City of Oregon
Oregon, Ohio

FINAL Report
Structure Foundation Exploration
Proposed Bridge Replacement
LUC-Seaman Road, PID 107148
Seaman Road over Otter Creek
Oregon, Lucas County, Ohio

February 2020

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February 14, 2020

TTL Project No. 1686001

Mr. Rodney Shultz, P.E.
City of Oregon
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Oregon, Ohio 43616-2633

FINAL Report
Structure Foundation Exploration
Proposed Bridge Replacement
LUC-Seaman Road, PID 107148
Seaman Road over Otter Creek
Oregon, Lucas County, Ohio

Dear Mr. Shultz:

Following is the report of the structure foundation exploration performed by TTL Associates, Inc. (TTL) at the site of the referenced project. This exploration was performed in general accordance with TTL Proposal No. 1686001R, dated April 3, 2018, and was authorized with City of Oregon Purchase Order No. 16081-00, dated April 18, 2018.

This report was submitted as "Draft" on September 11, 2018 for review and comments by City of Oregon and/or ODOT. This 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.

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

Sincerely,

TTL Associates, Inc.

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Senior Geotechnical Engineer

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c.c.: Mr. David T. Charville, P.E. – Tetra Tech

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FINAL REPORT
STRUCTURE FOUNDATION EXPLORATION
PROPOSED BRIDGE REPLACEMENT
LUC-SEAMAN ROAD, PID 107148
SEAMAN ROAD OVER OTTER CREEK
OREGON, LUCAS COUNTY, OHIO

FOR

CITY OF OREGON
5330 SEAMAN ROAD
OREGON, OHIO 43616-2633

SUBMITTED

FEBRUARY 14, 2020
TTL PROJECT NO. 1686001

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EXECUTIVE SUMMARY

This structure foundation exploration report has been prepared for the replacement of the existing Seaman Road bridge over Otter Creek in Oregon, Lucas County, Ohio. This exploration included two structure borings. A summary of the conclusions and recommendations of this study are as follows:

1. The borings were performed within the roadway drive lanes. At the location of Boring B-1, the surface materials consisted of approximately 15 inches of asphalt underlain by approximately 20 inches of crushed stone. At the location of Boring B-2, the surface materials consisted of approximately 6 inches of asphalt underlain by 6 inches of crushed stone.

2. The subsoils encountered underlying the pavement materials can be generally described as six strata of cohesive soils exhibiting varying strength and moisture characteristics. Zones of granular soils were encountered within the predominantly cohesive soil profile. **Stratum I** consisted of medium stiff to stiff cohesive soils underlying the pavement materials in Borings B-001 and B-002 to a depth of 6 feet below existing grade (Elevs. 580± to 579±). **Stratum II** consisted of **soft** to medium stiff cohesive soils underlying Stratum I to depths of 13 feet and 16 feet (Elevs. 572± and 570±), respectively. A zone of **peat** was encountered within Stratum II in Boring B-002 from 6 to 8½ feet (Elevs. 580± to 577±). A zone of **loose** granular soils was encountered within Stratum II in Boring B-002 from 14½ to 16 feet (Elevs. 571± to 570±). **Stratum III** consisted of stiff to very stiff cohesive soils underlying Stratum II in Borings B-001 and B-002 to a depth of 23½ feet (Elev. 562±). **Stratum IV** consisted of medium stiff to stiff cohesive soils underlying Stratum III in Boring B-001 to a depth of 33½ feet (Elev. 552±). An approximately 2½ feet thick very dense granular soil zone was encountered within the Stratum IV layer in Boring B-002 from approximately 23½ feet (Elev. 562±) to the depth of encountered sanitary sewer in this boring at approximately 26 feet (Elev. 560±). **Stratum V** consisted of stiff to very stiff cohesive soils underlying Stratum IV in Boring B-001 to a depth of 48½ feet (Elev. 537±). **Stratum VI** consisted of cohesive "hardpan" underlying Stratum V in Boring B-001 to termination at a depth of approximately 59 feet (Elev. 526±).

3. It is our opinion that "normal" groundwater level at this structure location will generally occur at or slightly above the stream flow water level in Otter Creek, which was at a depth of approximately 9 feet below existing roadway grade (Elev. 576±) during our exploration. Temporary sheet-pile cutoff walls or cofferdams to direct streamflow may be required to manage groundwater in addition to pumping from prepared sumps.

4. The proposed structure is planned as a three-sided culvert supported on spread footings bearing at Elev. 571. Based on the conditions encountered in the borings, the soils at the indicated bearing elevation are anticipated to consist of **loose** granular soils or stiff to very stiff cohesive soils. The cohesive soils are considered generally suitable for support of spread footing foundations. Based on Boring B-002, the granular soils extended to Elev. 569.5±, corresponding to 1½ feet below planned bearing elevation. We recommend that granular soils be overexcavated and replaced with lean concrete or other flowable controlled-density fill. At the service limit state, we recommend a nominal (unfactored) bearing resistance (q_n) of 3 kips per square foot (ksf) 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 ksf. At the strength limit state, we recommend a nominal bearing resistance (q_n) of 8 ksf for 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 3.6 ksf.

5. Settlement of the culvert foundation was calculated by conventional consolidation theory utilizing the recompression index, based on empirical relations using moisture content. Based on the bearing pressure of 3 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 4 feet. If the foundation width is increased for overturning resistance, it is anticipated that the average bearing pressure would be less than 4 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.

This executive summary highlights our evaluations and recommendations and should only be utilized in conjunction with the accompanying report, including the detailed findings, analysis and recommendations, and qualifications presented herein.

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1.0 INTRODUCTION

This structure foundation exploration report has been prepared for the replacement of the existing Seaman Road bridge over Otter Creek in Oregon, Lucas County, Ohio. The project area is located approximately between Wheeling Street and Whittlesey Avenue, as shown on the Site Location Map (Plate 1.0).

This exploration was performed in general accordance with TTL Proposal No. 1686001R, dated April 3, 2018, and was authorized with City of Oregon Purchase Order No. 16081-00, dated April 18, 2018.

1.1 Purpose and Scope of Exploration

The purpose of this exploration was to evaluate the subsurface conditions relative to the design and construction of foundations for a new bridge structure. To accomplish this, TTL performed two test borings, field and laboratory soil testing, a geotechnical engineering evaluation of the test results, and a review of available geologic and soils data for the project area.

This report summarizes our understanding of the proposed construction, describes the investigative and testing procedures utilized to evaluate the subsurface conditions at the site, and presents our findings from the field and laboratory testing. This report also provides design and construction recommendations for foundations for the proposed bridge replacement structure.

This report includes:

- A description of the subsurface soils and groundwater conditions encountered in the borings.
- Design recommendations for foundations associated with the bridge replacement.
- Recommendations concerning soil- and groundwater-related construction procedures such as site preparation, earthwork, foundation construction, as well as related field testing.

Appendix B includes pertinent ODOT Geotechnical Engineering Design Checklists that apply to the scope of this report.

This exploration did not include an environmental assessment of the surface or subsurface materials at the site.

1.2 Proposed Construction

It is our understanding that the existing bridge will be replaced with a new three-sided culvert. Final roadway grades are anticipated to approximate existing grades. Shallow spread foundations were indicated to bear at approximately Elev. 571. Loads associated with the structure were not indicated.

2.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1 General Geology and Hydrogeology

Published geologic maps from the Ohio Department of Natural Resources (ODNR) Division of Geological Survey indicate that the project site is located in the Maumee Lake Plains Physiographic Region. This region is within the Huron-Erie Lake Plains Section.

Within the Maumee Lake Plains Physiographic Region, the upper profile geology includes predominantly Pleistocene-age silts and clays that were lake-laid (lacustrine) sediments, underlain by glacial till deposits, which are underlain by sedimentary bedrock. It should be noted that soils at the site also consist of alluvial deposits associated with Otter Creek.

The USDA Natural Resource Conservation Service (NRCS) Web Soil Survey indicates that soils in the project area are predominantly mapped as Toledo silty clay. These soils consist of lacustrine deposits. These soils are considered very poorly drained.

The lacustrine sediments were deposited in historic glacial lakes following retreat and melting of glacial ice. The glacial till, also referred to as moraine, was deposited by the advance and retreat of glacial ice. Due to the weight of the ice mass, the till deposits are moderately to highly over-consolidated, that is, the existing soil deposits have experienced a previous vertical stress significantly higher than the effective vertical stress presently caused by the remaining overlying soil strata in the profile. Additionally, within the glacial till, it is not uncommon to encounter cobbles, boulders, and seams of granular soils, which may or may not be water bearing.

Bedrock in the project area is broadly mapped on the "Geologic Map of Ohio" as Monroe limestone. Specific to the project site, the uppermost carbonate rock formation is mapped as Greenfield dolomite. Bedrock in the project vicinity is mapped at approximately Elev. 520.

2.2 Observations of the Project

TTL performed a site reconnaissance on May 2, 2018. The bridge abutments were scoured and cracked. Additionally, the bridge beams were cracked. Sewer outlets to Otter Creek were present south of Seaman Road. Sediment buildup was present along the west side under the bridge.

3.0 EXPLORATION

3.1 Historic Borings

A review of the ODOT Transportation Information Mapping System (TIMS) indicated no historic test borings were performed in the vicinity of the project structure.

3.2 Project Exploration Program

This exploration included two test borings, designated as Borings B-001-0-18 and B-002-0-18, drilled by Envirocore under the direction of TTL on May 30 and July 10, 2018, respectively. These borings are fully designated as Borings B-001-0-18 and B-002-0-18 in accordance with ODOT protocol, but the "-0-18" portion of the nomenclature is generally omitted for ease of identification in the discussions within this report. The borings were located in the field by TTL based on direction from City of Oregon. Boring B-001 was performed in the westbound lane, west of the bridge. Boring B-002 was performed in the eastbound lane, east of the bridge. The approximate locations of the borings are shown on the Test Boring Location Plan (Plate 2.0).

Ground surface elevations and coordinates for the as-drilled boring locations were surveyed by City of Oregon. Stations and offsets were provided by Tetra Tech. Stations, offsets, coordinates, ground surface elevations, as well as boring termination depths, are summarized in the following table.

Table 3.2 General Boring Information

Boring Number	Coordinates	Approximate Station (feet)	Approximate Offset (feet)	Ground Surface Elevation (feet)	Boring Termination Depth (feet)
B-001	724572.9 N, 1699172.0 E	14+46	11 LT	585.2	59.2[1]
B-002	724557.6 N, 1699241.0 E	15+15	6 RT	585.7	29.0[2]

[1]Terminated after encountering approximately 10 feet of hard cohesive soils.
[1]Terminated after penetrating an existing underground utility, later determined to be a sanitary sewer.

