GEOTECHNICAL SUBSURFACE EXPLORATION

City of Port Clinton
Port Clinton, Ohio

Geotechnical Subsurface Exploration
Proposed Jefferson Street Reconstruction
East Perry Street to East Third Street
Port Clinton, Ohio

August 2018

TTL associates inc

Environmental, Geotechnical Engineering & Testing

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August 17, 2018

TTL Project No. 1654701

Mr. Olen F. Martin
Safety-Service Director
City of Port Clinton
1868 East Perry Street
Port Clinton, Ohio 43452

Geotechnical Subsurface Exploration
Proposed Jefferson Street Reconstruction
East Perry Street to East Third Street
Port Clinton, Ohio

Dear Mr. Martin:

Following is the report of the geotechnical subsurface exploration performed by TTL Associates, Inc. (TTL) for the referenced project. This study was performed in general accordance with TTL Proposal No. 1654701, dated June 6, 2018, and authorized by you on June 25, 2018, referencing Purchase Order No. 45636.

This final report contains the results of our study, our engineering interpretation of the results with respect to the project characteristics, as well as our soils-related design and construction recommendations for pavements, underground utilities, and a pump station.

The pavement cores and soil samples collected during this exploration 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.

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GEOTECHNICAL SUBSURFACE EXPLORATION
PROPOSED JEFFERSON STREET RECONSTRUCTION
EAST PERRY STREET TO EAST THIRD STREET
PORT CLINTON, OHIO

FOR

CITY OF PORT CLINTON
1868 EAST PERRY STREET
PORT CLINTON, OHIO 43452

SUBMITTED

AUGUST 17, 2018
TTL PROJECT NO. 1654701

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1.0 INTRODUCTION

This geotechnical subsurface exploration report has been prepared for the proposed pavement reconstruction and underground utility installation along an approximately 1,100 lineal feet portion of Jefferson Street, from East Perry Street to East Third Street, in Port Clinton, Ohio. The project will also include installation of a pump station north of the intersection of Jefferson Street and East Perry Street. The general project area 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 in accordance with Ohio Department of Transportation (ODOT) GB-1 “Plan Subgrades” (July 20, 2018), as well as provides our design and construction recommendations for pavements, underground utilities, and a pump station.

This study was performed in general accordance with TTL Proposal No. 1654701, dated June 6, 2018, and authorized by Mr. Olen F. Martin, Safety-Service Director for the City of Port Clinton, on June 25, 2018, referencing Purchase Order No. 45636.

The purpose of this exploration was to evaluate the subsurface conditions and laboratory data relative to pavement reconstruction, new underground utility installation, and pump station installation. To accomplish this, TTL performed five test borings that included pavement cores, field and laboratory soil testing, and a geotechnical engineering evaluation of the test results.

This report includes:

- A description of the type and thickness of existing pavement conditions at the boring locations.
- A description of the subsurface soil and groundwater conditions encountered in the borings.
- Design recommendations for pavements, subsurface utilities, and a pump station related to the proposed project.
- Recommendations concerning soil- and groundwater-related construction procedures such as subgrade preparation in accordance with ODOT GB-1 criteria, earthwork, pavement construction, underground utility and pump station installation, 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 exploration included five test borings (each with a pavement core), designated as Borings B-1 through B-5, drilled by TTL on July 23 and 24, 2018. The borings were located in the field by TTL. Ground surface elevations at the boring locations, which are shown on the logs of test borings, were estimated using Google Earth. The approximate locations of the borings are shown on the Test Boring Location Plan (Plate 2.0), and are summarized in the following table.

Table 2.0. Test Boring Locations

Boring Number	Approximate Boring Location along Jefferson Street
B-001	Approximately 40 feet north of 3rd Street in drive lane
B-002	Approximately 100 feet north of 2nd Street in parking area
B-003	Approximately 50 feet south of Perry Street in drive lane
B-004	Approximately 45 feet north of Perry Street in drive lane
B-005	Approximately 250 feet north of Perry Street, near existing restrooms building in entrance/exit to parking lot

The test borings were performed in general accordance with geotechnical investigative procedures outlined in ODOT "Specifications for Geotechnical Explorations" and GB-1 "Plan Subgrades" sampling criteria, as well as ASTM Standards D 1452 and D 5434. The pavement cores were obtained with a nominal 4-inch diameter coring barrel. The test borings performed during this exploration were advanced using a truck-mounted drilling rig utilizing 3½-inch diameter solid-stem augers and 3¼-inch inside diameter hollow-stem augers. Borings B-1, B-2, and B-3 were terminated at the target completion depth of 17½ feet below existing grades. The remaining borings were terminated at the target completion depth of 30 feet.

During auger advancement in the test borings, soil samples were obtained continuously using an 18-inch split-spoon sample drive to a depth of 6 feet below bottom of existing pavement cross-section, and at 2½-foot intervals thereafter to boring termination. Split-spoon 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 number of blows per increment was recorded at each depth interval, and these data are presented under the "SPT" column on the Logs of Test Borings attached to this report. 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, or N_m-value, and is typically reported in blows per foot (bpf). The N_m-values were corrected to an equivalent rod energy ratio of 60 percent, N_{60}. The calibrated hammer/rod energy ratio for the CME 75 truck-mounted drill rig utilized in this project was 75.4

percent based on calibration on February 8, 2018. The N_{60}-values are presented on the attached Logs of Test Borings.

A Shelby tube sample, designated ST on the Log of Test Boring, was obtained from Boring B-004 (15 to 17 feet). The Shelby tube sample was 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. The Shelby tube was then extracted from the subsoils, and the ends were capped and sealed. The sample was transported to our laboratory where it was extruded, classified, and tested. An attempt to retrieve a Shelby tube sample from 20 to 22 feet in Boring B-004 resulted in no recovered soil specimen.

The pavement materials and soil conditions encountered in the test borings are presented in the Logs of Test Borings, along with information related to sample data, Standard Penetration Test results, groundwater 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 on field logs of the encountered soils. Photographic logs of the recovered pavement cores are also attached to this report.

All samples of the subsoils were visually or manually classified in accordance with the Ohio Department of Transportation (ODOT) system of soil classification. Where gradation and plasticity tests were not performed for a complete ODOT classification, the soils were classified using visual-manual procedures. All recovered samples were also tested for moisture content (ASTM D 2216). Dry density determinations and unconfined compressive strength tests by the constant rate of strain method (ASTM D 2216) were performed on the recovered Shelby tube sample and selected split-spoon samples from the pump station borings. Unconfined compressive strength estimates were obtained for the remaining intact cohesive samples using a calibrated hand penetrometer. Atterberg limits tests (ASTM D 4318) and particle size analyses (ASTM D 422) were performed on selected soil samples from the borings to determine soil classification and index properties. Test results are shown on the Logs of Test Borings attached to this report.

Experience indicates that the actual soil conditions at a site could vary from those generalized on the basis of test borings made at specific locations, especially at previously developed sites such as this site. Therefore, it is essential that a geotechnical engineer be retained to provide soil engineering services during the site preparation and excavation 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

Based on the provided information, it is our understanding that the proposed project consists of reconstruction of approximately 1,100 lineal feet of existing pavement along Jefferson Street, from East Perry Street to East Third Street in Port Clinton, Ohio. It is anticipated that pavement remediation will consist of full-depth removal and replacement. Traffic loads and volumes were not available at the time of preparing this report.

Prior to pavement reconstruction, new waterline and sanitary sewer will be installed in the project area. Underground utility installation is assumed to be approximately 15 feet or less below existing grades using open-cut methods.

As part of the roadway reconstruction project, a new pump station is planned along Jefferson Street, between Perry Street and Portage River. Consideration is being given to replacing an existing restrooms building located east of Jefferson Street, just south of Portage River, with a new restroom building overlying the new pump station. The pump station will bear 20 feet or less below existing grades. A storm sewer outlet will be installed from the pump station to Portage River.

4.0 GENERAL SITE AND SUBSURFACE CONDITIONS

4.1 General Site Conditions

The project encompasses approximately 1,100 lineal feet of Jefferson Street roadway, from East Perry Street to East Third Street. Additionally, a pump station will be installed north of the intersection of Jefferson Street and East Perry Street, south of Portage River. At the time of this exploration, the existing pavements consisted of asphalt at the surface. Depending on location south of Perry Street, the asphalt was underlain by brick pavers or concrete, which was underlain by granular base. North of Perry Street, the asphalt was underlain by crushed stone base. The type and thicknesses of the pavement materials, as well as the subgrade material type, encountered at the boring locations are summarized in the following table:

Table 4.1. Encountered Pavement Materials

Boring Number	Pavement Material Thickness (inches)					Subgrade Materials
	Asphalt	Brick	Concrete	Sand and Gravel / Crushed Stone	Sand	
B-1	4	4	-	6½	-	Silt and Clay (A-6a)
B-2	1½	-	8	-	2¼	Silt and Clay (A-6a)
B-3	1¼	-	7¼	-	4	Silt and Clay (A-6a)
B-4	8	-	-	10	-	Crushed Stone Fill
B-5	3	-	-	15	-	Gravel with Sand and Silt Fill

Underlying the pavement materials in Borings B-004 and B-005, which were performed between Perry Street and Portage River, granular existing **fill** materials were encountered to depths of 3½ feet and 14 feet below top of pavement, respectively. In Boring B-004, the fill materials consisted of crushed stone with sand. In Boring B-005, the fill materials consisted of gravel with sand and varying amounts of silt, fine sand with trace gravel and silt, as well as coarse and fine sand with little silt and gravel. The fill encountered in Boring B-005 contained varying amounts of cinders, coal, and metal. Within the upper 3 to 3½ feet, SPT N_{60}-values within the granular fills ranged from 11 to 18 blows per foot (bpf), indicating medium dense compactness. Below a depth of 3 feet in Boring B-005, SPT N_{60}-values within the granular fills ranged from 3 to 5 bpf, indicating **very loose** to **loose** compactness. Wet, free water conditions were noted for the granular samples obtained from Boring B-005 below a depth of 3 feet. As such, SPT results within this zone may have been adversely affected by drive sample collection within saturated granular soils. Moisture contents were on the order of 12 to 13 percent for the samples obtained above depths of 3 to 3½ feet. Below a depth of 3 feet in Boring B-005, moisture contents ranged from 34 to 76 percent for the saturated granular fill samples.

4.2 General Soil Conditions

Based on the results of our field and laboratory tests, the subsoils encountered underlying the pavement and fill materials at the site can be generally characterized as three strata of cohesive soils with varying strength and moisture characteristics. Due to location closer to Portage River, Borings B-004 and B-005 also encountered apparent alluvial deposits.

In Boring B-004, saturated granular apparent alluvial deposits were encountered underlying the crushed stone fill to a depth of 6½ feet below existing grade. The granular soils consisted of coarse and fine sand (A-3a) with little silt, gravel, and trace clay. SPT N_{60}-values on the order of 13 and 14 blows per foot (bpf), indicating medium dense compactness, were determined for the recovered samples. Moisture contents ranged from 17 to 27 percent.

In Boring B-005, wet black/gray cohesive apparent alluvial deposits were encountered underlying the fill materials to a depth of 16 feet. The cohesive soils consisted of silt and clay (A-6a) with some sand and little gravel. An SPT N_{60}-value of 6 bpf, indicating medium stiff consistency, and a moisture content of 22 percent were determined for the recovered sample.

Stratum I consisted of predominantly medium stiff to stiff native cohesive soils encountered underlying the pavement materials in Borings B-001, B-002, and B-003, as well as the granular alluvial deposits in Boring B-004. Stratum I extended to depths ranging from 2½ to 8½ feet. These cohesive soils consisted of silt and clay (A-6a) with varying amounts of sand and gravel. SPT N_{60}-values ranged from 5 to 14 bpf. Unconfined compressive strengths typically ranged from 2,000 to 6,000 pounds per square foot (psf), although the strengths at the higher end of this range may have been affected by desiccation. Moisture contents ranged from 19 to 24 percent. A sample of Stratum I soils tested from Boring B-001 resulted in a liquid limit of 36 percent and a plasticity index of 14 percent. These values, along with gradation results, are indicative of silt and clay (ODOT A-6a) based on the ODOT system of soil classification.

Stratum II consisted of predominantly very stiff to hard cohesive soils encountered underlying Stratum I in Borings B-001 through B-004 to depths ranging from approximately 11½ to 13½ feet. These cohesive soils consisted of sandy silt (A-4a) with little clay, silt (A-4b) with varying amounts of clay, sand, and gravel, as well as silt and clay (A-6a) with varying amounts of sand and gravel. SPT N_{60}-values ranged from 18 to 50 bpf. Unconfined compressive strengths ranged from 6,000 psf to greater than 9,000 psf (the highest attainable reading using the hand penetrometer). Moisture contents generally ranged from 15 to 22 percent. Tested Stratum II samples from Borings B-002 and B-003 resulted in liquid limits ranging from 27 to 36 percent, and plasticity indices ranging from 2 to 14 percent. These values, along with gradation results, are indicative of silt (A-4b) as well as silt and clay (A-6a).

Stratum III consisted of predominantly stiff to very stiff cohesive soils encountered underlying the cohesive alluvial deposits in Boring B-005, as well as Stratum II in the remaining borings. Stratum III extended to termination at depths of 17½ feet or 30 feet. These cohesive soils consisted of silt and clay (A-6a) with varying amounts of sand and trace gravel. SPT N_{60}-values generally ranged from 11 to 28 bpf. Unconfined compressive strengths generally ranged from 2,000 to 5,000 psf. Moisture contents ranged from 15 to 19 percent.

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-004 and B-005 at depths of approximately 4½ feet and 5½ feet below existing grade, respectively. Groundwater was observed upon completion of drilling in these same two borings at depths of approximately 26 feet and 23 feet, respectively. Groundwater was not encountered during drilling or observed upon completion of drilling operations in the remaining borings. It should be noted that each boring was drilled and backfilled within the same day. As such, stabilized water levels may not have occurred over this limited time period. Instrumentation was not installed to observe long-term groundwater levels.

Based on the soil characteristics and groundwater conditions encountered in the borings, it is our opinion that the "normal" groundwater table will be generally encountered at a depth of approximately 11 feet or greater below existing grades for the portion of the project area south of Perry Street. Closer to Portage River, groundwater may be present shallower, likely meeting the river level along the shoreline. It should be noted that groundwater elevations can fluctuate with seasonal and climatic influences. In particular, "perched" water may be encountered in the granular alluvial deposits and granular existing fill materials. Therefore, the groundwater conditions may vary at different times of the year from those encountered during this exploration.

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 exploration. If the project information or location as previously described 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 Pavement Evaluation and Design

It is our understanding that the project plans call for the full-depth removal and replacement of the pavement along the project corridor. In some areas, this would require removal of asphalt and underlying brick pavers, while in other areas this would require removal of asphalt and underlying concrete. If roadway replacement will be performed north of Perry Street, only asphalt underlain by crushed stone was encountered in the borings performed in this area. As part of the removal and replacement process, pavement remediation will also require modification of any unstable subgrades, consisting of proof rolling or re-compaction of the granular base materials, and possibly undercutting of any marginal subgrade soils and replacement with dense-graded aggregate.

For the pavement reconstruction corridor between 3rd Street and Perry Street, it should be noted that **the thickness and type of the granular base course appears to be variable**, based on the limited borings performed during this exploration. Within the borings, the sand layer underlying concrete ranged from 2¼ to 4 inches in thickness, and the sand/gravel layer underlying brick was on the order of 6½ inches. While the thickness at the lower end of this range may have been considered adequate for placement and support of composite asphalt and concrete pavements in the past, this would generally be deemed marginal in terms of thickness for current design of typical municipal streets. Depending on final pavement design grades, additional stone could be added (replacing all or part of the “thickness” of the existing brick and concrete zones) to complete the new selected design pavement section.

5.1.1 Flexible (Asphalt) Pavement Design

ODOT Geotechnical Bulletin GB-1 "Plan Subgrades" (July 2018) was utilized to evaluate the subgrade soils at the site, as well as the subgrade design CBR value. It was assumed that the "subgrade" depth beneath the new asphalt and crushed stone pavement cross-section would be 1 foot below top of existing pavement. Based on the GB-1 analysis, a design CBR value of 6 percent was determined for the pavement reconstruction project corridor from 3rd Street to Perry Street using the data from Borings B-001 through B-003. This CBR is based on the "average" subgrade condition for soils with ODOT Group Indices (GI) ranging from 8 to 10.

If pavement reconstruction will extend north of Perry Street, GB-1 analyses indicate a design CBR value of 13 percent in this area due to the encountered granular fill materials at the anticipated subgrade elevations in Borings B-004 and B-005. Design of pavement sections north of Perry Street can consider this CBR value, but should cohesive subgrade soils be encountered in during construction, they would require over-excavation a minimum of 12 inches below top of subgrade elevation. Otherwise, the CBR value presented above for the project corridor from 3rd Street to Perry Street could be also utilized for the portion of the project area north of Perry Street such that over-excavation of cohesive soils would not be required unless they were unstable.

The CBR value(s) presented above are 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 in accordance with the "Site and Subgrade Preparation" section of this report.

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.

5.1.2 Pavements (General)

All paving operations should conform to 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 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 weather conditions.

It is recommended that proof-rolling/compaction, placement of aggregate base, and placement of asphalt or concrete be performed within as short a time period as possible. Exposure of the aggregate base to rain, snow, or freezing conditions may lead to deterioration of the subgrade and/or aggregate base due to excessive moisture conditions and to difficulties in achieving the required compaction.

Based on the poorly-drained nature of the cohesive subgrade 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. A system of “finger drains” should also be installed near catch basins within the pavement areas to collect surface water infiltration, thus reducing the potential for adverse freeze-thaw effects on the pavement.

5.2 Subgrades

5.2.1 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 suitable for support of the proposed pavements, although some subgrade areas may be appreciably wet of optimum. Based on field and laboratory data developed during this exploration, the subgrade soils along the pavement reconstruction project corridor from 3rd Street to Perry Street consist of predominantly native cohesive soils. Laboratory analyses performed for samples from Borings B-1 (SS-2), B-2 (SS-2 and SS-3), and B-3 (SS-2), as well as visual descriptions of the upper profile soils, indicate that the upper profile cohesive subgrade soils may be generally classified as Group A-6a (silt and clay) or Group A-4b (silt) in accordance with the ODOT system of soil classification. 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. In particular, ODOT A-4b soils are susceptible to frost heaving and are recommended by ODOT to be removed where present within 36 inches of top of subgrade. Based on the borings, the A-4b soils are generally anticipated to be present at least 36 inches below top of subgrade, considering top of subgrade 1 foot below existing top of pavement.

At the time of this exploration, the moisture contents determined for the cohesive subgrade soils in the upper 6 feet of the subsurface profile generally ranged from 16 to 27 percent. These moisture contents are estimated to range from near to appreciably above the optimum moisture content (OMC) for these soils. Remedial action may be required to adjust the moisture contents of the existing materials to achieve proper compaction of the subgrade soils, especially if construction is performed during a particularly wet seasonal period.

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. The method of subgrade modification should be determined at the time of construction (See Section 6.1, "Construction Recommendations - Site and Subgrade Preparation").

If pavement reconstruction will be performed north of Perry Street, granular fill materials are anticipated at pavement subgrade elevations based on Borings B-004 and B-005. The granular soils are considered generally suitable for subgrade support. However, they may require in-place re-compaction, and may be significantly wet of optimum. Saturated granular soils may require significant dewatering measures, or removal and replacement with new granular engineered fill.

5.2.2 GB-1 "Plan Subgrades" Evaluation

An evaluation of the existing subgrade soils at each of the boring locations was completed in general accordance with ODOT Geotechnical Bulletin GB-1 "Plan Subgrades" (July 2018), albeit with limited laboratory testing compared to standard ODOT GB-1 guidelines. As part of this evaluation, the ODOT "Subgrade Analysis" worksheet (V14.3) was completed, and is attached to this report. As mentioned in Section 5.1.1, for our evaluations, we have assumed that the new pavement section will be on the order of 12 inches in thickness, such that the subgrade will consist of those soils encountered in the borings at a depth of 1 foot below top of existing pavement. Due to varying subgrade soil conditions in Borings B-001 through B-003 compared to the conditions encountered in Borings B-004 and B-005, separate GB-1 evaluations were performed for the conditions from 3rd Street to Perry Street, and the conditions north of Perry Street.

The subgrade materials encountered during this exploration between 3rd Street and Perry Street were found to consist of predominantly ODOT A-6a soils, along with A-4b soils at depths greater than 3 feet below top of subgrade elevation. North of Perry Street, granular subgrade soils were encountered, consisting of ODOT A-1-b, A-2-4, and A-3a. Based on GB-1, soils classified as ODOT A-4b, A-2-5, A-5, A-7-5, A-8a, A-8b, or rock have been designated as being problematic with respect to pavement subgrade support. Of these soil types, only A-4b soils were encountered in the borings performed for this exploration. However, they were encountered at depths that do not necessarily require modification per GB-1 criteria. In any case, if A-4b soils are encountered during construction at subgrade elevations, they would require removal to 36 inches below the subgrade elevation.

Moisture contents for 17 of the 20 evaluated samples from the upper 6 feet of the subgrade soils were greater than 3 percent higher than optimum as determined using GB-1 criteria. These elevated moisture contents were recorded in each of the borings. Based on GB-1 criteria, subgrade soils with moisture contents greater than 3 percent above optimum are likely to require modification. Therefore, the subgrade soils may warrant modification according to GB-1 criteria based on moisture contents. In fact, all but one of the tested samples with moisture contents greater than 3 percent above optimum had moisture contents greater than or equal to 5 percent above optimum. Thus, where moisture contents were wet of optimum, they were appreciably wet of optimum. These data indicate that scarification and aeration methods may not be feasible to achieve satisfactory proof rolling and stabilization of the predominantly cohesive subgrades. However, scarification and aeration methods may be utilized in areas where granular subgrades wet of optimum are present, provided weather conditions and construction schedule will allow such soil modification. Additionally, dewatering operations may be required for removal of free water from saturated granular soils, which were encountered north of Perry Street in closer proximity to the Portage River.

