KS Associates
Elyria, Ohio

Geotechnical Subsurface Investigation
OTIC 71-19-02 -Bridge Demolition
MP 98.9 over Abandoned Norfolk Southern Railroad (NSRR)
Sandusky County, Ohio

November 2019

TTL associates inc
Environmental, Geotechnical Engineering & Testing

1915 North 12th Street
Toledo, OH 43604-5305
T 419-324-2222
F 419-241-1808
www.ttlassoc.com

November 18, 2019 **TTL Project No. 1852901**

Mr. Hamid V. Homaee, P.E.
KS Associates Inc.(KS)
260 Burns Road, Suite 100
Elyria, OH 44035

Geotechnical Subsurface Investigation
OTIC 71-19-02 -Bridge Demolition
MP 98.9 over Abandoned Norfolk Southern Railroad (NSRR)
Sandusky County, Ohio

Dear Mr. Hamid:

Following is the report of the geotechnical subsurface investigation performed by TTL Associates, Inc. (TTL) at the site of the referenced project. This investigation was performed in general accordance with TTL Proposal No. 1852901-rev1, dated July 23, 2019 and subsequent authorization.

This final report contains the results of our study, incorporates furnished ground surface elevations into the boring logs, and provides our construction recommendations for embankment fill placement.

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

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

Sincerely,

TTL Associates, Inc.

Imad El Hajjar
Geotechnical Project Manager

David M. Vovak, P.E.
Cleveland Operations Director

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GEOTECHNICAL SUBSURFACE INVESTIGATION
OTIC 71-19-02 -BRIDGE DEMOLITION
MP 98.9 OVER ABANDONED NORFOLK SOUTHERN RAILROAD (NSRR)
SANDUSKY COUNTY, OHIO

FOR

KS ASSOCIATES INC
260 BURNS ROAD, SUITE 100
ELYRIA, OH 44035

SUBMITTED

November 18, 2019
TTL PROJECT NO. 1852901

TTL ASSOCIATES, INC.
1915 NORTH 12TH STREET
TOLEDO, OHIO 43604
(419) 324-2222
(419) 321-6257 (FAX)

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1.0 INTRODUCTION

This geotechnical subsurface investigation report has been prepared for the bridge demolition project in Sandusky County, Ohio. The project is located at mile post (MP) 34.2 of the James W. Shocknessy Ohio Turnpike (Interstate Route 90 [IR-90]) approximately ¼ mile west of State Route (SR) 501, as 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 utilized to evaluate the subsurface conditions at the site, presents our findings from the field and laboratory testing, and provides our design and construction recommendations for embankment construction.

This investigation was performed in general accordance with TTL Proposal No. 1852901-rev1, dated July 23, 2019 and subsequent authorization.

The purpose of this investigation was to evaluate the subsurface conditions relative to the design and construction of embankments at the referenced location. To accomplish this, TTL performed two test borings, field and laboratory soil testing, a geotechnical engineering evaluation of the test results, and review of available geologic and soils data for the project area. TTL also performed 10 pavement cores for an assessment of the shoulder conditions across the project length.

This report includes:

- A description of the subsurface soil and groundwater conditions encountered in the borings.
- A description of the existing pavement cross-section encountered at the coring locations.
- Design recommendations for embankments.
- Recommendations concerning soil and groundwater-related construction procedures such as site preparation, earthwork, foundation construction, and related field testing.

The scope of this study did not include an environmental assessment of the subsurface materials at this site.

2.0 INVESTIGATIVE PROCEDURES

Two test borings, designated as Borings B-1 and B-2, were drilled by TTL on July 24, 2019. Boring B-1 was performed on the north side of the existing twin bridges, and Boring B-2 was performed on the south side. The boring locations were established in the field by TTL based on a site plan provided by KS. Ground surface elevations at the boring locations were provided by KS. The approximate locations of the test borings are shown on the attached Test Boring Location Plan (Plate 2.0).

10 Pavement cores, located on the shoulders along the project site were obtained using a 4-inch diameter single-wall, diamond-tipped core barrel attached to the drill rig. After pavement coring was completed, pavement thickness was measured along the sidewall of the core-hole.

The test borings were performed in general accordance with geotechnical investigative procedures outlined in ASTM Standards D 1586 and D 5434. The test borings performed during this investigation were drilled with a ATV-mounted rotary drilling rig utilizing 3¼-inch inside diameter hollow-stem augers. Borings B-1 and B-2 were terminated upon split spoon and auger refusal and extended to the depth of 53.8 and 51.2 feet below existing grades, respectively.

During auger advancement, soil samples were collected at 2½-foot intervals to a depth of 10 feet and at 5-foot intervals thereafter using a split-spoon sampler. The soil samples were sealed in jars and transported to our laboratory for further classification and testing.

Split-spoon (SS) samples were obtained by the Standard Penetration Test (SPT) Method (ASTM D 1586), which consists of driving a 2-inch outside diameter split-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. The sum of the number of blows required to advance the sampler the second and third 6-inch increments is termed the Standard Penetration Resistance (N-value) and is presented on the Logs of Test Boring attached to this report. The samples were sealed in jars and shipped to our laboratory for further classification and testing.

Shelby tube samples, designated ST on the Logs of Test Borings, were obtained from 16 to 18 feet and from 36 to 38 feet in Boring B-1, as well as from 21 to 23 feet and from 31 to 33 feet in Boring B-2. Each of the Shelby tube samples were obtained by hydraulically advancing a 3-inch diameter, thin walled sampler approximately 24 inches beyond the hollow stem auger into relatively undisturbed soil in accordance with ASTM D 1587. The Shelby tubes were then extracted from the subsoils, and the ends were capped and sealed. The samples were transported to our laboratory where they were 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, water conditions observed in the borings, and laboratory test data. It should be noted that these logs have been prepared on the basis of laboratory classification and testing, as well as field logs of the encountered soils.

All of the recovered samples of the subsoils were visually or manually classified in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 and D 2488). All samples of the subsoils were tested in our laboratory for moisture content (ASTM D 2216). The Shelby tube samples and selected intact cohesive split-spoon samples were tested for dry density and unconfined compressive strength utilizing constant rate of strain methods (ASTM D 2166). 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 four selected samples from each boring. The results of these tests are presented on the Logs of Test Borings, and Tabulation of Test Data sheets attached to this report.

One-dimensional consolidation tests (ASTM D 2435) were performed on samples from Boring B-1 (ST-2) and B-2 (ST-1). The results of these tests are attached to this report.

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, 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, excavation, and foundation phases of the proposed project. This is to observe compliance with the design concepts, specifications, and recommendations, and to allow design changes in the event subsurface conditions differ from those anticipated prior to the start of construction.

3.0 PROPOSED CONSTRUCTION

We understand that it is planned to demolish the existing twin bridges located at milepost (MP) 98.9 of the James W. Shocknessy Ohio Turnpike (Interstate Route 90 [IR-90]), approximately ¼ mile west of State Route (SR) 510, in Sandusky County, Ohio. The bridges formerly served as an overpass over the NSRR, and the railroad has since been demolished. Prior to bridge demolition, grades below the bridges will be raised to match existing embankment grades. We further understand that this work will be phased to accommodate traffic as needed, and that the Ohio Turnpike Infrastructure Commission (OTIC) can accommodate construction over multiple years if needed to allow for settlement of the new embankment.

We further understand that the existing bridges are 3-span structures, with abutments and piers supported on 12-inch, cast-in-place concrete pile foundations with an allowable design load of 40 tons, as noted in the table below as complied from a review of the Original Construction Plans provided:

Element	Pile Cap Bearing Elev. (ft msl)	Estimated Avg. Pile Length (ft)	Est. Pile Tip Elev. (ft msl)
Abutment Upper	630.5	67.32	563.18
Abutment Lower	623.5	61.76	561.74
Piers	599.5	39.68	559.82

Based upon the Original Construction Plans, Spans 1 and 3 of the duel bridges are approximately 49 feet in length and the middle span (Span 2) is approximately 55.5 feet in length. Existing grades below the bridges are roughly Elev. 606 feet, and roadway elevation across the existing bridges range from roughly Elev. 633 to 634 feet; therefore, approximately 26 to 27 feet of fill is required to raise the existing grades below the bridges to match the roadway elevations.

4.0 GENERAL SITE AND SUBSURFACE CONDITIONS

4.1 General Site Conditions

Existing grades below the bridges are on the order of Elev. 606. Grades along the bridges are on the order of Elev. 633 to 634.

The surface materials encountered in Borings B-1 and B-2 consisted of crushed stone on the order of 10 inches thick.

4.2 Site Geology

Published geologic maps from the Ohio Department of Natural Resources (ODNR) indicate that the project site is located within the glaciated portion of Ohio. These regions include upper profile soils consisting of **Lacustrine Silt (LL)**, deposited in low-velocity water of glacial and slack-water lakes; may contain fine sand or clay; well-laminated in distal portions of deltas, poorly-laminated elsewhere. These lacustrine deposits are underlain by predominantly silty and clayey glacial till, before encountering bedrock.

Bedrock at the site consist of the Upper and Lower Silurian Salina Group formation. The formation consists **of interbedded layers of Dolomite, anhydrite, gypsum, salt, and shale**. ODNR references indicate top of rock at elevations roughly elevations 550 roughly 560 feet msl.

4.3 Pavement Along Existing Shoulders

All the pavement cores were performed within the existing shoulder areas along the project length. The encountered surface materials at the core locations are summarized in the following table.

Table 4.3. Encountered Pavement Thickness and Conditions			
Core Number	Pavement Thickness (Inches)		
	Asphalt	Concrete	Crushed Stone
L-1	3.25	9.5	15.0
L-2	2.75	10.5	15.0
L-3	4.5	8.25	15.0
L-4	3.5	9.0	15.0
L-5	3.5	10.5	15.0
R-1	3.0	9.0	16.0
R-2	3.75	11.5	16.0
R-3	3.5	9.0	14.0
R-4	4.75	9.5	14.5
R-5	4.0	8.0	13.5

Photographic pavement core logs are attached to this report.

In summary, the asphalt thickness encountered ranged between approximately 2.75 to 4.75 inch-thick. A layer of concrete was encountered underlying the asphalt, this layer was observed to be 8.0 to 11.5 inch-thick. The encountered crushed stone subbase was observed to be 13.5 to 15 inch-thick. The total pavement cross-section encountered in the pavement cores extended to depths ranging from 2.1 feet to 2.6 feet below existing grades.

4.4 General Soil Conditions

Based on the results of our field and laboratory tests, the subsoils encountered underlying the crushed stone can generally be characterized by a layer of fill material overlying native lacustrine soils overlying a layer of cohesive glacial till overlying Dolomite bedrock. It should be noted that a layer of unconsolidated clay was encountered in B-2, these type-deposits are known to be highly compressible upon loading. Additional descriptions of the soil stratigraphy encountered in the borings are presented on the Logs of Test Borings attached to the report.

Cohesive Fill was encountered underlying the crushed stone in Borings B-1 and B-2 to depths of 2 feet and 3 feet below existing grades (approximate Elevs. 604 and 603 feet), respectively. The cohesive fill was predominantly clayey with varying amounts of sand, gravel, slag, bricks and organics. SPT N-values for the cohesive fill materials generally ranged from 13 to 15 blows per foot (bpf), indicating to stiff consistency. Unconfined compressive strengths ranged from 7,000 to 8,000 pounds per square foot (psf). Moisture contents generally ranged from 15 to 22 percent.

Stratum I consisted of predominantly medium stiff to stiff native cohesive soils encountered underlying the cohesive fill to depths of 12 feet below existing grade in Borings B-1 and B-2 (approximate Elev. 594 feet). Stratum I consisted of lean clay (CL) with sand and varying amounts of gravel. SPT N-values for the Stratum I soils ranged from 5 to 12 blows per foot (bpf). A Localized 2 foot-thick layer of very stiff native cohesive soils were encountered within this stratum in Boring B-2. Unconfined compressive strengths ranged from 4,000 to greater than 9,000 pounds per square foot (psf) were recorded within this stratum using a hand penetrometer. Moisture contents ranged from 19 to 28 percent. Liquid limits ranged from 42 to 33 percent and plasticity index ranged from 12 to 18 percent were determined for two Stratum I samples obtained from Borings B-1 (SS-2) and B-2 (SS-3). These values, along with gradation results, are indicative of lean clay (CL) in accordance with USCS designations. Based correlations from moisture content results and Atterberg limits testing, these soils are considered over-consolidated and slightly compressible.

Stratum II consisted of predominantly **very soft** to **soft** native cohesive soils encountered underlying Stratum I to depths of 23 feet below existing grade in Boring B-2 (approximate Elev. 583 feet). Stratum II consisted predominantly of lean clay (CL) with sand and varying amounts of gravel. SPT N-values for the Stratum I soils ranged from 0 to 3 blows per foot (bpf). Unconfined compressive strengths ranged from less than 500 to 500 pounds per square foot (psf) were recorded within this stratum using a hand penetrometer. Moisture contents ranged from 16 to 32 percent. Liquid limits ranged from 24 to 29 percent and plasticity index

ranged from 9 to 12 percent were determined for two Stratum I samples obtained from Borings B-2 (SS-6 and ST-1). These values, along with gradation results, are indicative of lean clay (CL) in accordance with USCS designations. **Based correlations from moisture content results and Atterberg limits testing, these soils are considered normally-consolidated and highly compressible.**

Stratum III consisted of predominantly **very soft** to **soft** native cohesive soils encountered underlying Stratum I to depths of 33½ feet below existing grade in Boring B-2 (approximate Elev. 572.5 feet). Stratum II consisted predominantly of lean clay (CL) with sand and varying amounts of gravel. SPT N-values for the Stratum III soils ranged from 2 to 3 blows per foot (bpf). Unconfined compressive strengths ranged from less than 500 to 500 pounds per square foot (psf) were recorded within this stratum using a hand penetrometer. Moisture contents ranged from 17 to 22 percent. A liquid limit of 30 percent and plasticity index of 13 percent were determined for a Stratum III sample obtained from Boring B-2 (SS-8). These values, along with gradation results, are indicative of lean clay (CL) in accordance with USCS designations. Based correlations from moisture content results and Atterberg limits testing, these soils are considered slightly over-consolidated and moderately compressible.

Stratum IV consisted of medium stiff to stiff cohesive glacial till deposits encountered underlying Stratum I to depth of 44 feet in Boring B-1 and Stratum III to depth of 43½ in Boring B-2 (approximate Elevs. 564 feet). SPT N-values ranged from 5 to 10 blows per foot (bpf). Unconfined compressive strengths ranged from 1,500 to 3,000 pounds per square foot (psf) were recorded within this stratum using a hand penetrometer. Moisture contents ranged from 13 to 19 percent. Liquid limits ranged from 29 to 30 percent and plasticity index ranged from 11 to 12 percent were determined for two Stratum I samples obtained from Borings B-1 (SS-9 and ST-2). These values, along with gradation results, are indicative of lean clay (CL) in accordance with USCS designations. Based correlations from moisture content results and Atterberg limits testing, these soils are considered over-consolidated and slightly compressible.

Stratum V consisted of hard to very hard cohesive glacial till deposits encountered underlying Stratum IV to depth of 48 and 51.5 feet in Borings B-1 and B-2 (approximate Elevs. 558 and 554.5 feet), respectively. SPT N-values ranged from 31 to 54 blows per foot (bpf). Unconfined compressive strengths ranged from 7,000 to greater than 9,000 pounds per square foot (psf) were recorded within this stratum using a hand penetrometer. Moisture contents ranged from 15 to 17 percent. These values, along with gradation results, are indicative of lean clay (CL) in accordance with USCS designations. Based correlations from moisture content results and

Atterberg limits testing, these soils are considered over-consolidated and somewhat compressible.

Highly Weathered Bedrock consisted of Dolomite was encountered underlying Stratum IV to depth of 51.2 and 53.8 feet in Borings B-1 and B-2 (approximate Elevs. 555 and 557.5feet), respectively. Both boring were terminated within layer.

4.5 Groundwater Conditions

Groundwater was initially encountered during drilling at depths of 16.5 feet and 28.2 feet below existing grades (approximate Elevs. 589.6 and 577.9) in Borings B-1 and B-2, respectively. Groundwater was observed upon completion of drilling operations at depths of 48.5 feet and 48.0 feet below existing grades (approximate Elevs. 557.5 and 558) in Borings B-1 and B-2, respectively. It should be noted that each of the borings 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. The groundwater conditions encountered in the borings are summarized in the following table.

Table 4.5 Encountered Groundwater Conditions

Boring Number	Approximate Ground Surface Elevation	Groundwater Initially Encountered During Drilling		Groundwater Observed Upon Completion of Drilling Operations	
		Depth (feet)	Approx. Elevation (feet)	Depth (feet)	Approx. Elevation (feet)
B-1	606.1	16.5	589.6	48.5	557.5
B-2	606.1	28.2	577.9	48.0	558.0

Based on the soil characteristics and groundwater conditions encountered in the borings, it is our opinion that the “normal” long-term groundwater table will be generally encountered at depths of approximately 12 feet or lower, corresponding to approximate Elev. 594 or deeper. However, groundwater elevations can fluctuate with seasonal and climatic influences. In particular, “perched” water may be encountered in fill materials or granular soils that are underlain by relatively impermeable cohesive soils. Therefore, the groundwater conditions may vary at different times of the year from those encountered during this investigation.

5.0 DESIGN RECOMMENDATIONS

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

5.1 New Embankment Fill

Fill will be placed below the existing bridge to create a new embankment at the project location. Maximum fill heights are anticipated to be on the order of 25 to 28 feet.

For each of the encountered soil strata, soil compressibility parameters were evaluated for use in embankment settlement calculations. The compressibility parameters were evaluated using one-dimensional consolidation test results, as well as correlations with moisture contents and Atterberg limits test results. As discussed in Section 4.3, Stratum II contains compressible normally consolidated soils and, as such, controls the settlement calculations with potential settlement exceeding 3 inches in some scenarios. To control settlement, light weight fill such as Foam Cellular Concrete Fill (FCCF) or EPS Geofoam may be utilized. Calculated settlements utilizing conventional fill and light weight fill options are provided in the table below, and our calculations are attached to this report.

Table 5.1 Summary of Settlement Calculations

Boring Number	Location	Embankment Unit Weight - Conventional γ_{avg} = 135 pcf		Embankment Unit Weight - FCCF γ_{avg} = 50 pcf		Embankment Unit Weight -Geofoam γ_{avg} = 25 pcf	
		Total (in)	Differential Abut/Pier (in)	Total (in)	Differential Abut/Pier (in)	Total (in)	Differential Abut/Pier (in)
B-1	Abutment	0.1	N/A	0	N/A	0	N/A
	Pier	1.7		1.1		0.8	
B-2	Abutment	0.3	N/A	0	N/A	0	N/A
	Pier	3.1		0.8		0.1	

In lieu of, or in combination with, the light-weight fill noted above, the compressible soils may be bridged (thus significantly limiting settlement) utilizing various proprietary soil improvements methods designed by specialty contractors.

Based on consolidation test results and correlations with soil index properties, as well as the indicated fill heights and range of compressible cohesive soil layer thicknesses, the time required limit additional settlement over the first year to roughly 1 inch or less was calculated to be on the order of 8 to 12 weeks. The estimated 8-to 12-week time rate of settlement is based on the "clock" starting at time t=0 when all of the fill is in place (in effect, assuming the surcharge load is applied instantly over the area). In reality, construction of the fill is expected to require 2 to 4 two weeks (or more), so some of the consolidation and settlement will be initiated and occurring during the fill placement period, thereby shortening the post-fill-placement waiting period. For this reason, it is imperative that the planned settlement platforms are installed and surveyed at the very prior to any fill operations.

For the higher settlement scenarios, wick drains become a cost-effective alternative as they significantly increase the settlement rates. Wick drain systems are proprietary in nature; however, a system installed 5-foot intervals to depths of 35 feet would generally cut the settlement time by a factor of 4 or greater: reducing a 6 month wait to 6 weeks or less.

Due to the above-noted concerns with settlement, we recommend that settlement be monitored throughout construction with settlement platform as detailed in Section 6.2.1 of this report.

5.1.1 Embankment Loading on Pile Cap

The pier structures are supported on a deep foundation system with a pile cap structure transferring the pier loads to the 12-inch CIP piles driven presumably to bedrock. Therefore, unlike a pier supported on a shallow foundation system which settles and proportionally gains capacity with the increased embankment load, a pier supported on a deep foundation system experiences negligible settlement and gains no capacity; therefore, the embankment load must be considered in design. Assuming 28 feet of fill and an average unit weight of 135 pcf, a uniform load of 3780 psf will be applied to the pier cap structure and transferred proportionally to the piles.

5.1.2 Downdrag

Downdrag on the foundations due to settlement could result in additional loading on the exposed foundation elements in excess of 400 kips per pier. Therefore, we recommend protecting the exposed foundation elements from the effect of downdrag.

Therefore, we recommend that the exposed portions of the bridge substructures be isolated from the proposed fill by coating the portions of the abutments and piers that are above existing grade with low viscosity bituminous asphalt and then covering or wrapping those components with a durable thick plastic (visqueen) which significantly reduces downdrag along the exposed portions.

As per Section 3.11.8 of the AASHTO LRFD Bridge Design Specifications (BDS), negative skin friction should be factored in layers where there is 0.4 inches or greater of relative soil movement between the piles and surrounding soil. Therefore, due to the settlements noted above coupled with the foundations bearing in rock, the above-noted criteria will be exceeded at the pier locations and the resulting negative skin friction may be calculated utilizing the information provided in the table below:

Table 5.1.1 Downdrag Skin Friction Values using Engineered Fill

Boring Number	Elevation Range (feet)	Undrained Shear Strength S_u (psf)	Adhesion Factor (α)	Negative Skin Friction F_s (psf)
B-1	606 - 594	2750	0.35	965
	594 - 567	1000	0.85	850
	567 - 565.5	1500	0.80	1200
B-2	606 - 594	2750	0.35	965
	594 - 577	250	1.00	250
	577-573	750	1.00	750
	573 - 567	1000	0.85	850

The net load on the underlying soils and pier cap can be reduced by utilizing a combination of conventional and light-weight fill. In regards to the above-noted table, the unit negative skin friction values would remain constant regardless of the net load applied by the embankment; however, the length of the pile where the unit loads are applied will increase or decrease as a function of increasing or decreasing settlement, respectively.

5.2 Groundwater Control

As stated previously, groundwater was initially encountered during drilling at depths of 16.5 feet and 28.2 feet below existing grades (approximate Elevs. 589.6 and 577.9) in Borings B-1 and B-2, respectively. Groundwater was observed upon completion of drilling operations at depths of 48.5 feet and 48.0 feet below existing grades (approximate Elevs. 557.5 and 558) in Borings B-1 and B-2, respectively.

Based on the soil and groundwater conditions encountered in the borings, it is our opinion that the "normal" long-term groundwater table will be generally encountered at depths of approximately 12 feet or lower, corresponding to approximate Elev. 594 or deeper. Therefore, construction planning should include potential remedial measures to be implemented where excessive groundwater seepage or unstable subgrades are encountered if excavations extend to approximate Elev. 594. Dewatering methods may include multiple sumps or a system of well points. The type of dewatering system utilized will depend on construction practices, soil conditions encountered in the foundation excavations, seasonal conditions, and the depth of excavation. Additionally, the contractor will need to exercise diligence to control seepage and runoff to maintain a stable subgrade.

5.3 Excavations and Slopes

The sides of temporary excavations for construction should be adequately sloped to provide stable sides and safe working conditions. Otherwise, the excavation must be properly braced against lateral movements. In any case, applicable OSHA safety standards must be followed.

Based on the test borings, it is likely that excavations will encounter a range of soil conditions that include 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

For temporary excavations in Type A and B soils, side slopes must be no steeper than 1 horizontal to 1 vertical (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, we recommend that grades be no steeper than 3H:1V without a more extensive geotechnical evaluation of the proposed construction plans and site conditions.

New embankment slopes are anticipated to match existing embankment slopes which are indicated at 3 horizontal to 1 vertical (3H:1V). Regardless of overall global slope stability, slopes graded steeper than 3H:1V may be prone to shallow surface sloughing. This type of shallow sliding is generally not problematic (by itself), but left unchecked, it can lead to progressive slope movements that eventually impact overall performance of the embankment.

In addition to slope protection, such as well-established vegetative cover and rock-lined channels in surface run-off collection ditches and swales, we recommend that surface drainage from pavement areas on the crest of the embankment should be directed to catch basins or storm drains and not allowed to sheet flow over the slope.

5.4 Slope Stability Analysis

We performed global slope stability analyses using the 2-D Limit Equilibrium Slope Stability Program Slide 6.0 by Rocscience to evaluate proposed 3H:1V embankment slope throughout the Site, and to ensure permanent embankment slope designs having factors of safety greater than 1.3 for static conditions. Using this program, a myriad of potential failure surfaces can be generated theoretically, from which the factor of safety can be determined as to whether sufficient resisting soil strength can be mobilized to counteract the driving forces (weight of soil, seepage, and surcharge loads) that would cause the slope to move downward. The factor of safety is the ratio of the resisting forces to the driving forces.

Global instability typically is manifested by pronounced movements of a large arc or wedge of soil that result in bulging at the toe of the slope as well as observable displacement of soil at or near the crest of the slope. This crest displacement may be exhibited by a near-vertical tension crack at the back edge of the displaced soil mass, or may be significant enough to exhibit a downward movement of soil that creates a “scarp” such that a sharp drop occurs in an otherwise level ground surface. Global instability of the embankment at this site could create a significant impact due to the potential for such movement to encompass a portion of the roadway.

We analyzed two cases on one typical embankment section located along the center span of the existing twin bridges. The first case simulated the short term conditions using total stress soil parameters (TSSP) and the second case simulated the long term conditions using effective stress soil parameters (ESSP). It should be noted that the properties of the embankment and foundation strata vary somewhat with layer and depth; the layers and assigned soil properties used in the analyses are detailed in the attached figures that illustrate the assumed embankment geometry and potential critical failure surfaces associated with the global stability of the embankment. The soil parameters utilized for analysis are presented on the slope stability outputs attached to this report.

The results are summarized in Table 5.4 below.

Table 5.4 Summary of Settlement Calculations

Model	Method	Slope	Factor of Safety
1	Total Stress Soil Parameters (TSSP)	3H:1V fill slope	1.4
2	Effective Stress Soil Parameters (ESSP)	3H: 1V fill slope	1.5

6.0 CONSTRUCTION RECOMMENDATIONS

6.1 Site Preparation

Site preparation activities should include the removal of topsoil, root mats, vegetation, pavements, and other deleterious non-soil materials from all proposed construction areas. Suitable topsoil may be stockpiled for later use in landscaped areas. The actual amount of required stripping should be determined in the field by a geotechnical engineer or qualified representative.

6.2 Embankment Construction for Replacement of Bridge Structures

Prior to placement of fill, the exposed portions of the bridge substructures shall be isolated from the proposed fill by coating the portions of the abutments and piers that are above existing grade with low viscosity bituminous asphalt and then covering or wrapping those components with a durable thick plastic (visqueen). Fill placement can be placed as possible and deemed practical by the contractor under the existing bridge superstructures with special provisions as defined below. It has been assumed that this stage of fill placement will reach within 8 feet of the bottom of the existing bridge beams.

6.2.1 Instrumentation

With both the pile and abutments supported on deep foundation systems, settlement of the structural units should be negligible. However, settlement of the underlying soils could impact the pavement profile. Moreover, due to the concerns with additional loading to the piles, it is recommended that the structure be monitored for settlement for precautionary reasons.

Settlement platforms should be installed prior to the beginning of the placement of the backfill. In addition, survey points at each substructure unit should be established and monitored on a regular basis. Bridge structure settlement criteria should be established by the structural engineer and monitored during all phases of embankment construction. The structural engineer may also elect to jack the bridge if differential settlement levels exceeds pre-established tolerances. Fill placement limitations will be based on actual settlement monitoring.

TTL would be pleased to review construction plans to provide a recommendation on the quantity and locations of settlement platforms.

As noted above, settlement platforms are recommended for this project to monitor settlement as fill is placed. Settlement platforms shall be fabricated and installed in general accordance with ASTM D 6598. We recommend that each platform be surveyed by the contractor's surveyor three times per week during fill operations and an average of once per week throughout the monitoring period which should be a minimum of 4 weeks.

Surveys of the platforms will also need to be performed immediately prior to and immediately after installing extensions during fill placement activities. Each settlement monitor survey record should include a record of the top of fill elevation adjacent to the settlement monitor. A fill settlement rate of roughly 0.2 inches/week is generally considered acceptable for pavement construction for the specific soil profile in question.

6.3 Fill

Material for engineered fill or backfill required to achieve design grades should meet ODOT Item 203 "Embankment Fill" placement and compaction requirements. In general, suitable fills may consist of any non-organic soils having a maximum dry density as determined by Supplement 1015 of 100 pounds per cubic foot (pcf) or greater. On-site soils may be used as engineered fill materials provided that they are free of organic matter, debris, excessive moisture, and rock or stone fragments larger than 3 inches in diameter. Depending on seasonal conditions, the on-site soils may be wet of optimum and may require scarification and aeration to achieve satisfactory compaction. If the construction schedule does not allow for scarification and aeration activities, it may be more practical or economical to utilize imported granular fill.

Fill should be placed in uniform layers not more than 8 inches thick (loose measure) and adequately keyed into stripped and scarified soils. All fill placed within pavement areas should be compacted to a dry density consistent with the requirements of ODOT Item 203, based on the maximum dry density as determined by Supplement 1015.

Fill placement shall be performed as symmetrical as possible across the entire site to prevent lateral stresses from developing on the existing bridge piers and their foundation components. Compaction of the new fill placement around the existing bridge piers and abutments shall be accomplished through the use of portable compaction equipment (hand operated tampers or other equipment approved by the engineer). Furthermore, it is recommended to that self-propelled heavy compaction equipment be kept at least 5 feet away from the existing substructure elements.

The on-site soils consist of native cohesive soils. For the cohesive soils, a sheepsfoot roller should provide the most effective soil compaction. For granular soils, or if new granular engineered fill is placed, a vibratory smooth-drum roller would be required to provide effective compaction.

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.

7.0 QUALIFICATIONS OF RECOMMENDATIONS

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

The evaluations and recommendations presented in this report have been formulated on the basis of reported or assumed data relating to the location and finished grades for the proposed structure. Any significant change in this data in the final design plans should be brought to our attention for review and evaluation with respect to the prevailing subsoil conditions.

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 and our findings have been derived 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

Plate 1.0	**Site Location Map**
Plate 2.0	**Test Boring Location Plan**
Plate 3.0	**Pavement Core Location Plan**
Plate 3.1	**Pavement Core Location Plan**

LEGEND

APPROXIMATE SITE LOCATION

N

APPROXIMATE SCALE – FEET

0 800 1,600

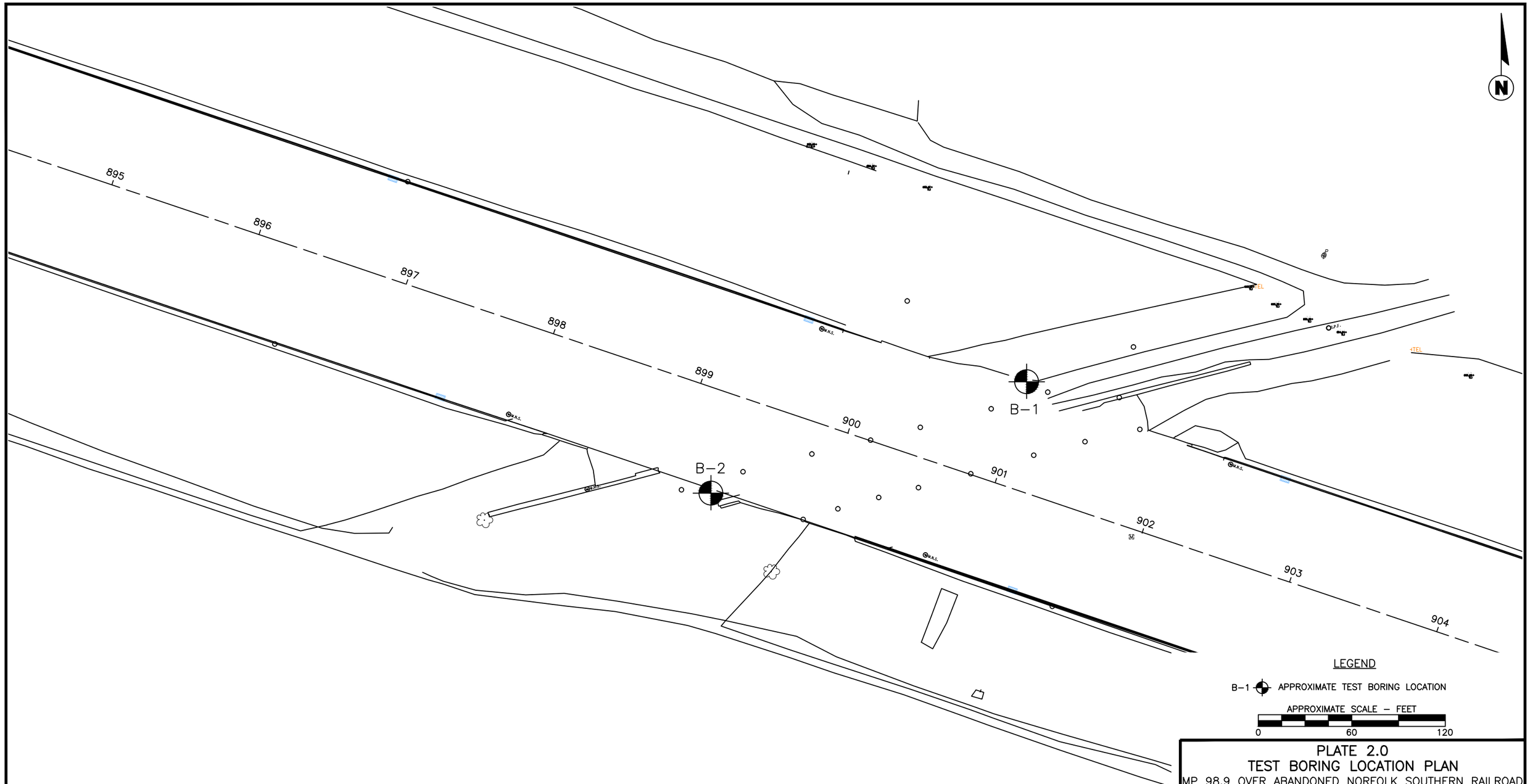
PLATE 1.0
SITE LOCATION MAP

MP 98.9 OVER ABANDONED NORFOLK SOUTHERN RAILROAD
OHIO TURNPIKE MP 98.9
SANDUSKY COUNTY, OHIO

PREPARED FOR
KS ASSOCIATES
ELYRIA, OHIO

DRAWN	TRR/10-24-19	CHECKED	
REVISED		APPROVED	
JOB NO.	1852901		
DRAWING NUMBER	**1852901-01G**		

TTL associates inc
Environmental, Geotechnical Engineering & Testing



LEGEND

B-1 APPROXIMATE TEST BORING LOCATION

APPROXIMATE SCALE – FEET

0 60 120

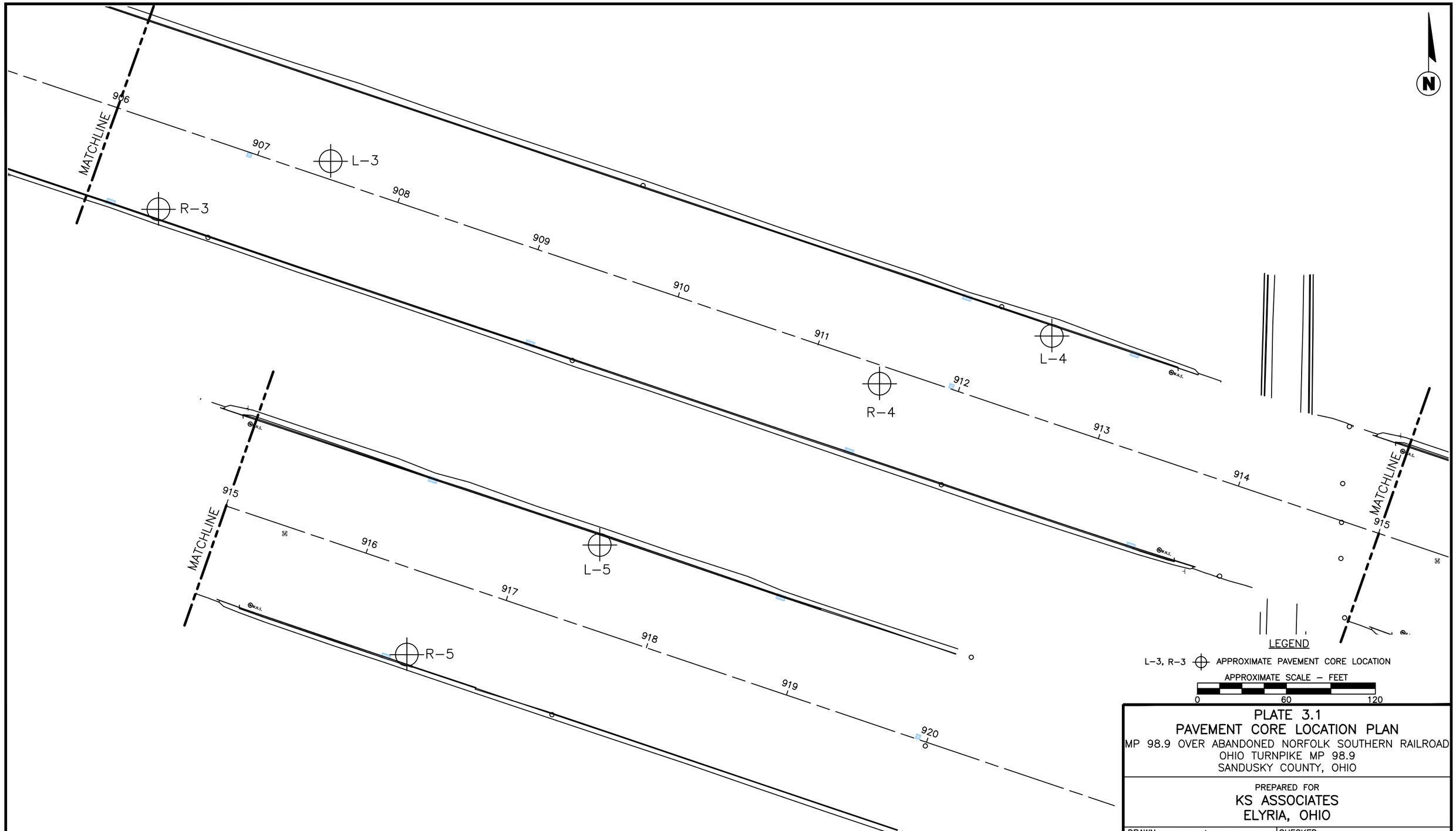
BASE DRAWING PROVIDED BY KS ASSOCIATES.

PLATE 2.0
TEST BORING LOCATION PLAN
MP 98.9 OVER ABANDONED NORFOLK SOUTHERN RAILROAD
OHIO TURNPIKE MP 98.9
SANDUSKY COUNTY, OHIO

PREPARED FOR
KS ASSOCIATES
ELYRIA, OHIO

DRAWN TRR/10-24-19	CHECKED
REVISED	APPROVED
JOB NO. 1852901	
DRAWING NUMBER 1852901-02G	

associates inc
Environmental, Geotechnical Engineering & Testing



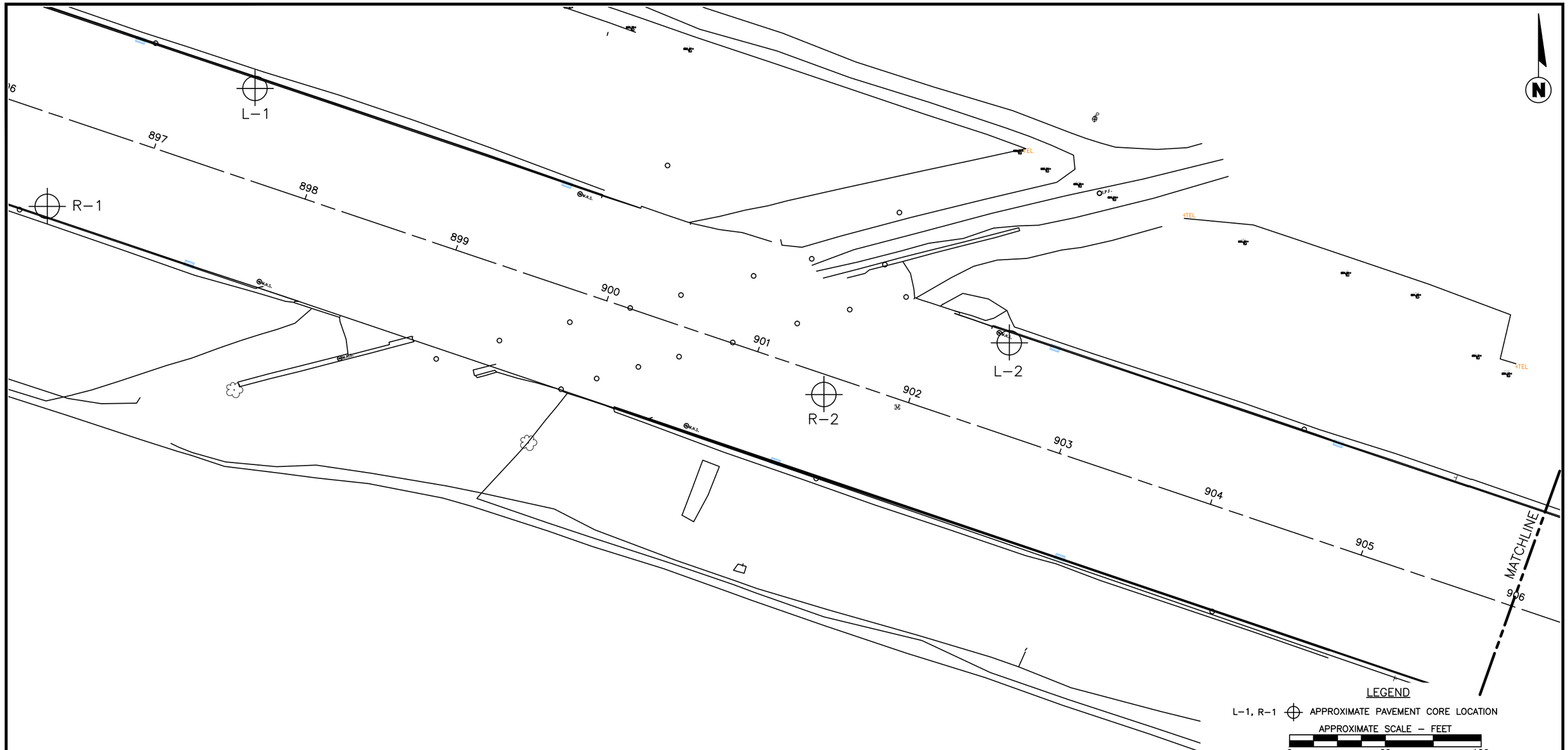
BASE DRAWING PROVIDED BY KS ASSOCIATES.

PLATE 3.1
PAVEMENT CORE LOCATION PLAN
MP 98.9 OVER ABANDONED NORFOLK SOUTHERN RAILROAD
OHIO TURNPIKE MP 98.9
SANDUSKY COUNTY, OHIO

PREPARED FOR
KS ASSOCIATES
ELYRIA, OHIO

DRAWN	TRR/10-24-19	CHECKED	
REVISED		APPROVED	
JOB NO.	1852901		
DRAWING NUMBER	1852901-03.1G		

ITL associates inc
Environmental, Geotechnical Engineering & Testing



BASE DRAWING PROVIDED BY KS ASSOCIATES.

PLATE 3.0
PAVEMENT CORE LOCATION PLAN
MP 98.9 OVER ABANDONED NORFOLK SOUTHERN RAILROAD
OHIO TURNPIKE MP 98.9
SANDUSKY COUNTY, OHIO

PREPARED FOR
KS ASSOCIATES
ELYRIA, OHIO

DRAWN TRR/10-24-19	CHECKED
REVISED	APPROVED
JOB NO. 1852901	
DRAWING NUMBER 1852901-03G	

ITL associates inc
Environmental, Geotechnical Engineering & Testing

APPENDIX A

Log of Test Borings

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

BORING NUMBER B-1

CLIENT KS Associates
PROJECT NAME OTIC 71-19-02 Bridge Demolition
PROJECT NUMBER 1852901
PROJECT LOCATION OTIC MP 98.9, Sandusky County, OH
DRILLING CONTRACTOR TTL Associates IC MB/TB
RIG NO. 550
GROUND ELEVATION 606.11 ft
DRILLING METHOD 3-1/4 in. HSA
GROUND WATER LEVELS:
DATE STARTED 9/24/19 **COMPLETED** 9/24/19
AT TIME OF DRILLING 48.5 ft / Elev 557.6 ft
LOGGED BY KKC **CHECKED BY**
AT END OF DRILLING 16.8 ft / Elev 589.3 ft
NOTES Split spoon refusal encountered at a depth of 53.8'.
0hrs AFTER DRILLING Backfilled w/Cuttings and Bentonite Chips

ELEVATION (ft)	DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	UNCONF. COMP. STR. (tsf)	DRY UNIT WT. (pcf)	PL MC LL (20 40 60 80) / SPT N VALUE (20 40 60 80)
605	0	CRUSHED STONE - 10 Inches (0.8')						
		FILL - Moist Stiff Gray/Brown LEAN CLAY w/Sand, Trace Gravel, Slag, Brick, and Organics (2.0')	SS 1	67	3-6-9 (15)	3.75		22
	5	Moist Medium Stiff Brown/Gray GRAVELLY LEAN CLAY w/Sand and Trace Organics (CL) (6.0')	SS 2	78	2-2-3 (5)	0.50		26
600		Moist Stiff Brown/Gray LEAN CLAY w/Sand and Trace Gravel (CL)	SS 3	78	3-4-6 (10)	>4.5	108	23
	10	@8': Brown, w/Trace Iron Oxide Stain Seam	SS 4	100	4-7-8 (15)	3.50		23
595		(12.0') Moist Medium Stiff Gray LEAN CLAY w/Sand and Trace Gravel (CL)						
	15		SS 5	100	2-4-4 (8)	1.25	107	17
590			ST 1	100		1.02	113	17
	20		SS 6	100	2-3-4 (7)	1.00		18
585	25		SS 7	100	0-2-3 (5)	0.75		19
580	30		SS 8	100	0-2-3 (5)	0.75	105	19

(Continued Next Page)

TTL_GEOTECH_STANDARD 1852901.GPJ TTL DATATMPL.GDT 10/24/19

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

BORING NUMBER B-1

CLIENT KS Associates **PROJECT NAME** OTIC 71-19-02 Bridge Demolition

PROJECT NUMBER 1852901 **PROJECT LOCATION** OTIC MP 98.9, Sandusky County, OH

ELEVATION (ft)	DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	UNCONF. COMP. STR. (tsf)	DRY UNIT WT. (pcf)	PL MC LL 20 40 60 80 ▲ SPT N VALUE ▲ 20 40 60 80
575		Moist Medium Stiff Gray LEAN CLAY w/Sand and Trace Gravel (CL) *(continued)*						
	35		SS 9	28	3-3-5 (8)	0.75		18
570			ST 2	92		0.88	114	17
		38.5' Moist Stiff Gray LEAN CLAY w/Sand and Trace Gravel (CL)	SS 10	100	3-4-5 (9)	1.50		18
	40							
565		44.0' Moist Hard Gray LEAN CLAY w/Sand and Trace Gravel (CL)	SS 11	78	5-10-21 (31)	>4.5		10
	45							
560		48.5' Moist Very Hard Gray SANDY LEAN CLAY w/Trace Gravel and Gypsum Fragments (CL)	SS 12	100	11-16-50/2"	0.75		17 >>
	50							
555		51.5' Moist Very Dense Gray WEATHERED DOLOMITE w/Sand and Clay						
		53.8' Bottom of hole at 53.8 feet.	SS 13	100	50/3"	NP		19 >>

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

BORING NUMBER B-2

CLIENT KS Associates
PROJECT NAME OTIC 71-19-02 Bridge Demolition
PROJECT NUMBER 1852901
PROJECT LOCATION OTIC MP 98.9, Sandusky County, OH
DRILLING CONTRACTOR TTL Associates IC MB/TB
RIG NO. 550
GROUND ELEVATION 606.11 ft
DRILLING METHOD 3-1/4 in. HSA
GROUND WATER LEVELS:
DATE STARTED 9/24/19 **COMPLETED** 9/24/19
AT TIME OF DRILLING 48.0 ft / Elev 558.1 ft
LOGGED BY KKC **CHECKED BY**
AT END OF DRILLING 28.2 ft / Elev 577.9 ft
NOTES Split spoon refusal encountered at a depth of 51.2'.
0hrs AFTER DRILLING Backfilled w/Cuttings and Bentonite Chips

ELEVATION (ft)	DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	UNCONF. COMP. STR. (tsf)	DRY UNIT WT. (pcf)	PL / MC / LL (20 40 60 80) ▲ SPT N VALUE ▲ (20 40 60 80)
	0	CRUSHED STONE - 10 Inches (0.8')						
605		Moist Stiff Gray/Brown SANDY LEAN CLAY w/Trace Gravel and Calcite Stain Seam (CL) (Possible Fill) (3.0')	SS 1	56	4-8-5 (13)	4.00	116	15
	5	Moist Medium Stiff Gray/Brown LEAN CLAY w/Sand and Trace Gravel (CL) (6.0')	SS 2	67	3-3-5 (8)	2.00		23
600		Moist Very Stiff Brown LEAN CLAY w/Trace Sand (CL)	SS 3	89	4-8-10 (18)	3.00		20
	10	@8': Stiff, Gray/Brown, w/Trace Iron Oxide Stain Seam	SS 4	100	4-6-6 (12)	2.00	99	28
595		Moist Soft Brown/Gray LEAN CLAY w/Sand and Trace Gravel (CL) (12.0')						
	15		SS 5	89	0-1-2 (3)	0.25		32
590	20	@18.5': Very Soft, w/Trace Sand	SS 6	100	0-0-0 (0)	<0.25		31
585		Moist Very Soft Gray SANDY LEAN CLAY w/Trace Gravel (CL) (21.0')	ST 1	42				16
	25	Moist Very Soft Gray LEAN CLAY w/Sand and Trace Gravel (CL) (23.0')	SS 7	78	0-0-2 (2)	<0.25	102	22
580	30	@28.5': Soft	SS 8	100	0-1-2 (3)	0.50		21

(Continued Next Page)

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

BORING NUMBER B-2

CLIENT KS Associates
PROJECT NAME OTIC 71-19-02 Bridge Demolition
PROJECT NUMBER 1852901
PROJECT LOCATION OTIC MP 98.9, Sandusky County, OH

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	UNCONF. COMP. STR. (tsf)	DRY UNIT WT. (pcf)	PL MC LL (20 40 60 80) / ▲ SPT N VALUE ▲ (20 40 60 80)
575			Moist Very Soft Gray LEAN CLAY w/Sand and Trace Gravel (CL) *(continued)*	ST 2	100		1.35	114	17
			33.5'						
	35		Moist Medium Stiff Gray LEAN CLAY w/Sand and Trace Gravel (CL) 35.0'	SS 9	100	1-3-3 (6)	1.25		18
570			Moist Stiff Gray LEAN CLAY w/Sand, Trace Gravel, and Shale Fragments (CL)						
	40			SS 10	17	3-4-6 (10)	NI		13
565			43.5'						
	45		Moist Very Hard Gray LEAN CLAY w/Sand and Trace Gravel (CL)	SS 11	89	7-11-43 (54)	3.50		15
560			48.0'						
	50		Moist Very Dense Gray WEATHERED DOLOMITE w/Sand, Clay, and Trace Gypsum Fragments	SS 12	100	50/3"	NP		21 >>▲
555			51.2'	SS 13	100	50/2"	NP		16 >>▲

Bottom of hole at 51.2 feet.

APPENDIX B

Legend Key

LEGEND KEY

Unified Soil Classification System Soil Symbols

- GW - WELL GRADED GRAVEL Includes Gravel-Sand mixtures, little or no fines.
- GP - POORLY GRADED GRAVEL Includes Gravel-Sand mixtures, little or no fines.
- GM - SILTY GRAVEL Includes Gravel-Sand-Silt mixtures.
- GC - CLAYEY GRAVEL Includes Gravel-Sand-Clay mixtures.
- SW - WELL GRADED SAND Includes Gravelly Sands, little or no fines.
- SP - POORLY GRADED SAND Includes Gravelly Sands, little or no fines.
- SM - SILTY SAND Includes Sand-Silt mixtures.
- SC - CLAYEY SAND Includes Sand-Clay mixtures.
- ML - SILT Includes Silt with Sand and Sandy Silt.
- CL - LEAN CLAY Includes Sandy Lean Clay and Lean Clay with Sand and Gravel.
- MH - ELASTIC SILT Includes Sandy Elastic Silt and Elastic Silt with Sand.
- CH - FAT CLAY Includes Sandy Fat Clay and Fat Clay with Sand.
- CL-ML - SILTY CLAY Includes Clayey Silt of low plasticity.
- OL - ORGANIC SILT and ORGANIC CLAY of low plasticity.
- OH - ORGANIC SILT and ORGANIC CLAY of medium to high plasticity.
- Pt - PEAT Includes humus, swamp and other soils with high organic content.
- FILL MATERIAL - Includes controlled and non-controlled soil and non-soil materials.
- TOPSOIL
- ASPHALT - Bituminous Asphalt
- CONCRETE - Includes broken concrete rubble.

Sample Symbols

- SS - Split Spoon
- ST - Shelby Tube
- RC - Rock Core
- GS - Geoprobe Sleeve
- AU - Auger Cuttings
- GB - Grab

Notes:

1. Exploratory borings were drilled on September 24, 2019, using 3¼ -inch 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 test borings were located in the field by TTL Associates, Inc. in accordance with a proposed boring location plan provided by KS Associates.

4. Unconfined Compressive Strength (tsf):
   NP = Non-Plastic
   NI = Not Intact

APPENDIX C

Pavement Core Photographic Logs

CORE LOG for L-1

Project: OTIC 71-19-02 -Bridge Demolition

Project Location: Sandusky County, Ohio

TTL Project No. 1852901-rev1

Core Date: October 10, 2019

ASPHALT THICKNESS (in)	=	3.25
CONCRETE THICKNESS (in)	=	9.5
AGGREGATE BASE THICKNESS (in)	=	15.0
CORE BARREL DIAMETER (in)	=	4.0

VISUAL DESCRIPTION:

CORE LOG for L-2

Project: OTIC 71-19-02 -Bridge Demolition

Project Location: Sandusky County, Ohio

TTL Project No. 1852901-rev1

Core Date: October 10, 2019

ASPHALT THICKNESS (in)	=	2.75
CONCRETE THICKNESS (in)	=	10.5
AGGREGATE BASE THICKNESS (in)	=	15.0
CORE BARREL DIAMETER (in)	=	4.0

VISUAL DESCRIPTION:

CORE LOG for L-3

Project: OTIC 71-19-02 -Bridge Demolition

Project Location: Sandusky County, Ohio

TTL Project No. 1852901-rev1

Core Date: October 10, 2019

ASPHALT THICKNESS (in)	=	4.5
CONCRETE THICKNESS (in)	=	8.25
AGGREGATE BASE THICKNESS (in)	=	15.0
CORE BARREL DIAMETER (in)	=	4.0

VISUAL DESCRIPTION:

#3 Steel reinforcement bar observed 3¾ inches below top of concrete.

CORE LOG for L-4

Project: OTIC 71-19-02 -Bridge Demolition
Project Location: Sandusky County, Ohio
TTL Project No. 1852901-rev1
Core Date: October 10, 2019

ASPHALT THICKNESS (in)	=	3.5
CONCRETE THICKNESS (in)	=	9.0
AGGREGATE BASE THICKNESS (in)	=	15.0
CORE BARREL DIAMETER (in)	=	4.0

VISUAL DESCRIPTION:

CORE LOG for L-5

Project: OTIC 71-19-02 -Bridge Demolition

Project Location: Sandusky County, Ohio

TTL Project No. 1852901-rev1

Core Date: October 10, 2019

ASPHALT THICKNESS (in)	=	3.5
CONCRETE THICKNESS (in)	=	10.5
AGGREGATE BASE THICKNESS (in)	=	15.0
CORE BARREL DIAMETER (in)	=	4.0

VISUAL DESCRIPTION:

Visible layering in asphalt pavement 1¼ inches below pavement surface.

CORE LOG for R-1

Project: OTIC 71-19-02 -Bridge Demolition
Project Location: Sandusky County, Ohio
TTL Project No. 1852901-rev1
Core Date: October 10, 2019

ASPHALT THICKNESS (in)	=	3.0
CONCRETE THICKNESS (in)	=	9.0
AGGREGATE BASE THICKNESS (in)	=	16.0
CORE BARREL DIAMETER (in)	=	4.0

VISUAL DESCRIPTION:

Visible layering and change in aggregate gradation in asphalt pavement 1¼ inches below pavement surface.

CORE LOG for R-2

Project: OTIC 71-19-02 -Bridge Demolition

Project Location: Sandusky County, Ohio

TTL Project No. 1852901-rev1

Core Date: October 10, 2019

ASPHALT THICKNESS (in)	=	3.75
CONCRETE THICKNESS (in)	=	11.5
AGGREGATE BASE THICKNESS (in)	=	16.0
CORE BARREL DIAMETER (in)	=	4.0

VISUAL DESCRIPTION:

#2 Steel reinforcement bar observed 5½ inches below top of concrete.

#3 Steel reinforcement bar observed 5¾ inches below top of concrete.

CORE LOG for R-3

Project: OTIC 71-19-02 -Bridge Demolition

Project Location: Sandusky County, Ohio

TTL Project No. 1852901-rev1

Core Date: October 10, 2019

ASPHALT THICKNESS (in)	=	3.5
CONCRETE THICKNESS (in)	=	9.0
AGGREGATE BASE THICKNESS (in)	=	14.0
CORE BARREL DIAMETER (in)	=	4.0

VISUAL DESCRIPTION:

Visible layering and change in aggregate gradation in asphalt pavement 1¼ inches below pavement surface.

CORE LOG for R-4

Project: OTIC 71-19-02 -Bridge Demolition
Project Location: Sandusky County, Ohio
TTL Project No. 1852901-rev1
Core Date: October 10, 2019

ASPHALT THICKNESS (in)	=	4.75
CONCRETE THICKNESS (in)	=	9.5
AGGREGATE BASE THICKNESS (in)	=	14.5
CORE BARREL DIAMETER (in)	=	4.0

VISUAL DESCRIPTION:

#3 Steel reinforcement bar observed 4¾ inches below top of concrete.

CORE LOG for R-5

Project: OTIC 71-19-02 -Bridge Demolition

Project Location: Sandusky County, Ohio

TTL Project No. 1852901-rev1

Core Date: October 10, 2019

ASPHALT THICKNESS (in)	=	4.0
CONCRETE THICKNESS (in)	=	8.0
AGGREGATE BASE THICKNESS (in)	=	13.5
CORE BARREL DIAMETER (in)	=	4.0

VISUAL DESCRIPTION:

APPENDIX D

Tabulation of Test Data

PROJECT: OTIC 71-19-02 Bridge Demolition, OTIC MP 98.9, Sandusky County, OH	**TTL Associates, Inc.**	PROJECT NO: 1852901

TABULATION OF TEST DATA

Boring Number	Sample Number	Sample Interval Depth (Feet)		Standard Penetration (Blows per Foot)	Natural Moisture Content (% of Dry Weight)	In-Place Dry Density (Pounds per Cubic Foot)	Unconfined Compressive Strength (Pounds per Square Foot)		Particle Size Distribution (%)						Atterberg Limits (%)			Unified Soil Classification
									Gravel	Coarse Sand	Medium Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index	
B-1	SS-1	1.0-2.5		15	21.7		*7,500											
	SS-2	3.5-5.0		5	26.3		*1,000		21	5	5	6	22	41	42	24	18	CL
	SS-3	6.0-7.5		10	22.6	108.4	*9,000+											
	SS-4	8.5-10.0		15	23.2		*7,000											
	SS-5	13.5-15.0		8	17.5	106.7	*2,500											
	ST-1	16.0-18.0			17.3	113.3	2,040											
	SS-6	18.5-20.0		7	17.6		*2,000		5	5	8	8	19	55	29	17	12	CL
	SS-7	23.5-25.0		5	19.1		*1,500											
	SS-8	28.5-30.0		5	18.7	105.4	*1,500											
	SS-9	33.5-35.0		8	18.3		*1,500		2	5	8	12	21	52	30	18	12	CL
	ST-2	36.0-38.0			17.3	113.8	1,750		5	3	8	13	25	46	29	18	11	CL
	SS-10	38.5-40.0		9	17.6		*3,000											
	SS-11	43.5-45.0		31	9.7		*9,000+											
	SS-12	48.5-49.7		SSR	16.6		*1,500											
	SS-13	53.5-53.8		SSR	18.7													

SSR = Split-Spoon Refusal

*Unconfined compressive strength derived from a calibrated hand penetrometer

PROJECT: OTIC 71-19-02 Bridge Demolition, OTIC MP 98.9, Sandusky County, OH	**TTL Associates, Inc.**	PROJECT NO: 1852901

TABULATION OF TEST DATA

Boring Number	Sample Number	Sample Interval Depth (Feet)		Standard Penetration (Blows per Foot)	Natural Moisture Content (% of Dry Weight)	In-Place Dry Density (Pounds per Cubic Foot)	Unconfined Compressive Strength (Pounds per Square Foot)		Particle Size Distribution (%)						Atterberg Limits (%)			Unified Soil Classification
									Gravel	Coarse Sand	Medium Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index	
B-2	SS-1	1.0-2.5		13	14.7	116.0	*8,000											
	SS-2	3.5-5.0		8	23.2		*4,000											
	SS-3	6.0-7.5		18	19.7		*6,000		0	0	2	7	23	68	33	21	12	CL
	SS-4	8.5-10.0		12	27.6	99.1	*4,000											
	SS-5	13.5-15.0		3	32.4		*500											
	SS-6	18.5-20.0		0	31.4		*<500		1	2	5	6	20	66	29	17	12	CL
	ST-1	21.0-23.0			16.0				6	8	13	19	28	26	24	15	9	CL
	SS-7	23.5-25.0		2	21.8	101.9	*<500											
	SS-8	28.5-30.0		3	20.7		*1,000		3	7	4	9	19	58	30	17	13	CL
	ST-2	31.0-33.0			17.0	114.4	2,705											
	SS-9	33.5-35.0		6	17.9		*2,500											
	SS-10	38.5-40.0		10	13.2													
	SS-11	43.5-45.0		54	15.0		*7,000											
	SS-12	48.5-48.8		SSR	21.4													
	SS-13	50.0-50.2																
		51.0			16.2													

SSR = Split-Spoon Refusal

*Unconfined compressive strength derived from a calibrated hand penetrometer

APPENDIX E

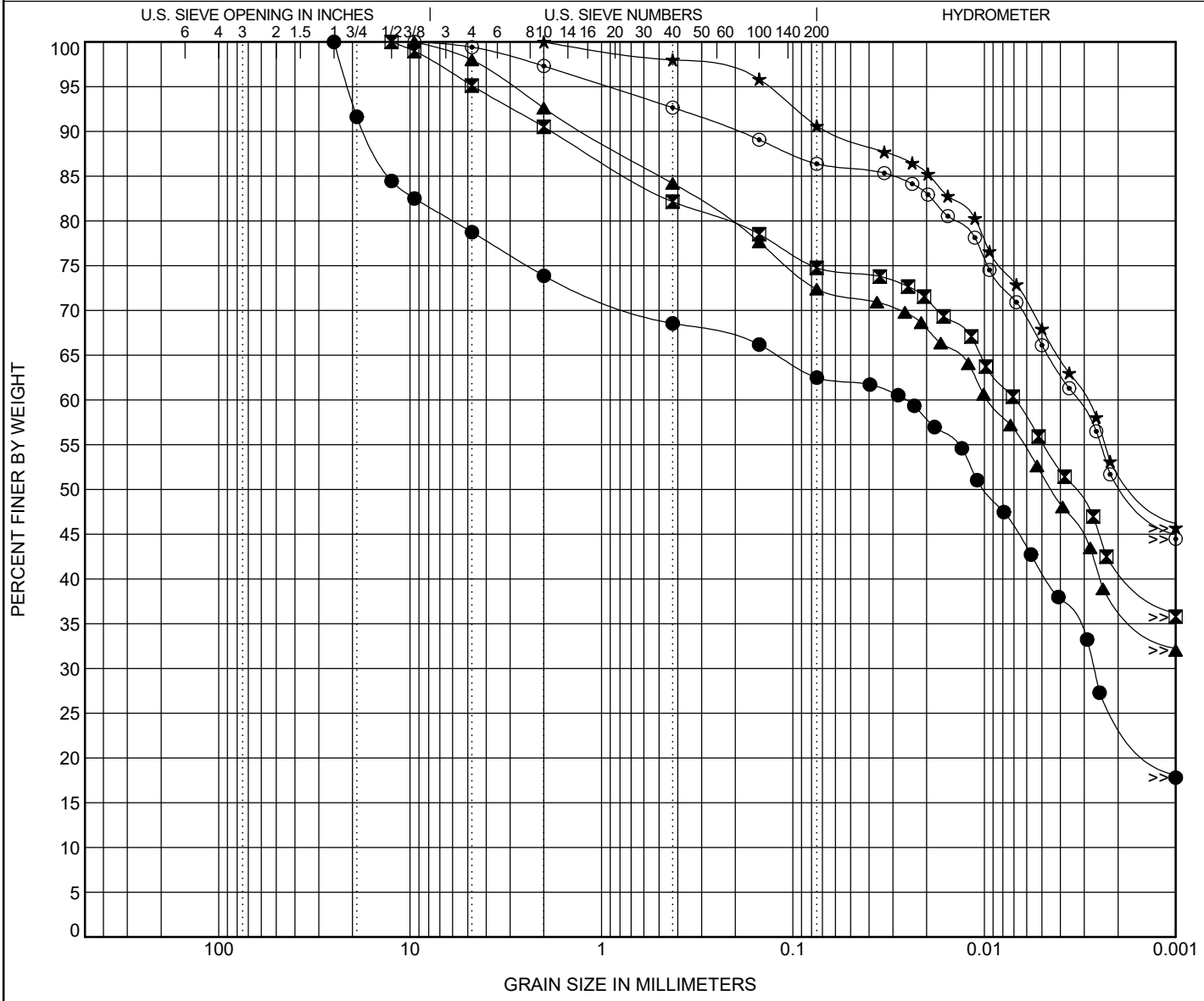
Laboratory Test Results

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

GRAIN SIZE DISTRIBUTION

CLIENT KS Associates **PROJECT NAME** OTIC 71-19-02 Bridge Demolition

PROJECT NUMBER 1852901 **PROJECT LOCATION** OTIC MP 98.9, Sandusky County, OH



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		USCS Classification	LL	PL	PI	Cc	Cu
●	B-1	3.5	**GRAVELLY LEAN CLAY with SAND (CL)**	42	24	18		
⊠	B-1	18.5	**LEAN CLAY with SAND (CL)**	29	17	12		
▲	B-1	33.5	**LEAN CLAY with SAND (CL)**	30	18	12		
★	B-2	6.0	**LEAN CLAY (CL)**	33	21	12		
⊙	B-2	18.5	**LEAN CLAY (CL)**	29	17	12		

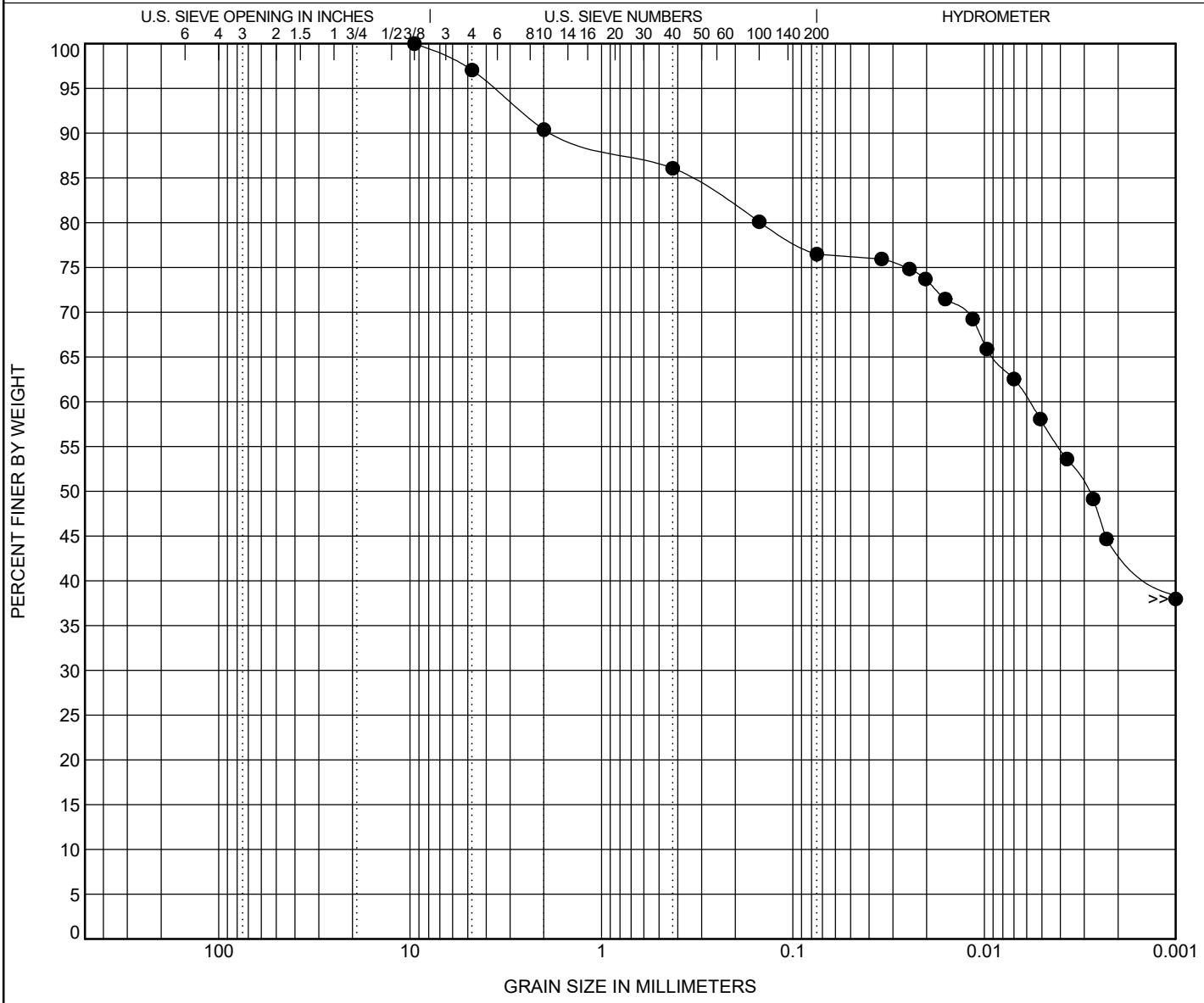
	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-1	3.5	25	0.026	0.003		21.3	16.3	21.7	40.8
⊠	B-1	18.5	12.5	0.007			4.9	20.4	19.4	55.3
▲	B-1	33.5	9.5	0.01			2.0	25.6	20.6	51.8
★	B-2	6.0	2	0.003			0.0	9.4	22.7	67.9
⊙	B-2	18.5	9.5	0.003			0.6	13.0	20.3	66.1

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

GRAIN SIZE DISTRIBUTION

CLIENT KS Associates **PROJECT NAME** OTIC 71-19-02 Bridge Demolition

PROJECT NUMBER 1852901 **PROJECT LOCATION** OTIC MP 98.9, Sandusky County, OH



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		USCS Classification	LL	PL	PI	Cc	Cu
●	**B-2**	**28.5**	**LEAN CLAY with SAND (CL)**	**30**	**17**	**13**		

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-2**	**28.5**	**9.5**	**0.006**			**3.0**	**20.6**	**18.7**	**57.8**

Project No.: 1852901
Date: 10/1/2019
Client: KS Associates
Project: OTIC 71-19-02 Bridge Demolition
Sandusky County, OH
Boring No.: B-1
Sample No.: ST-2
Depth: 36.0 - 38.0'

Initial H= 1.012 inches

Pressure tsf	Final Height (in)	Initial Height (in)	DH	Average H (in)	e	Ave P (tsf)
0.125	1.01200	1.01200	0.00000	1.0120	0.476	0.0625
0.25	1.00890	1.01200	0.00310	1.0105	0.472	0.1875
0.5	1.00650	1.00890	0.00550	1.0077	0.468	0.375
1	0.99980	1.00650	0.01220	1.0032	0.458	0.75
2	0.98830	0.99980	0.02370	0.9941	0.442	1.5
4	0.97640	0.98830	0.03560	0.9824	0.424	3
8	0.96070	0.97640	0.05130	0.9686	0.401	6
16	0.94130	0.96070	0.07070	0.9510	0.373	12
4	0.94570	0.94130	0.06630	0.9435	0.380	10
1	0.95410	0.94570	0.05790	0.9499	0.392	2.5
0.25	0.96430	0.95410	0.04770	0.9592	0.407	0.625

Estimated Cc: 0.094
Estimated Cr: 0.019

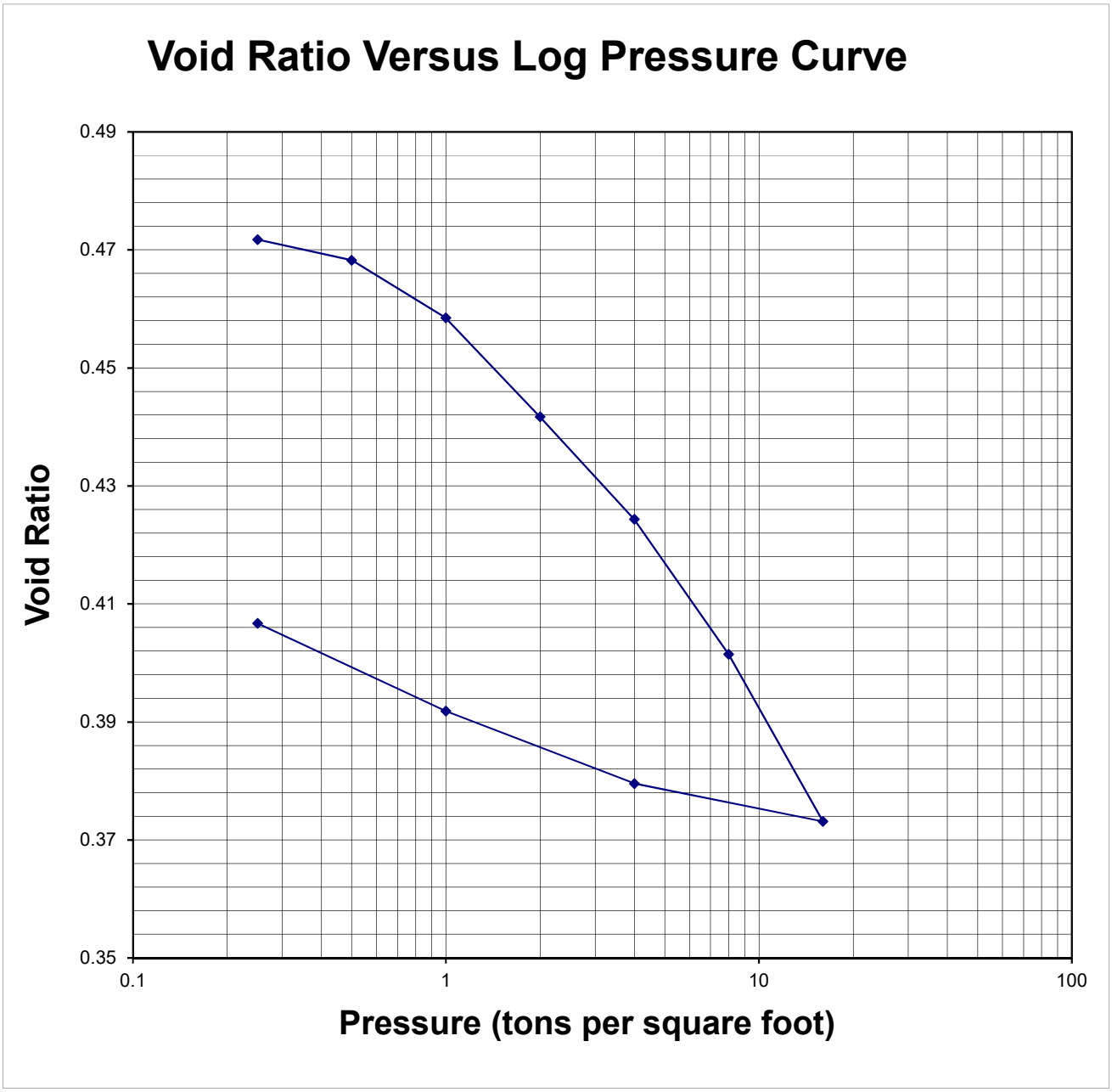
Soil Description: Gray LEAN CLAY w/Sand and Trace Gravel (CL)
Specific Gravity: 2.73
Liquid Limit: 29
Plastic Limit: 18
Plasticity Index: 11

Initial Water Content:	17.6 %	Final Water Content:	15.0 %
Inital Dry Density:	115.4 pcf	Final Dry Density:	121.2 pcf
Initial Void Ratio:	0.476	Final Void Ratio:	0.407
Initial Degree of Saturation:	101.1 %	Final Degree of Saturation	100.9 %

Estimated Preconsolidation Pressure: tsf

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

Project No.: 1852901
Date: 10/1/2019
Client: KS Associates
Project: OTIC 71-19-02 Bridge Demolition
Sandusky County, OH
Boring No.: B-1
Sample No.: ST-2
Depth: 36.0 - 38.0'



Project No.: 1852901
Date: 10/1/2019
Client: KS Associates
Project: OTIC 79-19-02 Bridge Demolition
Sandsusky County, OH
Boring No.: B-2
Sample No.: ST-1
Depth: 21.0 - 23.0'

Initial H= 1.022 inches

Pressure tsf	Final Height (in)	Initial Height (in)	DH	Average H (in)	e	Ave P (tsf)
0.125	1.02200	1.02200	0.00000	1.0220	0.429	0.0625
0.25	1.00710	1.02200	0.01490	1.0146	0.408	0.1875
0.5	0.99970	1.00710	0.02230	1.0034	0.397	0.375
1	0.99085	0.99970	0.03115	0.9953	0.385	0.75
2	0.97840	0.99085	0.04360	0.9846	0.368	1.5
4	0.96630	0.97840	0.05570	0.9724	0.351	3
8	0.95200	0.96630	0.07000	0.9592	0.331	6
16	0.93380	0.95200	0.08820	0.9429	0.305	12
4	0.93700	0.93380	0.08500	0.9354	0.310	10
1	0.94295	0.93700	0.07905	0.9400	0.318	2.5
0.25	0.95060	0.94295	0.07140	0.9468	0.329	0.625

Estimated Cc: 0.085
Estimated Cr: 0.013

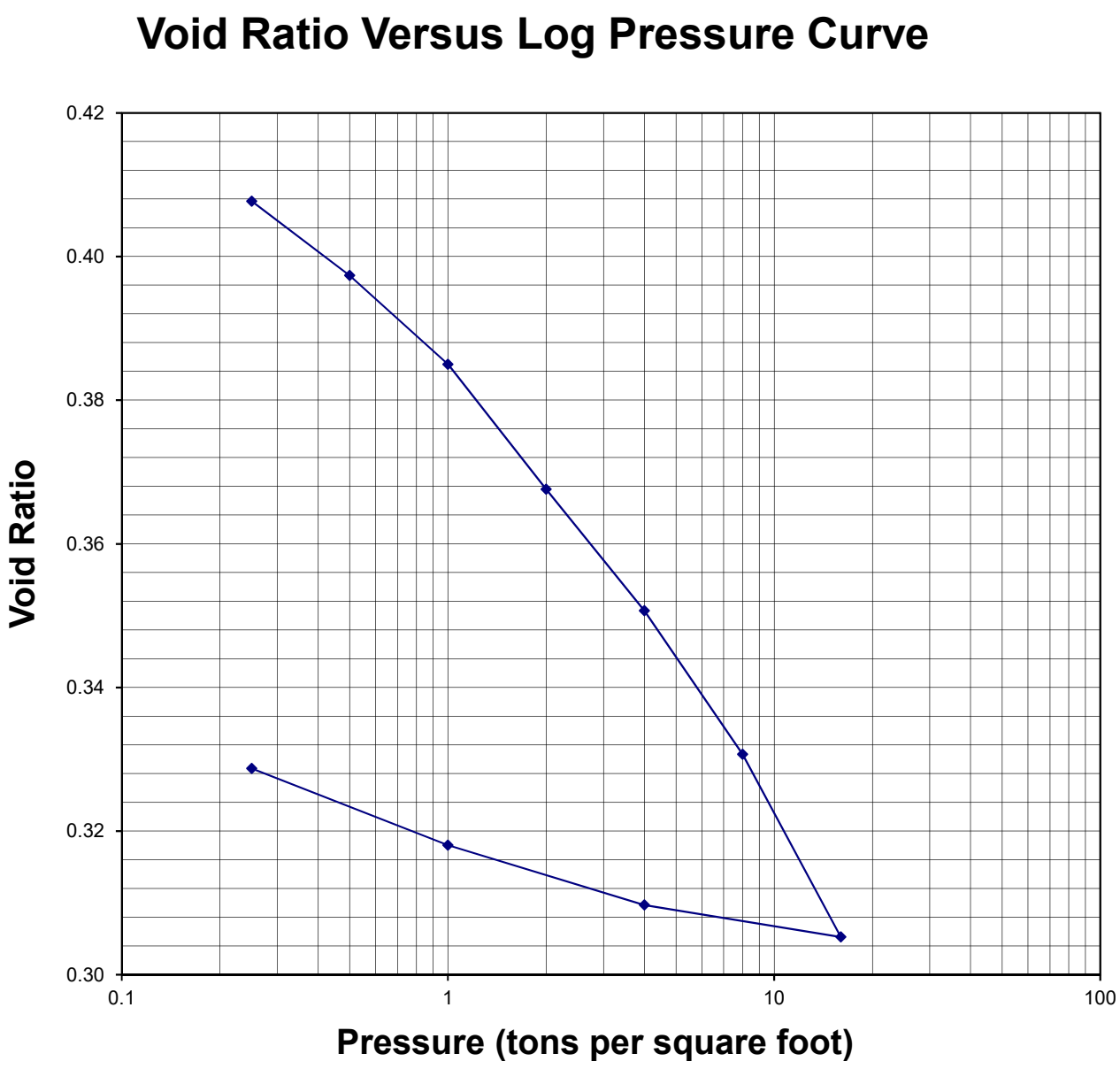
Soil Description: Gray SANDY LEAN CLAY w/Trace Gravel (CL)
Specific Gravity: 2.705
Liquid Limit: 24
Plastic Limit: 15
Plasticity Index: 9

Initial Water Content:	15.9 %	Final Water Content:	12.7 %
Inital Dry Density:	118.2 pcf	Final Dry Density:	127.1 pcf
Initial Void Ratio:	0.429	Final Void Ratio:	0.329
Initial Degree of Saturation:	100.3 %	Final Degree of Saturation	104.7 %

Estimated Preconsolidation Pressure: tsf

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

Project No.: 1852901
Date: 10/1/2019
Client: KS Associates
Project: OTIC 79-19-02 Bridge Demolition
Sandsusky County, OH
Boring No.: B-2
Sample No.: ST-1
Depth: 21.0 - 23.0'



APPENDIX F

Settlement Analysis Data

Project Name: OTIC 71-19-02
Project Number: 1852901
Calculated by: IJH

Embank Software Input

Engineered Fill - 135 pcf														
Boring	**Structure Location**	**H**	**W**	**X**	**x_1**	**y_2**	**x_2**	**y_2**	**Y_1**	**Y_2**	**δ_i**	**δ_1**	**δ_2**	**δ_{net}**
B-2	Abutment	28	235	201.5	0	100	84	156	179	21	14.28	13.98	0	**0.3**
	Pier	28	235	201.5	0	100	84	156	128	72	14.28	9.29	1.9	**3.09**
B-1	Abutment	28	235	201.5	0	100	84	156	179	21	5.41	5.34	0	**0.07**
	Pier	28	235	201.5	0	100	84	156	128	72	5.41	3.59	0.12	**1.7**

LIGHTWEIGHT FILL (FCCF) - 50 pcf														
Boring	**Structure Location**	**H**	**W**	**X**	**x_1**	**y_2**	**x_2**	**y_2**	**Y_1**	**Y_2**	**δ_i**	**δ_1**	**δ_2**	**δ_{net}**
B-2	Abutment	28	235	201.5	0	100	84	156	179	21	7.82	7.74	0	**0.08**
	Pier	28	235	201.5	0	100	84	156	128	72	7.82	5.26	1.73	**0.83**
B-1	Abutment	28	235	201.5	0	100	84	156	179	21	2.95	2.91	0	**0.04**
	Pier	28	235	201.5	0	100	84	156	128	72	2.95	1.8	0.05	**1.1**

GEOFOAM - 25pcf														
Boring	**Structure Location**	**H**	**W**	**X**	**x_1**	**y_2**	**x_2**	**y_2**	**Y_1**	**Y_2**	**δ_i**	**δ_1**	**δ_2**	**δ_{net}**
B-2	Abutment	28	235	201.5	0	100	84	156	179	21	5.26	5.21	0	**0.05**
	Pier	28	235	201.5	0	100	84	156	128	72	5.26	3.66	1.6	**0**
B-1	Abutment	28	235	201.5	0	100	84	156	179	21	1.8	1.77	0	**0.03**
	Pier	28	235	201.5	0	100	84	156	128	72	1.8	1.04	0.02	**0.74**

B-1				
Stratum	**Elevation Range (feet)**	**C_C**	**C_R**	**Past Consolidation Pressure (psf)**
Existing Embankment	635 - 606	0.09	0.02	16616
Upper Stiff Soils	606 - 594			16988
Medium Stiff Clay	594-567			6908
Stiff Clays	567-562			9715
Stiff to Hard (In compressible)	562 - ---			24169

B-1				
Stratum	**Elevation Range (feet)**	**C_C**	**C_R**	**Past Consolidation Pressure (psf)**
Existing Embankment	635 - 606	0.09	0.02	16616
Upper Stiff Soils	606 - 594			17811
V Soft Clay	594 - 577			1659
Soft Clay	577-573			4744
Medium Stiff Clay	573 - 562.5			6325
Stiff to Hard (In compressible)	562.5 - ---			2148

Project Name: OTIC 71-19-02
Project Number: 1852901
Calculated by: IJH

Embankment Parameters

Height	Pressure @135 pcf	
28 feet	3780 psf	1.89 tsf

Coefficient of Consolidation from NAVFAC Figure 4 (7.1-144)

Stratum	LL	Virgin Compression		Recompression		Average
		C_v (cm^2/sec)	C_v (ft^2/day)	C_v (cm^2/sec)	C_v (ft^2/day)	C_v (ft^2/day)
I Upper Stiff Clay	42	0.03	2.4	0.1	13	3
	33	0.04	4.0	0.3	28	
II - V Soft Clay	29	>0.05	>0.5	>0.3	>2	2
III - Soft Clay	24	>0.05	>0.5	>0.3	>2	2
IV - M Stiff Clay	30	0.05	4.7	>0.3	>2	4.5

Coefficient of Consolidation from Tested Values

Stratum	Pressure (tsf)	Virgin Compression		Recompression		Cv for 1.89 tsf
		C_v (cm^2/sec)	C_v (ft^2/day)	C_v (cm^2/sec)	C_v (ft^2/day)	
B-2 (ST-1) III	1.0	-		-	0.08	0.14
	2.0	-		-	0.15	

Project Nai OTIC 71-19-02
Project Nui 1852901
Calculated IJH

Encountered Conditions

	Low H_{dr} (feet)	High H_{dr} (feet)
Stratum I layer thickness	5	6.5
Stratum II and III layer thickness	5	8.5
Stratum IV layer thickness	2	2

Assume double drainage between strata layers

Time for 90% Consolidation

$$t = \frac{T\ (H_{dr})^2}{C_v}$$

where T = 0.848 for 90% consolidation

Results Based on Low End H_{dr}

Stratum	From NAVFAC Cv Values			From Lab Cv Values			
	t (days)	t (weeks)	t (months)	t (days)	t (weeks)	t (months)	t (years)
I	6.68	0.955	0.223				
II and III	10.6	1.514	0.353	149	21.3	5.0	0
IV	0.8	0.11	0.03				

Results Based on High End H_{dr}

Stratum	From NAVFAC Cv Values			From Lab Cv Values			
	t (days)	t (weeks)	t (months)	t (days)	t (weeks)	t (months)	t (years)
I	11.3	1.61	0.377				
II and III	31	4	1	431	62	14	1
IV	0.8	0.11	0.03				

Final Conclusions

High end results not realistic for Stratum II and III
Expect up to 6 months for 90% consolidation

Project No.: 1852901
Date: 10/1/2019
Client: KS Associates
Project: OTIC 79-19-02 Bridge Demolition
Sandsusky County, OH
Boring No.: B-2
Sample No.: ST-1
Depth: 21.0 - 23.0'

Initial H= 1.022 inches

Pressure tsf	Final Height (in)	Initial Height (in)	DH	Average H (in)	e	t50 (min)	Ave P (tsf)	Cv (in2/s)	Cv (ft2/d)
0.125	1.02200	1.02200	0.00000	1.0220	0.429		0.0625		
0.25	1.00710	1.02200	0.01490	1.0146	0.408	11.5	0.1875	0.000073	0.044
0.5	0.99970	1.00710	0.02230	1.0034	0.397	8	0.375	0.000103	0.062
1	0.99085	0.99970	0.03115	0.9953	0.385	6	0.75	0.000136	0.081
2	0.97840	0.99085	0.04360	0.9846	0.368	3.1	1.5	0.000257	0.154
4	0.96630	0.97840	0.05570	0.9724	0.351	3	3	0.000259	0.155
8	0.95200	0.96630	0.07000	0.9592	0.331	1.5	6	0.000503	0.302
16	0.93380	0.95200	0.08820	0.9429	0.305	0.95	12	0.000768	0.461
4	0.93700	0.93380	0.08500	0.9354	0.310		10		
1	0.94295	0.93700	0.07905	0.9400	0.318		2.5		
0.25	0.95060	0.94295	0.07140	0.9468	0.329		0.625		

Estimated Cc: 0.085
Estimated Cr: 0.013

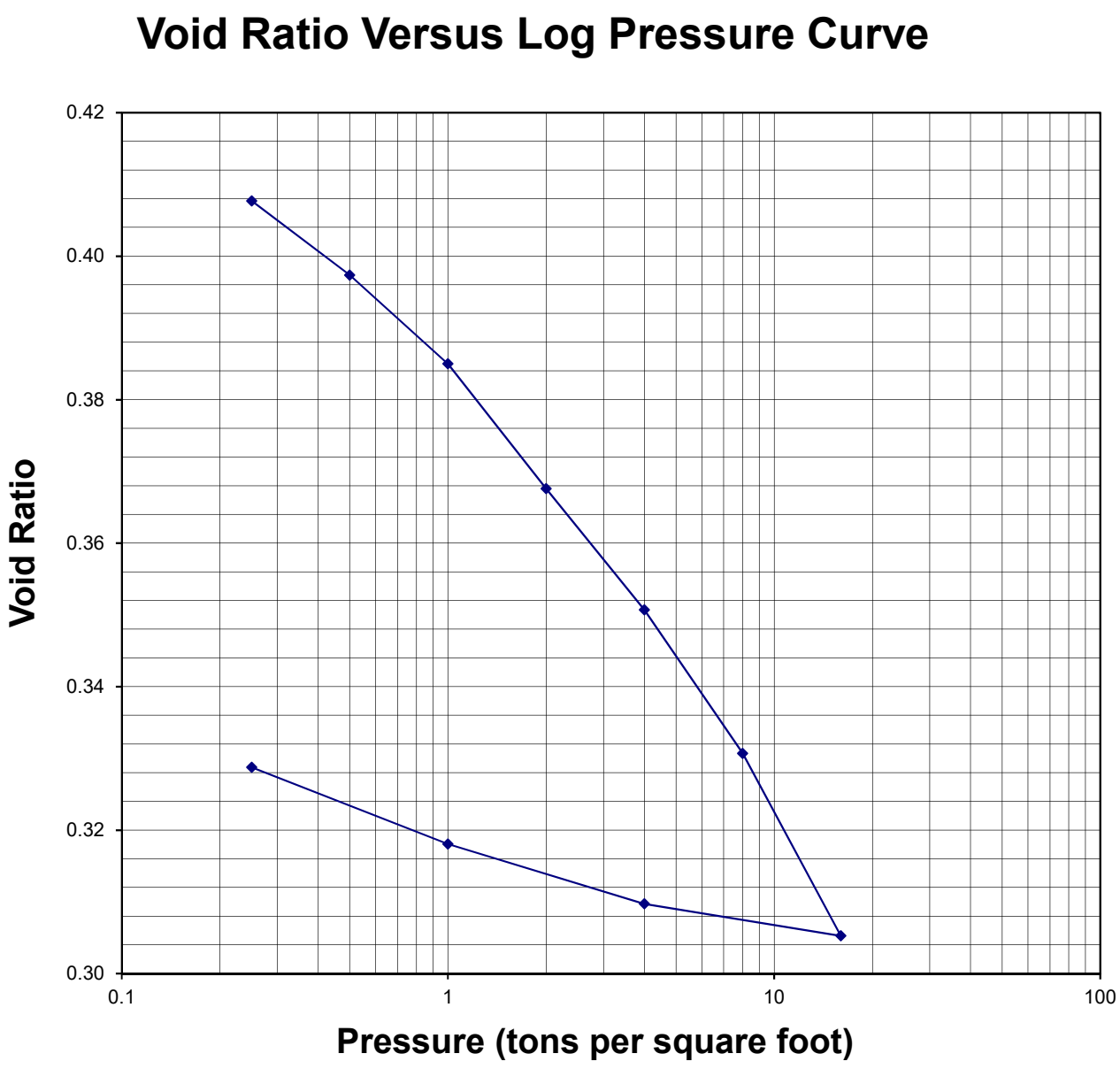
Soil Description: Gray SANDY LEAN CLAY w/Trace Gravel (CL)
Specific Gravity: 2.705 zz
Liquid Limit: 24
Plastic Limit: 15
Plasticity Index: 9

Initial Water Content:	15.9 %	Final Water Content:	12.7 %
Inital Dry Density:	118.2 pcf	Final Dry Density:	127.1 pcf
Initial Void Ratio:	0.429	Final Void Ratio:	0.329
Initial Degree of Saturation:	100.3 %	Final Degree of Saturation	104.7 %

Estimated Preconsolidation Pressure: tsf

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

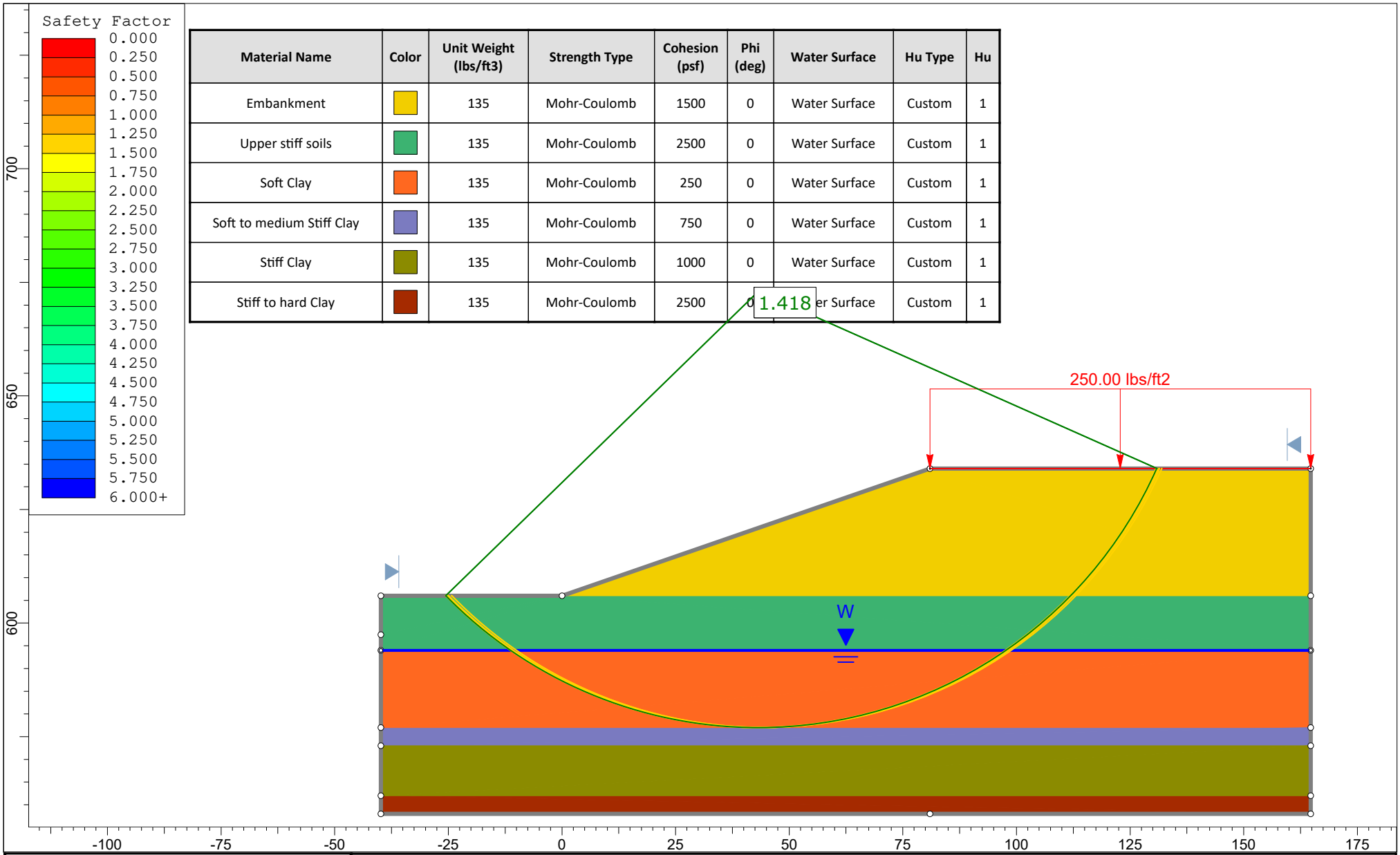
Project No.: 1852901
Date: 10/1/2019
Client: KS Associates
Project: OTIC 79-19-02 Bridge Demolition
Sandsusky County, OH
Boring No.: B-2
Sample No.: ST-1
Depth: 21.0 - 23.0'



APPENDIX G

Slope Stability Analysis Outputs

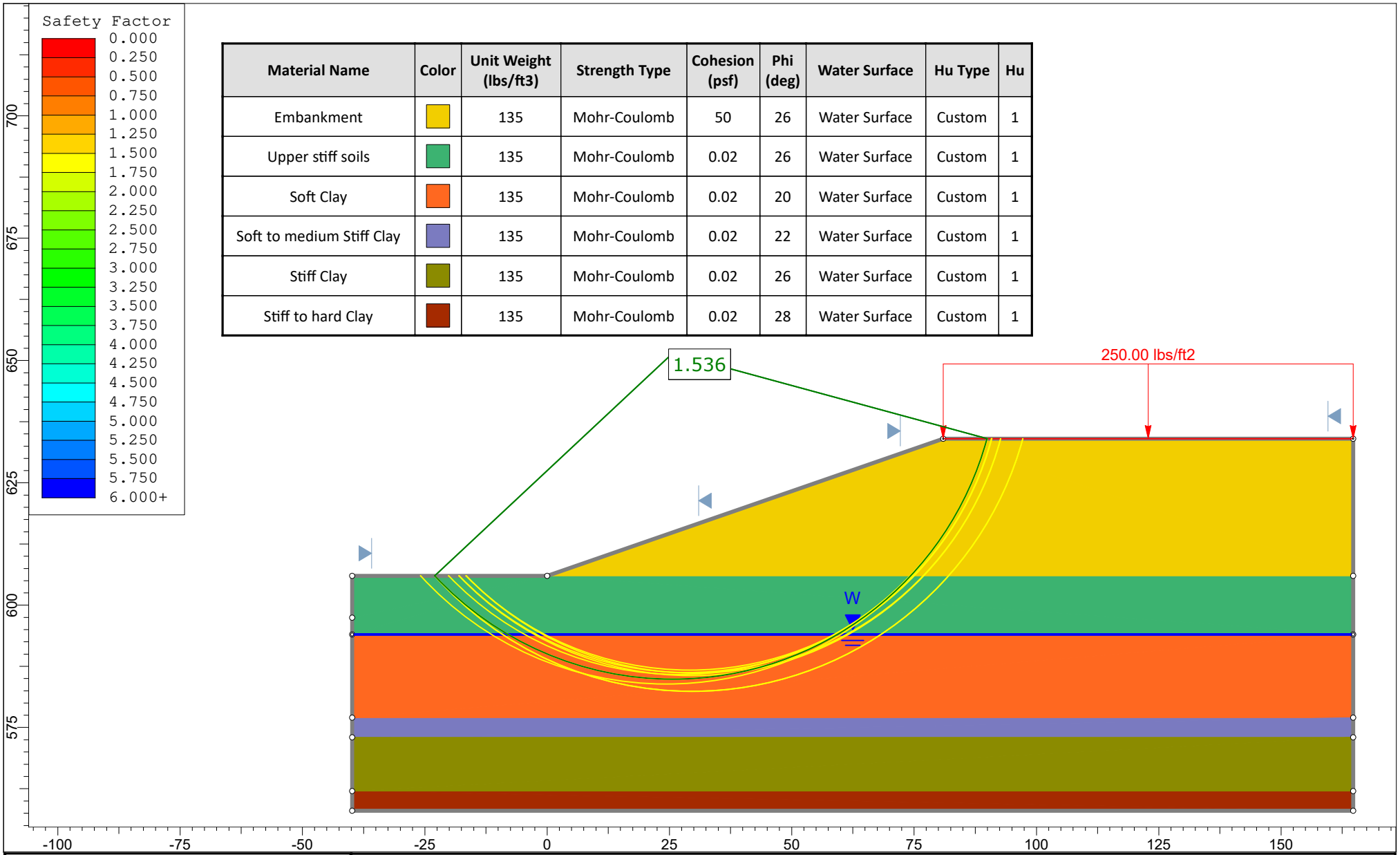
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Embankment		135	Mohr-Coulomb	1500	0	Water Surface	Custom	1
Upper stiff soils		135	Mohr-Coulomb	2500	0	Water Surface	Custom	1
Soft Clay		135	Mohr-Coulomb	250	0	Water Surface	Custom	1
Soft to medium Stiff Clay		135	Mohr-Coulomb	750	0	Water Surface	Custom	1
Stiff Clay		135	Mohr-Coulomb	1000	0	Water Surface	Custom	1
Stiff to hard Clay		135	Mohr-Coulomb	2500	0	Water Surface	Custom	1



TTL associates inc
Environmental, Geotechnical Engineering & Testing

Project	OTIC 71-19-02 - Deck Replacement EB & WB Mainline Bridges MP 98.9 - Sandusky County
Analysis Description	**Total Stress Soil Parameters (TSSP)**
Drawn By	
Scale	1:350
Company	
Date	11/14/2019, 7:01:07 PM
File Name	Slide Model 1.slim

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Embankment		135	Mohr-Coulomb	50	26	Water Surface	Custom	1
Upper stiff soils		135	Mohr-Coulomb	0.02	26	Water Surface	Custom	1
Soft Clay		135	Mohr-Coulomb	0.02	20	Water Surface	Custom	1
Soft to medium Stiff Clay		135	Mohr-Coulomb	0.02	22	Water Surface	Custom	1
Stiff Clay		135	Mohr-Coulomb	0.02	26	Water Surface	Custom	1
Stiff to hard Clay		135	Mohr-Coulomb	0.02	28	Water Surface	Custom	1



TTL associates inc
Environmental, Geotechnical Engineering & Testing

Project	OTIC 71-19-02 - Deck Replacement EB & WB Mainline Bridges MP 98.9 - Sandusky County		
Analysis Description	**Effective Stress Soil Parameters (ESSP)**		
Drawn By		*Scale* 1:325	*Company*
Date	11/14/2019, 7:01:07 PM		*File Name* Slide Model 2- Drained.slim