The borings were performed in general accordance with the ODOT Specifications for Geotechnical Explorations (SGE, 2007 and current revisions). Boring B-001 was terminated after extending through approximately 10 feet of hard cohesive soils. Boring B-002 was terminated after encountering a void from 26 to 29 feet, which was later determined to be an existing 54-inch diameter sanitary sewer. A temporary PVC pipe was installed in the B-002 borehole to maintain an open borehole until the sewer was repaired. Following repair of the sewer, the PVC pipe was removed and the borehole was sealed with cement-bentonite grout.

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 and inspection 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.

3.3 Boring Methods

The test borings performed during this exploration were drilled with a GeoProbe® 3230 with drilling capabilities and a Mobile B-57 drilling rig. The borings were extended utilizing 3¼-inch inside diameter hollow-stem augers. During borehole advancement, samples were generally taken at 2½-foot intervals to a depth of 30 feet and at 5-foot intervals thereafter. Continuous sampling to obtain additional samples for evaluation of potential scour was performed from 10 to 16 feet using 18-inch sample drives. The samples were sealed in jars and transported to our laboratory for further classification and testing.

Split-spoon (SS) soil samples were obtained by the Standard Penetration Test Method (ASTM D 1586). The Standard Penetration Test (SPT) consists of driving a 2-inch outside diameter split-spoon 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, 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 hammer/rod energy ratio was 82.5 percent for the GeoProbe® 3230 rig and 82.7 percent for the Mobile B-57 rig, respectively, which were utilized on this project. Calibration was performed for both rigs on February 23, 2018. The N_{60}-values are presented on the attached Logs of Test Borings, as well as the Tabulation of Test Data sheets attached to this report. In conjunction with published data and typical correlations, the N_{60}-values can be evaluated as a measure of soil compactness/consistency as well as shear strength and bearing capacity.

A Shelby tube sample, designated ST on the Log of Test Boring, was obtained in Boring B-001 (25 to 27 feet) 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.

Soil conditions encountered in the test borings are presented in the Logs of Test Borings along with information related to sample data, SPT results and corresponding N_{60}-values, water conditions observed in the borings, and laboratory test data. Field and laboratory data were incorporated into gINT™ software for presentation purposes. 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.

3.4 Laboratory Testing Program

All soil samples were visually or manually classified in accordance with the ODOT Soil Classification System. All samples of the subsoils were also tested in our laboratory for moisture content (ASTM D 2216). Atterberg limits tests (ASTM D 4318) and particle size analyses (ASTM D 422) were performed on selected samples to determine soil classification and index properties. Dry density determinations and unconfined compressive strength tests (ASTM D 2166) by the constant rate of strain method were performed on the Shelby tube sample and selected intact cohesive split-spoon samples. Unconfined compressive strength estimates were obtained for the remaining intact cohesive samples using a calibrated hand penetrometer. These test results are presented on the Logs of Test Borings and Tabulation of Test Data sheets attached to this report. Additionally, a stress-strain curve for the unconfined compressive strength test performed on the Shelby tube sample is attached to this report.

It should be noted that dry density determinations and unconfined compressive strength tests were performed on intact split-spoon (SS) samples as well as Shelby tube (ST) samples. Due to the sample size and sampling disturbance effects on split-spoon samples, dry density and unconfined compressive strengths may be less precise than for Shelby tube samples. Typically, unconfined compressive strength results from SS samples will tend to be lower than those from ST samples, and this trend is often more pronounced for soft clays than for stiff to hard clays. Nevertheless, test results from SS samples can be used as “index” parameters to compare and contrast soil properties within similar strata and between different strata.

4.0 FINDINGS

4.1 General Site Conditions

The project site is located along Seaman Road, at the crossing of Otter Creek, between Wheeling Street and Whittlesey Avenue, in Oregon, Lucas County, Ohio. Roadway grades in the project area are generally level, with ground surface elevations at the boring locations on the order of Elev. 586 to 585.

At the time of our site reconnaissance, the water level in the creek was approximately 9 feet below roadway grades, at Elev. 576±. The creek bottom was approximately 9½ feet below roadway grades, at Elev. 575±.

The borings were performed within the roadway. At the location of Boring B-1, the surface materials consisted of approximately 15 inches of asphalt underlain by approximately 20 inches of crushed stone. At the location of Boring B-2, the surface materials consisted of approximately 6 inches of asphalt underlain by 6 inches of crushed stone.

4.2 General Soil Conditions

The subsoils encountered underlying the roadway materials can be generally described as six strata of predominantly cohesive soils exhibiting varying strength and moisture characteristics. Zones of granular soils were encountered within the predominantly cohesive soil profile.

Stratum I consisted of predominantly medium stiff to stiff cohesive soils encountered underlying the pavement materials in Borings B-001 and B-002 to a depth of 6 feet below existing grade (Elevs. 580± to 579±). These cohesive soils consisted of silt and clay (ODOT A-6a) and sand, as well as silty clay (ODOT A-6b) with varying amounts of sand and gravel. SPT N_{60}-values ranged from 7 to 12 blows per foot (bpf). Unconfined compressive strengths were on the order of 3,000 pounds per square foot (psf). Moisture contents generally ranged from 20 to 22 percent.

Stratum II consisted of predominantly **soft** to medium stiff cohesive soils encountered underlying Stratum I to depths of 13 feet and 16 feet (Elevs. 572± and 570±), respectively. These cohesive soils consisted of silty clay (ODOT A-6b) with varying amounts of sand and gravel. SPT N_{60}-values ranged from 3 to 8 bpf. Unconfined compressive strengths generally ranged from 420 to 2,000 psf. Moisture contents ranged from 23 to 28 percent.

A zone of **peat** was encountered within Stratum II in Boring B-002 from 6 to 8½ feet (Elevs. 580± to 577±). An SPT N_{60}-value of 7 bpf and a moisture content of approximately 123 percent were determined for the recovered sample.

A zone of **loose** granular soils was encountered within Stratum II in Boring B-002 from 14½ to 16 feet (Elevs. 571± to 570±). The granular soils consisted of gravel with sand and silt (ODOT A-2-4) with trace clay. An SPT N_{60}-value of 6 bpf and a moisture content of approximately 16 percent were determined for the recovered sample.

Stratum III consisted of predominantly stiff to very stiff cohesive soils encountered underlying Stratum II in Borings B-001 and B-002 to a depth of 23½ feet (Elev. 562±). These cohesive soils consisted of silt and clay (ODOT A-6a) with varying amounts of sand and trace gravel, as well as silty clay (ODOT A-6b) with some sand and trace gravel. SPT N_{60}-values generally ranged from 12 to 21 bpf. Within the deeper portion of this stratum in Boring B-002, the SPT resulted in split-spoon refusal (SSR, 50 or more blows for 6 inches or less penetration), indicating very stiff to hard consistency. Unconfined compressive strengths ranged from 4,000 psf to greater than 9,000 psf (maximum reading obtainable with a hand penetrometer). Moisture contents ranged from 14 to 17 percent.

Stratum IV consisted of predominantly medium stiff to stiff cohesive soils encountered underlying Stratum III in Boring B-001 to a depth of 33½ feet (Elev. 552±). These cohesive soils consisted of silt and clay (ODOT A-6a) with some sand and trace gravel. SPT N_{60}-values ranged from 10 to 14 bpf. Unconfined compressive strengths ranged from 1,180 to 2,500 psf. Moisture contents were on the order of 18 to 19 percent.

An approximately 2½ feet thick very dense granular soil zone was encountered within the Stratum IV layer in Boring B-002 from approximately 23½ feet (Elev. 562±) to the depth of encountered sanitary sewer in this boring at approximately 26 feet (Elev. 560±). The granular soils consisted of coarse and fine sand (ODOT A-3a) with some gravel and little silt. Within this zone, the SPT resulted in SSR. A moisture content of 15 percent was determined for the recovered sample.

Stratum V consisted of stiff to very stiff cohesive soils encountered underlying Stratum IV in Boring B-001 to a depth of 48½ feet (Elev. 537±). These cohesive soils consisted of silt and clay (ODOT A-6a) with some sand and trace gravel. SPT N_{60}-values within this stratum were each 22 bpf. Unconfined compressive strengths ranged from 2,440 to 3,000 psf. Moisture contents ranged from 15 to 18 percent.

Stratum VI consisted of predominantly very stiff to hard cohesive soils (commonly referred to as "hardpan" in this region) encountered underlying Stratum V in Boring B-001 to termination at a depth of approximately 59 feet (Elev. 526±). These cohesive soils consisted of silty clay (ODOT A-6b) with little sand and trace gravel. The uppermost samples from this stratum exhibited SPT N_{60}-values on the order of 55 to 56 bpf. The SPT performed just prior to boring termination resulted in SSR. Unconfined compressive strengths ranged from 6,600 psf to greater than 9,000 psf. Moisture contents ranged from 9 to 19 percent.

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

4.3 Groundwater Conditions

As previously mentioned, at the time of our site reconnaissance, the water level in the creek was approximately 9 feet below roadway grades, at Elev. 576±. Groundwater was initially encountered during drilling in Boring B-002 at a depth of 14½ feet below existing grade (Elev. 571±). Groundwater was observed at the completion of drilling in Boring B-002 at a depth of 20 feet (Elev. 566±). Groundwater was not initially encountered during drilling or observed upon completion of drilling operations in Boring B-002. It should be noted that the Boring B-001 borehole was drilled and backfilled within the same day. Therefore, 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 moisture conditions encountered in the borings, it is our opinion that "normal" groundwater levels at this structure location will generally occur at or slightly above the streamflow levels in Otter Creek, which were at a depth of approximately 9 feet below existing roadway grade (Elev. 576±) during our exploration. It should be noted that groundwater elevations can also fluctuate with seasonal and climatic influences, as well as streamflow conditions in the creek. Therefore, the groundwater conditions may vary at different times of the year from those encountered during this exploration.

4.4 Gradation Results for Potential Scour Evaluations

Particle size analyses were performed on selected samples from Borings B-001 and B-002, obtained within a depth of approximately 6 feet below the creek bottom elevation of Elev. 575±. The particle size analyses were performed to determine D_{50} values of the soils to facilitate scour analysis. Based on the tested samples, D_{50} values generally ranged from 0.01 millimeters (mm) to less than 0.004 mm. The shallowest tested sample considered for scour evaluation from Boring B-002 (SS-5) exhibited a gradation with 52 percent material finer than

0.004 millimeters, at the termination of the hydrometer test. The deepest evaluated scour sample from Boring B-002 (SS-8) was a gravel sample with a D_{50} value of 0.57 mm. The results for the samples within the estimated potential scour zone are summarized as follows:

Table 4.4. Gradation Results for Potential Scour Evaluation

Boring Number	Sample Number	Sample Depth (feet)	Approximate Sample Elevation (feet)	D_{50} (mm)
B-001	SS-5	10 – 11.5	575 – 573.5	0.026
	SS-6	11.5 – 13	573.5 – 572	0.081
	SS-7	13 – 14.5	572 – 570.5	0.01
B-002	SS-5	10 – 11.5	575.5 – 574	<0.004
	SS-7	13 – 14.5	572.5 – 571	0.061
	SS-8	14.5 – 16	571 – 569.5	0.57

4.5 Remedial Measures

Based on the conditions encountered in the borings, the subsoils are expected to be generally conducive for support of the bridge culvert and headwall using conventional spread foundations. Based on Boring B-002, **loose** granular soils were encountered at the anticipated foundation bearing elevation of Elev. 571, and extended to Elev. 569.5±. Due to their loose compactness and occurrence below the stream bottom, we recommend that granular soils be overexcavated and replaced with lean concrete having a minimum compressive strength of 1,500 pounds per square inch (psi) or other flowable controlled-density fill (CDF) 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 one foot for every foot of over-excavation below the original bearing elevation, and centered along the footing. Replacement of these soils with lean concrete or CDF would also reduce the risk of scour through the granular zone.