The type and thickness of subgrade modification is determined by GB-1 criteria based on the average, low SPT N_{60}-value (N_{60L}) of the subgrade soils in a particular portion of the project area, soil type, and moisture content (relative to estimated optimum moisture content). Based on these criteria, each of the borings contained subgrade soils which indicated the potential for subgrade modification. Using GB-1 criteria based on the encountered conditions, possible alternatives for modification of the subgrade soils could include:

- For granular subgrade soils, scarification and re-compaction,
- For cohesive subgrade soils, undercut and replacement with granular engineered fill, or 14 inches of global chemical stabilization using cement.

A summary of the GB-1 recommended depths of undercut are presented in the following table.

Table 5.2.2. GB-1 Subgrade Analysis Indicated Undercut Depths

Boring Number	Approximate Location along Jefferson Street	GB-1 Recommended Depth of Undercut and Replacement with Granular Engineered Fill (inches)
B-001	Approximately 40 feet north of 3rd Street	12
B-002	Approximately 100 feet north of 2nd Street	24
B-003	Approximately 50 feet south of Perry Street	18
B-004	Approximately 45 feet north of Perry Street	Re-Compact Granular Soils
B-005	Approximately 250 feet north of Perry Street	Re-Compact Granular Soils

For Boring B-002, GB-1 indicated an undercut on the order of 33 inches for the soils encountered in sample SS-1. However, these problematic soils were encountered only to a depth of 24 inches below anticipated top of subgrade elevation, and were underlain by soils indicated by GB-1 to be suitable. As such, 24 inches of undercut is indicated in the above table.

Based on the GB-1 analysis, each of the borings indicate subgrade treatment is required. GB-1 indicates that, if it is determined that 30 percent or more of the subgrade area must be stabilized, consideration should be given to stabilizing the entire project (global stabilization). Given the comparatively short length of the project, the likelihood that the construction sequencing may not provide large contiguous areas for economical chemical stabilization, and the possibility that soil conditions during construction demonstrate to be more favorable during proof-rolling operations than projected, it is our recommendation that undercutting and replacement using granular engineered fill material be considered for subgrade modification, as needed.

Where undercut and replacement is utilized, all fill should consist of ODOT Item 304 Aggregate Base or Item 703.16C, Granular Material Type B or Type C. It is recommended that, geotextile fabric (referenced in ODOT Item 204, and specified as ODOT Item 712.09, Type D) be utilized on the subgrade at the bottom of the undercut zone. If particularly unstable subgrades are encountered during construction, a geogrid could be used to reduce the total undercut and replacement of the unsuitable soils.

It should be noted that GB-1 analyses are used as a pre-construction tool to plan subgrade modification alternatives. **Actual subgrade modification will depend on field observations of proof-rolling conditions at the time of construction.** Changes in soil moisture content could create more or less favorable subgrade conditions that may result in adjustments to subgrade modification or soil stabilization requirements at the time of construction.

5.3 Subsurface Utility Support

Details of the proposed utility improvements were not provided at the time of preparing this report. However, waterlines are expected to bear at 4 to 5 feet below grade, while storm sewers and sanitary sewers may extend as deep as 15 feet below grades.

For utilities bearing at these depths, the subsoils encountered during this exploration consisted of predominantly stiff to hard cohesive soils. These soils are considered generally suitable for pipe support, provided that sufficient bedding and haunching is maintained below and above the proposed utility lines. Based on the borings performed north of Perry Street, pipe installations may encounter **very loose** to medium dense water-bearing granular soils at the pipe invert elevations. Dewatering, as discussed in Section 5.6, and in-place densification using a backhoe-mounted vibratory compactor (hoe-pac) should be anticipated for these pipe support soils.

The bedding and haunching should consist of properly placed aggregate in accordance with the pipe manufacturer's recommendations or specifications. In the absence of specific requirements, we recommend that bedding or haunching consist of ODOT Item 304 crushed stone, or in areas of saturated soils or minor seepage conditions, No. 57 or 67 stone may be utilized. If unsuitable soft soils or loose granular soils that cannot be suitably re-compacted are encountered at pipe invert elevations, undercutting and replacement with additional bedding stone may be required.

5.4 Pump Station

5.4.1 Foundations

We understand that the pump station will bear at a depth of approximately 20 feet or less below existing grade. For our evaluations, we have assumed a foundation bearing at approximately 18 to 20 feet below existing grades. Based on the results of the field and laboratory testing in Boring B-004, the soils encountered at the anticipated foundation bearing depths are expected to consist of medium stiff to hard native cohesive soils, which are generally suitable for support of the proposed structure.

Due to its location below the groundwater table, the foundation subgrade may be prone to disturbance and loss of subgrade strength, particularly if construction occurs during a wet seasonal period. As such, the contractor should be prepared to use a thin “mud mat” of lean concrete, or alternately, an undercut and replacement stone layer, to maintain a stable bottom of excavation for placement of slab reinforcing steel and concrete. If groundwater seepage and its associated flow gradients are not controlled, it is likely that the foundation subgrade will become unstable and the bearing capacity will be compromised. Additionally, due to the water-bearing upper-profile granular soils, steel sheet piling extending into the underlying cohesive soils may be required as a groundwater cut-off. Use of sheet piling would also reduce the required lateral extent of excavation. Additional discussions regarding dewatering and excavations are provided in Section 5.7 and Section 5.6, respectively.

Following satisfactory completion of the site preparation and excavation inspections, the proposed pump station may be supported on a spread foundation. It is our experience that these types of structures are often nearly “floating” structures for which foundation bearing pressures due to the new loads do not exceed those soil pressures associated with the removal of the overlying soils. If loads are such that the pump station is essentially a “floating” structure, settlement should be negligible. This would be the case for an average bearing pressure of not more than 1,775 pounds per square foot (psf) at the B-004 location or 1,265 psf at the B-005 location.

If average bearing pressures are greater than those indicated above for the “floating” negligible settlement condition, the spread foundation may be designed using a gross allowable bearing pressure capacity of 3,000 psf at the B-004 location or 3,500 psf at the B-005 location, based on a nominal factor of safety of 3 applied to the theoretical bearing capacity. If using these bearing pressures for design, the foundation bearing materials should be field verified as consisting of native lean clay (CL) with a minimum unconfined compressive strength of 2,000 psf at the B-004 location or 2,500 psf at the B-005 location.

The gross allowable bearing pressures presented above are greater than the effective overburden stress associated with the soil that will be removed to construct the pump station. However, due to the appreciable depth of overburden soil being removed to construct the structure, the effective or net pressure contribution increase to the bearing soils is only approximately 1,225 psf at the B-004 location and 2,235 psf at the B-005 location. The net increase in pressure on the bearing soils will be responsible for settlement, which was calculated to be less than 1 inch assuming a maximum 20 feet by 20 feet foundation.

Upon completion of the pump station excavation, we recommend that the soils be inspected by a TTL geotechnical engineer or qualified representative. The purpose of this inspection is to verify that the exposed soil and groundwater conditions at the bearing elevation are consistent with the subsurface conditions encountered in the test borings. Additionally, the presence of our engineer will help facilitate the timely remediation of unsuitable soil conditions. If overexcavation and backfill is required, the area should also be inspected during undercutting and backfilling operations to provide verification that engineered fill has been properly placed and compacted.

The pump station foundation should not be placed on subgrade that has been left open to ponded water in the excavation. If the bearing surface becomes unstable or excessively saturated the bearing soils should be over-excavated. The base of the over-excavation should be widened one foot for every foot of depth and centered along the footing. The over-excavated areas should be backfilled with dense-graded aggregate in controlled lifts and compacted to not less than 100 percent of the maximum dry density as determined by ASTM D 698 (Standard Proctor). For particular soft or wet subgrades, an initial lift of open-graded stone may be required to provide a stable base on which to compact the dense-graded aggregate and to provide a working platform. In this circumstance, the minimum undercut should be 18 inches, to allow for a minimum thickness of open-graded stone to provide a stable base of 12 inches, with a minimum of 6 inches of dense-graded aggregate placed overlying the open-graded stone.

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.

Consideration should be given to buoyancy to evaluate whether the structure, when operating empty, will remain stable under high water conditions. When considering buoyancy of the structure, the weight of the structure, including the spread foundation, can be used to resist uplift. In addition, a side wall friction factor of 0.30 may be utilized along the face of the vertical concrete walls to mobilize sliding resistance between the installed structure and the backfill, conservatively assuming predominantly cohesive soils based on the encountered soil profile in the borings. If needed, the bottom slab and spread foundations can be widened beyond the pump station wall lines to mobilize the weight of the associated backfill to help resist buoyancy. In this case, use of side friction along the wall of the structure would not be incorporated into uplift resistance.

5.4.2 Mat Foundation Modulus

Generally, mat foundations are designed using a modulus of subgrade reaction (k). In addition, mat foundations are typically designed using finite element method (FEM) analyses or similar methodologies that allow for evaluations of contact pressure, deflection, shear and bending moment for structural reinforcement determinations, and thickness/rigidity considerations. For mat design, we recommend a subgrade modulus (k) of 75 pounds per cubic inch (pci).

The modulus of subgrade reaction value indicated above is based on a unit k-value (k_{v1} or k_{s1}) assuming an equivalent 1-foot by 1-foot plate load test. Depending on the method of analysis used to model the mat, a correction or adaptation is typically made to the k_{v1} modulus value based on the width and shape of the loaded area, as well as whether the bearing soils are sands or clays. Care should be taken by the structural designer to understand whether the analytical input requires the k_{v1} or k_{s1} modulus value based on a 1-foot by 1-foot plate, or the modulus of subgrade reaction (k_s), sometimes identified as k_b, which is a corrected value based on foundation width B. For foundations bearing on clays, k_s for a full-sized footing or mat is equal to k_{v1}/B. For a mat foundation, this B may not be the entire width of the mat, but the effective width of where the mat is acted upon by line loads or point loads spaced a distance B apart. For typical mat design that does not have uniform load intensity, the point loads or line loads and the associated shear and moment distribution in the mat will result in zones where deflection is at or near zero, and the effective width can be taken as the distance between these zones of "zero deflection." This is valid as long as the contact pressures associated with the areas of concentrated loads are less than ½ of the ultimate bearing capacity of the soil, the latter of which is independent of foundation width. For the anticipated design loads associated with the proposed pump station, this contact pressure criterion is expected to be met (i.e., less than ½ of the ultimate bearing capacity of the soil).

We recommend that the design of the mat consider what is the actual effective width B of the foundation (in this case, B is taken as 10 feet), but in no case incorporate a k_s or k_b value less than 10 pci. The design should also consider that the contact pressure is not likely to be uniform within all areas of the mat, and deflection may not be uniform unless the mat is indeed a rigid structural element. In the case of non-uniform contact pressure, localized areas of allowable bearing pressure (based on ½ of the ultimate bearing capacity) could be as high as 4,500 psf, assuming the resultant shear and moment could be accommodated in the mat design.

With respect to determination of k_s, it is difficult for the geotechnical engineer to determine accurate elastic design parameters for the soil (i.e., E_s, p, or k_s) as applicable to design of a large structural mat. It is our experience that bending moments and computed soil pressures are usually not very sensitive to k_{v1} values or k_b values because the structural member (concrete mat) stiffness or rigidity is generally much greater than the soil stiffness as measured by k of the subgrade.

Regarding subgrade stiffness and mat design, the American Concrete Institute (ACI) recognizes that the structural designer and geotechnical engineer may do a parametric study, varying the value of k_s over a range of one-half the furnished value up to five times this value. The results of the parametric study should be reviewed by the geotechnical engineer during the course of the design. If no satisfactory solution is found, then adjustments in the development concept may be appropriate. Adjustments to the mat design may include enlarging the mat in plan or deepening the mat base to reduce the net applied pressure. Such adjustments should be made with the concurrence of the geotechnical engineer. During the final design stage, TTL would be pleased to review analyses and coordinate such efforts with the structural engineer.

5.5 Lateral Earth Pressure

5.5.1 Cohesive Soil Model

Based on the conditions encountered in the borings performed within the new roadway reconstruction project area from 3rd Street to Perry Street, the soils along lateral bracing for excavation support are anticipated to consist of predominantly native cohesive soils. The borings performed north of Perry Street for the proposed pump station encountered upper-profile granular soils underlain by cohesive soils. Below-grade walls for the pump station and lateral bracing for excavation support in this area could conservatively be modeled using a simplified subsoils condition based on cohesive soils.

Below-grade walls are anticipated to be restrained from rotation and are considered rigid and non-yielding. As such, lateral earth pressures should be assumed for “at-rest” conditions. For the encountered subsurface soils, an at-rest lateral earth pressure coefficient (k_o) of 0.5 should be used along with a total soil unit weight of 140 pounds per cubic foot (pcf) in determining the lateral pressure acting on the walls. Alternately, an equivalent fluid weight of 70 pcf may be used for the at-rest case design.

For retaining walls and temporary sheetpile walls that are not restrained from rotation, lateral earth pressures should be assumed for “active” conditions. For the encountered subsurface soils, an active lateral earth pressure coefficient (k_a) of 0.33 should be used along with a total soil unit weight of 140 pounds per cubic foot (pcf) in determining the lateral pressure acting on the walls. Alternately, an equivalent fluid weight of 50 pcf may be used for the active case design.

5.5.2 Layered Soil Model

As indicated in the previous section, granular soils and granular fill materials were encountered in the upper soil profile of the pump station borings performed between Perry Street and the Portage River. Lateral earth pressure design for these structures may consider lower lateral earth pressures than what would be determined from modeling strictly cohesive soils such as discussed in Section 5.5.1.

Below-grade walls are anticipated to be restrained from rotation and are considered rigid and non-yielding. As such, lateral earth pressures should be assumed for “at-rest” conditions. For retaining walls and temporary sheetpile walls that are not restrained from rotation, lateral earth pressures should be assumed for “active” conditions. Based on Borings B-004 and B-005 at the potential pump station locations, lateral earth pressure design parameters are presented in the following table.

Table 5.5.2. Layered Lateral Earth Pressure Design Parameters							
Soil Layer	**Total Unit Weight (pcf)**	**At-Rest Case**		**Active Case**		**Design Elevation**	
		Lateral Earth Pressure Coefficient (k_o)	**Equivalent Fluid Weight (pcf)**	**Lateral Earth Pressure Coefficient (k_a)**	**Equivalent Fluid Weight (pcf)**	**B-004**	**B-005**
Granular Materials	120	0.50	60	0.33	40	575 – 569	575 – 561
Cohesive	140	0.50	70	0.33	50	569-	561-

Since the granular materials are generally loose to medium dense, and the cohesive soils are generally medium stiff to very stiff, the recommended design earth pressure coefficients are taken to be similar. Inasmuch as the unit weights vary by 20 pcf, design using a "single-layer" cohesive soil model may be only somewhat more conservative than design using a layered approach soil model, particularly for the B-004 location where the granular soils do not extend as deep as at the B-005 location.

5.5.3 General

For below-grade wall design considerations, if lower lateral earth pressures are preferred for structural design considerations, a select granular backfill material should be specified, and earth pressure coefficients can be adjusted accordingly. However, this would require a substantial wedge of granular fill.

Additionally, lateral loading due to hydrostatic pressures below the design groundwater depth should be included in design of below-grade walls and retaining walls. Depending on the design methodology, total lateral pressures would be the resultant of the hydrostatic pressures in combination with submerged (or "effective") unit weights of the soil. Effective unit weights of 80 pcf and 60 pcf should be used for cohesive soils and granular soils, respectively, for lateral earth pressure design below the design groundwater depth.

If design of below-grade walls will consider the normal Portage River level in the project vicinity and the 100-year flood elevation for the Portage River, structural load factors and geotechnical factors of safety (if pertinent to the design methodology) may consider the flood elevation as the "unusual" groundwater condition with a lesser load factor or lower required factor of safety than for the normal river level as the "normal design-basis" groundwater elevation. We would be pleased to review any geotechnical aspects of applicability of geotechnical load-resistance factors or geotechnical factors of safety in the context of your design.

It should be noted that the above k-parameters in the preceding sections may be used for general design of subsurface structures, retaining walls, and excavation support systems associated with the project. However, certain types of braced excavations may account for method-specific earth pressure distributions, for which the above parameters should be reviewed and utilized in the proper context of the design method/system.

A passive earth pressure coefficient (k_p) of 3.0 may be utilized for the portion of temporary walls (e.g., sheet pile walls) that is below the excavation bottom. In the case of permanent structures, a k_p value of 3.0 should only be utilized below the frost depth of 3½ feet below toe grades. It should be noted that some wall movement or horizontal displacement is typically needed to mobilize the full passive pressure of the soil.

It should also be noted that the earth pressure coefficients in the preceding sections are based on a level backfill condition behind the retaining wall. In areas where appreciable sloping materials are present behind the top of the wall, surcharge loading or equivalent higher earth pressure coefficients should be evaluated, based on the sloping material, backfill slope, and proximity to the wall. In general, 50 percent of the vertical surcharge load should be used for lateral loading in the design of the wall.

5.6 Open-Cut Excavations

The sides of the temporary excavations for utility line installation should be adequately sloped to provide stable sides and safe working conditions. Otherwise, the excavation must be properly braced against lateral movements.

Due to the required depth of excavation below the groundwater table for the proposed pump station, as well as the presence of upper profile saturated granular soils, we anticipate use of sheet-pile cutoff walls as the optimal method to manage groundwater and control of seepage gradients, as well as to avoid an excessively large, open excavation.

Design of sheet-pile cutoff walls should be the responsibility of the contractor, since their installation and performance is integrally tied to the contractor's means and methods of construction. In any case, applicable Occupational Safety and Health Administration (OSHA) standards must be followed. It is the responsibility of the installation contractor to develop appropriate installation methods and equipment specifications prior to commencement of work, and to obtain the services of a qualified engineer to design or approve sloped or benched excavations and/or lateral bracing systems as required by OSHA criteria. **In addition, OSHA requires that excavations with open-cut slopes higher than 20 feet, or braced excavation support systems such as sheetpiling or cofferdams be reviewed and designed by a registered professional engineer.**

If the excavation is to be performed with sloped banks, adequate stable slopes must be provided. Based on the soil conditions encountered in the test borings, utility excavations may encounter the following types of 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 (fill materials and granular soils).

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 for 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, grades should be no steeper than 3H:1V.

If a portable trench box (also known as a sliding trench shield) system is utilized, vertical side slopes may be used up to 18 inches below the top of the shield. The sides should be sloped from that point to the ground surface in accordance with the criteria presented in the preceding paragraph.

Construction traffic and excavated material stockpiles should be kept away from the excavation a minimum distance equal to the full depth of the excavation. The construction excavation should not be left open any longer than necessary. As soon as a section of the utility installation is completed, the area should be backfilled to final grade.

We emphasize the need for placing the fill in lifts and compacting each lift to the specified density, especially where the trenches and excavations will be directly beneath roadway pavement. The installation contractor should not be allowed to push or end-dump several feet of backfill into the trench as a single layer or lift, because the lower portion of a thick lift will not achieve significant densification from compaction equipment operating at the surface of that lift. Utility trenches and excavations beneath roadways should be backfilled with ODOT 304 aggregate for the full depth of excavation to avoid post-construction roadway settlement.

5.7 Groundwater Control and Drainage

As stated previously, groundwater was initially encountered during drilling in Borings B-004 and B-005 at depths of approximately 4½ feet and 5½ feet below existing grade, respectively. Groundwater was observed upon completion of drilling in these same two borings at depths of approximately 26 feet and 23 feet, respectively. Groundwater was not encountered during drilling or observed upon completion of drilling operations in the remaining borings. Based on the soil characteristics and groundwater conditions encountered in the borings, it is our opinion that the "normal" groundwater table will be generally encountered at a depth of 11 feet or greater below existing grades for the portion of the project area south of Perry Street. Closer to Portage River, groundwater may be present shallower, likely meeting the river level along the shoreline.

It should be noted that "perched" water may be encountered in the granular alluvial deposits and granular existing fill materials. It is our experience that adequate control of groundwater seepage, perched water, or surface water run-off into shallow excavations in predominantly cohesive soil profiles should be achievable by minor dewatering systems, such as pumping from prepared sumps.

For pump station installation, due to the depth of excavation below the groundwater depth and the presence of upper profile water-bearing granular soils, we anticipate that the most effective means of temporary groundwater management and control of seepage gradients in the bottom of the excavation will consist of a system of sheetpiling driven into the underlying clays acting as a cut-off wall, in conjunction with a prepared sump-and-pump operation in the bottom of the excavation.

Where excavations extend below ambient groundwater conditions, there is potential for clayey soils to become soft when saturated and/or exposed to seepage pressures. If diligence and care is taken to maintain a stable subgrade upon excavation, significant modification of the bearing surface is not likely to be required. If the excavation will remain open for a period of time prior to installation of slabs, a mud mat should be placed at the base of the excavation to maintain a suitable working surface. If soft or saturated cohesive soils are encountered, or if seepage and surface runoff result in an unstable excavation bottom, the subgrade will need to be undercut and replaced with granular fill to provide a firm stratum on which to construct the structure slabs. It is our experience that the undercut will need to be a minimum of 12 inches to provide a stable bridging layer of granular material. Additional discussion regarding granular engineered fill placement to maintain a stable working subgrade is presented in Section 5.4.1 for the pump station.

In the event excessive seepage is encountered during construction, TTL should be notified to evaluate whether other dewatering methods are required.

6.0 CONSTRUCTION RECOMMENDATIONS

6.1 Site and Subgrade Preparation

For the new pavements to be located along Jefferson Street, site preparation activities should include the removal of existing pavements, brick, concrete, and other deleterious non-soil materials from all proposed roadway areas.

Upon completion of pavement, brick, and concrete removal, 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 subgrade soils, which should be performed with a 20- to 30-ton loaded truck or other pneumatic-tired vehicle of similar size and weight. Proof rolling/compaction of the granular subgrades should be performed using a vibratory, smooth- drum roller. The roller or truck should make a minimum of two passes covering the proposed development area, with additional passes as necessary to achieve required compaction and/or subgrade stabilization.

The purpose of proof rolling is to locate any weak, soft, loose, or excessively wet soils that may be present at the time of construction. The purpose of vibratory compaction for the 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.

Depending on construction sequence and incorporation of the existing aggregate base or granular fill into the new pavement section, the proof-rolling/compaction operations may be performed on a comparatively thin layer of granular material. If the underlying subgrade is found to be unstable, it will be necessary to remove this granular zone as part of the undercut and replacement discussed in Section 5.2.2 of this report.

Once the proof-rolling operations are completed to demonstrate the stability of the subgrade, and/or subgrade undercuts and replacement are completed, any remaining aggregate base layer(s) should be re-compacted utilizing a vibratory smooth-drum roller.

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. 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 may 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.

6.2 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.

As mentioned in Section 5.6, utility trenches and excavations beneath roadways should be backfilled with ODOT 304 aggregate for the full depth of excavation to avoid post-construction roadway settlement.

Fill should be placed in uniform layers not more than 8 inches thick (loose measure) and adequately keyed into stripped and scarified soils. All fill within pavement subgrades should be compacted to a density of not less than 100 percent of the maximum dry density as determined by ASTM D 698 (Standard Proctor).

The on-site soils consist of predominantly cohesive soils. The contractor should be prepared to use a sheepsfoot roller to compact the on-site cohesive soils. Compaction for aggregate base and existing granular materials should be performed with a vibratory smooth-drum roller. In narrow utility excavations, the on-site clays 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.

6.3 Foundation Excavations

As mentioned in Section 5.4.1, pump station foundations 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, 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, or over-excavation and replacement with granular engineered fill be performed to reduce damage to the surface from weather or construction. Foundation concrete should not be placed on frozen or saturated subgrade.

Additional pump station foundation subgrade inspection and preparation recommendations are provided in Section 5.4.1.

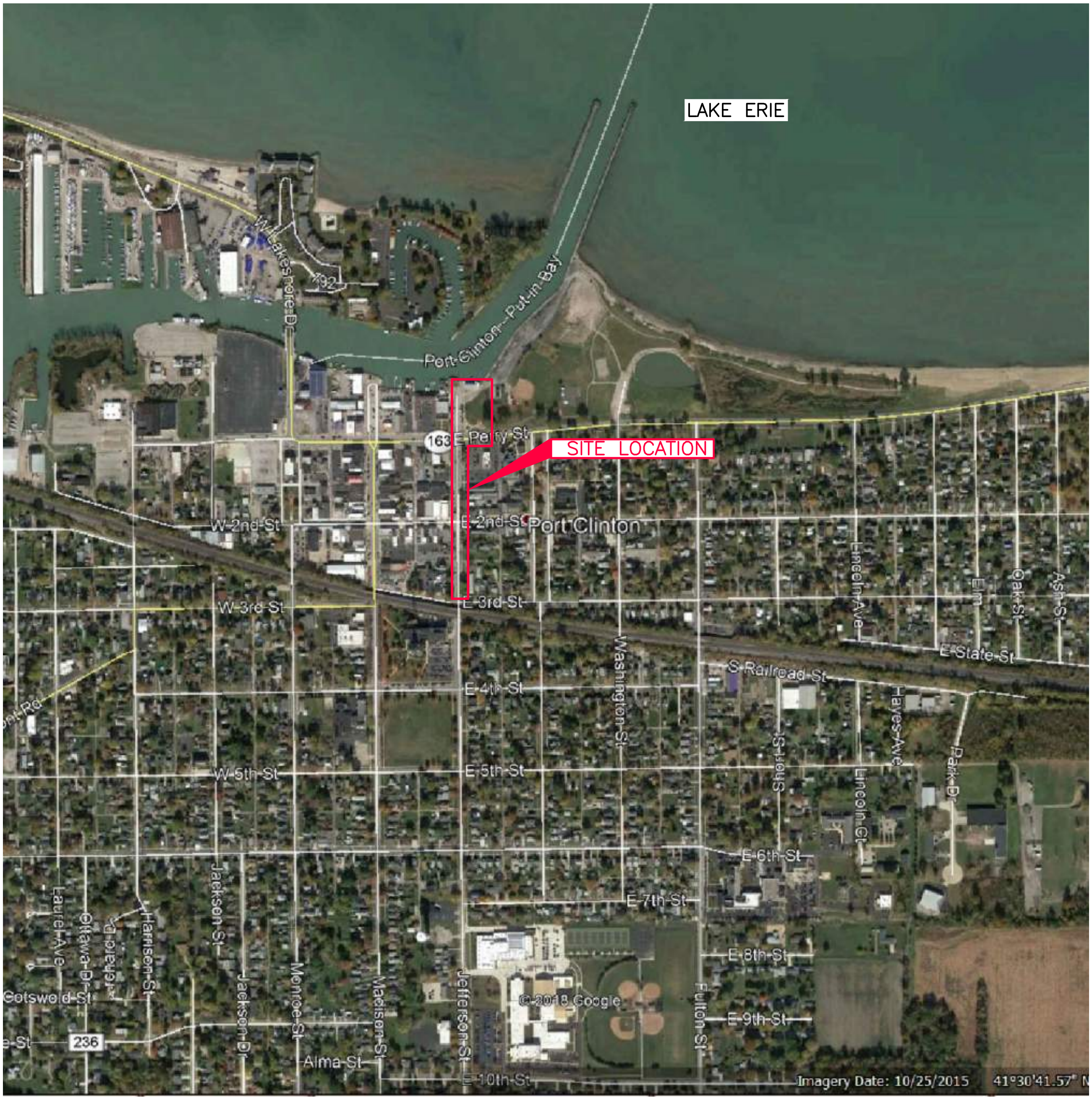
7.0 QUALIFICATION OF RECOMMENDATIONS

Our evaluation of soils-related pavement, subsurface utility, and pump station design and construction conditions has been based on our understanding of the site and project information, and the data obtained during our field exploration. The general subsurface conditions were based on interpretation of the subsurface data obtained at specific boring locations. Regardless of the thoroughness of a subsurface exploration, 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. This is especially true for previously developed sites. Therefore, experienced geotechnical engineers should observe earthwork 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 1,000 2,000

PLATE 1.0
SITE LOCATION MAP

PROPOSED JEFFERSON STREET RECONSTRUCTION
PORT CLINTON, OHIO

PREPARED FOR
CITY OF PORT CLINTON
PORT CLINTON, OHIO

DRAWN	TRR/7-31-18	CHECKED	CAS/8-1-18
REVISED		APPROVED	
JOB NO.	1654701		
DRAWING NUMBER	**1654701-01G**		

TTL associates inc
Environmental, Geotechnical Engineering & Testing

BASE IMAGE RETRIVED FROM GOOGLE EARTH 2018.

LEGEND

B-1 ⊕ APPROXIMATE TEST BORING LOCATION

APPROXIMATE SCALE – FEET

0 150 300

PLATE 2.0
TEST BORING LOCATION PLAN

PROPOSED JEFFERSON STREET RECONSTRUCTION
PORT CLINTON, OHIO

PREPARED FOR
CITY OF PORT CLINTON
PORT CLINTON, OHIO

DRAWN	TRR/7-31-18	CHECKED	CAS/8-1-18
REVISED		APPROVED	
JOB NO.	1654701		
DRAWING NUMBER	1654701-02G		

TTL associates inc
Environmental, Geotechnical Engineering & Testing

PROJECT: OTT - JEFFERSON STREET TYPE: ROADWAY PID: 106850 SFN: N/A START: 7/24/18 END: 7/24/18	DRILLING FIRM / OPERATOR: TTL / CW SAMPLING FIRM / LOGGER: TTL / KKC DRILLING METHOD: 3.5" SSA SAMPLING METHOD: SPT	DRILL RIG: CME 75 TRUCK 111 HAMMER: CME AUTOMATIC CALIBRATION DATE: 2/8/18 ENERGY RATIO (%): 75.4	STATION / OFFSET: ALIGNMENT: JEFFERSON STREET ELEVATION: 577.0 (NAVD88) EOB: 17.5 ft. COORD: Not Recorded

EXPLORATION ID: B-001-0-18
PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 577.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
ASPHALT - 4 INCHES	576.7																	
BRICK - 4 INCHES	576.3	1																
SAND AND GRAVEL - 6.5 INCHES	575.8	2	4 4 4	10	89	SS-1	3.00	-	-	-	-	-	-	-	-	20	A-6a (V)	
STIFF, GRAY/BROWN, **SILT AND CLAY**, SOME SAND AND TRACE GRAVEL, DAMP @3': MOIST		3–4	4 4 7	14	100	SS-2	NI	1	7	17	25	50	36	22	14	24	A-6a (10)	
	572.0																	
VERY STIFF TO HARD, GRAY/BROWN, **SILT**, SOME CLAY, LITTLE SAND, AND TRACE GRAVEL, DAMP		5–6	7 9 10	24	100	SS-3	4.25	-	-	-	-	-	-	-	-	16	A-4b (V)	
		6–7	9 14 14	35	100	SS-4	4.00	-	-	-	-	-	-	-	-	16	A-4b (V)	
		8																
		9–10	5 9 13	28	100	SS-5	4.50	-	-	-	-	-	-	-	-	15	A-4b (V)	
	565.2	11																
STIFF TO VERY STIFF, GRAY, **SILT AND CLAY**, LITTLE SAND AND TRACE GRAVEL, DAMP		11–12	10 14 14	35	100	SS-6	2.50	-	-	-	-	-	-	-	-	17	A-6a (V)	
		13																
		14–15	3 4 5	11	100	SS-7	2.00	-	-	-	-	-	-	-	-	15	A-6a (V)	
		16–17	5 6 8	18	100	SS-8	1.75	-	-	-	-	-	-	-	-	16	A-6a (V)	
	559.5	EOB																

NOTES: "NI" - NOT INTACT

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; AUGER CUTTINGS MIXED WITH 1 BAG BENTONITE CHIPS

PROJECT: OTT - JEFFERSON STREET	DRILLING FIRM / OPERATOR: TTL / CW	DRILL RIG: CME 75 TRUCK 111	STATION / OFFSET:	EXPLORATION ID B-002-0-18
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: TTL / KKC	HAMMER: CME AUTOMATIC	ALIGNMENT: JEFFERSON STREET	
PID: 106850 SFN: N/A	DRILLING METHOD: 3.5" SSA	CALIBRATION DATE: 2/8/18	ELEVATION: 577.0 (NAVD88) EOB: 17.5 ft.	PAGE 1 OF 1
START: 7/24/18 END: 7/24/18	SAMPLING METHOD: SPT	ENERGY RATIO (%): 75.4	COORD: Not Recorded	

MATERIAL DESCRIPTION AND NOTES	ELEV. 577.0
ASPHALT - 1.5 INCHES	576.9
CONCRETE - 8 INCHES	576.2
SAND - 2.25 INCHES	576.0
MEDIUM STIFF, BROWN, **SILT AND CLAY**, TRACE SAND, DAMP	574.0
VERY STIFF, GRAY/BROWN, **SILT AND CLAY**, SOME SAND AND TRACE GRAVEL, DAMP	572.5
VERY STIFF TO HARD, GRAY/BROWN, **SILT**, "AND"CLAY, TRACE SAND, GRAVEL, DAMP	570.5
HARD, BROWN, **SANDY SILT**, LITTLE CLAY, DAMP	568.0
VERY STIFF TO HARD, BROWN, **SILT**, SOME CLAY, LITTLE SAND, TRACE GRAVEL, DAMP	566.0
HARD, GRAY/BROWN, **SILT**, SOME CLAY, LITTLE SAND, AND TRACE GRAVEL, DAMP	563.2
STIFF TO VERY STIFF, GRAY, **SILT AND CLAY**, LITTLE SAND AND TRACE GRAVEL, DAMP	559.5

EOB

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)
3 2 2	5	89	SS-1	3.00	-	-	-	-	-	-	-	-	19	A-6a (V)
5 6 8	18	100	SS-2	4.25	3	5	17	25	50	35	21	14	20	A-6a (10)
7 10 11	26	100	SS-3	3.50	1	2	7	53	37	27	25	2	21	A-4b (8)
13 19 17	45	100	SS-4	NI	-	-	-	-	-	-	-	-	19	A-4a (V)
6 11 17	35	100	SS-5	3.00	-	-	-	-	-	-	-	-	17	A-4b (V)
17 21 19	50	100	SS-6	4.50	-	-	-	-	-	-	-	-	17	A-4b (V)
5 5 6	14	100	SS-7	2.00	-	-	-	-	-	-	-	-	17	A-6a (V)
6 7 7	18	100	SS-8	1.50	-	-	-	-	-	-	-	-	17	A-6a (V)

NOTES: "NI" - NOT INTACT

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; AUGER CUTTINGS MIXED WITH 1 BAG BENTONITE CHIPS

PROJECT: OTT - JEFFERSON STREET	DRILLING FIRM / OPERATOR: TTL / CW	DRILL RIG: CME 75 TRUCK 111	STATION / OFFSET:
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: TTL / KKC	HAMMER: CME AUTOMATIC	ALIGNMENT: JEFFERSON STREET
PID: 106850 SFN: N/A	DRILLING METHOD: 3.5" SSA	CALIBRATION DATE: 2/8/18	ELEVATION: 576.0 (NAVD88) EOB: 17.5 ft.
START: 7/24/18 END: 7/24/18	SAMPLING METHOD: SPT	ENERGY RATIO (%): 75.4	COORD: Not Recorded

EXPLORATION ID B-003-0-18

PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 576.0	DEPTHS	SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
ASPHALT - 1.25 INCHES	575.9																
CONCRETE - 7.25 INCHES	575.3	1															
SAND - 4 INCHES	575.0	2	2 3 3	8	89	SS-1	2.75	-	-	-	-	-	-	-	-	24	A-6a (V)
MEDIUM STIFF, BROWN, **SILT AND CLAY**, TRACE SAND, MOIST	573.5	3	6 8 10	23	100	SS-2	3.75	0	2	8	25	65	36	22	14	20	A-6a (10)
VERY STIFF, GRAY/BROWN, **SILT AND CLAY**, LITTLE SAND, DAMP	572.0	4–5	6 8 12	25	100	SS-3	3.00	-	-	-	-	-	-	-	-	22	A-6a (V)
VERY STIFF, BROWN, **SILT AND CLAY**, TRACE SAND, DAMP TO MOIST	570.0	6	11 12 13	31	100	SS-4	3.00	-	-	-	-	-	-	-	-	27	A-4b (V)
VERY STIFF TO HARD, BROWN, **SILT**, SOME CLAY AND LITTLE SAND, MOIST @6.8': VERY STIFF, BROWN/GRAY, TRACE GRAVEL, DAMP		7–10	12 11 9	25	100	SS-5	3.50	-	-	-	-	-	-	-	-	18	A-4b (V)
VERY STIFF, GRAY, **SILT AND CLAY**, LITTLE SAND AND TRACE GRAVEL, DAMP	564.5	11–12	7 8 8	20	100	SS-6	NI	-	-	-	-	-	-	-	-	17	A-6a (V)
@14': STIFF, GRAY		13–15	3 4 5	11	100	SS-7	1.25	-	-	-	-	-	-	-	-	18	A-6a (V)
@16': STIFF TO VERY STIFF, SOME SAND	558.5	16–17 EOB	7 7 9	20	100	SS-8	1.50	-	-	-	-	-	-	-	-	17	A-6a (V)

NOTES: "NI" - NOT INTACT

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; AUGER CUTTINGS MIXED WITH 1 BAG BENTONITE CHIPS

PROJECT: OTT - JEFFERSON STREET
TYPE: ROADWAY
PID: 106850 SFN: N/A
START: 7/23/18 END: 7/23/18

DRILLING FIRM / OPERATOR: TTL / CW
SAMPLING FIRM / LOGGER: TTL / KKC
DRILLING METHOD: 3.25" HSA
SAMPLING METHOD: SPT / ST

DRILL RIG: CME 75 TRUCK 111
HAMMER: CME AUTOMATIC
CALIBRATION DATE: 2/8/18
ENERGY RATIO (%): 75.4

STATION / OFFSET:
ALIGNMENT: JEFFERSON STREET
ELEVATION: 575.0 (NAVD88) EOB: 30.0 ft.
COORD: Not Recorded

EXPLORATION ID B-004-0-18
PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 575.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
ASPHALT - 8 INCHES	574.3																
CRUSHED STONE - 10 INCHES	573.5																
MEDIUM DENSE, BROWN, **CRUSHED STONE WITH SAND**, DAMP **FILL**	571.5		4 5 4	11	89	SS-1	NP	-	-	-	-	-	-	-	-	13	A-1-b (V)
MEDIUM DENSE, BROWN, **COARSE AND FINE SAND**, LITTLE SILT, GRAVEL, AND TRACE CLAY, WET (FREE WATER NOTED) @4.8': (FREE WATER NOTED)		W 570.2	4 5 6	14	89	SS-2	NP	17	22	41	19	1	NP	NP	NP	27	A-3a (0)
	568.5		4 5 5	13	89	SS-3	NP	-	-	-	-	-	-	-	-	17	A-3a (V)
MEDIUM STIFF, GRAY/BROWN, **SILT AND CLAY**, LITTLE SAND AND TRACE GRAVEL, DAMP TO MOIST	566.5		2 2 2	5	100	SS-4	1.00	-	-	-	-	-	-	-	-	22	A-6a (V)
VERY STIFF TO HARD, BROWN, **SILT AND CLAY**, LITTLE SAND AND TRACE GRAVEL, DAMP			6 11 14	31	100	SS-5	3.25	-	-	-	-	-	-	-	-	19	A-6a (V)
@11': GRAY/BROWN	562.0		6 10 15	31	100	SS-6	3.00	-	-	-	-	-	-	-	-	20	A-6a (V)
MEDIUM STIFF TO STIFF, GRAY, **SILT AND CLAY**, SOME SAND AND TRACE GRAVEL, DAMP			3 4 5	11	100	SS-7	1.00	-	-	-	-	-	-	-	-	18	A-6a (V)
					33	ST-8	1.59*	-	-	-	-	-	-	-	-	16	A-6a (V)
@18': LITTLE SAND			5 6 6	15	100	SS-9	1.00	-	-	-	-	-	-	-	-	18	A-6a (V)
					0	ST-10	NR	-	-	-	-	-	-	-	-	-	A-6a (V)
@23.5': SOME SAND, WET (FREE WATER NOTED)	549.0	▼ 549.0	4 5 6	14	100	SS-11	0.94*	-	-	-	-	-	-	-	-	18	A-6a (V)
STIFF TO VERY STIFF, GRAY, **SILT AND CLAY**, SOME SAND AND TRACE GRAVEL, DAMP			7 8 10	23	100	SS-12	1.50	-	-	-	-	-	-	-	-	17	A-6a (V)
@28.5': STIFF	545.0	EOB 30	4 5 7	15	100	SS-13	1.00	-	-	-	-	-	-	-	-	19	A-6a (V)

NOTES: "*" - UNCONFINED STRENGTH DETERMINED BY ASTM D 2166. "NP" - NON PLASTIC "NR" - NO RECOVERY

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; AUGER CUTTINGS MIXED WITH 2 BAGS BENTONITE CHIPS

PROJECT: OTT - JEFFERSON STREET	DRILLING FIRM / OPERATOR: TTL / CW	DRILL RIG: CME 75 TRUCK 111	STATION / OFFSET:
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: TTL / KKC	HAMMER: CME AUTOMATIC	ALIGNMENT: JEFFERSON STREET
PID: 106850 SFN: N/A	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 2/8/18	ELEVATION: 575.0 (NAVD88) EOB: 30.0 ft.
START: 7/23/18 END: 7/23/18	SAMPLING METHOD: SPT	ENERGY RATIO (%): 75.4	COORD: Not Recorded

EXPLORATION ID B-005-0-18

PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 575.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
ASPHALT - 3 INCHES	574.7																	
CRUSHED STONE - 15 INCHES (WET)	573.5																	
MEDIUM DENSE, DARK BROWN, **GRAVEL WITH SAND AND SILT**, DAMP **FILL**	572.0		7 7 7	18	72	SS-1	NP	-	-	-	-	-	-	-	-	12	A-2-4 (V)	
LOOSE, BLACK, **GRAVEL WITH SAND**, SOME SILT, TRACE CLAY, WET **FILL** (FREE WATER NOTED)			3 2 2	5	89	SS-2	NP	40	21	15	23	1	NP	NP	NP	42	A-1-b (0)	
@5.5': LITTLE CINDERS, COAL, METAL, (ODOR AND FREE WATER NOTED)		W 569.5	3 1 1	3	89	SS-3	NP	-	-	-	-	-	-	-	-	54	A-1-b (V)	
@6': (ODOR AND FREE WATER NOTED)	566.5		1 1 1	3	89	SS-4	NP	-	-	-	-	-	-	-	-	34	A-1-b (V)	
VERY LOOSE, GRAY, **FINE SAND**, TRACE GRAVEL AND SILT, WET **FILL** (FREE WATER NOTED)	564.0		1 1 1	3	11	SS-5	NP	-	-	-	-	-	-	-	-	76	A-3 (V)	
VERY LOOSE, BLACK/BROWN, **COARSE AND FINE SAND**, LITTLE SILT, GRAVEL, AND CINDERS, WET **FILL** (FREE WATER NOTED)	561.0		1 1 1	3	78	SS-6	NP	-	-	-	-	-	-	-	-	59	A-3a (V)	
MEDIUM STIFF, BLACK/GRAY, **SILT AND CLAY**, SOME SAND AND LITTLE GRAVEL, WET (FREE WATER NOTED)	559.0		0 2 3	6	100	SS-7	NI	-	-	-	-	-	-	-	-	22	A-6a (V)	
STIFF, GRAY, **SILT AND CLAY**, SOME SAND AND TRACE GRAVEL, DAMP			3 4 5	11	100	SS-8	1.21*	-	-	-	-	-	-	-	-	17	A-6a (V)	
@18.5': STIFF TO VERY STIFF	555.0		4 6 9	19	100	SS-9	1.27*	-	-	-	-	-	-	-	-	15	A-6a (V)	
VERY STIFF TO HARD, GRAY, **SILT AND CLAY**, SOME SAND AND TRACE GRAVEL, DAMP	551.5	551.7	8 11 14	31	100	SS-10	2.00	-	-	-	-	-	-	-	-	15	A-6a (V)	
STIFF, GRAY, **SILT AND CLAY**, LITTLE SAND AND TRACE GRAVEL, DAMP			8 4 7	14	100	SS-11	1.50	-	-	-	-	-	-	-	-	18	A-6a (V)	
@26': STIFF TO VERY STIFF			8 10 12	28	100	SS-12	1.75	-	-	-	-	-	-	-	-	18	A-6a (V)	
	545.0	EOB 30	4 6 10	20	100	SS-13	2.00	-	-	-	-	-	-	-	-	15	A-6a (V)	

NOTES: "*" - UNCONFINED STRENGTH DETERMINED BY ASTM D 2166. "NI" - NOT INTACT "NP" - NON PLASTIC

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; AUGER CUTTINGS MIXED WITH 2 BAGS BENTONITE CHIPS

LEGEND KEY

Ohio Department of Transportation Soil Symbols

A-1-a - Gravel and/or Stone Fragments	A-1-b - Gravel and/or Stone Fragments with Sand	A-2-4,A-2-5 - Gravel and/or Stone Fragments with Sand and Silt	A-2-6,A-2-7 - Gravel and/or Stone Fragments with Sand, Silt and Clay
A-3 - Fine Sand	A-3a - Coarse and Fine Sand	A-4a - Sandy Silt	A-4b - Silt
A-5 - Elastic Silt and Clay	A-6a - Silt and Clay	A-6b - Silty Clay	A-7-5 - Elastic Clay
A-7-6 - Clay	A-8a - Organic Silt	A-8b - Organic Clay	Asphalt
Sod and/or Topsoil	Concrete	Random Fill	Peat
Dolomite	Weathered Dolomite	Limestone	Weathered Limestone
Sandstone	Weathered Sandstone	Shale	Weathered Shale

Notes:

1. Pavement cores and exploratory borings were performed on July 23 and 24, 2018, using a 4-inch diameter pavement core barrel, 3½-inch outside diameter solid stem augers, as well as 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. The borings were field located by TTL Associates, Inc. based on direction from CT Consultants. Ground surface elevations presented on the boring logs were estimated from Google Earth.

4. Unconfined Compressive Strength (tsf):
   NI = Not Intact.
   NP = Non-Plastic.
   NR = No Recovery.
   "*" = Unconfined compressive strength per ASTM D 2166.

OHIO DEPARTMENT OF TRANSPORTATION

OFFICE OF GEOTECHNICAL ENGINEERING

PLAN SUBGRADES
Geotechnical Bulletin GB1

OTT-Jefferson Street Reconstruction
PID 106850
Third Street to Perry Street
Port Clinton, Ohio

TTL Associates, Inc.

Prepared By: Katherine C. Hennicken, P.E.
Date prepared: Thursday, August 09, 2018

Katherine C. Hennicken, P.E.
TTL Associates, Inc.
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Toledo, Ohio 43604
419-324-2222
khennicken@ttlassoc.com

NO. OF BORINGS: **3**

#	Boring ID	Alignment	Station	Offset	Dir	Drill Rig	ER	Boring EL.	Proposed Subgrade EL	Cut Fill
1	B-001-0-18	Just North of 3rd St.				CME 75 Truck 111	75	577.0	576.0	1.0 C
2	B-002-0-18	Just North of 2nd St.				CME 75 Truck 111	75	577.0	576.0	1.0 C
3	B-003-0-18	Just South of Perry St.				CME 75 Truck 111	75	576.0	575.0	1.0 C

OHIO DEPARTMENT OF TRANSPORTATION

Subgrade Analysis

V. 14.3 7/20/2018

#	Boring	Sample	Sample Depth		Subgrade Depth		Standard Penetration		HP (tsf)	Physical Characteristics						Moisture		Ohio DOT		Sulfate Content (ppm)	Problem		Excavate and Replace (Item 204)		Recommendation
			From	To	From	To	N_{60}	N_{60L}		LL	PL	PI	% Silt	% Clay	P200	M_C	M_{OPT}	Class	GI		Unsuitable	Unstable	Unsuitable	Unstable	
1	B	SS-1	1.2	3.0	0.2	2.0	10		3							20	14	A-6a	10			N_{60} & Mc		12"	U.C. 12"
	001-0	SS-2	3.0	5.0	2.0	4.0	14		NI	36	22	14	25	50	75	24	17	A-6a	10						
	18	SS-3	5.0	6.0	4.0	5.0	24		4.25							16	10	A-4b	8						
		SS-4	6.0	8.5	5.0	7.5	35	10	4							16	10	A-4b	8						
2	B	SS-1	1.0	3.0	0.0	2.0	5		3							19	14	A-6a	10			N_{60} & Mc		33"	U.C. 24"
	002-0	SS-2	3.0	4.5	2.0	3.5	18		4.25	35	21	14	25	50	75	20	16	A-6a	10			Mc			
	18	SS-3	4.5	6.5	3.5	5.5	26		3.5	27	25	2	53	37	90	21	20	A-4b	8						
		SS-4	6.5	9.0	5.5	8.0	45	5	NI							19	10	A-4b							
3	B	SS-1	1.0	2.5	0.0	1.5	8		2.75							24	14	A-6a	10			N_{60} & Mc		18"	U.C. 18"
	003-0	SS-2	2.5	4.0	1.5	3.0	23		3.75	36	22	14	25	65	90	20	17	A-6a	10			Mc			
	18	SS-3	4.0	6.0	3.0	5.0	25		3							22	14	A-6a	10						
		SS-4	6.0	8.5	5.0	7.5	31	8	3							27	10	A-4b	8						

TLL Project No. 1654701
OTT-Jefferson Street Reconstruction, PID 106850
Third Street to Perry Street
Port Clinton, Ohio

PID: PID 106850

County-Route-Section: OTT-Jefferson Street Reconstruction
No. of Borings: 3

Geotechnical Consultant: TTL Associates, Inc.
Prepared By: Katherine C. Hennicken, P.E.
Date prepared: 8/9/2018

Chemical Stabilization Options		
320	Rubblize & Roll	No
206	Cement Stabilization	Option
	Lime Stabilization	No
206	Depth	14''

Excavate and Replace Stabilization Options	
Global Geotextile Average(N60L): Average(HP):	15'' 0''
Global Geogrid Average(N60L): Average(HP):	0'' 0''

Design CBR	6

% Samples within 6 feet of subgrade			
$N_{60} \leq 5$	8%	HP ≤ 0.5	0%
$N_{60} < 12$	25%	0.5 < HP ≤ 1	0%
$12 \leq N_{60} < 15$	8%	1 < HP ≤ 2	0%
$N_{60} \geq 20$	58%	HP > 2	83%
M+	42%		
Rock	0%		
Unsuitable	42%		

Excavate and Replace at Surface	
Average	0''
Maximum	0''
Minimum	0''

% Proposed Subgrade Surface	
Unstable & Unsuitable	83%
Unstable	83%
Unsuitable	0%

	N_{60}	N_{60L}	HP	LL	PL	PI	Silt	Clay	P 200	M_C	M_{OPT}	GI
Average	22	8	3.45	34	23	11	32	51	83	21	14	9
Maximum	45	10	4.25	36	25	14	53	65	90	27	20	10
Minimum	5	5	2.75	27	21	2	25	37	75	16	10	8

Classification Counts by Sample

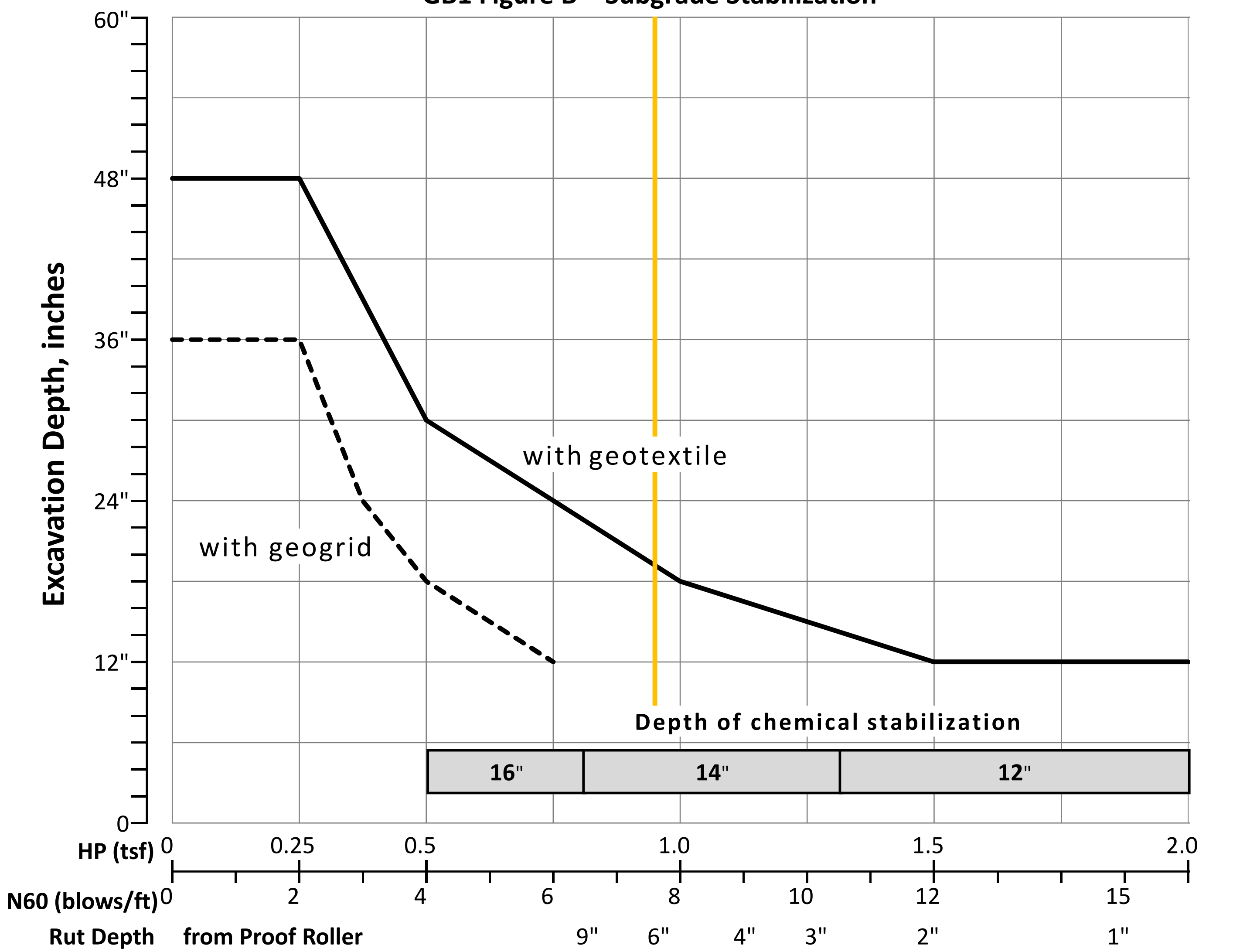
ODOT Class	Rock	A-1-a	A-1-b	A-2-4	A-2-5	A-2-6	A-2-7	A-3	A-3a	A-4a	A-4b	A-5	A-6a	A-6b	A-7-5	A-7-6	A-8a	A-8b	Totals
Count	0	0	0	0	0	0	0	0	0	0	5	0	7	0	0	0	0	0	12
Percent	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	42%	0%	58%	0%	0%	0%	0%	0%	100%
% Rock|Granular|Cohesive	0%	0%									100%								100%
Surface Class Count	0	0	0	0	0	0	0	0	0	0	0	0	6	0	0	0	0	0	6
Surface Class Percent	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	100%	0%	0%	0%	0%	0%	100%

OHIO DEPARTMENT OF TRANSPORTATION

Subgrade Analysis

V. 14.3 7/20/2018

GB1 Figure B – Subgrade Stabilization



OVERRIDE TABLE

Calculated Average	New Values	Check to Override
3.45		☐ HP
7.67		☐ N60L

TLL Project No. 1654701
OTT-Jefferson Street Reconstruction, PID 106850
Third Street to Perry Street
Port Clinton, Ohio

OHIO DEPARTMENT OF TRANSPORTATION

OFFICE OF GEOTECHNICAL ENGINEERING

PLAN SUBGRADES
Geotechnical Bulletin GB1

OTT-Jefferson Street Reconstruction
PID 106850
North of Perry Street
Port Clinton, Ohio

TTL Associates, Inc.

Prepared By: Katherine C. Hennicken, P.E.
Date prepared: Thursday, August 09, 2018

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NO. OF BORINGS: **2**

#	Boring ID	Alignment	Station	Offset	Dir	Drill Rig	ER	Boring EL.	Proposed Subgrade EL	Cut Fill
1	B-004-0-18	Just North of Perry St.				CME 75 Truck 111	75	575.0	574.0	1.0 C
2	B-005-0-18	Near Existing Restroom Building				CME 75 Truck 111	75	575.0	574.0	1.0 C

OHIO DEPARTMENT OF TRANSPORTATION

Subgrade Analysis

V. 14.3 7/20/2018

#	Boring	Sample	Sample Depth		Subgrade Depth		Standard Penetration		HP (tsf)	Physical Characteristics						Moisture		Ohio DOT		Sulfate Content (ppm)	Problem		Excavate and Replace (Item 204)		Recommendation
			From	To	From	To	N_{60}	N_{60L}		LL	PL	PI	% Silt	% Clay	P200	M_C	M_{OPT}	Class	GI		Unsuitable	Unstable	Unsuitable	Unstable	
1	B	SS-1	1.5	3.5	0.5	2.5	11		NP							13	6	A-1-b	0						Recompact
	004-0	SS-2	3.5	4.5	2.5	3.5	14		NP	NP	NP	NP	19	1	20	27	8	A-3a	0						
	18	SS-3	4.5	6.5	3.5	5.5	13		NP							17	8	A-3a	0						
		SS-4	6.5	8.5	5.5	7.5	5	5	1							22	14	A-6a							
2	B	SS-1	1.5	3.0	0.5	2.0	18		NP							12	10	A-2-4	0						If Unstable, Recompact
	005-0	SS-2	3.0	5.5	2.0	4.5	5		NP	NP	NP	NP	23	1	24	42	6	A-1-b	0						
	18	SS-3	5.5	6.0	4.5	5.0	3		NP							54	6	A-1-b	0						
		SS-4	6.0	8.5	5.0	7.5	3	3	NP							34	6	A-1-b	0						

TLL Project No. 1654701
OTT-Jefferson Street Reconstruction, PID 106850
North of Perry Street
Port Clinton, Ohio

OHIO DEPARTMENT OF TRANSPORTATION

Subgrade Analysis

V. 14.3 7/20/2018

PID: PID 106850

County-Route-Section: OTT-Jefferson Street Reconstruction
No. of Borings: 2

Geotechnical Consultant: TTL Associates, Inc.
Prepared By: Katherine C. Hennicken, P.E.
Date prepared: 8/9/2018

Chemical Stabilization Options		
320	Rubblize & Roll	No
206	Cement Stabilization	No
	Lime Stabilization	Option
206	Depth	16''

Excavate and Replace Stabilization Options	
Global Geotextile Average(N60L): Average(HP):	 24'' 18''
Global Geogrid Average(N60L): Average(HP):	 18'' 0''

Design CBR	13

% Samples within 6 feet of subgrade			
$N_{60} \leq 5$	50%	$HP \leq 0.5$	0%
$N_{60} < 12$	63%	$0.5 < HP \leq 1$	13%
$12 \leq N_{60} < 15$	25%	$1 < HP \leq 2$	0%
$N_{60} \geq 20$	0%	$HP > 2$	0%
M+	0%		
Rock	0%		
Unsuitable	0%		

Excavate and Replace at Surface	
Average	0''
Maximum	0''
Minimum	0''

% Proposed Subgrade Surface	
Unstable & Unsuitable	0%
Unstable	0%
Unsuitable	0%

	N_{60}	N_{60L}	HP	LL	PL	PI	Silt	Clay	P 200	M_C	M_{OPT}	GI
Average	9	4	1.00				21	1	22	28	8	0
Maximum	18	5	1.00	0	0	0	23	1	24	54	14	0
Minimum	3	3	1.00	0	0	0	19	1	20	12	6	0

Classification Counts by Sample

ODOT Class	Rock	A-1-a	A-1-b	A-2-4	A-2-5	A-2-6	A-2-7	A-3	A-3a	A-4a	A-4b	A-5	A-6a	A-6b	A-7-5	A-7-6	A-8a	A-8b	Totals
Count	0	0	4	1	0	0	0	0	2	0	0	0	1	0	0	0	0	0	8
Percent	0%	0%	50%	13%	0%	0%	0%	0%	25%	0%	0%	0%	13%	0%	0%	0%	0%	0%	100%
% Rock|Granular|Cohesive	0%	88%									13%								100%
Surface Class Count	0	0	2	1	0	0	0	0	1	0	0	0	0	0	0	0	0	0	4
Surface Class Percent	0%	0%	50%	25%	0%	0%	0%	0%	25%	0%	0%	0%	0%	0%	0%	0%	0%	0%	100%

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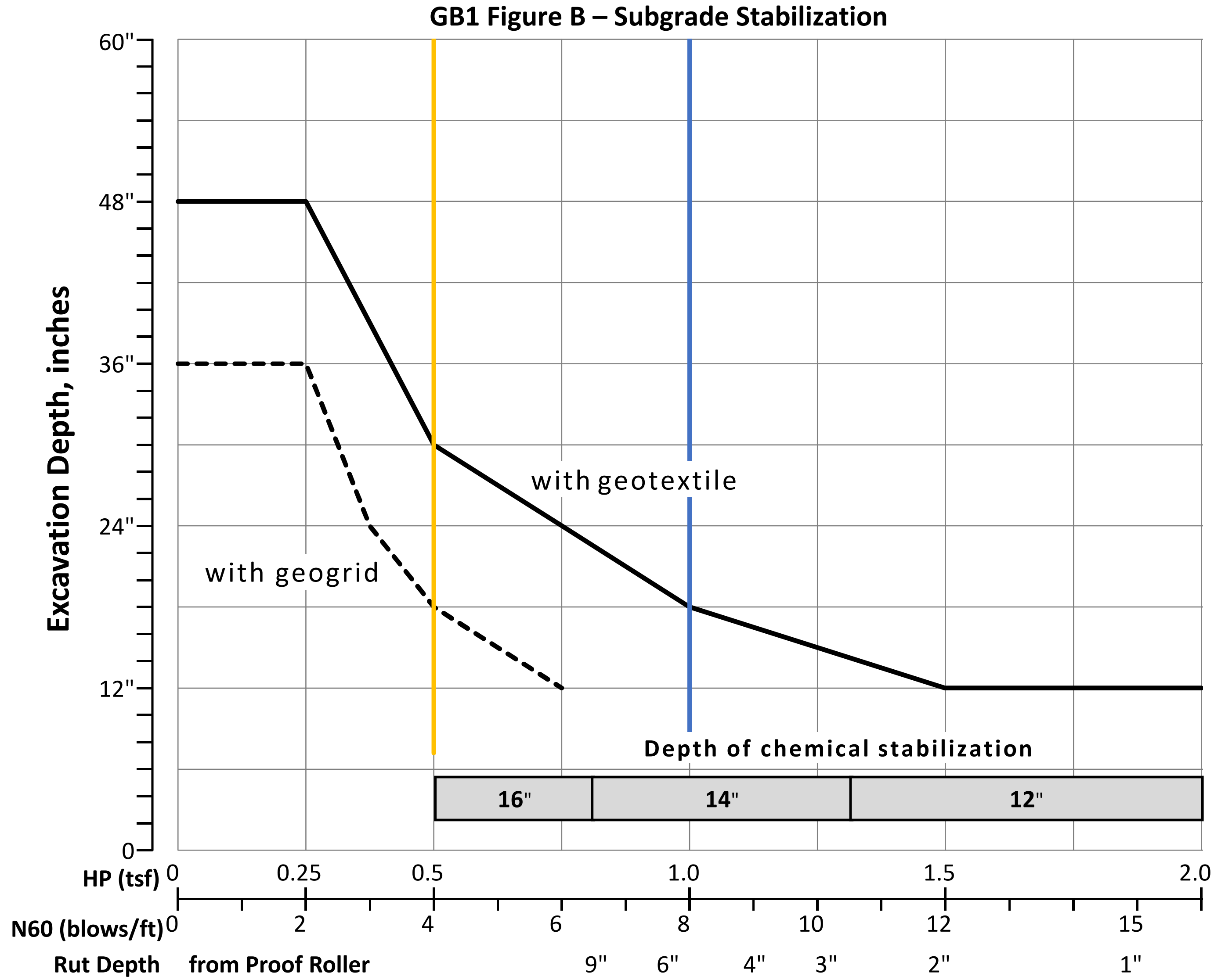
GB1 Figure B – Subgrade Stabilization



OVERRIDE TABLE

Calculated Average	New Values	Check to Override
1.00		☐ HP
4.00		☐ N60L

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CORE LOG for B-001-0-18

Project: OTT-Jefferson Street Reconstruction
Project Location: Port Clinton, Ohio
TTL Project No. 1654701
Core Date: July 24, 2018

ASPHALT THICKNESS (in)	=	4.0
BRICK THICKNESS (in)	=	4.0
CORE BARREL DIAMETER (in)	=	4.0

VISUAL DESCRIPTION:

CORE LOG for B-002-0-18

Project: OTT-Jefferson Street Reconstruction

Project Location: Port Clinton, Ohio

TTL Project No. 1654701

Core Date: July 24, 2018

ASPHALT THICKNESS (in)	=	1.5
CONCRETE THICKNESS (in)	=	8.0
CORE BARREL DIAMETER (in)	=	4.0

VISUAL DESCRIPTION:

3/16-inch steel bars at 4½ inches below top of concrete.

CORE LOG for B-003-0-18

Project: OTT-Jefferson Street Reconstruction
Project Location: Port Clinton, Ohio
TTL Project No. 1654701
Core Date: July 24, 2018

ASPHALT THICKNESS (in)	=	1.25
CONCRETE THICKNESS (in)	=	7.25
CORE BARREL DIAMETER (in)	=	4.0

VISUAL DESCRIPTION:

3/16-inch steel bars at 4¾ inches below top of concrete.

CORE LOG for B-004-0-18

Project: OTT-Jefferson Street Reconstruction

Project Location: Port Clinton, Ohio

TTL Project No. 1654701

Core Date: July 23, 2018

ASPHALT THICKNESS (in)	=	8.0
CORE BARREL DIAMETER (in)	=	4.0

VISUAL DESCRIPTION:

CORE LOG for B-005-0-18

Project: OTT-Jefferson Street Reconstruction
Project Location: Port Clinton, Ohio
TTL Project No. 1654701
Core Date: July 23, 2018

ASPHALT THICKNESS (in)	=	3.0
CORE BARREL DIAMETER (in)	=	4.0

VISUAL DESCRIPTION: