg Limits (%)			Unified Soil Classification
									Gravel	Coarse Sand	Medium Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index	
B-7	SS-1	1.0-2.5		3	20.5		*6,500											
	SS-2	3.5-5.0		5	7.7													
	SS-3	6.0-7.5		8	101.9													
	SS-4	8.5-10.0		14	15.7		*9,000+											
	SS-5	11.0-12.5		20	16.4		*9,000+											
	SS-6	13.5-15.0		11	16.9	101.1	4,775											
	SS-7	16.0-17.5		15	17.9	105.4	5,145											
	SS-8	18.5-20.0		11	16.8		*4,000											
	SS-9	21.0-22.5		13	17.6		*3,000											
	SS-10	23.5-25.0		19	15.1		*4,000											
	SS-11	26.0-27.5		22	13.8	119.7	4,660											
	SS-12	28.5-30.0		20	13.6		*9,000+											
	SS-13	33.5-35.0		14	19.6		*6,500											
	SS-14	38.5-40.0		12	26.9	104.3	2,020											
	ST-1	41.0-43.0			26.1	97.5	3,585											
	SS-15	43.5-45.0		10	22.1		*3,500											
	SS-16	48.5-50.0		10	30.1		*3,000											

*Unconfined compressive strength derived from a calibrated hand penetrometer

PROJECT: Proposed Connector Trail, Toledo, Ohio | **TTL Associates, Inc.** | PROJECT NO: 1726801

TABULATION OF TEST DATA

Boring Number	Sample Number	Sample Interval Depth (Feet)		Standard Penetration (Blows per Foot)	Natural Moisture Content (% of Dry Weight)	In-Place Dry Density (Pounds per Cubic Foot)	Unconfined Compressive Strength (Pounds per Square Foot)		Particle Size Distribution (%)						Atterberg Limits (%)			Unified Soil Classification
									Gravel	Coarse Sand	Medium Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index	
B-8	SS-1	1.0-2.5		16	14.7		*9,000+											
	SS-2	3.5-5.0		16	8.1		*7,000											
	SS-3	6.0-7.5		18	17.6		*9,000+											
	SS-4	8.5-10.0		12	22.4		*9,000+											
	SS-5	11.0-12.5		16	20.8		*9,000+											
	SS-6	13.5-15.0		9	16.8		*3,000											
	SS-7	16.0-17.5		9	17.4		*3,000											
	ST-1	18.0-20.0			17.9	110.9	2,235											
	SS-8	21.0-22.5		6	18.1		*1,500											
	SS-9	23.5-25.0		7	17.4	108.1	1,310											
	SS-10	26.0-27.5		8	18.1		*1,500											
	SS-11	28.5-30.0		6	17.6		*2,000											
	SS-12	33.5-35.0		8	18.3		*2,000											
	SS-13	38.5-40.0		9	18.1		*3,000											
	SS-14	43.5-45.0		15	17.7	111.8	4,355											
	SS-15	48.5-50.0		18	19.4		*6,500											
	SS-16	53.5-55.0		16	24.5		*6,000											

*Unconfined compressive strength derived from a calibrated hand penetrometer

PROJECT: Proposed Connector Trail, Toledo, Ohio	TTL Associates, Inc.	PROJECT NO: 1726801

TABULATION OF TEST DATA

Boring Number	Sample Number	Sample Interval Depth (Feet)		Standard Penetration (Blows per Foot)	Natural Moisture Content (% of Dry Weight)	In-Place Dry Density (Pounds per Cubic Foot)	Unconfined Compressive Strength (Pounds per Square Foot)		Particle Size Distribution (%)						Atterberg Limits (%)			Unified Soil Classification
									Gravel	Coarse Sand	Medium Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index	
B-8	SS-17	58.5-60.0		10	27.2		*2,500											
	SS-18	63.5-65.0		6	24.8	109.7	580											
	SS-19	68.5-70.0		7	20.4		*2,000											
B-9	SS-1	1.0-2.5		5	15.2													
	SS-2	3.5-5.0		6	9.6													
	SS-3	6.0-7.5		4	21.6													
	SS-4	8.5-10.0		4	18.5		*500											
	SS-5	11.0-12.5		10	21.6													
	SS-6	13.5-15.0		12	18.8	117.9	3,395											
	SS-7	16.0-17.5		25	22.9		*5,500											
	SS-8	18.5-20.0		11	18.8	111.9	2,260											
	SS-9	21.0-22.5		14	24.2		*2,000		2	1	2	6	23	66	29	18	11	CL
	SS-10	23.5-25.0		11	24.4		*4,000											
	ST-1	26.0-28.0			27.8	95.1	2,015											
	SS-11	28.5-30.0		8	28.0		*2,500											
	SS-12	33.5-35.0		19	11.3		*3,000											

*Unconfined compressive strength derived from a calibrated hand penetrometer

PROJECT: Proposed Connector Trail, Toledo, Ohio | **TTL Associates, Inc.** | PROJECT NO: 1726801

TABULATION OF TEST DATA

Boring Number	Sample Number	Sample Interval Depth (Feet)		Standard Penetration (Blows per Foot)	Natural Moisture Content (% of Dry Weight)	In-Place Dry Density (Pounds per Cubic Foot)	Unconfined Compressive Strength (Pounds per Square Foot)		Particle Size Distribution (%)						Atterberg Limits (%)			Unified Soil Classification
									Gravel	Coarse Sand	Medium Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index	
B-10	SS-1	1.0-2.5		5	18.3		*6,000											
	SS-2	3.5-5.0		4	22.9		*500		0	0	5	33	44	18	25	22	3	ML
	SS-3	6.0-7.5		4	23.9		*500											
	SS-4	8.5-10.0		4	34.1													

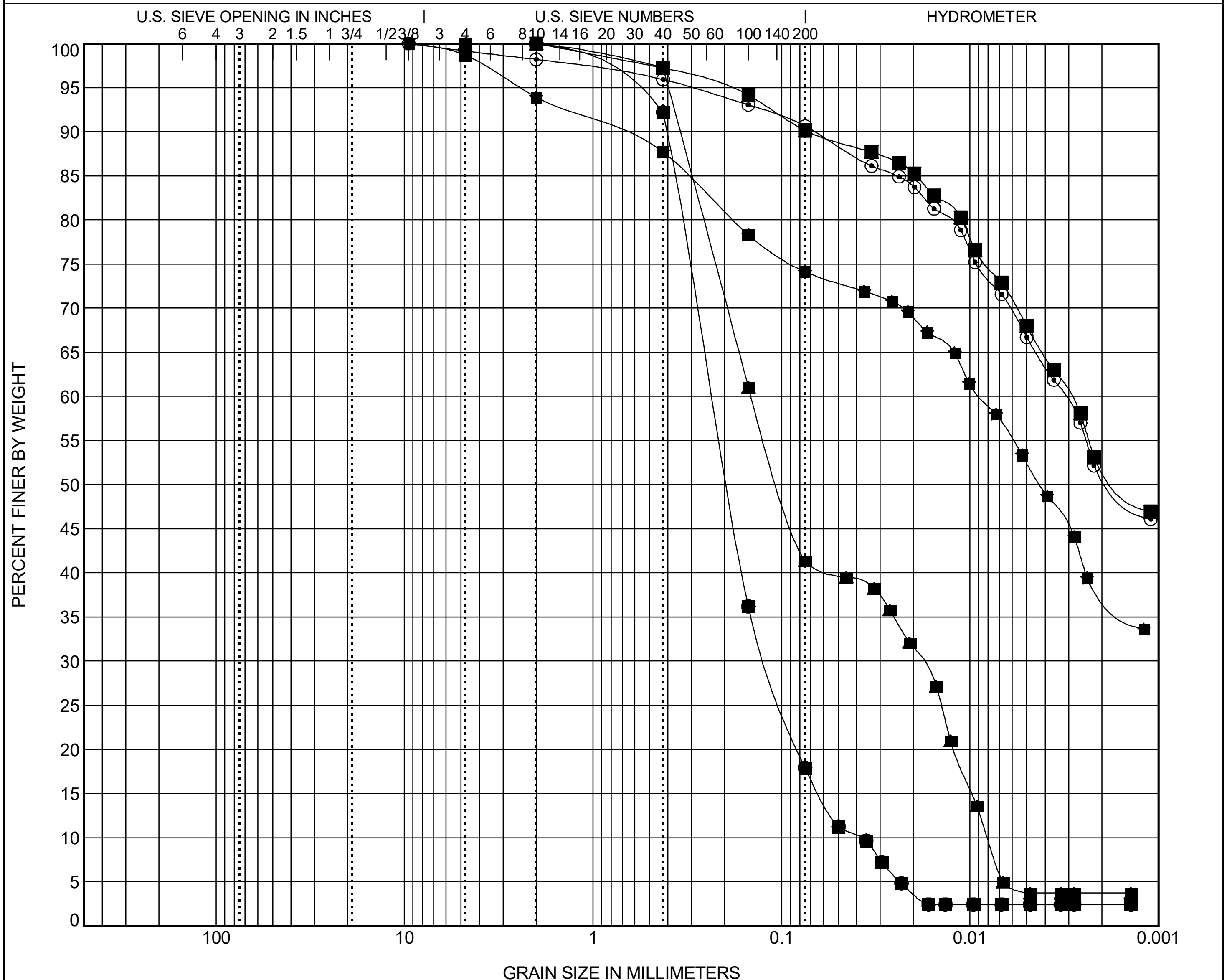
*Unconfined compressive strength derived from a calibrated hand penetrometer

TTL Associates, Inc.
1915 N 12th Street
Toledo, Ohio 43624
Telephone: 419-324-2222
Fax: 419-241-1808

GRAIN SIZE DISTRIBUTION

CLIENT DGL Consulting Engineers **PROJECT NAME** Proposed Connector Trail

PROJECT NUMBER 1726801 **PROJECT LOCATION** Toledo, OH



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification		USCS Classification	LL	PL	PI	Cc	Cu
■ B-2	1.0	**SILTY SAND (SM)**	**NP**	**NP**	**NP**	**1.6**	**6.1**
■ B-3	1.0	**LEAN CLAY (CL)**	**36**	**22**	**14**		
■ B-4	1.0	**SILTY SAND (SM)**	**19**	**16**	**3**	**0.3**	**17.9**
■ B-5	18.0	**LEAN CLAY with SAND (CL)**	**27**	**17**	**10**		
⊙ B-5	38.5	**LEAN CLAY (CL)**	**30**	**19**	**11**		

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
■ B-2	1.0	**2**	**0.234**	**0.119**	**0.038**	**0.0**	**82.1**	**15.5**	**2.4**
■ B-3	1.0	**2**	**0.003**			**0.0**	**9.8**	**22.2**	**67.9**
■ B-4	1.0	**4.75**	**0.145**	**0.018**	**0.008**	**0.0**	**58.6**	**37.5**	**3.9**
■ B-5	18.0	**9.5**	**0.009**			**1.3**	**24.6**	**21.6**	**52.5**
⊙ B-5	38.5	**9.5**	**0.003**			**0.8**	**8.5**	**23.9**	**66.7**

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PROJECT NAME Proposed Connector Trail
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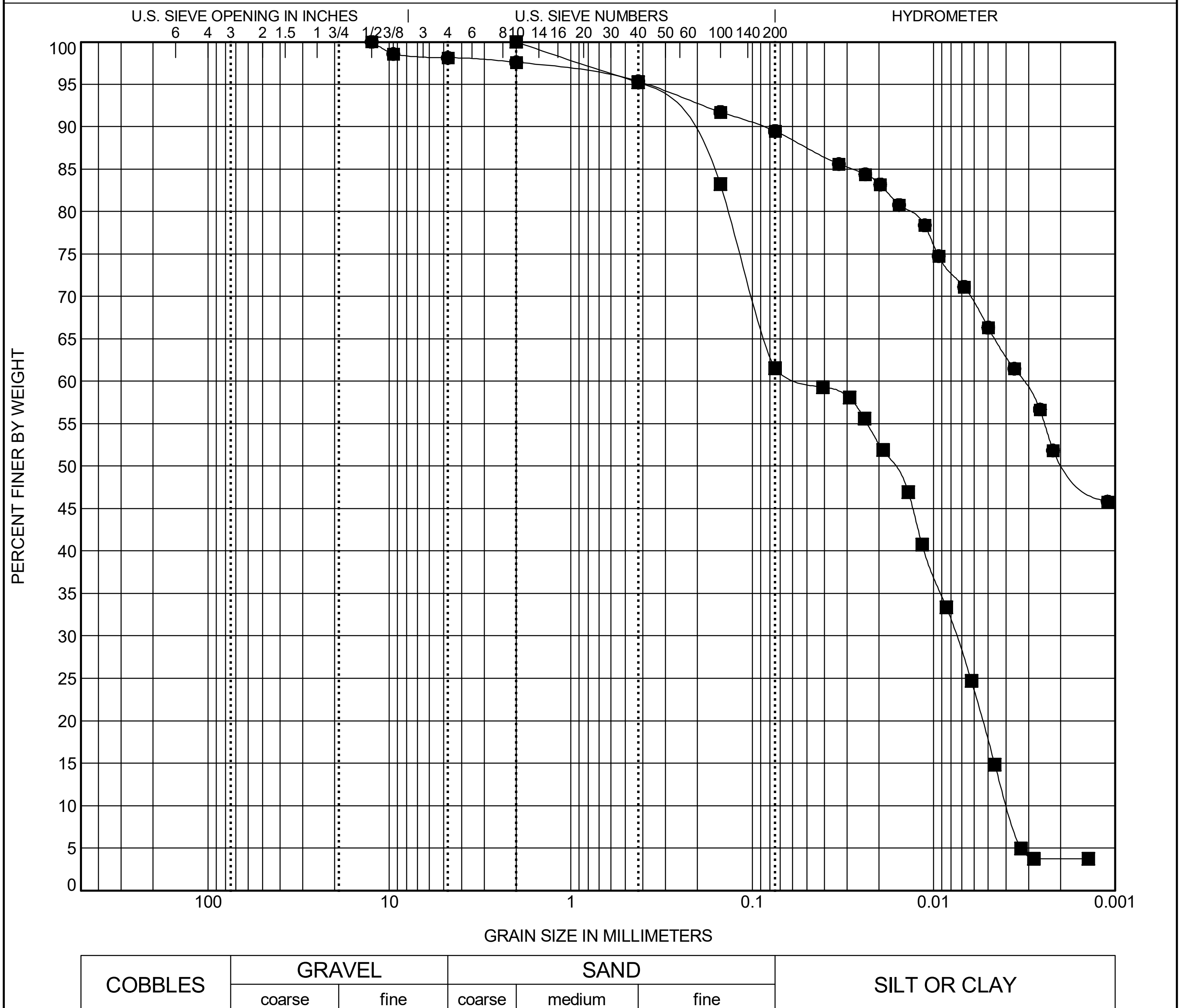


Specimen Identification		USCS Classification	LL	PL	PI	Cc	Cu
■ B-9	21.0	**LEAN CLAY (CL)**	29	18	11		
■ B-10	3.5	**SANDY SILT (ML)**	25	22	3	0.3	12.6

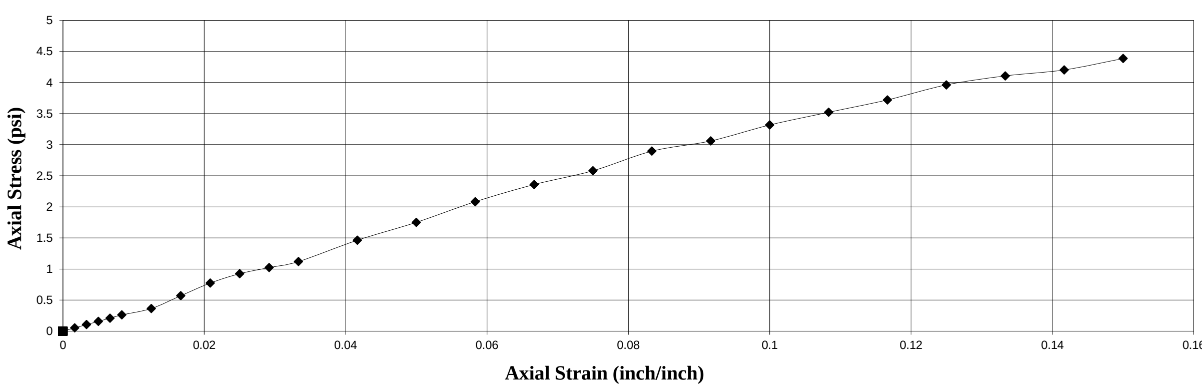
Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
■ B-9	21.0	12.5	0.003			1.9	8.6	23.2	66.3
■ B-10	3.5	2	0.049	0.008	0.004	0.0	38.5	44.0	17.6

Unconsolidated - Undrained Triaxial Shear Strength Test

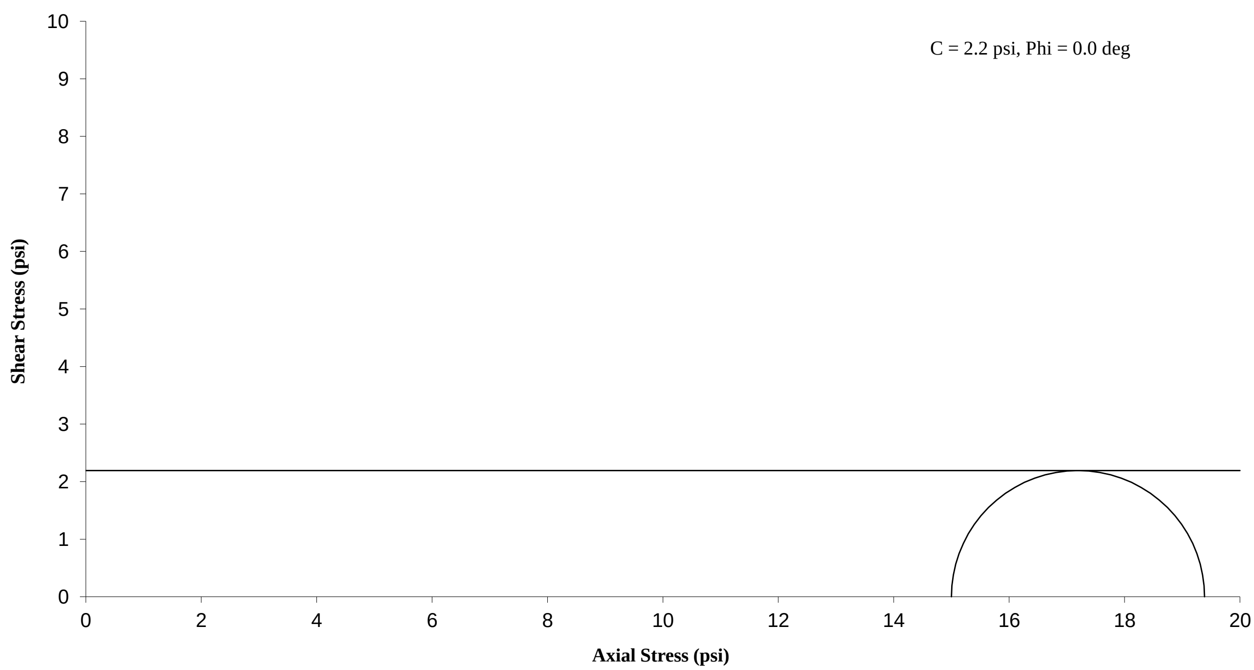
ASTM D 2850

General Sample Data		Triaxial Specimen Data			
TTL Project No.:	17268 01	Symbol	◆	■	●
Project:	Proposed Connector Trail	Init. Specimen Height (in.)	6.00	-	-
Sample ID:	B-5 ST-1	Init. Specimen Diameter (in.)	2.88	-	-
Sample Interval:	18.0 - 20.0'	Init. Moisture Content* (%)	19.1	-	-
Soil Description:	Brown/Gray LEAN CLAY w/Sand and Trace Gravel (CL)	Init. Dry Unit Weight (pcf)	113.1	-	-
Liquid Limit:	27	Init. Void Ratio	0.52	-	-
Plastic Limit:	17	Init. Degree of Saturation (%)	101	-	-
Plasticity Index:	10	Minor Principal Stress (psi)	15.0	-	-
Specific Gravity:	2.75 (Assumed)	Deviator Stress at Failure (psi)	4.4	-	-
Rate of Strain:	0.03 Inches per Minute	Major Principal Stress (psi)	19.4	-	-
Failure Criteria:	Peak Deviator Stress or Deviator Stress at 15% Axial Strain	Axial Strain at Failure (%)	15.0	-	-

Stress/Strain



Mohr Circle Plot



UNCONSOLIDATED, UNDRAINED COMPRESSIVE STRENGTH OF COHESIVE SOILS IN TRIAXIAL COMPRESSION (ASTM D 2850)

Project:	Proposed Connector Trail	Date:	10/30/2018
Client:	DGL Consulting Engineers	File:	1726801B-5ST-1
Sample ID	B-5 ST-1	Depth:	18.0 - 20.0'
TTL Project No.	17268 01	Specimen ID:	"B" (18.5 - 19.0 Feet)

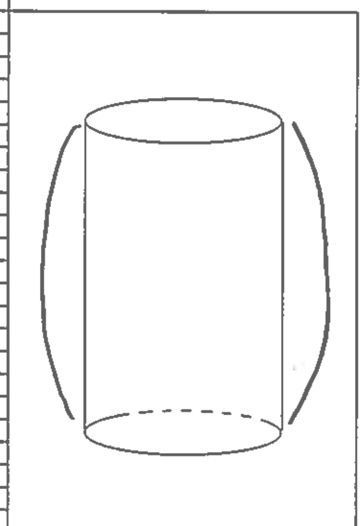
SAMPLE PROPERTIES

Visual Description: Brown/Gray LEAN CLAY w/Sand and Trace Gravel (CL)

Diameter:	2.88 in.	Initial Dry Unit Weight of Sample:	113.1 pcf
Area:	6.514 in^2	Initial Moisture Content:	19.1 %
Length:	6.00 in	Specific Gravity (assumed):	2.75
Initial Void Ratio:	0.52	Initial Degree of Saturation:	101 %
Chamber Pressure:	15 psi	Proving Ring Number	1155-12-13322

STRESS-STRAIN DATA

Speciman Deformation (in)	Vertical Strain	Proving Ring Reading	Piston Load (lbs)	Corrected Area (in^2)	Deviator Stress (psi)
0.000	0.000	0.0	0.0	6.514	0.0
0.010	0.002	0.5	0.3	6.525	0.1
0.020	0.003	1.0	0.7	6.536	0.1
0.030	0.005	1.5	1.0	6.547	0.2
0.040	0.007	2.0	1.4	6.558	0.2
0.050	0.008	2.5	1.7	6.569	0.3
0.075	0.013	3.5	2.4	6.597	0.4
0.100	0.017	5.5	3.8	6.625	0.6
0.125	0.021	7.5	5.1	6.653	0.8
0.150	0.025	9.0	6.2	6.681	0.9
0.175	0.029	10.0	6.9	6.710	1.0
0.200	0.033	11.0	7.5	6.739	1.1
0.250	0.042	14.5	9.9	6.798	1.5
0.300	0.050	17.5	12.0	6.857	1.8
0.350	0.058	21.0	14.4	6.918	2.1
0.400	0.067	24.0	16.5	6.980	2.4
0.450	0.075	26.5	18.2	7.043	2.6
0.500	0.083	30.0	20.6	7.107	2.9
0.550	0.092	32.0	22.0	7.172	3.1
0.600	0.100	35.0	24.0	7.238	3.3
0.650	0.108	37.5	25.7	7.306	3.5
0.700	0.117	40.0	27.4	7.375	3.7
0.750	0.125	43.0	29.5	7.445	4.0
0.800	0.133	45.0	30.9	7.517	4.1
0.850	0.142	46.5	31.9	7.590	4.2
0.900	0.150	49.0	33.6	7.664	4.4



Sketch of Tested Specimen

RESULTS

Maximum Deviator Stress 4.4 psi

Consolidation Laboratory Calculations

Consolidometer:	**1**
Method:	ASTM D 2435 Method B
Project No. :	17268 01
Client:	DGL Consulting Engineers
Project:	Proposed Connector Trail
Location:	Toledo, OH
Boring No. :	B-5
Sample No.:	ST-1
Depth:	18.0 - 20.0'
Date of Test:	10/29/2018

Visual Description:	Gray LEAN CLAY with Sand and Trace Gravel (CL)
Liquid Limit:	27 %
Plastic Limit:	17 %
Plasticity Index:	10 %

Initial Sample Data			***Final Sample Data***		
Initial Height	1.006 in.		Final Height	0.920 in.	
Ring Dia.	2.493 in.		Ring Dia.	2.493 in.	
Area of Ring	4.8813 in^2		Area of Ring	4.8813 in^2	
Initial Volume	4.9106 in^3	0.00284 ft^3	Final Volume	4.4898 in^3	0.00260 ft^3
Specific Gravity	2.742				
			Final wet mass soil, pan & ring	365.6 g	
			Wt of Pan	51.3 g	
Initial wet mass soil & ring	319.4 g		Final wet mass soil & ring	314.3	
Mass of ring	146.4 g		Mass of ring	146.4 g	
			Final dry mass of soil, pan & ring	341.8 g	
Initial wet mass soil	173 g	0.38140 lb	Final wet mass soil	167.9 g	0.37016 lb
			Weight of water	23.8 g	0.05247 lb
Initial Water Content					
Mass can & wet soil	347.4 g				
Mass can & dry soil	300.5 g				
Mass of can	51.5 g				
Mass of water	46.9 g				
Mass of soil	249 g				
Initial water content	18.84 % (trimmings)				
Initial water content	20.06 % (based on final dry weight)		Final water content	16.52 % (based on final dry weight)	
			Final weight of solids (Md)	144.1 g	0.31769 lb
Initial dry density	111.8 pcf		Final dry density	122.3 pcf	
			Final volume of solids (Vs)	3.2069 in^3	0.00186 ft^3
			Final height of solids (Hs)	0.6570 in.	
Initial void ratio (eo)	0.531		Final void ratio (ef)	0.400	
Initial volume of voids (Vvo)	1.7037 in^3	0.00099 ft^3	Final volume of voids (Vvf)	1.2829 in^3	0.00074 ft^3
Initial volume of water (Vwo)	1.7635 in^3	0.00102 ft^3	Final volume of water (Vwf)	1.4523 in^3	0.00084 ft^3
Initial degree of saturation (So)	103.51 %		Final degree of saturation (Sf)	113.20 %	

Checks:
Final DD >= Initial DD TRUE

1726801B-5ST-1

Project No.: 17268 01
Date: 10/29/2018
Client: DGL Consulting Engineers
Project: Proposed Connector Trail
Toledo, OH
Boring No.: B-5
Sample No.: ST-1
Depth: 18.0 - 20.0'

Initial H= 1.006 inches

Pressure tsf	Final Height (in)	Initial Height (in)	DH	Average H (in)	e
0.25	0.98815	1.00600	0.01785	0.9971	0.504
0.5	0.97790	0.98815	0.02810	0.9830	0.488
1	0.96390	0.97790	0.04210	0.9709	0.467
2	0.95000	0.96390	0.05600	0.9570	0.446
4	0.92960	0.95000	0.07640	0.9398	0.415
8	0.91090	0.92960	0.09510	0.9203	0.387
16	0.88750	0.91090	0.11850	0.8992	0.351
4	0.89255	0.88750	0.11345	0.8900	0.359
1	0.90320	0.89255	0.10280	0.8979	0.375
0.25	0.91980	0.90320	0.08620	0.9115	0.400

Estimated Cc: 0.118
Estimated Cr: 0.027

Soil Description: Gray LEAN CLAY with Sand and Trace Gravel (CL)
Specific Gravity: 2.742
Liquid Limit: 27
Plastic Limit: 17
Plasticity Index: 10

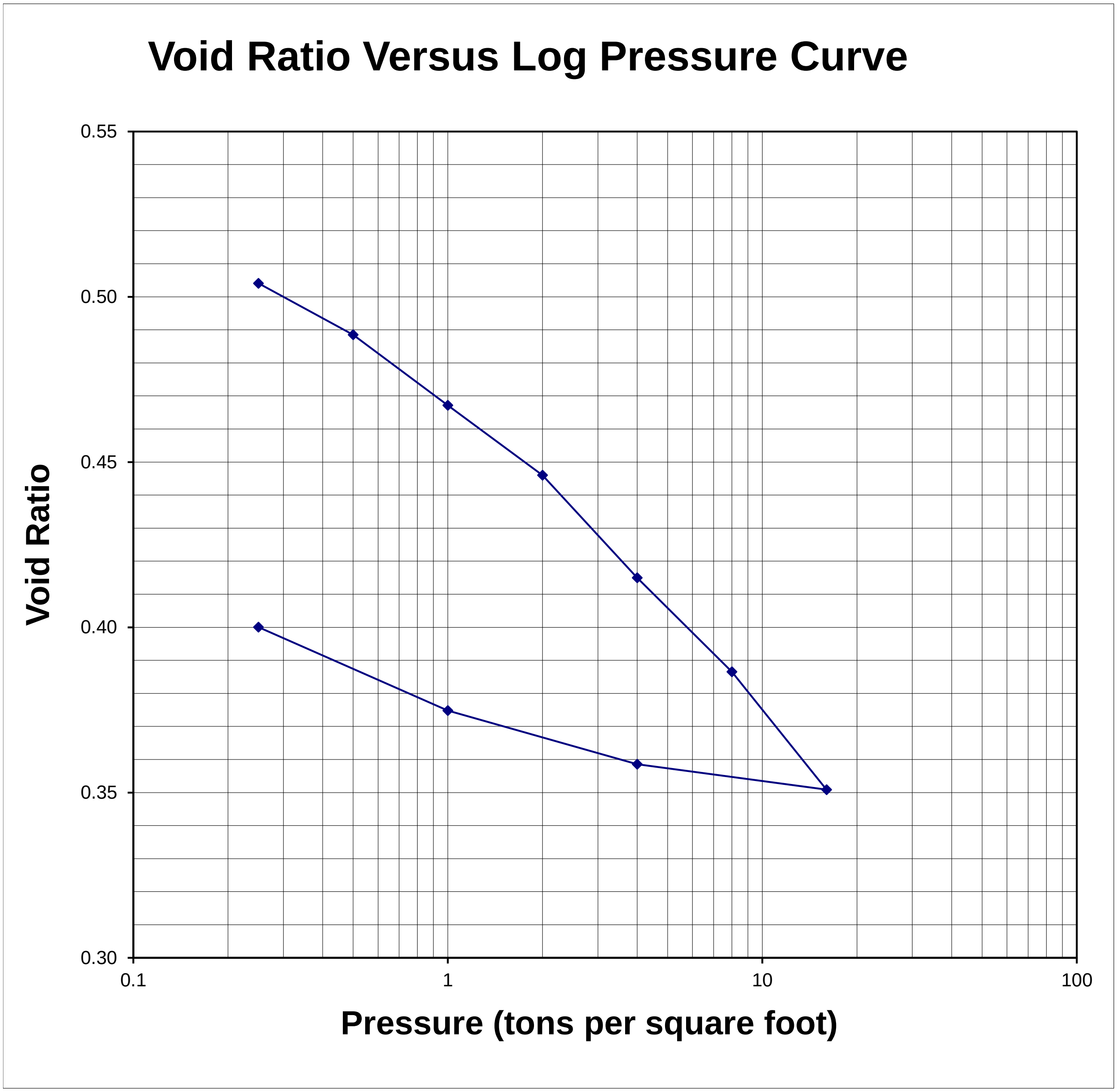
Initial Water Content:	20.1 %	Final Water Content:	16.5 %
Inital Dry Density:	111.8 pcf	Final Dry Density:	122.3 pcf
Initial Void Ratio:	0.531	Final Void Ratio:	0.400
Initial Degree of Saturation:	103.5 %	Final Degree of Saturation	113.2 %

Estimated Preconsolidation Pressure: 1.3 tsf

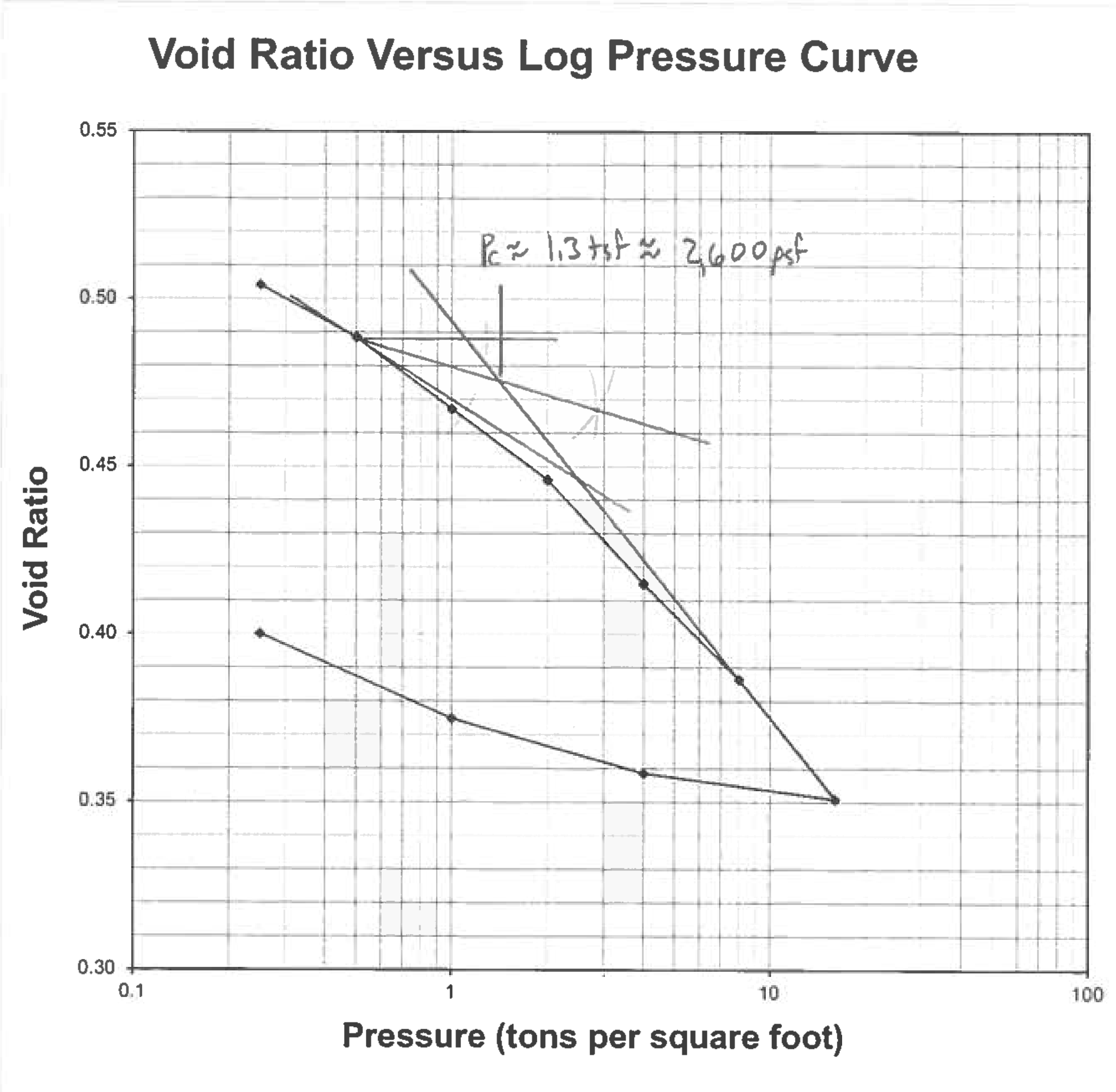
The sample for the test was trimmed from a Shelby tube sample using a cutting shoe. Test Method B was used with the specimen inundated during testing. Coefficients of consolidation were computed by log of time method.

TTL associates inc

Project No.: 17268 01
Date: 10/29/2018
Client: DGL Consulting Engineers
Project: Proposed Connector Trail
Toledo, OH
Boring No.: B-5
Sample No.: ST-1
Depth: 18.0 - 20.0'



Project No.: 17268 01
Date: 10/29/2018
Client: DGL Consulting Engineers
Project: Proposed Connector Trail
Toledo, OH
Boring No.: B-5
Sample No.: ST-1
Depth: 18.0 - 20.0'



DRIVEN 1.2

GENERAL PROJECT INFORMATION

B-8
Rear Abutment
HP 10x42

Filename: T:\GEOTECH\DRIVEN\1726801\B-8ABUTH.DVN
Project Name: Swan Creek Trail Project Date: 01/28/2019
Project Client: DGL
Computed By: CPI
Project Manager: CPI

PILE INFORMATION

Pile Type: H Pile - HP10X42
Top of Pile: 3.00 ft — Elev 616 (Bottom of Pile Cap)
Perimeter Analysis: Box
Tip Analysis: Box Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	16.00 ft
	- Driving/Restrike	16.00 ft
	- Ultimate:	16.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	4.50 ft	0.00%	125.00 pcf	2000.00 psf	T-79 Steel
2	Cohesionless	1.50 ft	0.00%	120.00 pcf	33.7/33.7	Nordlund
3	Cohesive	7.00 ft	0.00%	125.00 pcf	2000.00 psf	T-79 Steel
4	Cohesive	8.00 ft	0.00%	130.00 pcf	1200.00 psf	T-79 Steel
5	Cohesive	17.50 ft	0.00%	130.00 pcf	850.00 psf	T-79 Steel
6	Cohesive	8.50 ft	0.00%	130.00 pcf	1500.00 psf	T-79 Steel
7	Cohesive	11.50 ft	0.00%	130.00 pcf	2100.00 psf	T-79 Steel
8	Cohesive	3.50 ft	0.00%	130.00 pcf	1250.00 psf	T-79 Steel
9	Cohesive	8.00 ft	0.00%	130.00 pcf	700.00 psf	T-79 Steel

ULTIMATE - SKIN FRICTION

Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
2.99 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
3.00 ft	Cohesive	N/A	N/A	1165.00 psf	0.00 Kips
4.49 ft	Cohesive	N/A	N/A	1165.00 psf	5.72 Kips
4.51 ft	Cohesionless	563.10 psf	24.74	N/A	5.77 Kips
5.99 ft	Cohesionless	651.90 psf	24.74	N/A	7.01 Kips
6.01 ft	Cohesive	N/A	N/A	1165.00 psf	7.06 Kips
12.99 ft	Cohesive	N/A	N/A	1228.84 psf	35.33 Kips
13.01 ft	Cohesive	N/A	N/A	937.71 psf	35.41 Kips
20.99 ft	Cohesive	N/A	N/A	996.00 psf	61.61 Kips
21.01 ft	Cohesive	N/A	N/A	768.24 psf	61.67 Kips
30.01 ft	Cohesive	N/A	N/A	817.82 psf	85.93 Kips
38.49 ft	Cohesive	N/A	N/A	837.50 psf	109.92 Kips
38.51 ft	Cohesive	N/A	N/A	1295.00 psf	109.99 Kips
46.99 ft	Cohesive	N/A	N/A	1295.00 psf	146.18 Kips
47.01 ft	Cohesive	N/A	N/A	1555.00 psf	146.28 Kips
56.01 ft	Cohesive	N/A	N/A	1555.00 psf	192.40 Kips
58.49 ft	Cohesive	N/A	N/A	1555.00 psf	205.11 Kips
58.51 ft	Cohesive	N/A	N/A	1122.50 psf	205.20 Kips
61.99 ft	Cohesive	N/A	N/A	1122.50 psf	218.07 Kips
62.01 ft	Cohesive	N/A	N/A	700.00 psf	218.13 Kips
69.99 ft	Cohesive	N/A	N/A	700.00 psf	236.54 Kips

ULTIMATE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
2.99 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
3.00 ft	Cohesive	N/A	N/A	N/A	12.22 Kips
4.49 ft	Cohesive	N/A	N/A	N/A	12.22 Kips
4.51 ft	Cohesionless	563.70 psf	53.42	43.98 Kips	13.45 Kips
5.99 ft	Cohesionless	741.30 psf	53.42	43.98 Kips	17.69 Kips
6.01 ft	Cohesive	N/A	N/A	N/A	12.22 Kips
12.99 ft	Cohesive	N/A	N/A	N/A	12.22 Kips
13.01 ft	Cohesive	N/A	N/A	N/A	7.33 Kips
20.99 ft	Cohesive	N/A	N/A	N/A	7.33 Kips
21.01 ft	Cohesive	N/A	N/A	N/A	5.19 Kips
30.01 ft	Cohesive	N/A	N/A	N/A	5.19 Kips
38.49 ft	Cohesive	N/A	N/A	N/A	5.19 Kips
38.51 ft	Cohesive	N/A	N/A	N/A	9.16 Kips
46.99 ft	Cohesive	N/A	N/A	N/A	9.16 Kips
47.01 ft	Cohesive	N/A	N/A	N/A	12.83 Kips
56.01 ft	Cohesive	N/A	N/A	N/A	12.83 Kips
58.49 ft	Cohesive	N/A	N/A	N/A	12.83 Kips
58.51 ft	Cohesive	N/A	N/A	N/A	7.64 Kips
61.99 ft	Cohesive	N/A	N/A	N/A	7.64 Kips
62.01 ft	Cohesive	N/A	N/A	N/A	4.28 Kips
69.99 ft	Cohesive	N/A	N/A	N/A	4.28 Kips

B-B Rear Abutment
HP 10x42

ULTIMATE - SUMMARY OF CAPACITIES

Depth		Skin Friction	End Bearing	Total Capacity	
0.01 ft	– GSE 619	0.00 Kips	0.00 Kips	0.00 Kips	
2.99 ft		0.00 Kips	0.00 Kips	0.00 Kips	
3.00 ft		0.00 Kips	12.22 Kips	12.22 Kips	
4.49 ft		5.72 Kips	12.22 Kips	17.94 Kips	
4.51 ft		5.77 Kips	13.45 Kips	19.22 Kips	
5.99 ft		7.01 Kips	17.69 Kips	24.71 Kips	
6.01 ft		7.06 Kips	12.22 Kips	19.28 Kips	
12.99 ft		35.33 Kips	12.22 Kips	47.55 Kips	
13.01 ft		35.41 Kips	7.33 Kips	42.74 Kips	← 61k @ 18.5'
20.99 ft		61.61 Kips	7.33 Kips	68.94 Kips	Elev 600
21.01 ft		61.67 Kips	5.19 Kips	66.86 Kips	
30.01 ft		85.93 Kips	5.19 Kips	91.12 Kips	← 92k @ 30½'
38.49 ft		109.92 Kips	5.19 Kips	115.11 Kips	Elev. 588
38.51 ft		109.99 Kips	9.16 Kips	119.15 Kips	
46.99 ft		146.18 Kips	9.16 Kips	155.34 Kips	
47.01 ft		146.28 Kips	12.83 Kips	159.10 Kips	
56.01 ft		192.40 Kips	12.83 Kips	205.23 Kips	
58.49 ft		205.11 Kips	12.83 Kips	217.94 Kips	
58.51 ft		205.20 Kips	7.64 Kips	212.83 Kips	
61.99 ft		218.07 Kips	7.64 Kips	225.71 Kips	
62.01 ft		218.13 Kips	4.28 Kips	222.41 Kips	
69.99 ft		236.54 Kips	4.28 Kips	240.82 Kips	

HP 10x42 Max Rndr = 350k

$TFL = 128k \rightarrow Rndr = \frac{128k}{0.7} = 183k$

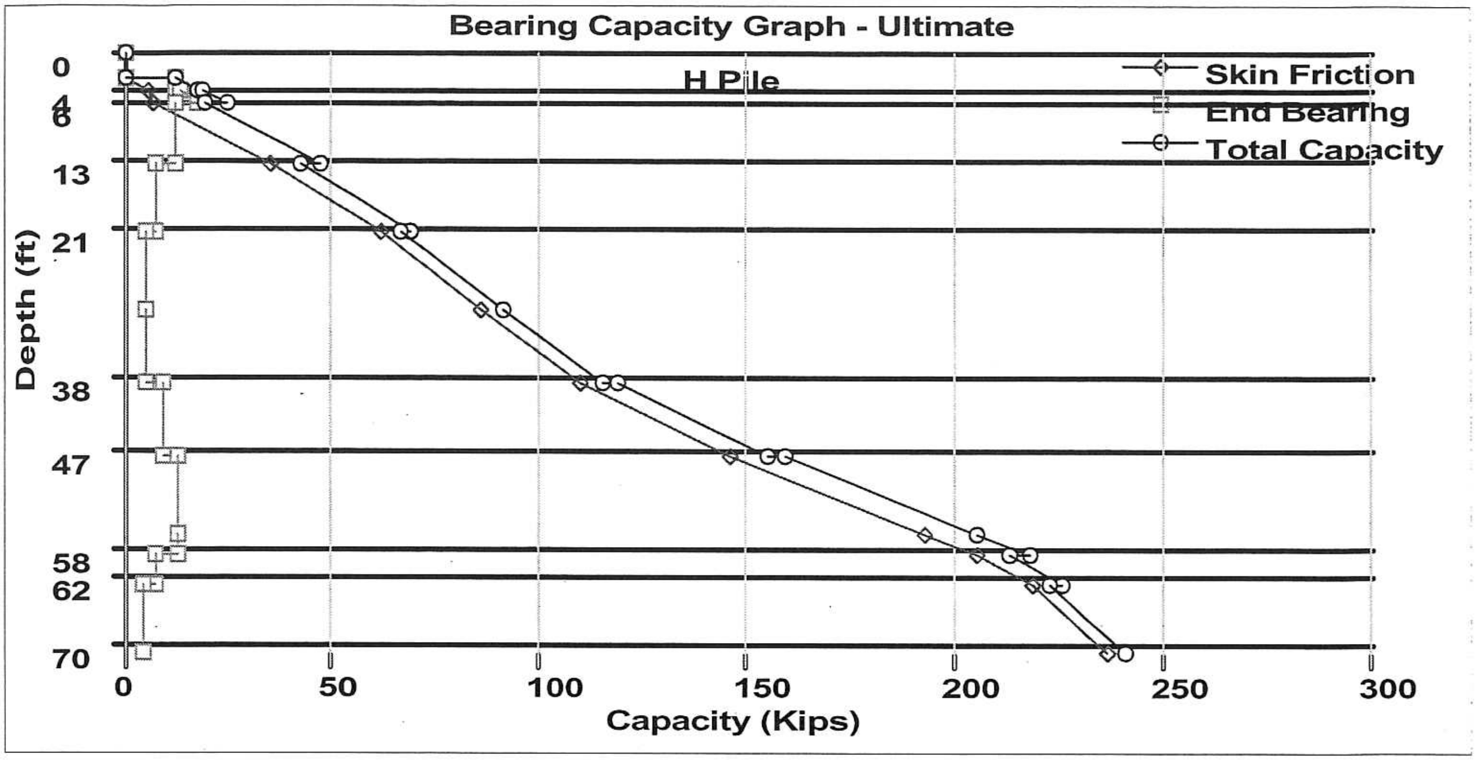
Say 3 piles → 61k/pile
Say 2 piles → 92k/pile

Footing = 616 + 2' shelf = 618

	618	618
	− 600	− 588
	18'	30'
	Est 20'	Est 35'
	order 25'	order 40'
	for 3 piles	for 2 piles

TTL associates inc
Environmental, Geotechnical Engineering & Testing

Filename: T:\GEOTECH\DRIVEN\1726801\B-8ABUTH.DVN



Soil Profile

0.0 ft
4.7 ft
9.3 ft
14.0 ft
18.7 ft
23.3 ft
28.0 ft
32.7 ft
37.3 ft
42.0 ft
46.7 ft
51.3 ft
56.0 ft
60.7 ft
65.3 ft
70.0 ft

Clay: Unit Weight 125 -- Undrained Shear Strength 2000 -- Driving Loss 0%

Sand: Unit Weight 120 -- Friction Angles 34/34 -- Driving Loss 0%

Clay: Unit Weight 125 -- Undrained Shear Strength 2000 -- Driving Loss 0%

Clay: Unit Weight 130 -- Undrained Shear Strength 1200 -- Driving Loss 0%

Clay: Unit Weight 130 -- Undrained Shear Strength 850 -- Driving Loss 0%

Clay: Unit Weight 130 -- Undrained Shear Strength 1500 -- Driving Loss 0%

Clay: Unit Weight 130 -- Undrained Shear Strength 2100 -- Driving Loss 0%

Clay: Unit Weight 130 -- Undrained Shear Strength 1250 -- Driving Loss 0%

Clay: Unit Weight 130 -- Undrained Shear Strength 700 -- Driving Loss 0%

DRIVEN 1.2

GENERAL PROJECT INFORMATION

B-9
Intermediate Pier
HP 10x42

Filename: T:\GEOTECH\DRIVEN\1726801\B-9PIERH.DVN
Project Name: Swan Creek Trail Project Date: 01/28/2019
Project Client: DGL
Computed By: CPI
Project Manager: CPI

PILE INFORMATION

Pile Type: H Pile - HP10X42
Top of Pile: 5.50 ft – Elev. 581 (Bottom of Pile Cap)
Perimeter Analysis: Box
Tip Analysis: Box Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	8.00 ft
	- Driving/Restrike	8.00 ft
	- Ultimate:	8.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	8.00 ft	0.00%	120.00 pcf	29.0/29.0	Nordlund
2	Cohesive	3.00 ft	0.00%	125.00 pcf	500.00 psf	T-79 Steel
3	Cohesive	2.00 ft	0.00%	125.00 pcf	1000.00 psf	T-79 Steel
4	Cohesive	15.50 ft	0.00%	130.00 pcf	1500.00 psf	T-79 Steel
5	Cohesive	5.50 ft	0.00%	130.00 pcf	1000.00 psf	T-79 Steel
6	Cohesive	2.00 ft	0.00%	130.00 pcf	2000.00 psf	T-79 Steel
7	Cohesionless	4.00 ft	0.00%	150.00 pcf	43.0/43.0	Nordlund

ULTIMATE - SKIN FRICTION

Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesionless	0.00 psf	0.00	N/A	0.00 Kips
5.49 ft	Cohesionless	0.00 psf	0.00	N/A	0.00 Kips
5.50 ft	Cohesionless	660.00 psf	21.24	N/A	0.00 Kips
7.99 ft	Cohesionless	809.40 psf	21.24	N/A	1.78 Kips
8.01 ft	Cohesive	N/A	N/A	410.00 psf	1.80 Kips
10.99 ft	Cohesive	N/A	N/A	419.27 psf	5.92 Kips
11.01 ft	Cohesive	N/A	N/A	815.57 psf	5.96 Kips
12.99 ft	Cohesive	N/A	N/A	827.36 psf	11.36 Kips
13.01 ft	Cohesive	N/A	N/A	1103.05 psf	11.43 Kips
22.01 ft	Cohesive	N/A	N/A	1187.02 psf	46.64 Kips
28.49 ft	Cohesive	N/A	N/A	1247.48 psf	75.08 Kips
28.51 ft	Cohesive	N/A	N/A	919.79 psf	75.15 Kips
33.99 ft	Cohesive	N/A	N/A	950.00 psf	92.31 Kips
34.01 ft	Cohesive	N/A	N/A	1515.00 psf	92.39 Kips
35.99 ft	Cohesive	N/A	N/A	1515.00 psf	102.28 Kips
36.01 ft	Cohesionless	2828.24 psf	31.52	N/A	102.40 Kips
39.99 ft	Cohesionless	3002.56 psf	31.52	N/A	130.01 Kips

ULTIMATE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesionless	0.00 psf	26.32	9.04 Kips	0.00 Kips
5.49 ft	Cohesionless	0.00 psf	26.32	9.04 Kips	0.00 Kips
5.50 ft	Cohesionless	660.00 psf	26.32	9.04 Kips	6.57 Kips
7.99 ft	Cohesionless	958.80 psf	26.32	9.04 Kips	9.04 Kips
8.01 ft	Cohesive	N/A	N/A	N/A	3.05 Kips
10.99 ft	Cohesive	N/A	N/A	N/A	3.05 Kips
11.01 ft	Cohesive	N/A	N/A	N/A	6.11 Kips
12.99 ft	Cohesive	N/A	N/A	N/A	6.11 Kips
13.01 ft	Cohesive	N/A	N/A	N/A	9.16 Kips
22.01 ft	Cohesive	N/A	N/A	N/A	9.16 Kips
28.49 ft	Cohesive	N/A	N/A	N/A	9.16 Kips
28.51 ft	Cohesive	N/A	N/A	N/A	6.11 Kips
33.99 ft	Cohesive	N/A	N/A	N/A	6.11 Kips
34.01 ft	Cohesive	N/A	N/A	N/A	12.22 Kips
35.99 ft	Cohesive	N/A	N/A	N/A	12.22 Kips
36.01 ft	Cohesionless	2828.68 psf	307.00	459.87 Kips	459.87 Kips
39.99 ft	Cohesionless	3177.32 psf	307.00	459.87 Kips	459.87 Kips

ULTIMATE - SUMMARY OF CAPACITIES

B-9 Intermediate Pier HP10x42

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
5.49 ft	0.00 Kips	0.00 Kips	0.00 Kips
5.50 ft	0.00 Kips	6.57 Kips	6.57 Kips
7.99 ft	1.78 Kips	9.04 Kips	10.82 Kips
8.01 ft	1.80 Kips	3.05 Kips	4.86 Kips
10.99 ft	5.92 Kips	3.05 Kips	8.98 Kips
11.01 ft	5.96 Kips	6.11 Kips	12.07 Kips
12.99 ft	11.36 Kips	6.11 Kips	17.47 Kips
13.01 ft	11.43 Kips	9.16 Kips	20.59 Kips
22.01 ft	46.64 Kips	9.16 Kips	55.80 Kips
28.49 ft	75.08 Kips	9.16 Kips	84.24 Kips
28.51 ft	75.15 Kips	6.11 Kips	81.26 Kips
33.99 ft	92.31 Kips	6.11 Kips	98.42 Kips
34.01 ft	92.39 Kips	12.22 Kips	104.61 Kips
35.99 ft	102.28 Kips	12.22 Kips	114.50 Kips
36.01 ft	102.40 Kips	459.87 Kips	562.27 Kips
39.99 ft	130.01 Kips	459.87 Kips	589.88 Kips

— GSE B-9 586½

96k @ 33½' Elev 553

144k @ ≥36'

AR on Rock or Boulder

HP 10x42 Max Rndr = 350k

TFL = 201k → Rndr = $\frac{201}{0.7}$ = 287k

For 3 piles, 96k/pile

For 2 Piles 144k/pile

Footing 581 + 2' stickup = 583
− 553
30'
Est 35'
Order 40'
w/3 piles

Extrapolate method at 34-36'

Side = 102.3 − 92.4 = 9.9k/2'
= 4.95k/ft

End = 12.2k

144k − 12.2k = 131.8 in side

@ 36' side = 102.4k

need 131.8 − 102.4k = 29.4k

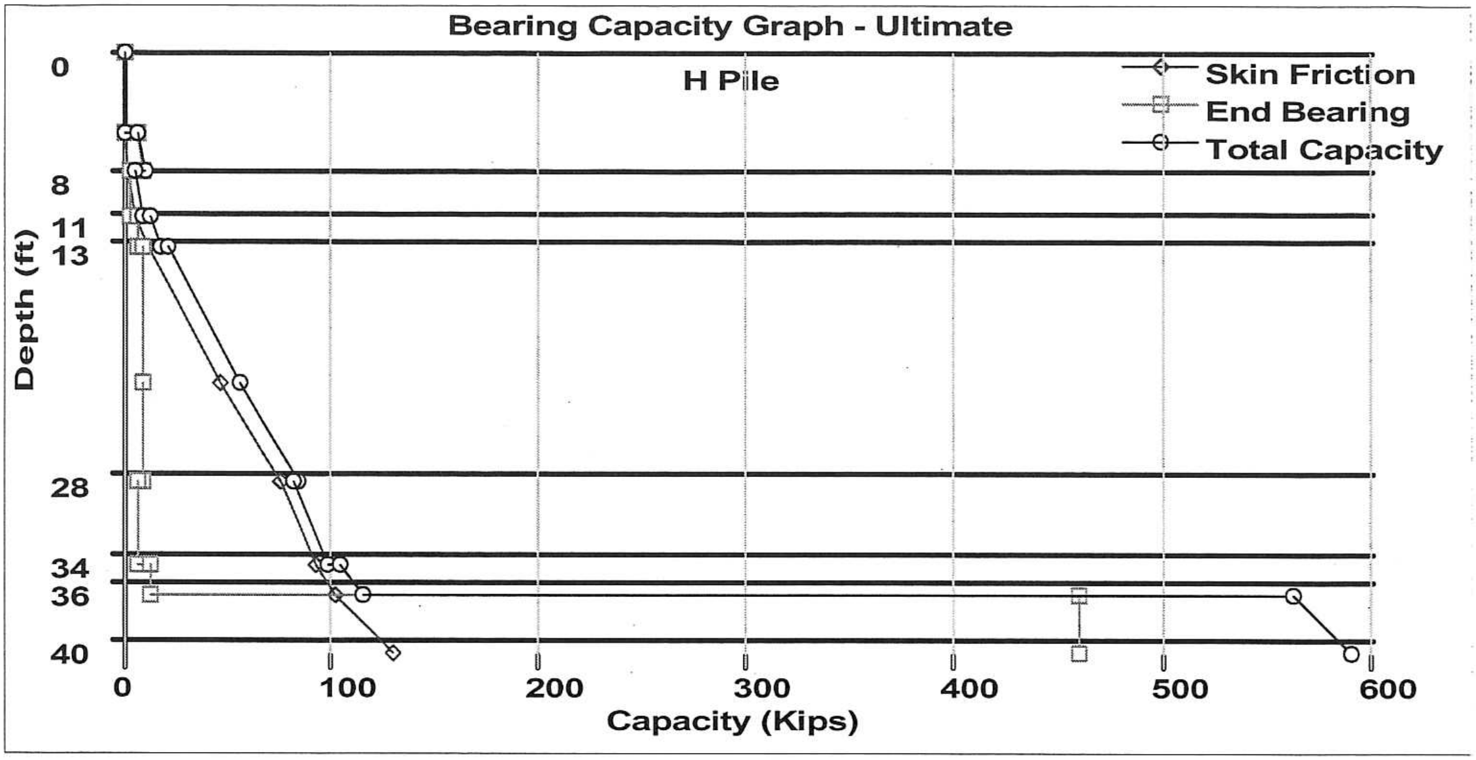
$\frac{29.4k}{4.95k/ft}$ = 5.9' ~ 6'

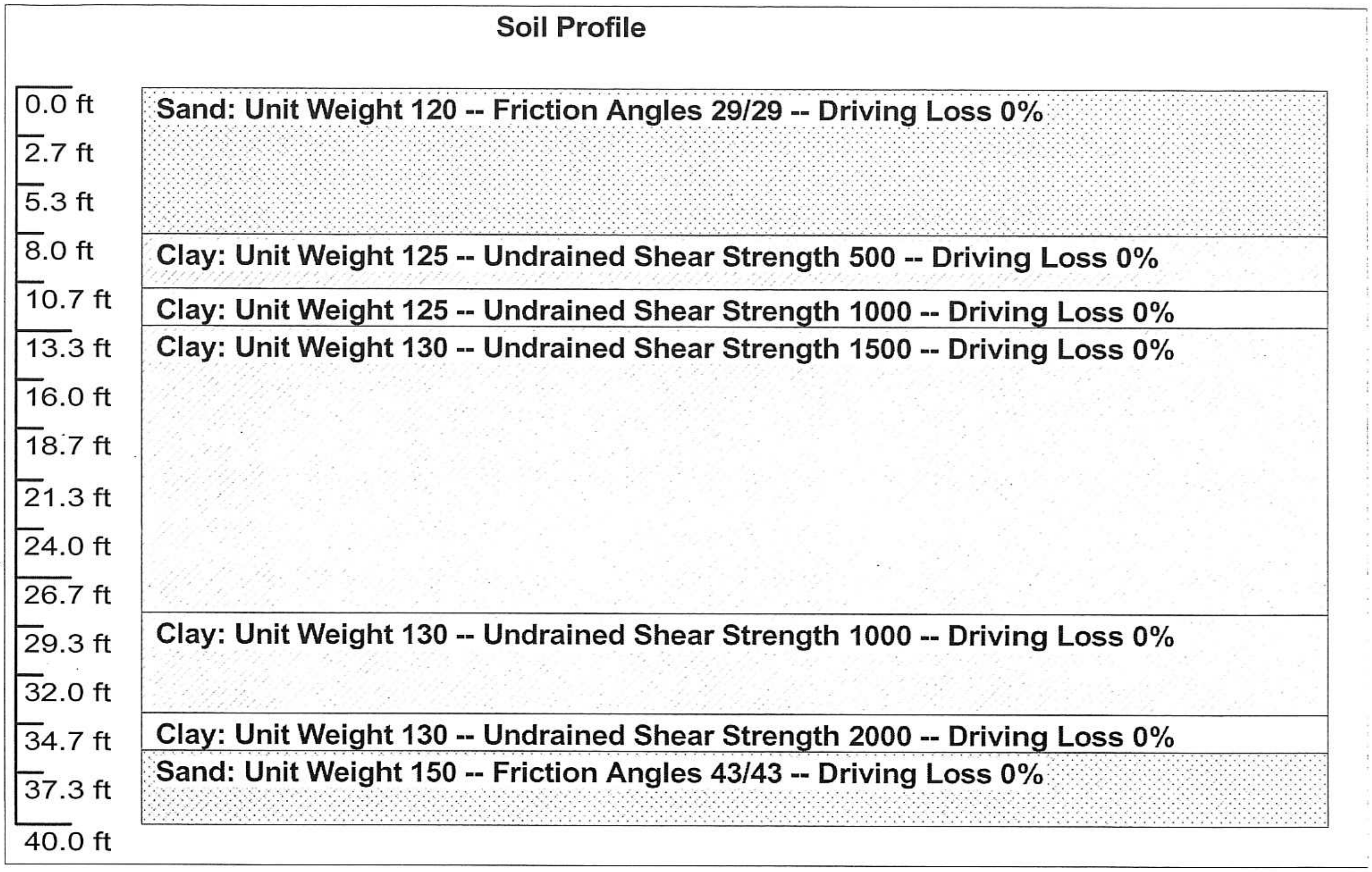
36+6 = 42' (Elev 544±)

Still shallower than maybe rock @ 530±

583
− 544
39'
Est 40'
Order 45'

Filename: T:\GEOTECH\DRIVEN\1726801\B-9PIERH.DVN




Soil Profile
0.0 ft
2.7 ft
5.3 ft
8.0 ft
10.7 ft
13.3 ft
16.0 ft
18.7 ft
21.3 ft
24.0 ft
26.7 ft
29.3 ft
32.0 ft
34.7 ft
37.3 ft
40.0 ft
Sand: Unit Weight 120 -- Friction Angles 29/29 -- Driving Loss 0%
Clay: Unit Weight 125 -- Undrained Shear Strength 500 -- Driving Loss 0%
Clay: Unit Weight 125 -- Undrained Shear Strength 1000 -- Driving Loss 0%
Clay: Unit Weight 130 -- Undrained Shear Strength 1500 -- Driving Loss 0%
Clay: Unit Weight 130 -- Undrained Shear Strength 1000 -- Driving Loss 0%
Clay: Unit Weight 130 -- Undrained Shear Strength 2000 -- Driving Loss 0%
Sand: Unit Weight 150 -- Friction Angles 43/43 -- Driving Loss 0%

DRIVEN 1.2

GENERAL PROJECT INFORMATION

B-9
Fwd Abutment
HP 10x42
+ Boardwalk Piers
HP 10x42

Filename: T:\GEOTECH\DRIVEN\1726801\B-9ABUTH.DVN
Project Name: Swan Creek Trail — Project Date: 01/28/2019
Project Client: DGL
Computed By: CPI
Project Manager: CPI

PILE INFORMATION

Pile Type: H Pile - HP10X42
Top of Pile: 0.00 ft — Elev. 586.5 (Bottom of Pile Cap)
Perimeter Analysis: Box
Tip Analysis: Box Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	8.00 ft
	- Driving/Restrike	8.00 ft
	- Ultimate:	8.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	8.00 ft	0.00%	120.00 pcf	29.0/29.0	Nordlund
2	Cohesive	3.00 ft	0.00%	125.00 pcf	500.00 psf	T-79 Steel
3	Cohesive	2.00 ft	0.00%	125.00 pcf	1000.00 psf	T-79 Steel
4	Cohesive	15.50 ft	0.00%	130.00 pcf	1500.00 psf	T-79 Steel
5	Cohesive	5.50 ft	0.00%	130.00 pcf	1000.00 psf	T-79 Steel
6	Cohesive	2.00 ft	0.00%	130.00 pcf	2000.00 psf	T-79 Steel
7	Cohesionless	4.00 ft	0.00%	150.00 pcf	43.0/43.0	Nordlund

ULTIMATE - SKIN FRICTION

Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesionless	0.60 psf	21.24	N/A	0.00 Kips
7.99 ft	Cohesionless	479.40 psf	21.24	N/A	3.39 Kips
8.01 ft	Cohesive	N/A	N/A	410.00 psf	3.41 Kips
10.99 ft	Cohesive	N/A	N/A	419.27 psf	7.53 Kips
11.01 ft	Cohesive	N/A	N/A	815.57 psf	7.57 Kips
12.99 ft	Cohesive	N/A	N/A	827.36 psf	12.97 Kips
13.01 ft	Cohesive	N/A	N/A	1103.05 psf	13.03 Kips
22.01 ft	Cohesive	N/A	N/A	1187.02 psf	48.25 Kips
28.49 ft	Cohesive	N/A	N/A	1247.48 psf	76.68 Kips
28.51 ft	Cohesive	N/A	N/A	919.79 psf	76.76 Kips
33.99 ft	Cohesive	N/A	N/A	950.00 psf	93.92 Kips
34.01 ft	Cohesive	N/A	N/A	1515.00 psf	94.00 Kips
35.99 ft	Cohesive	N/A	N/A	1515.00 psf	103.89 Kips
36.01 ft	Cohesionless	2828.24 psf	31.52	N/A	104.00 Kips
39.99 ft	Cohesionless	3002.56 psf	31.52	N/A	131.62 Kips

ULTIMATE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesionless	1.20 psf	26.32	9.04 Kips	0.01 Kips
7.99 ft	Cohesionless	958.80 psf	26.32	9.04 Kips	9.04 Kips
8.01 ft	Cohesive	N/A	N/A	N/A	3.05 Kips
10.99 ft	Cohesive	N/A	N/A	N/A	3.05 Kips
11.01 ft	Cohesive	N/A	N/A	N/A	6.11 Kips
12.99 ft	Cohesive	N/A	N/A	N/A	6.11 Kips
13.01 ft	Cohesive	N/A	N/A	N/A	9.16 Kips
22.01 ft	Cohesive	N/A	N/A	N/A	9.16 Kips
28.49 ft	Cohesive	N/A	N/A	N/A	9.16 Kips
28.51 ft	Cohesive	N/A	N/A	N/A	6.11 Kips
33.99 ft	Cohesive	N/A	N/A	N/A	6.11 Kips
34.01 ft	Cohesive	N/A	N/A	N/A	12.22 Kips
35.99 ft	Cohesive	N/A	N/A	N/A	12.22 Kips
36.01 ft	Cohesionless	2828.68 psf	307.00	459.87 Kips	459.87 Kips
39.99 ft	Cohesionless	3177.32 psf	307.00	459.87 Kips	459.87 Kips

ULTIMATE - SUMMARY OF CAPACITIES

B-9 Fwd Abutment + Boardwalk Piers HP 10x42

Depth		Skin Friction	End Bearing		Total Capacity	
0.01 ft	- GSE = 586½	0.00 Kips	0.01 Kips		0.01 Kips	
7.99 ft		3.39 Kips	9.04 Kips		12.43 Kips	
8.01 ft		3.41 Kips	3.05 Kips		6.46 Kips	
10.99 ft		7.53 Kips	3.05 Kips		10.58 Kips	
11.01 ft		7.57 Kips	6.11 Kips		13.68 Kips	
12.99 ft		12.97 Kips	6.11 Kips		19.08 Kips	
13.01 ft		13.03 Kips	9.16 Kips		22.20 Kips	
22.01 ft		48.25 Kips	9.16 Kips		57.41 Kips	58k @ 22½' Elev 564
28.49 ft		76.68 Kips	9.16 Kips		85.85 Kips	64k @ 23½' Elev 563
28.51 ft		76.76 Kips	6.11 Kips		82.87 Kips	95k @ 32½' Elev 554
33.99 ft		93.92 Kips	6.11 Kips	86k @ 29½' Elev. 557	100.03 Kips	
34.01 ft		94.00 Kips	12.22 Kips		106.22 Kips	
35.99 ft		103.89 Kips	12.22 Kips		116.10 Kips	
36.01 ft	AR on apparent bedrock	104.00 Kips	459.87 Kips		563.88 Kips	
39.99 ft		131.62 Kips	459.87 Kips		591.49 Kips	

HP 10x42 Max Rndr = 350k

$\boxed{\text{Bridge Forward Abutment}}$

TFL = 133k (max) → Rndr = $\frac{133k}{0.7}$ = 190k

w/ Box Beam Boardwalk

(Less if Double Tee Boardwalk)

If 3 piles, ~64 k/pile

If 2 piles, ~95 k/pile

Footing @ 586.5 + 2' Stickup = 588.5

$$\begin{array}{r} 588.5 \\ -\ 563 \\ \hline 25.5' \end{array} \qquad \begin{array}{r} 588.5 \\ -\ 554 \\ \hline 34.5' \end{array}$$

Boardwalk Piers

TFL = 120k (max for Box Beam)

Rndr = $\frac{120}{0.7}$ = 172k

If 3 piles, 58 kips/pile

If 2 piles, 86 kips/pile

Say 30' Est
35' order
W/ 3 piles

Est 35'
Order 40'
W/ 2 piles

Assume Footing @ 586.5 + 2' Stickup = 588.5

$$\begin{array}{r} 588.5 \\ -\ 564 \\ \hline 24.5' \end{array} \qquad \begin{array}{r} 588.5 \\ -\ 557 \\ \hline 31.5' \end{array}$$

Due to uncertainty w/ Footing Elev., Round up to 30' = Est and 35' order (Same as for Fwd Abutment)

Est = 35'
Order = 40'

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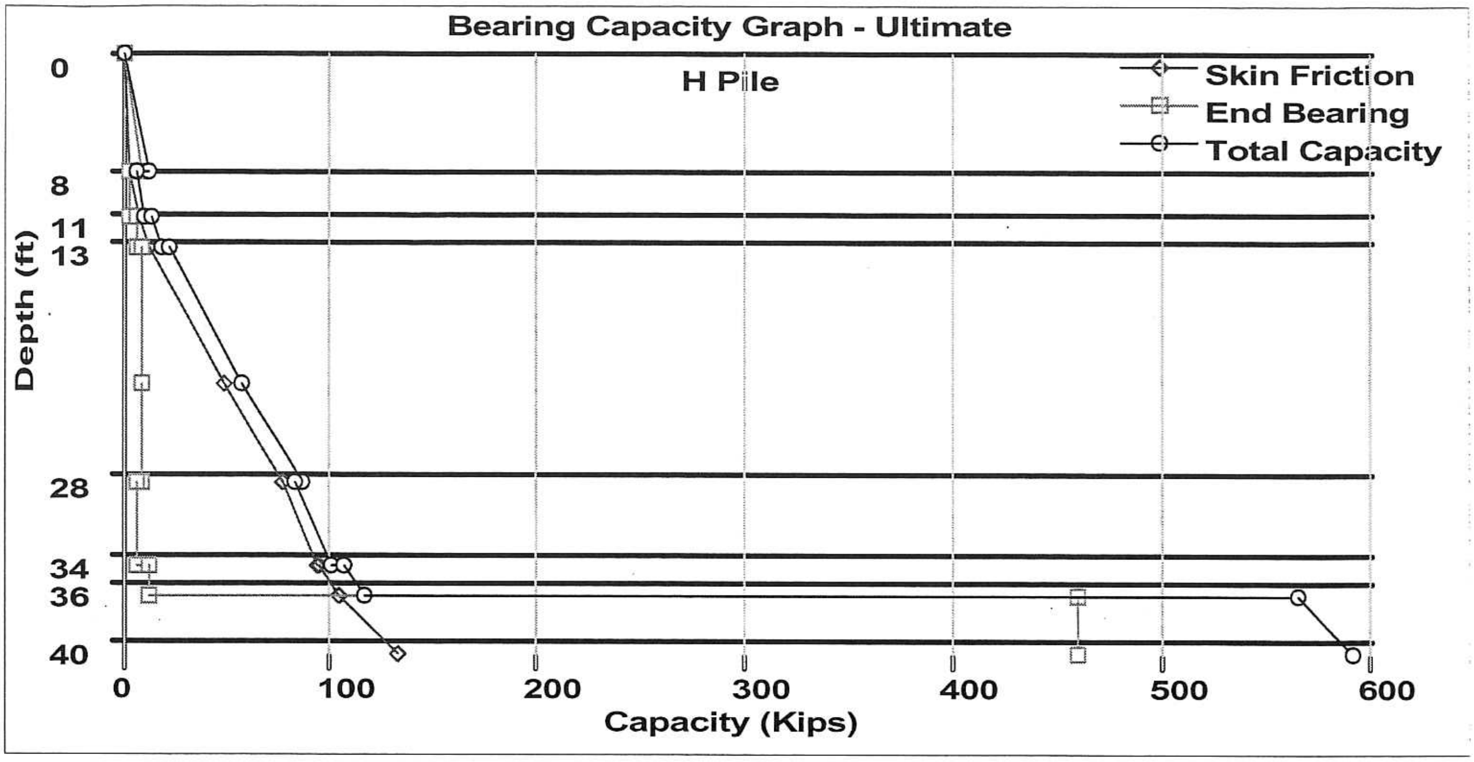

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Soil Profile