During construction, temporary sheet-pile cutoff walls or cofferdams to direct streamflow may be required to manage groundwater in addition to pumping from prepared sumps.

5.0 ANALYSES AND RECOMMENDATIONS

The following analyses and recommendations are based on our understanding of the proposed construction and upon the data obtained during our field exploration. If the project information or location as outlined is incorrect or should change significantly, a review of these recommendations should be made by TTL.

5.1 Bridge Culvert Foundations

The proposed structure is planned as a three-sided culvert supported on spread footings. 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. The bottom of the footings was indicated to be planned at Elev. 571, corresponding to 4 feet below creek bottom elevation.

Based on the conditions encountered in the borings, the soils at the indicated bearing elevation are anticipated to consist of **loose** granular soils or stiff to very stiff cohesive soils. The cohesive soils are considered generally suitable for support of spread footing foundations. Based on Boring B-002, the granular soils extended to Elev. 569.5±, corresponding to 1½ feet below planned bearing elevation. Due to their loose compactness and occurrence below the stream bottom, we recommend that granular soils be overexcavated and replaced with lean concrete having a minimum compressive strength of 1,500 pounds per square inch (psi) or other flowable controlled-density fill (CDF) 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 one foot for every foot of over-excavation below the original bearing elevation, and centered along the footing. Replacement of these soils with lean concrete or CDF would also reduce the risk of scour through the granular zone.

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 3 kips per square foot (ksf) 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 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 8 ksf for 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 3.6 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 stiff to very stiff bearing stratum exhibiting a minimum unconfined compressive strength of 3,000 psf.

Settlement of the culvert foundation was calculated by conventional consolidation theory utilizing the recompression index, based on empirical relations using moisture content. Based on the bearing pressure of 3 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 4 feet. If the foundation width is increased for overturning resistance, it is anticipated that the average bearing pressure would be less than 3 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 foundations extend through the loose granular soil layer. 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 lean concrete or CDF.

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.

5.2 Bridge Culvert Walls and Headwalls

Headwall foundations should be designed in accordance with the recommendations presented in Section 5.1.

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 stream 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, which is expected to be the case for this construction. 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 stiff to very stiff bearing soils of 1,500 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 1,275 psf.

5.3 Construction Dewatering and Groundwater Control

As previously mentioned, at the time of our site reconnaissance, the water level in the creek was approximately 9 feet below roadway grades, at Elev. 576±. Based on the soil characteristics and moisture conditions encountered in the borings, it is our opinion that "normal" groundwater levels at this structure location will generally occur at or slightly above the streamflow levels in Otter Creek.

Even for normal streamflow conditions, installation of the bridge culvert foundations is expected to require excavation below the "normal" groundwater level. Although the soils below the groundwater table will be predominantly native cohesive soils with relatively low permeability, some groundwater seepage into foundation excavations should be anticipated. In particular, excavation will be required through water-bearing granular zones, which were on the order of 1½ feet in thickness and encountered at approximately Elevs. 571 to 569 in Boring B-002. Therefore, significant groundwater seepage into footings would be expected where these conditions are encountered.

In the absence of a diversion of the stream, 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 temporary 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. We recommend that sheet-pile cutoff walls extend a minimum of 10 feet

below the planned bearing depth for control of groundwater seepage and flow gradients. Deeper embedment may be required for resistance of lateral earth pressures and stability of the excavation support system. In the event excessive seepage is encountered during construction, TTL should be notified to evaluate whether other dewatering methods are required.

5.4 Construction

5.4.1 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.

5.4.2 Site Preparation and Earthwork

Prior to proceeding with construction operations, site preparation activities should include the removal of any remaining structures or substructures, as well as topsoil, root systems, and vegetation from all proposed foundation areas.

5.4.3 Fill

Material for engineered fill or backfill required to achieve design grades should meet ODOT Item 203 “Embankment Fill” placement and compaction requirements.

The upper profile on-site soils consist of cohesive soils. As such, the contractor should be prepared to use a sheepsfoot roller to provide effective compaction of the cohesive soils. In narrow utility or footing excavations, the on-site cohesive soils may be difficult to compact; therefore, a clean granular material, lean concrete, or other flowable controlled-density fill (CDF) may be required in these areas.

5.4.4 Excavations and Slopes

The sides of temporary excavations for 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. Due to the required depth of excavation below the groundwater table and normal creek level, we anticipate sheet-pile cutoff walls may be necessary to manage groundwater and control of seepage gradients in combination with excavation support 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 in accordance with the criteria presented below. The soils encountered in the test borings within the anticipated depth of excavation may be classified as the following OSHA designations:

- Type A soils (cohesive soils with unconfined compressive strengths of 3,000 pounds per square foot (psf) or greater),
- Type B soils (cohesive soils with unconfined compressive strengths greater than 1,000 psf but less than 3,000 psf), and
- Type C soils (cohesive soils with unconfined compressive strengths of 1,000 psf or less 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 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. All slopes along the channel of Otter Creek should be lined with rip-rap or other channel erosion protection.

6.0 QUALIFICATION OF RECOMMENDATIONS

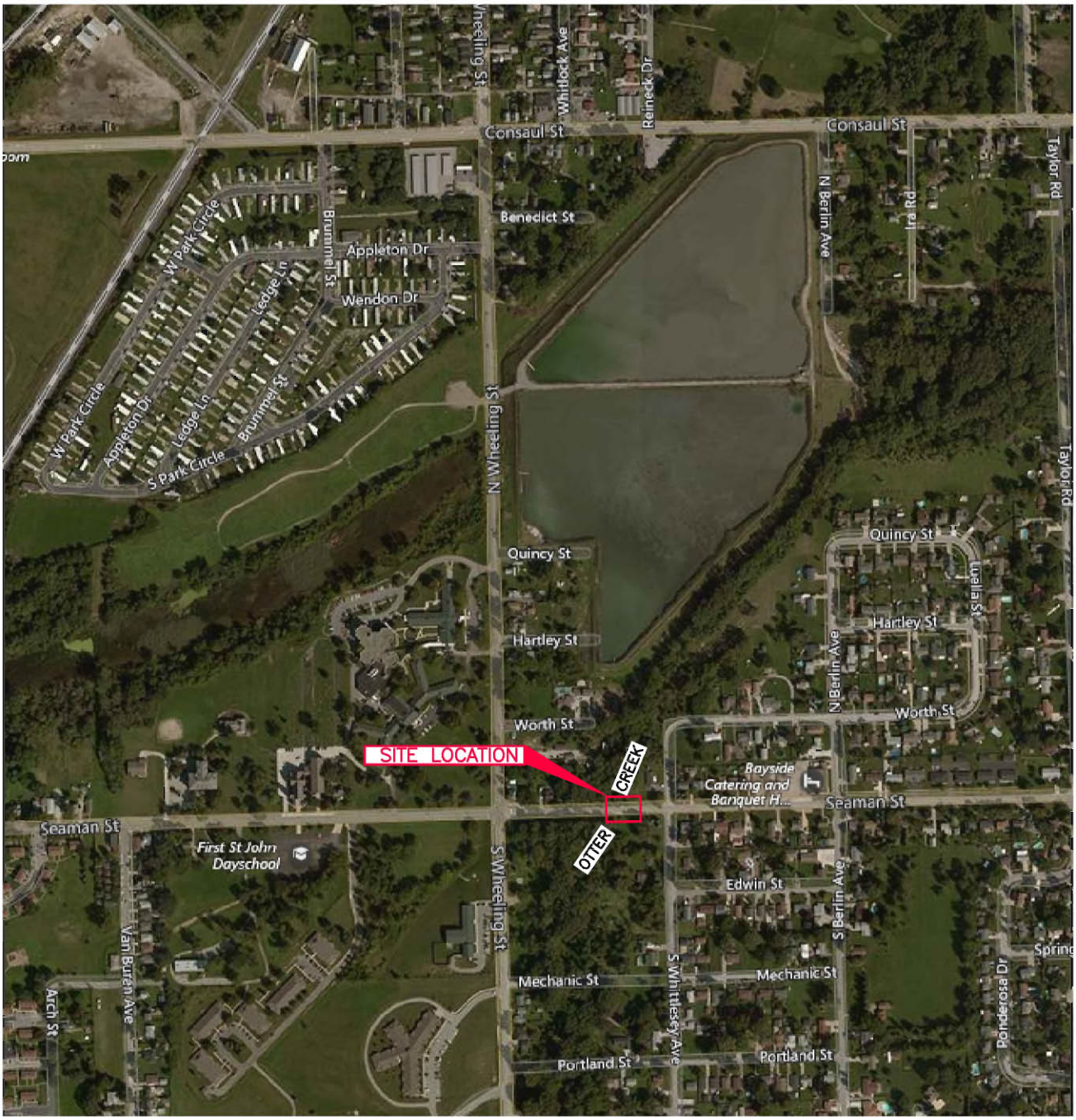
Our evaluation of bridge culvert and headwall foundation design and construction conditions has been based on our understanding of the site and project information and the data obtained during our field investigation. The general subsurface conditions used were based on interpretation of the subsurface 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.

PLATES



LEGEND

APPROXIMATE SITE LOCATION

N

APPROXIMATE SCALE – FEET

0 600 1,200

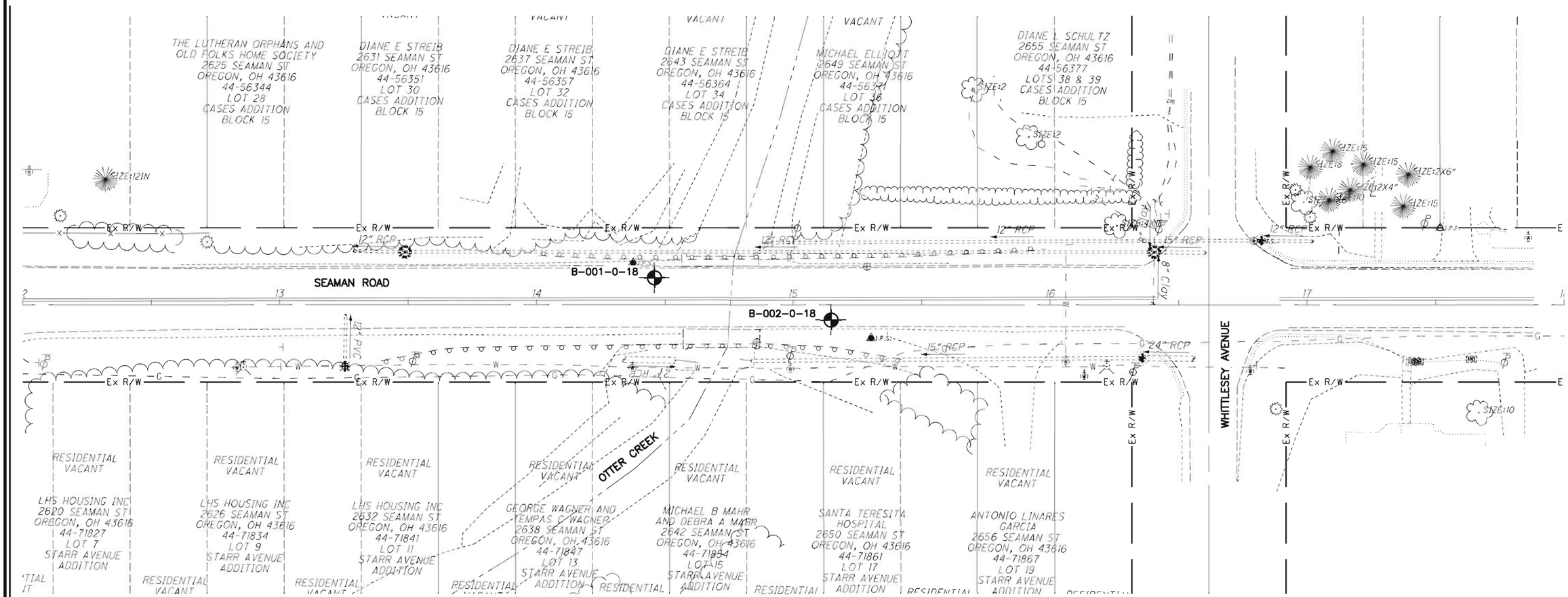
PLATE 1.0
SITE LOCATION MAP

PROPOSED BRIDGE REPLACEMENT
LUC–SEAMAN ROAD, PID 107148
SEAMAN ROAD OVER OTTER CREEK, OREGON, LUCAS COUNTY, OHIO

PREPARED FOR
CITY OF OREGEN
OREGON, OHIO

DRAWN TRR/9–4–18	CHECKED CPI/9–5–18
REVISED	APPROVED
JOB NO. 1686001	
DRAWING NUMBER **1686001–01G**	

TTL associates inc
Environmental, Geotechnical Engineering & Testing



LEGEND

B-001-0-18 APPROXIMATE TEST BORING LOCATION

APPROXIMATE SCALE – FEET

0 40 80

BASE PLAN PROVIDED BY TETRA TECH VIA EMAIL ON AUGUST 3, 2018.

PLATE 2.0
TEST BORING LOCATION PLAN
PROPOSED BRIDGE REPLACEMENT
LUC-SEAMAN ROAD OVER OTTER CREEK, PID 107148
SEAMAN ROAD OVER OTTER CREEK, OREGON, LUCAS COUNTY, OHIO

PREPARED FOR
CITY OF OREGON
OREGON, OHIO

DRAWN TRR/9-5-18	CHECKED CPI/9-5-18
REVISED	APPROVED
JOB NO. 1686001	
DRAWING NUMBER **1686001-02G**	

TTL associates inc
Environmental, Geotechnical Engineering & Testing

FIGURES

PROJECT: SEAMAN @ OTTER CREEK	DRILLING FIRM / OPERATOR: ENVIROCORE / JOE	DRILL RIG: GEOPROBE 3230	STATION / OFFSET: 14+46, 11 LT	EXPLORATION ID B-001-0-18
TYPE: STRUCTURE FOUNDATION	SAMPLING FIRM / LOGGER: ENVIROCORE / KKC	HAMMER: AUTOMATIC HAMMER	ALIGNMENT: SEAMAN ROAD	
PID: 107148 BR ID:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 2/23/18	ELEVATION: 585.2 (NAVD88) EOB: 59.2 ft.	PAGE 1 OF 2
START: 7/10/18 END: 7/10/18	SAMPLING METHOD: SPT / ST	ENERGY RATIO (%): 82.5	COORD: 724572.900 N, 1699172.000 E	

MATERIAL DESCRIPTION AND NOTES	ELEV. 585.2	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
ASPHALT - 15 INCHES	583.9	1	14 10 11	29	78	SS-1	NP	-	-	-	-	-	-	-	-	4	A-1-b (V)	
CRUSHED STONE - APPROXIMATELY 20 INCHES (TRANSITIONED TO COHESIVE SUBGRADE SOILS BETWEEN SS-1 AND SS-2 SAMPLES. TRANSITION SHOWN HALF WAY BETWEEN AT 3 FEET BELOW TOP OF PAVEMENT.)	582.2	2																
MEDIUM STIFF TO STIFF, BROWN, **SILTY CLAY**, SOME SAND, MOIST	579.2	3																
SOFT TO MEDIUM STIFF, BROWN, **SILTY CLAY**, LITTLE SAND, MOIST		4	6 3 2	7	78	SS-2	1.50	0	10	22	26	42	34	18	16	22	A-6b (9)	
@8.5': BROWN/GRAY, TRACE GRAVEL		5																
@10': SOFT, BROWN/BLUE/GRAY, SOME SAND, LITTLE GRAVEL	573.7	6	3 1 1	3	89	SS-3	0.50	-	-	-	-	-	-	-	-	25	A-6b (V)	
MEDIUM STIFF TO STIFF, GRAY/BROWN, **SILTY CLAY**, SOME SAND AND GRAVEL, MOIST	572.2	7																
STIFF TO VERY STIFF, GRAY, **SILT AND CLAY**, SOME SAND AND TRACE GRAVEL, DAMP		8																
@16': VERY STIFF, LITTLE SAND		9	0 1 1	3	78	SS-4	0.50	-	-	-	-	-	-	-	-	26	A-6b (V)	
@18': STIFF TO VERY STIFF		10	0 0 3	4	56	SS-5	-0.25	11	12	19	30	28	-	-	-	25	A-6b (V)	
@21': MOIST	561.7	11	1 2 3	7	78	SS-6	2.00	23	12	16	29	20	-	-	-	23	A-6b (V)	
MEDIUM STIFF TO STIFF, GRAY, **SILT AND CLAY**, SOME SAND AND TRACE GRAVEL, MOIST		12																
		13	3 3 6	12	89	SS-7	2.36*	5	12	18	25	40	28	16	12	15	A-6a (7)	
		14																
		15	3 4 7	15	100	SS-8	>4.5	-	-	-	-	-	-	-	-	14	A-6a (V)	
		16																
		17	4 6 9	21	78	SS-9	>4.5	-	-	-	-	-	-	-	-	14	A-6a (V)	
		18																
		19	2 4 5	12	78	SS-10	2.74*	-	-	-	-	-	-	-	-	15	A-6a (V)	
		20																
		21																
		22	3 3 6	12	78	SS-11	3.50	-	-	-	-	-	-	-	-	17	A-6a (V)	
		23																
		24	2 3 4	10	89	SS-12	1.00	-	-	-	-	-	-	-	-	18	A-6a (V)	
		25																
		26		96	ST-13	0.59*	9	8	17	25	41	28	15	13	18	A-6a (7)		
		27																
		28	3 3 5	11	89	SS-14	1.25	-	-	-	-	-	-	-	-	19	A-6a (V)	
		29	3 4 6	14	89	SS-15	0.76*	-	-	-	-	-	-	-	-	19	A-6a (V)	
		30																

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 9/10/18 13:56 - S:\PROJECTS\1686001.GPJ

PID: 107148	BR ID:	PROJECT: SEAMAN @ OTTER CREEK	STATION / OFFSET: 14+46, 11 LT	START: 7/10/18	END: 7/10/18	PG 2 OF 2	B-001-0-18

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
MEDIUM STIFF TO STIFF, GRAY, **SILT AND CLAY**, SOME SAND AND TRACE GRAVEL, MOIST *(continued)*	554.2	32 33																
	551.7																	
STIFF TO VERY STIFF, GRAY, **SILT AND CLAY**, SOME SAND AND TRACE GRAVEL, MOIST		34 35 36 37 38	4 7 9	22	89	SS-16	1.22*	0	9	19	29	43	29	17	12	18	A-6a (8)	
@38.5': DAMP		39 40 41 42 43	5 6 10	22	89	SS-17	1.50	-	-	-	-	-	-	-	-	15	A-6a (V)	
		44 45 46 47 48	4 10 6	22	89	SS-18	NI	-	-	-	-	-	-	-	-	16	A-6a (V)	
	536.7																	
VERY STIFF TO HARD, GRAY, **SILTY CLAY**, LITTLE SAND AND TRACE GRAVEL, MOIST		49 50 51 52 53	6 12 29	56	100	SS-19	3.30*	-	-	-	-	-	-	-	-	19	A-6b (V)	
@53.5': HARD, DAMP		54 55 56 57 58	8 16 24	55	100	SS-20	4.26*	-	-	-	-	-	-	-	-	13	A-6b (V)	
	526.0	EOB 59	16 50/2"	-	100	SS-21	>4.5	-	-	-	-	-	-	-	-	9	A-6b (V)	

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

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; AUGER CUTTINGS MIXED WITH 3 BAGS BENTONITE CHIPS

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 9/10/18 13:56 - S:\PROJECTS\1686001.GPJ

PROJECT: SEAMAN @ OTTER CREEK	DRILLING FIRM / OPERATOR: ENVIROCORE / ALEX	DRILL RIG: MOBILE B-57	STATION / OFFSET: 15+15, 6 RT
TYPE: STRUCTURE FOUNDATION	SAMPLING FIRM / LOGGER: ENVIROCORE / KKC	HAMMER: AUTOMATIC HAMMER	ALIGNMENT: SEAMAN ROAD
PID: 107148 BR ID:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 2/23/18	ELEVATION: 585.7 (NAVD88) EOB: 29.0 ft.
START: 5/30/18 END: 5/30/18	SAMPLING METHOD: SPT	ENERGY RATIO (%): 82.7	COORD: 724557.600 N, 1699241.000 E

EXPLORATION ID B-002-0-18

PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 585.7	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
ASPHALT - 6 INCHES	585.2																	
CRUSHED STONE - 6 INCHES	584.7																	
STIFF, BROWN, **SILTY CLAY**, LITTLE SAND AND TRACE GRAVEL, DAMP	582.2	1 2 3	3 4 5	12	6	SS-1	NI	-	-	-	-	-	-	-	-	8	A-6b (V)	
MEDIUM STIFF TO STIFF, GRAY/BROWN, **SILT AND CLAY**, "AND" SAND, MOIST	579.7	4 5	2 2 3	7	67	SS-2	1.50	0	11	28	28	33	30	19	11	20	A-6a (6)	
LOOSE, GRAY, **PEAT**, LITTLE SAND, WOOD, AND TRACE GRAVEL, WET (P)	577.2	6 7 8	3 2 3	7	67	SS-3	NP	-	-	-	-	-	-	-	-	123	Peat (V)	
SOFT TO MEDIUM STIFF, GRAY, **SILTY CLAY**, LITTLE SAND, MOIST		9	2 1 2	4	67	SS-4	1.00	-	-	-	-	-	-	-	-	28	A-6b (V)	
@10': BROWN/GRAY, TRACE SAND		10 11	2 1 2	4	67	SS-5	0.75	0	2	7	35	56	-	-	-	25	A-6b (V)	
@11.5': MEDIUM STIFF, GRAY/BROWN		12 13	1 2 2	6	67	SS-6	1.50	-	-	-	-	-	-	-	-	26	A-6b (V)	
@13': SOFT TO MEDIUM STIFF, "AND" SAND, TRACE GRAVEL	571.2	14	2 4 2	8	67	SS-7	0.21*	3	15	31	34	17	-	-	-	23	A-6b (V)	
LOOSE, BROWN/GRAY, **GRAVEL AND STONE FRAGMENTS WITH SAND AND SILT**, TRACE CLAY, DAMP	569.7	15	2 2 2	6	67	SS-8	NP	31	24	17	27	1	NP	NP	NP	16	A-2-4 (0)	
STIFF TO VERY STIFF, GRAY, **SILT AND CLAY**, SOME SAND AND TRACE GRAVEL, DAMP	567.2	16 17	2 4 5	12	67	SS-9	2.73*	6	9	17	26	42	29	17	12	15	A-6a (7)	
VERY STIFF TO HARD, GRAY, **SILTY CLAY**, SOME SAND AND TRACE GRAVEL, DAMP		18 19 20	3 4 50/4"	-	75	SS-10	3.50	-	-	-	-	-	-	-	-	15	A-6b (V)	
		21 22 23	50/3"	-	100	SS-11	NI	4	9	18	26	43	-	-	-	14	A-6b (V)	
	562.2																	
VERY DENSE, GRAY, **COARSE AND FINE SAND**, SOME GRAVEL AND LITTLE SILT, DAMP	559.7	24 25 26	50/2"	-	100	SS-12	NP	-	-	-	-	-	-	-	-	15	A-3a (V)	
VOID ENCOUNTERED (SANITARY SEWER PENETRATED)	556.7	27 28 29																
		EOB																

W

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

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.25 BAG ASPHALT PATCH; PUMPED BENTONITE GROUT

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 9/10/18 13:57 - S:\PROJECTS\1686001.GPJ

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

Sample Symbols

SS - Split Spoon

ST - Shelby Tube

RC - Rock Core

GS - Geoprobe Sleeve

AU - Auger Cuttings

GB - Grab

Notes:

1. The exploratory borings were drilled on May 30 and July 10, 2018, using 3¼-inch inside diameter hollow-stem augers and a tri-cone rotary bit.

2. These logs are subject to the limitations and encountered conditions presented in the report and should not be interpreted separate from the report.

3. The boring locations were established in the field by TTL Associates, Inc. based on a site plan provided by City of Oregon. Coordinates and ground surface elevations at the boring locations were obtained by City of Oregon using a hand-held GPS unit. Station and offset for each boring location were provided by Tetra Tech.

4. Unconfined Compressive Strength (tsf):
   NP = Non-Plastic
   NI = Not Intact
   "*" = Unconfined Compressive Strength per ASTM D 2166

PROJECT: LUC-Seaman Road Bridge over Otter Creek, Oregon, Ohio	**TTL Associates, Inc.**	PROJECT NO: 1686001

TABULATION OF TEST DATA

Boring Number	Sample Number	Sample Interval Depth (Feet)	Measured Standard Penetration, N_m (Blows per Foot)	Corrected Standard Penetration, N_{60} (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 (%)			ODOT Soil Classification
									Gravel	Coarse Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index	
B-001-0-1	SS-1	1.0-2.5	21	38	4.4												
	SS-2	3.5-5.0	5	9	22.0		3,000		0	10	22	26	42	34	18	16	A-6b (9)
	SS-3	6.0-7.5	2	4	25.1		1,000										
	SS-4	8.5-10.0	2	4	26.1		1,000										
	SS-5	10.0-11.5	3	5	25.0		9,000+		11	12	19	30	28				
	SS-6	11.5-13.0	5	9	22.7		4,000		23	12	16	29	20				
	SS-7	13.0-14.5	9	16	14.8	115.5	*4,715		5	12	18	25	40	28	16	12	A-6a (7)
	SS-8	14.5-16.0	11	20	13.6		9,000+										
	SS-9	16.0-17.5	15	27	14.0		9,000+										
	SS-10	18.5-20.0	9	16	14.7	117.5	*5,475										
	SS-11	21.0-22.5	9	16	17.2		7,000										
	SS-12	23.5-25.0	7	13	17.6		2,000										
	ST-13	25.0-27.0			18.0	111.8	*1,180		9	8	17	25	41	28	15	13	A-6a (7)
	SS-14	27.0-28.5	8	14	19.1		2,500										
	SS-15	28.5-30.0	10	18	18.7	115.2	*1,515										
	SS-16	33.5-35.0	16	29	18.1	116.7	*2,430		0	9	19	29	43	29	17	12	A-6a (8)

SSR = Split-Spoon Refusal

Unconfined compressive strength (UCS) generally derived from a calibrated hand penetrometer. UCS denoted with "*" determined by ASTM D 2166.

PROJECT: LUC-Seaman Road Bridge over Otter Creek, Oregon, Ohio	**TTL Associates, Inc.**	PROJECT NO: 1686001

TABULATION OF TEST DATA

Boring Number	Sample Number	Sample Interval Depth (Feet)	Measured Standard Penetration, N_m (Blows per Foot)	Corrected Standard Penetration, N_{60} (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 (%)			ODOT Soil Classification
									Gravel	Coarse Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index	
B-001-0-1	SS-17	38.5-40.0	16	29	15.0		3,000										
	SS-18	43.5-45.0	16	29	16.4	113.6	*1,135										
	SS-19	48.5-50.0	41	74	18.9	110.3	*6,600										
	SS-20	53.5-55.0	40	72	12.7	120.2	*8,520										
	SS-21	58.5-59.2	SSR	-	9.0		9,000+										

SSR = Split-Spoon Refusal

Unconfined compressive strength (UCS) generally derived from a calibrated hand penetrometer. UCS denoted with "*" determined by ASTM D 2166.

PROJECT: LUC-Seaman Road Bridge over Otter Creek, Oregon, Ohio	**TTL Associates, Inc.**	PROJECT NO: 1686001

TABULATION OF TEST DATA

Boring Number	Sample Number	Sample Interval Depth (Feet)	Measured Standard Penetration, N_m (Blows per Foot)	Corrected Standard Penetration, N_{60} (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 (%)			ODOT Soil Classification
									Gravel	Coarse Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index	
B-002-0-1	SS-1	1.0-2.5	9	16	8.4												
	SS-2	3.5-5.0	5	9	19.8		3,000		0	11	28	28	33	30	19	11	A-6a (6)
	SS-3	6.0-7.5	5	9	123.0												
	SS-4	8.5-10.0	3	5	27.7		2,000										
	SS-5	10.0-11.5	3	5	24.7		1,500		0	2	7	35	56				
	SS-6	11.5-13.0	4	7	25.9		3,000										
	SS-7	13.0-14.5	6	11	22.8	107.2	*425		3	15	31	34	17				
	SS-8	14.5-16.0	4	7	16.2				31	24	17	27	1	NON-PLASTIC			A-2-4 (0)
	SS-9	16.0-17.5	9	16	14.5	113.4	*5,470		6	9	17	26	42	29	17	12	A-6a (7)
	SS-10	18.5-19.8	SSR	-	15.2		7,000										
	SS-11	21.0-213	SSR	-	13.5				4	9	18	26	43				
	SS-12	23.5-23.7	SSR	-	15.0												

SSR = Split-Spoon Refusal

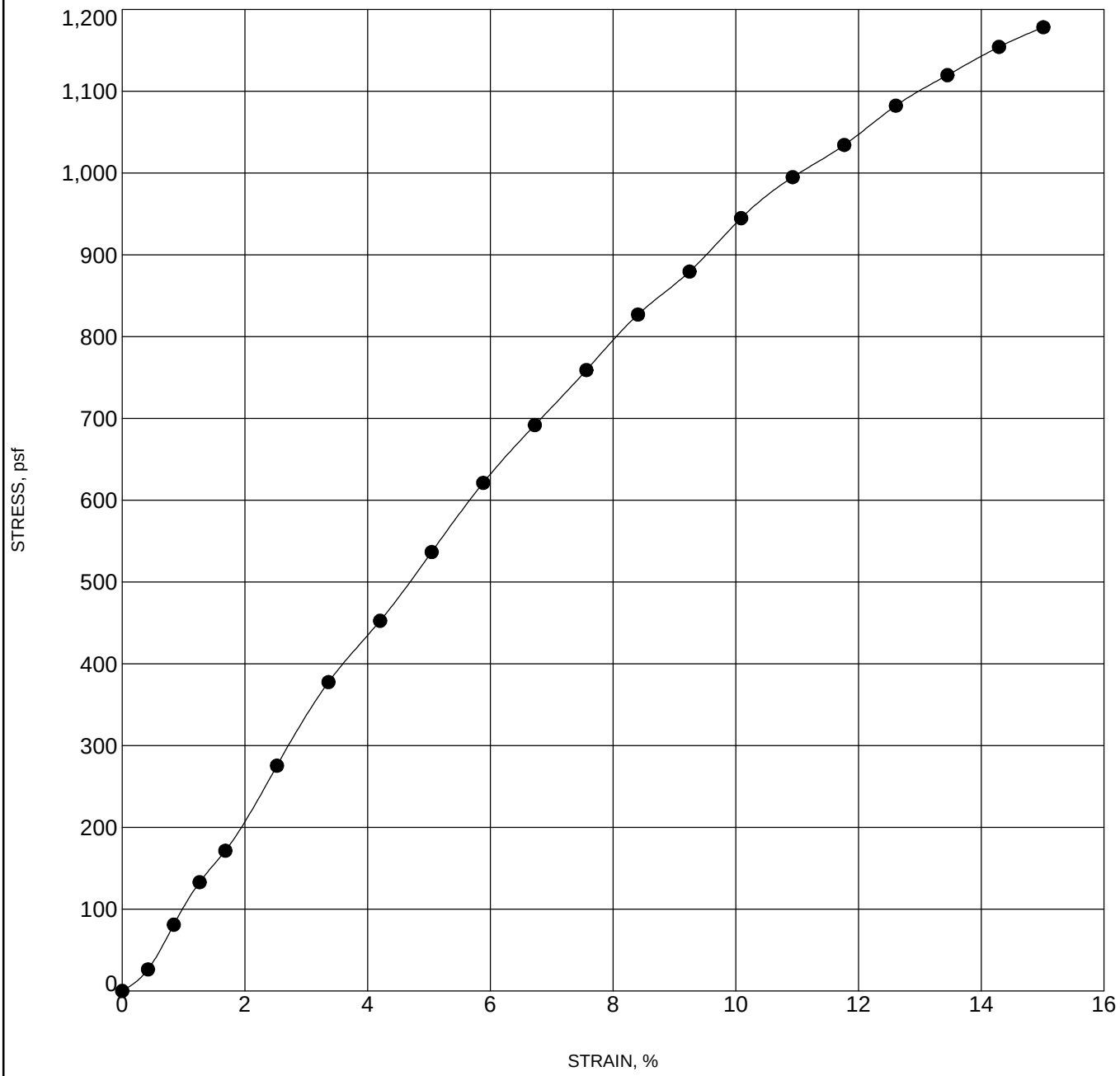
Unconfined compressive strength (UCS) generally derived from a calibrated hand penetrometer. UCS denoted with "*" determined by ASTM D 2166.

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

UNCONFINED COMPRESSION TEST

CLIENT City of Oregon **PROJECT NAME** LUC-Seaman Road over Otter Creek

PROJECT NUMBER 1686001 **PROJECT LOCATION** Oregon, Ohio



Specimen Identification		Classification	γ_d	MC%
● B-001-0-18 25.0		A-6a (7)	112	18

Appendix A

Engineering Calculations

	Stratum I	**Med Stiff to Stiff**				**Stratum II**	**Soft-Med**				**Stratum III**	**Stiff to Very Stiff**				**Stratum IV**	**Med to Stiff**	
	N	**UCS (tsf)**	**MC**			**N**	**UCS (tsf)**	**MC**			**N**	**UCS (tsf)**	**MC**			**N**	**UCS (tsf)**	**MC**
	7	1.5	22								12	2.73	15			10	1	18
	7	1.5	20			4	1	28				3.5	15				0.59	18
	12					4	0.75	25					14			11	1.25	19
						6	1.5	26			12	2.36	15			14	0.76	19
						8	0.21	23			15		14					
						6	0.5	25			21		14					
						3	0.5	26			12	2.74	15					
						3	0.25	25			12	3.5	17					
						4												
Avg	8.666667	1.5	21															
					Avg	4.8	0.672857	25		Avg	14.0	2.97	15					
UCS Nx250	2166.667	psf																
					UCS Nx250	1188	psf			UCS Nx250	3500	psf						
Su (psf)	1083.333	1500													Avg	11.7	0.90	19
					Su (psf)	594	672.8571			Su (psf)	1750	2966						
Design Su	**1100**	**psf**								12x250/2=	1500				UCS Nx250	2917	psf	
					Design Su	**600**	**psf**			**Design Su**	**1500**	**psf**						
															Su (psf)	1458	900	
															Design Su	**900**	**psf**	

	Stratum V	**Stiff to V Stiff**				**Stratum VI**	**Very Stiff to Hard**												
	N	**UCS (tsf)**	**MC**			**N**	**UCS (tsf)**	**MC**											
	22	1.22	18			56	3.3	19											
	22	1.5	15			55	4.26	13											
	22		16					9											
Avg	22.0	1.36	16		Avg	55.5	3.78	14											
UCS Nx250	5500	psf			UCS Nx250	13875	psf												
Su (psf)	2750	1360			Su (psf)	6938	3780												
Design Su	**1500**	**psf**			**Design Su**	**4000**	**psf**												

Project Name Seaman Rd over Otter Creek Project No. 1686001

By CPT Checked by/Date CPT 9/6/18

Subject LRFD Bearing & Settlement

3 Sided Culvert bearing 4' below Creek Bottom @ Elev. 571.

B-001 @ 571 : Stratum III 5-VS to Elev. 561± then IV med to stiff

B-002 @ 571 : 1 1/2' of loose ground then III to Elev. 562 V Dense Granular (to II)

Stratum III : C = 1500 psf

Stratum IV : C = 900 psf (See Spreadsheet)

Service Limit State : Presumptive Bearg Resistance

AASHTO LRFD BDM C10.6.2.6.1-1 (pg 10-86)

Very stiff to Hard CL (very dense in Table) = 6 to 12 ksf, say 6 ksf

Med stiff to stiff CL (med dense to dense in Table) = 2 to 6 ksf, say 3,000 psf.

If q_n = 3 ksf, and ϕ_b = 1.0 → $q_R = q_n \phi_b = 3 \times 1 = \underline{3 ksf}$

Look at Settlement using spreadsheet

@ 3 ksf, Total settlement < 1"

Strength Limit State : $q_R = \phi_b q_n$, q_n by bearing "capacity" equation

Look at 2-Layer System : Das 3rd Ed. Principles of Foundation Engineering

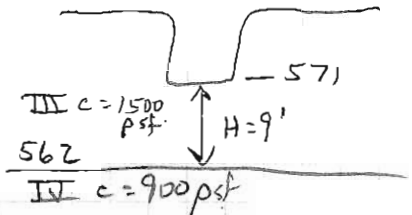
$$q_n = \left[1 + 0.2\left(\frac{B}{L}\right)\right] C_2 N_c + \left(1 + \frac{B}{L}\right)\left(\frac{2 c_a H}{B}\right) + \gamma D_f$$

$$\leq \left[1 + 0.2\left(\frac{B}{L}\right)\right] C_1 N_c + \gamma D_f \qquad \phi = 0 \rightarrow N_c = 5.14$$

Project Name Sherman Road over Otter Creek Project No. 1686001
By CJI Checked by/Date CAW 9/11/18
Subject LRFD Bearing & Settlement

Look @ B=4' & L=50'

$\frac{C_2}{C_1} = \frac{900}{1500} = 0.6 \rightarrow \frac{C_a}{C_1} = 0.97 \rightarrow C_a = 0.97(1500) = 1455$ psf



$q_n = \left[1+0.2\left(\frac{4}{50}\right)\right](900)(5.14) + \left(1+\frac{4}{50}\right)\left(\frac{2(1455)(9)}{4}\right) + (130-62.4)(4)$

$= 4700' + 7071' + 270' = 12,041$ psf ~ 12 ksf

$\leq \left[1+0.2\left(\frac{4}{50}\right)\right]1500(5.14) + (130-62.4)(4)$

$= 7833' + 270' = 8103$ psf ~ 8 ksf ✓

$q_n = 8$ ksf ✓

If bigger B for overturning, say up to B=10',

$q_n = \left[1+0.2\left(\frac{10}{50}\right)\right](900)(5.14) + \left(1+\frac{10}{50}\right)\left(\frac{2(1455)(9)}{10}\right) + (130-62.4)(4)$

$= 4811' + 3143' + 270' = 8224$ psf ~ 8 ksf

$\leq \left[1+0.2\left(\frac{10}{50}\right)\right]1500(5.14) + (130-62.4)(4) =$

$= 8018 + 270 = 8288$ psf ~ 8 ksf.

$q_n = 8$ ksf ✓

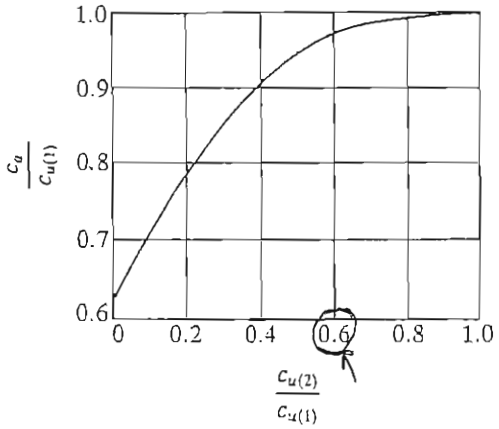
Regardless if B=4' to 10', $q_n = 8$ ksf $\phi_b = 0.45$

$q_R = B(0.45) = 3.6$ ksf

16B6001
Seaman R. over Otter Creek

where B = width of foundation
L = length of foundation
$N_c = 5.14$ (bearing capacity factor for $\phi = 0$; Table 3.4)
c_a = adhesion along the interface aa'

Figure 3.23 shows the variation of $c_a/c_{u(1)}$ with $c_{u(2)}/c_{u(1)}$.



▼ **FIGURE 3.23** Variation of $c_a/c_{u(1)}$ with $c_{u(2)}/c_{u(1)}$ (according to Meyerhof and Hanna, 1978)

Case II: $c_{u(1)}/c_{u(2)} < 1$. The ultimate bearing capacity for this condition (Meyerhof, 1974; Meyerhof and Hanna, 1978) is as follows:

$$q_u = q_t + (q_b - q_t)\left(1 - \frac{H}{H_f}\right)^2 \geq q_t \tag{3.66}$$

where

$$q_t = \left[1 + 0.2\left(\frac{B}{L}\right)\right]c_{u(1)}N_c + \gamma_1 D_f \tag{3.67}$$

$$q_b = \left[1 + 0.2\left(\frac{B}{L}\right)\right]c_{u(2)}N_c + \gamma_2 D_f \tag{3.68}$$

$$H_f \approx B \tag{3.69}$$

Note that $N_c = 5.14$ because $\phi = 0$ (Table 3.4).

In the opinion of some foundation engineers, Eq. (3.66) may be an unconservative relationship. Hence proper judgment should be exercised about using it in designing for field conditions.

Foundations on Dense or Compacted Sand Overlying Soft Clay

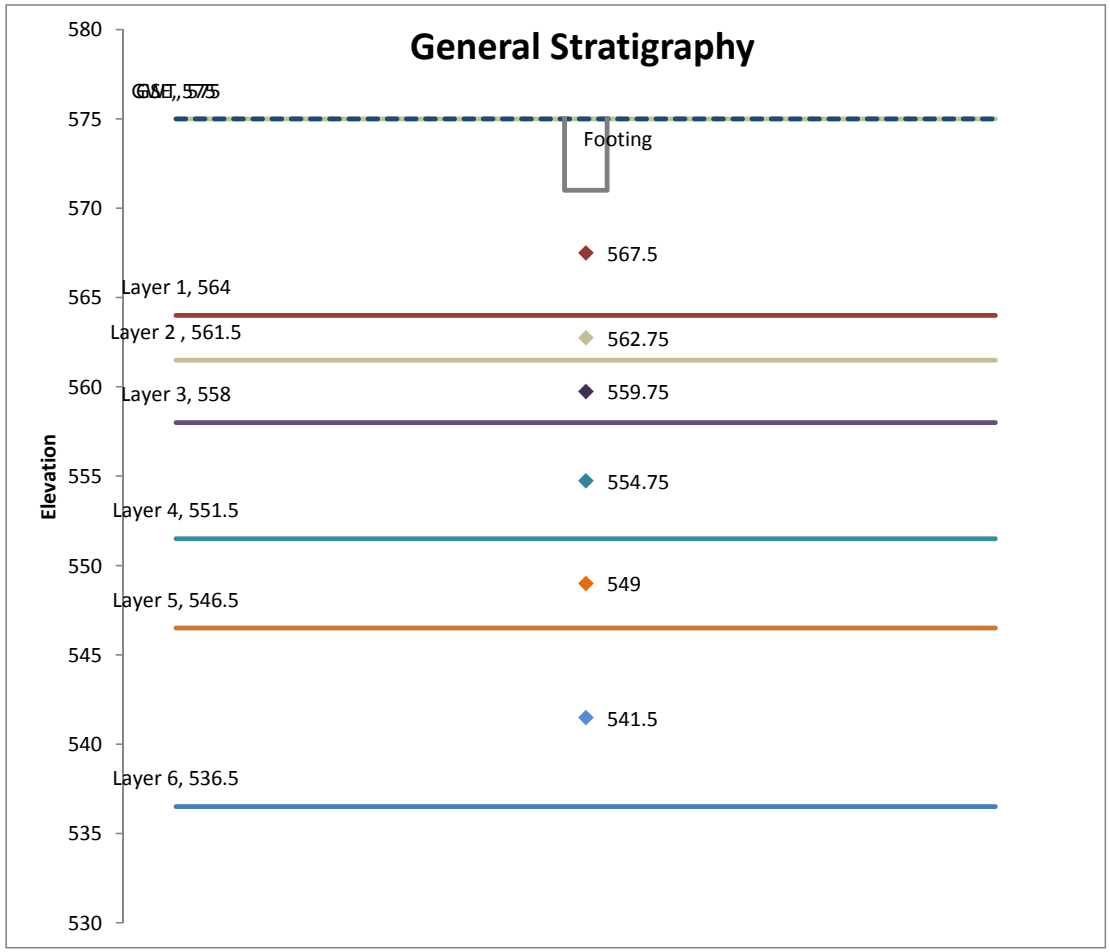
In some cases, foundations are constructed on compacted sand layers that overlie soft clay soils. If the thickness of the sand layer uner the foundation is relatively small, the failure surface may extend into the soft clay layer. This is shown in the left half of

Das : 3rd Ed. Principles of Foundation Engineering

Project Name: Seaman over Otter Creek
Project Number: 1686001
Calculated by: CPI 9-5-18

Boring Number B-001
Analysis Type Boussinesq Continuous

Layer	H (feet)	C_r	e_o	sigma v (psf)	z (feet)	b (feet)	(z-Df) / b	I_z	delta p@	3000 psf	(check) sigma v+ΔP	delta H (inches)		C'	delta H w/C'
Layer 1	7	0.015	0.49	507	3.5	4	0.9	0.6	1800		2307	0.56		0	#DIV/0!
Layer 2	2.5	0.017	0.52	828	8.25	4	2.1	0.3	900		1728	0.11		0	#DIV/0!
Layer 3	3.5	0.018	0.53	1031	11.25	4	2.8	0.2	600		1631	0.10		0	#DIV/0!
Layer 4	6.5	0.019	0.55	1369	16.25	4	4.1	0.1	300		1669	0.08		0	#DIV/0!
Layer 5	5	0.018	0.53	1758	22	4	5.5	0	0		1758	0.00		0	#DIV/0!
Layer 6	10	0.016	0.50	2265	29.5	4	7.4	N/A	N/A		N/A	N/A		0	#DIV/0!
											Total delta H (in.)	0.84			#DIV/0!
											+15%	0.97			#DIV/0!
											-15%	0.72			#DIV/0!



Project Name:	Seaman over Otter Creek	Boring Number	B-001
Project Number:	1686001	Analysis Type	Boussinesq Continuous
Calculated by:	CPI 9-5-18		(Homogeneous)

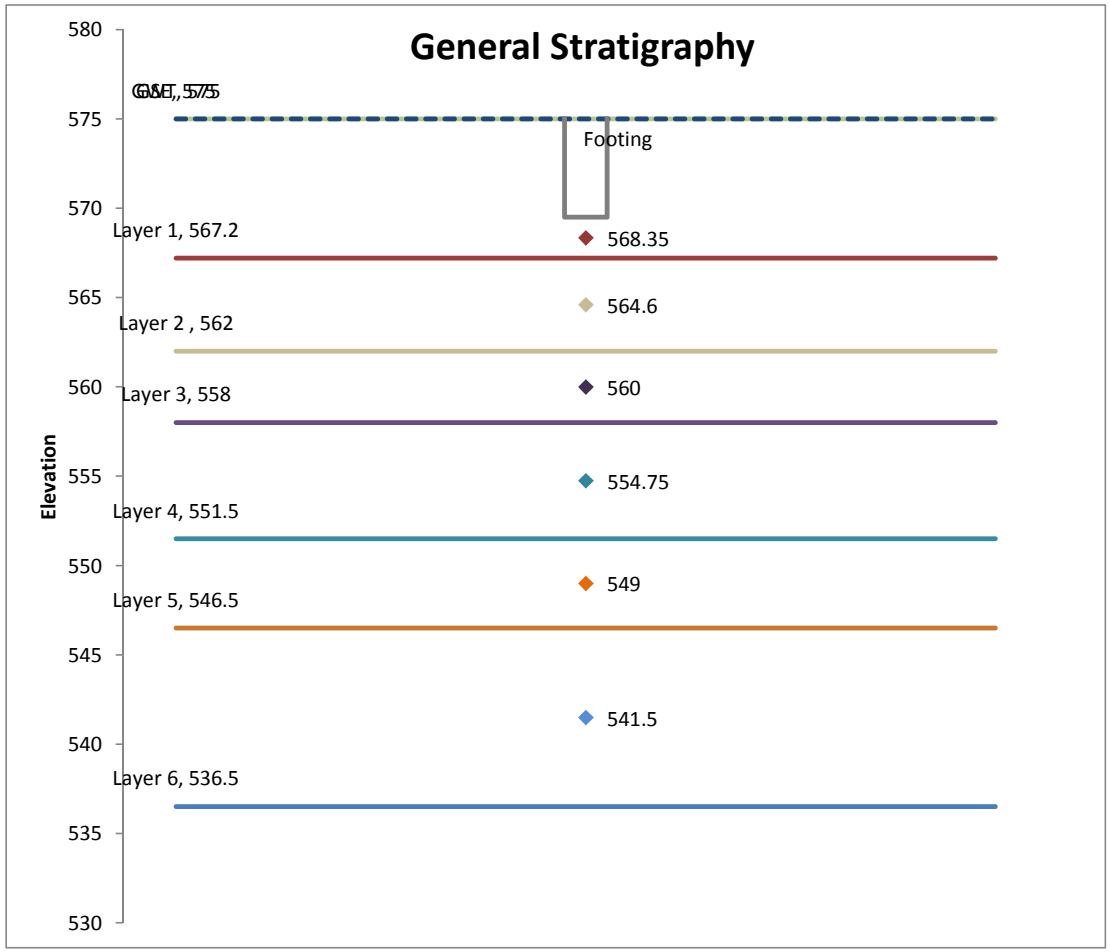
G (assumed)	2.7	
GSE	575	
GWT	575	
Bearing Elev	571	
D_f	4	ft
Footing Width, B	4	ft
P	3000	psf
Rig	Truck 111	ER 75.4

	Bot. Elev.	Centroid (C) Elev.	H (ft)	z below footing	z below GSE	γ_T (pcf)	γ_d (pcf)	H_{GWT-C}	w at C (%) (or C_rx1000)	e_o	Depth of Influence = $(z-D_f)/B$	I_z	σ_v' (psf)	N'/N	N_m	N_{60}	N'	C'
Layer 1	564	567.5	7	3.5	7.5	130	113	7.5	15	0.49	0.88	0.6	507	1.94		0	0	
Layer 2	561.5	562.75	2.5	8.25	12.25	130	111	12.25	17	0.52	2.1	0.3	828	1.5		0	0	
Layer 3	558	559.75	3.5	11.25	15.25	130	110	15.25	18	0.53	2.8	0.2	1031	1.3		0	0	
Layer 4	551.5	554.75	6.5	16.25	20.25	130	109	20.25	19	0.55	4.1	0.1	1369	1.05		0	0	
Layer 5	546.5	549	5	22	26	130	110	26	18	0.53	5.5	0	1758	0.96		0	0	
Layer 6	536.5	541.5	10	29.5	33.5	130	112	33.5	16	0.50	7.4	N/A	2265	1		0	0	

Project Name: Seaman over Otter Creek
Project Number: 1686001
Calculated by: CPI 9-5-18

Boring Number B-002
Analysis Type Boussinesq Continuous

Layer	H (feet)	C_r	e_o	sigma v (psf)	z (feet)	b (feet)	(z-Df) / b	I_z	delta p@	3000 psf	(check) sigma v+ΔP	delta H (inches)		C'	delta H w/C'
Layer 1	2.3	0.015	0.49	450	1.15	4	0.3	1	3000		3450	0.25		0	#DIV/0!
Layer 2	5.2	0.015	0.49	703	4.9	4	1.2	0.5	1500		2203	0.31		0	#DIV/0!
Layer 3	4	0.018	0.53	1014	9.5	4	2.4	0.2	600		1614	0.11		0	#DIV/0!
Layer 4	6.5	0.019	0.55	1369	14.75	4	3.7	0.2	600		1969	0.15		0	#DIV/0!
Layer 5	5	0.018	0.53	1758	20.5	4	5.1	0	0		1758	0.00		0	#DIV/0!
Layer 6	10	0.016	0.50	2265	28	4	7.0	N/A	N/A		N/A	N/A		0	#DIV/0!
											Total delta H (in.)	0.82			#DIV/0!
											+15%	0.95			#DIV/0!
											-15%	0.70			#DIV/0!



Soil properties for
Layers 3 to 6 from B-001

Project Name:	Seaman over Otter Creek	Boring Number	B-002
Project Number:	1686001	Analysis Type	Boussinesq Continuous
Calculated by:	CPI 9-5-18		(Homogeneous)

G (assumed)	2.7		
GSE	575		
GWT	575		
Bearing Elev	569.5		1.5 ft undercut of granular, replace w/CDF
D_f	5.5	ft	
Footing Width, B	4	ft	
P	3000	psf	
Rig	Truck 111		ER 75.4

	Bot. Elev.	Centroid (C) Elev.	H (ft)	z below footing	z below GSE	γ_T (pcf)	γ_d (pcf)	H_{GWT-C}	w at C (%) (or C_rx1000)	e_o	Depth of Influence = (z-D_f)/B	I_z	σ_v' (psf)	N'/N	N_m	N_{60}	N'	C'
Layer 1	567.2	568.35	2.3	1.15	6.65	130	113	6.65	15	0.49	0.29	1	450	2.05		0	0	
Layer 2	562	564.6	5.2	4.9	10.4	130	113	10.4	15	0.49	1.2	0.5	703	1.65		0	0	
Layer 3	558	560	4	9.5	15	130	110	15	18	0.53	2.4	0.2	1014	1.32		0	0	
Layer 4	551.5	554.75	6.5	14.75	20.25	130	109	20.25	19	0.55	3.7	0.2	1369	1.05		0	0	
Layer 5	546.5	549	5	20.5	26	130	110	26	18	0.53	5.1	0	1758	0.96		0	0	
Layer 6	536.5	541.5	10	28	33.5	130	112	33.5	16	0.50	7.0	N/A	2265	1		0	0	

Appendix B

Geotechnical Engineering Design Checklists

II. Reconnaissance and Planning Checklist

C-R-S: LUC-Seaman Road	PID: 107148	Reviewer: CPI	Date: 09-09-18

Reconnaissance			
Y N **X**	1	Based on Section 302.1 in the SGE, have the necessary plans been developed in the following areas prior to the commencement of the subsurface exploration reconnaissance: □ Roadway plans □ Structures plans □ Geohazards plans	Plans to be prepared by others.
Y N X	2	Based on Section 302.2 in the SGE, has the Geotechnical Red Flag Summary, or in its absence, the resources listed in Section 202 of the SGE, been reviewed as part of the office reconnaissance?	
Y N X	3	Have all the features listed in Section 302.3 of the SGE been observed and evaluated during the field reconnaissance?	
Y N **X**	4	If notable features were discovered in the field reconnaissance, were the GPS coordinates of these features recorded?	

II. Reconnaissance and Planning Checklist

Planning - General					
Y	N	X	5	In planning the geotechnical exploration program for the project, have the specific geologic conditions, the proposed work, and existing subsurface exploration work been considered?	
Y	N	X	6	Have the borings been located to develop the maximum subsurface information while using a minimum number of borings?	
Y	N	X	7	Has the topography, geologic origin of materials, surface manifestation of soil conditions, and any other special design considerations been utilized in determining the spacing and depth of borings?	
Y	N	X	8	Have the borings been located so as to provide adequate overhead clearance for the equipment, clearance of underground utilities, minimize damage to private property, and minimize disruption of traffic, without compromising the quality of the exploration?	
Y	N	**X**	9	Have any previous geotechnical explorations been utilized to the fullest extent possible?	
Y	N	**X**	10	Have the scaled boring plans, showing all project and historic borings, and a schedule of borings in tabular format, been submitted to the District Geotechnical Engineer?	These items are provided in the attached report.
				The schedule of borings should present the following information for each boring:	
Y	N	**X**		□ exploration identification number	
Y	N	**X**		□ location by station and offset	
Y	N	**X**		□ estimated amount of rock and soil, including the total for each for the entire program.	
Planning – Exploration Number					
Y	N	X	11	Have the coordinates, stations and offsets of all explorations (borings, probes, test pits, etc.) been identified?	
Y	N	X	12	Has each exploration been assigned a unique identification number, in the following format X-ZZZ-W-YY, as per Section 303.2 of the SGE?	
Y	N	**X**	13	When referring to historic explorations that did not use the identification scheme in 12 above, have the historic explorations been assigned identification numbers according to Section 303.2 of the SGE?	

Notes:

II. Reconnaissance and Planning Checklist

Planning – Boring Types		
Y N X 14	Based on Sections 303.3 to 303.76 of the SGE, have the location, depth, and sampling requirements for the following boring types been determined for the project? Check all boring types utilized for this project: □ Existing Subgrades (Type A) □ Roadway Borings (Type B) □ Embankment Foundations (Type B1) □ Cut Sections (Type B2) □ Sidehill Cut Sections (Type B3) □ Sidehill Cut-Fill Sections (Type B4) □ Sidehill Fill Sections on Unstable Slopes (Type B5) □ Geohazard Borings (Type C) □ Lakes, Ponds, and Low-Lying Areas (Type C1) □ Peat Deposits, Compressible Soils, and Low Strength Soils (Type C2) □ Uncontrolled Fills, Waste Pits, and Reclaimed Surface Mines (Type C3) □ Underground Mines (C4) □ Landslides (Type C5) □ Karst (Type C6) □ Proposed Underground Utilities (Type D) □ Structure Borings (Type E) □ Bridges (Type E1) X Culverts (Type E2 a,b,c) □ Retaining Walls (Type E3 a,b,c) □ Noise Barrier (Type E4) □ High Mast Lighting Towers (Type E5) □ Buildings and Salt Domes (Type E6)	

Notes:

IV.A Foundations/Structures - Non-bridge Applications

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If you do not have such a foundation or structure on the project, you do not have to fill out this checklist.

Soil and Bedrock Strength Data			
Y N X 1	Has the shear strength of the foundation soils been determined? Check method used: X laboratory shear tests X estimation from SPT or field tests		
Y N X 2	Have sufficient soil shear strength, consolidation, and other parameters been determined so that the required allowable loads for the foundation/structure can be designed?		
Y N **X** 3	Has the shear strength of the foundation bedrock been determined? Check method used: □ laboratory shear tests □ other List Other items:		

Notes:

Stage 1:

IV.A Foundations/Structures - Non-bridge Applications

Spread Footings						
Y	N		4		Are there spread footings on the project? If no, go to Question **11**	
Y	N	X	5		Has the recommended bottom of footing elevation and reason for this recommendation been provided?	
Y	N	X		a	Has the recommended bottom of footing elevation taken scour from streams or other water flow into account?	Data provided for scour evaluation by civil engineer.
			6		Were representative sections analyzed for the entire length of the structure for the following:	
Y	N	X		a	bearing capacity?	
Y	N	X		b	sliding?	
Y	N	**X**		c	overturning?	
Y	N	X		d	settlement?	
Y	**N**	X	7		Has the need for a shear key been evaluated?	
Y	N	**X**		a	If needed, have the details been included in the plans?	
Y	N	**X**	8		If special conditions exist (e.g. geometry, sloping rock, varying soil conditions), was the bottom of footing “stepped” to accommodate them?	
Y	N	X	9		Has the recommended allowable soil or rock bearing pressure been provided?	LRFD factored bearing resistance at strength and service limit states.
Y	N	**X**	10		If weak soil is present at the proposed foundation level, has the removal / treatment of this soil been developed and included in the plans?	Recommendations provided in report. Plans to be prepared by others.
Y	N	**X**		a	Have the procedure and quantities related to this removal / treatment been included in the plans?	

Notes:

Stage 1:

IV.A Foundations/Structures - Non-bridge Applications

Pile Structures				
Y **N**		11	Are there piles on the project? If no, go to Question **17**	
Y N		12	Has an appropriate pile type been selected? Check the type selected: □ H-pile (driven) □ H-pile (drilled) □ Cast In-place Concrete □ other List Other items:	
Y N	X	13	Have the estimated pile length or tip elevation and section (diameter) been specified? Check method used: □ SPILE, DRIVEN, or equivalent software □ hand calculations	
		14	If required for design, have sufficient soil parameters been provided and calculations performed to evaluate the:	
Y N	X		a Lateral load capacity and maximum deflection of the piles?	
Y N	X		b Vertical load capacity and maximum settlement of the piles?	
Y N	X		c Negative skin friction on piles driven through new embankment or soft foundation layers?	
Y N	X		d Potential for and impact of lateral squeeze from soft foundation soils?	
Y N	X	15	If piles are to be driven to bedrock, have “pile points” been recommended to assure secure contact with the rock surface, as per BDM 202.2.3.2.a?	
Y N	X	16	If subsurface obstacles exist, has preboring been recommended to avoid these obstructions?	

Notes:

Stage 1:

IV.A Foundations/Structures - Non-bridge Applications

Drilled Shafts					
Y	**N**		17	Are there drilled shafts on the project? If no, go to the next checklist.	
Y	N	X	18	Have the drilled shaft diameter and embedment length been specified?	
Y	N	X	19	Have the recommended drilled shaft diameter and embedment been developed based on side friction and end bearing for vertical loading situations?	
			20	For shafts undergoing lateral loading, have the following been determined:	
Y	N	X		a. maximum lateral shear	
Y	N	X		b. maximum bending moment	
Y	N	X		c. maximum deflection	
Y	N	X		d. reinforcement design	
Y	N	X	21	Generally, bedrock sockets are 6" smaller in diameter than the soil embedment section of the drilled shaft. Has this factor been accounted for in the drilled shaft design?	
Y	N	X	22	If a bedrock socket is required below soil embedment, have separate quantities been estimated based on shaft diameters and materials to be excavated?	
Y	N	X	23	Has the site been assessed for groundwater influence?	
Y	N	X		a If yes, if artesian flow is a potential concern, does the design address control of groundwater flow during construction?	
Y	N	X	24	If special construction features (e.g., slurry, casing, load tests) are required, have all the proper items been included in the plans?	

Notes:

Stage 1

VI.D. Geotechnical Reports

C-R-S: LUC-Seaman Road	PID: 107148	Reviewer: CPI	Date: 02-14-20

General		
Y N X 1	Has the first complete version of a geotechnical report being submitted been labeled as 'Draft'?	This is the final submittal.
Y N X 2	Subsequent to ODOT's review and approval, has the complete version of the revised geotechnical report being submitted been labeled 'Final'?	
Y N X 3	Have all geotechnical reports being submitted been titled correctly as prescribed in Section 705.1 of the SGE?	

Report Body		
Y N X 4	Do all geotechnical reports being submitted contain an Executive Summary as described in Section 705.2 of the SGE?	
Y N X 5	Do all geotechnical reports being submitted contain an Introduction as described in Section 705.3 of the SGE?	
Y N X 6	Do all geotechnical reports being submitted contain a section titled "Geology and Observations of the Project," as described in Section 705.4 of the SGE?	
Y N X 7	Do all geotechnical reports being submitted contain a section titled "Exploration," as described in Section 705.5 of the SGE?	
Y N X 8	Do all geotechnical reports being submitted contain a section titled "Findings," as described in Section 705.6 of the SGE?	
Y N X 9	Do all geotechnical reports being submitted contain a section titled "Analyses and Recommendations," as described in Section 705.7 of the SGE?	

VI.D. Geotechnical Reports

Appendices		
Y N X 10	Do all geotechnical reports being submitted contain all applicable Appendices as described in Section 705.8 of the SGE?	
Y N X 11	Do the Appendices present a site Boring Plan showing all boring locations as described in Section 705.8.1 of the SGE?	
Y N X 12	Do the Appendices include boring logs as described in Section 705.8.2 of the SGE?	
Y N X 13	Do the Appendices present reports of undisturbed test data as described in Section 705.8.3 of the SGE?	
Y N X 14	Do the Appendices present calculations in a logical format to support recommendations as described in Section 705.8.4 of the SGE?	

Notes: