



Subgrade and Structure Foundation
Exploration Report – Final (Rev. #1)
STA-241-17.37 Roundabout and Bridge Replacement
North Canton, Stark County, Ohio
S&ME Project No. 23170065A/B

PREPARED FOR:

ODOT District 4
2088 S. Arlington Road
Akron, OH 44306

PREPARED BY:

S&ME, Inc.
6190 Enterprise Court
Dublin, OH 43016

October 17, 2025



October 17, 2025

ODOT District 4
2088 S. Arlington Road
Akron, Ohio 44306

Attention: Mr. Thomas Powell, P.E.

Reference: **Subgrade and Structure Foundation Exploration Report – Final (Rev. #1)**
STA-241-17.37 Roundabout and Bridge Replacement (PID 119585)
North Canton, Stark County, Ohio
S&ME Project No. 23170065A/B, Task Order No. 4-01-1
District 4 Geotechnical Engineering Services Task Order
PID No. 117651; Agreement No. 38621

Mr. Powell:


In accordance with our revised proposal (Modification #1) dated December 22, 2023, which was authorized by Encumbrance No. 742130 on January 3, 2024, by ODOT District 4 (D04), S&ME, Inc. (S&ME) has completed a Geotechnical Exploration for the STA-241-17.37 Roundabout and Bridge Replacement (STA-00241-17.140) project in Stark County, Ohio. The proposed roundabout is at the intersection of SR 241 (Wales Avenue NW) and CR 230 (Shuffel Street NW), and the proposed bridge replacement is approximately 1,200 feet southwest of the intersection on SR 241. Please see the approximate project location on the Vicinity Map of Appendix I.

In accordance with Section 701 of the January 2023 ODOT *Specifications for Geotechnical Explorations (SGE)*, S&ME is herewith submitting a revised "final" version of our April 9, 2025, final report addressing review comments from the ODOT Office of Geotechnical Engineering dated April 17, 2025. S&ME has also prepared Geotechnical Profile report sheets which will be submitted under separate cover.


We appreciate the opportunity to be of service. Please do not hesitate to contact us if you have any questions concerning this report.

Respectfully,

S&ME, Inc.


Brian K. Sears, P.E.
Senior Engineer | Project Manager




Richard S. Weigand, P.E.
Principal Engineer | Senior Reviewer

Submitted: Email copy



Table of Contents

Plate

1.0	Executive Summary	1
2.0	Introduction	2
2.1	Geology.....	3
2.2	Site Reconnaissance	3
3.0	Exploration	4
3.1	Available Information	4
3.2	Field Exploration.....	4
3.3	Laboratory Testing Program	5
4.0	Findings	6
4.1	Bridge Structure Borings (B-002 & B-003).....	6
4.2	Roadway Embankment Borings (B-001 & B-004)	7
4.3	Roundabout Subgrade Borings (B-005 through B-008).....	7
4.4	Hand Auger Samples (D/H-006-1-23 and D/H-006-2-23)	8
4.5	Groundwater Observations	8
4.6	Soil Sulfate Test Results.....	8
4.7	Geotechnical Data for Scour Analysis.....	8
5.0	Analyses and Recommendations	9
5.1	Geotechnical Evaluation.....	9
5.2	Roundabout Subgrade Analyses (~Sta. 591+50 to North End of Project).....	9
5.2.1	<i>ODOT Subgrade Analysis</i>	<i>9</i>
5.2.2	<i>Subgrade Support Parameters</i>	<i>10</i>
5.2.3	<i>Subgrade Remediation Assessment & Recommendations</i>	<i>11</i>
5.2.4	<i>Embankment Foundation Preparation.....</i>	<i>12</i>
5.2.5	<i>“At-Grade”/“Cut” Soil Subgrade Areas</i>	<i>12</i>
5.2.6	<i>“Fill” Areas.....</i>	<i>12</i>
5.2.7	<i>Benching</i>	<i>13</i>
5.2.8	<i>Borrow Requirements and Compaction Criteria</i>	<i>13</i>
5.2.9	<i>Compaction/Moisture Conditioning Concerns.....</i>	<i>14</i>
5.2.10	<i>Groundwater Considerations for Roadway Subgrade</i>	<i>14</i>
5.3	Flexible Pavement Thickness Analysis	14
5.4	SR 241 Embankment Modifications (~Sta. 586+50 to Sta. 591+50).....	15
5.4.1	<i>Summary of Embankment Foundation Conditions</i>	<i>15</i>
5.4.2	<i>Settlement Analyses – Bridge Approach Embankments.....</i>	<i>16</i>
5.4.2.1	<i>Geometry</i>	<i>16</i>
5.4.2.2	<i>Strength and Settlement Parameters.....</i>	<i>17</i>
5.4.2.3	<i>Groundwater Modeling.....</i>	<i>17</i>
5.4.2.4	<i>Settlement Analyses</i>	<i>17</i>
5.4.3	<i>Embankment Design and Construction</i>	<i>19</i>

Subgrade and Structure Foundation Exploration Report – Final (Rev. #1)

STA-241-17.37 Roundabout and Bridge Replacement (PID 119585)

North Canton, Stark County, Ohio

S&ME Project No. 23170065A/B, Task Order No. 4-01-1



5.4.3.1	Embankment Foundation Preparation	19
5.4.3.2	Wick Drains Spacing and Waiting Period Duration.....	19
5.4.3.3	Embankment Fill Placement	21
5.4.3.4	Instrumentation and Monitoring.....	21
5.4.3.5	General Embankment Construction Considerations.....	21
5.4.3.6	Embankment Stability Analyses.....	22
5.5	Proposed Replacement Bridge Recommendations/Considerations.....	22
5.5.1	<i>Downdrag Considerations</i>	22
5.5.2	<i>Earth Pressures on Bridge Abutments</i>	22
5.6	Groundwater Considerations for Embankment and Bridge Construction.....	23
5.7	Temporary Excavation Considerations.....	24
6.0	Considerations and Report Limitations	24



Appendices

Appendix I

	<u>Plate No.</u>
Vicinity Map	1
Plan of Borings	2A-2B
Explanation of Symbols and Terms Used on Boring Logs.....	3
Boring Logs.....	4-19
Shelby Tube Log	20
DCP Test Data	21-22
Important Information About Your Geotechnical Engineering Report.....	23

Appendix II

	<u>Plate No.</u>
Sulfate Test Results.....	1
Consolidation Test Results	2-14

Appendix III

	<u>Plate No.</u>
ODOT OGE CPT Report.....	1-32

Appendix IV

	<u>Plate No.</u>
Subgrade Analysis Results	1-6
Settlement Analyses Parameter Estimation	7-19
Settlement Analyses Results Output.....	20-51
Global Stability Analyses Results Output	52-54

Appendix V

	<u>Plate No.</u>
OGE Design Checklists	1-21



1.0 Executive Summary

This project includes a roundabout to be constructed at the intersection of SR 241 (Wales Avenue NW) and Shuffel Street NW, which is immediately southwest of the runway of the Akron-Canton Airport. In addition to the proposed roundabout, roadway flooding near an existing single span bridge approximately 1,200 feet southwest of the intersection is planned to be addressed by replacing the existing bridge and raising the profile of SR 241 by up to 3 to 4 feet from approximately 250 feet southwest of the bridge to approximately 350 feet northeast of the bridge.

S&ME performed a field exploration program consisting of eight (8) soil borings. Four (4) borings (B-001-0-23 through B-004-0-23) were performed in the general area where the profile of SR 241 will be raised and included two (2) embankment borings (B-001-0-23 and B-004-0-23) and two bridge borings for the bridge replacement (B-002-0-23 and B-003-0-23). The embankment/bridge borings were extended to depths ranging from 15 to 100 feet below existing grades. The remaining four (4) soil borings (B-005-0-23 through B-008-0-23) were performed near the roundabout, with one near the intersection and one on each leg near the roundabout realignment limits. The subgrade borings performed for the roundabout were performed to depths ranging from 7.5 to 8 feet below existing grades.

S&ME also performed two (2) Kessler DCP soundings in the private property area to the west of SR 241. The DCP's were performed to a depth of approximately 7.5 feet. After the DCPs were completed, hand auger samples were collected from the upper 21 to 24 inches of soil present below the existing surficial materials (i.e. topsoil) at the DCP locations. In addition to S&ME's explorations, ODOT performed eight (8) CPT soundings within the general area where the profile of SR 241 will be raised to depths ranging from 19.65 to 127.11 feet below existing grades. The pavement cored at seven (7) of the CPT locations which were performed in the northbound travel lane of SR 241. During the soundings, a total of 11 dissipation tests were performed.

Within the area where the profile of SR 241 is to be raised, the borings or CPTs encountered the following conditions, described in descending order:

- 8 to 18 inches of pavement materials (asphalt, concrete, brick) with three of the borings encountering 3 to 6 inches of granular base and Boring B-004-0-23 encountering 2 inches of sandy underlayment followed by 10 inches of cobbles intermixed with clay.
- Thin (approximately 3 feet thick) layer of very-stiff to hard SANDY SILT (A-4a) or SILT AND CLAY (A-6a) or medium-dense GRAVEL WITH SAND (A-1-b) which was visually identified as fill in Borings B-002-0-23 through B-004-0-23.
- Thick deposits of saturated, very-soft to soft (with occasional medium-stiff to stiff zones) SANDY SILT (A-4a), SILT (A-4b), SILT AND CLAY (A-6a), SILTY CLAY (A-6b) and ORGANIC CLAY (A-8b) to the termination depths of Borings B-001-0-23 and B-004-0-23 (15 feet and 25 feet, respectively) and to depths of 23 to 28 feet in Borings B-002-0-23 and B-003-0-23. Similarly weak soils were encountered in the CPT soundings to depths ranging from 21 to 28 feet; however, the two CPTs performed closest to Boring B-004-0-23 encountered these weak deposits to an approximate depth of 42 feet.
- Predominantly granular deposits consisting of very-loose to loose GRAVEL WITH SAND (A-1-b), FINE SAND (A-3), and/or SANDY SILT (A-4a) to a depth of 57 feet in Borings B-002-0-23 and B-003-0-23. This stratum also included a 2.3-foot-thick layer of stiff SILT (A-4b) in Boring B-002-0-23.



- Loose to medium-dense (with few very-loose zones) GRAVEL WITH SAND (A-1-b), FINE SAND (A-3), COARSE AND FINE SAND (A-3a), and SILT (A-4b) to the termination depth of 100 feet in Borings B-002-0-23 and B-003-0-23. This stratum also included a 5.5-foot-thick layer of stiff SANDY SILT (A-4a) in Boring B-002-0-23.

The borings performed for the roundabout encountered 8 to 16 inches of asphalt over 5 to 6 inches of granular base Borings B-005-0-23 and B-007-0-23. One (1) to 4 inches of topsoil was encountered at Boring B-006-0-23 and at Hand Auger Borings H-006-1-23 and H-006-2-23. Existing fill and probable/possible fill consisting predominantly of medium-dense GRAVEL WITH SAND (A-1-b), GRAVEL WITH SAND AND SILT (A-2-4), COARSE AND FINE SAND (A-3a) or SANDY SILT (A-4a) with a layer of stiff SANDY SILT (A-4a) encountered in Boring B-007-0-23. to depths ranging from 5.0 to 6.2 feet below the existing grade. Natural soils encountered consisted of medium-stiff to very-stiff SANDY SILT (A-4a), SILT AND CLAY (A-6a), SILTY CLAY (A-6b), CLAY (A-7-6), or loose to medium-dense COARSE AND FINE SAND (A-3a).

Based on conditions encountered in the borings and the preliminary information provided to date, a summary of the recommendations provided in this report is provided below.

- Using the ODOT Subgrade Analysis spreadsheet (Ver. 14.6, dated 2/11/2022), the average California Bearing Ratio (CBR) of the existing subgrade soils encountered during this exploration is 7%.
- The ODOT Subgrade Analysis spreadsheet indicates that 1 of the 8 borings performed during this exploration encountered soil at or just below the proposed subgrade level with characteristics defined as problematic (excessive soil moisture content or a low N_{60} value) and which may require remediation by the "Excavate and Replace" procedures outlined in the ODOT Subgrade Analysis.
- Using traffic information provided by ODOT District 4, and following discussions with District 4, a recommended flexible pavement section is summarized in Section 5.3.
- Based upon observations made at the time of this investigation, significant groundwater problems are not anticipated in connection with the proposed roadway improvements for the roundabout.
- Settlement on the order of 7 to 7.5 inches is anticipated in areas of maximum fill placement near the bridge abutments for the revised profile of SR 241. See Section 5.4.2.4 for a summary of settlement analyses.
- Due to the anticipated magnitude and rate of settlement, S&ME recommends preloading the embankment area and installing wick drains to accelerate the settlement waiting period. See Section 5.4.3.2 for a discussion of analyses for the installation of wick drains.
- Due to the magnitude of secondary settlement, downdrag loads may be applied to the deep pile foundations supporting the proposed bridge. S&ME recommends that consideration be given to the piles being sleeved through the soft soils encountered near the bridge abutments.

2.0 Introduction

This project includes a roundabout to be constructed at the intersection of SR 241 (Wales Avenue NW) and Shuffel Street NW, which is immediately southwest of the runway of the Akron-Canton Airport. In addition to the proposed roundabout, roadway flooding near an existing single span bridge approximately 1,200 feet southwest of the intersection is planned to be addressed by replacing the existing bridge and raising the profile of SR 241 by 3 to 4 feet from approximately 250 feet southwest of the bridge to approximately 300 feet northeast of the



bridge. At ODOT D04's request, S&ME previously performed (August 2023) the field work for a geotechnical exploration to obtain information at the bridge abutments and within the limits of the planned vertical profile adjustments, and for most of the roundabout. S&ME provided ODOT D04 a geotechnical data report summarizing the findings of the borings and the results of the laboratory testing program for this exploration on October 31, 2023. Additionally, in November 2023, ODOT OGE completed eight (8) Cone Penetrometer Test (CPT) soundings in the vicinity of the bridge to obtain additional information to assist with designing deep foundations for the replacement bridge and estimating settlement of the soft soils encountered in the soil borings.

This Geotechnical Exploration has been performed in general accordance with the January 2025 update of the ODOT *Specifications for Geotechnical Investigations (SGE)*.

2.1 Geology

The site is in the Akron-Canton Interlobate Plateau Physiographic Region which generally consists of sandy Wisconsinan-age glacial drift with frequent isolated areas of kames, kettles, and bogs/fens. Bedrock topography mapping indicates the site is over a buried valley where bedrock is present at depths greater than 150 feet below the existing ground surface. Mapping of the surficial glacial overburden indicates the presence of a thick section of ice contact deposits consisting of highly variable deposits of poorly sorted gravel and sand with inclusions of silt, clay, and till. Below the ice contact deposits, outwash sand and gravel deposits are present to the top of bedrock. It is also noteworthy that multiple areas of organic deposits are present near this site, but do not appear to be present at the bridge structure. S&ME also reviewed the logs of multiple water wells in the vicinity of this bridge. These logs generally indicated the presence of granular soils consisting predominantly of sand and gravel; however, several logs did indicate the presence of "mud" or otherwise soft/organic materials to depths of roughly 40 feet.

Inactive surface sand, gravel and limestone mines are located to the east of SR 241 stretching from SR 241 on the west to Lake O Springs Ave NW on the east and Shuffel Street NW on the north to Strausser Street NW on the south. Some of the existing ponds to the east of SR 241 may be located over open pit quarries.

2.2 Site Reconnaissance

S&ME initially visited the site on August 4, 2023, to observe and document the site conditions and mark the boring locations. S&ME observed that the water level at the existing bridge spanning SR 241 was just above the bottom of the bridge deck. Multiple ponds were observed in the southern half of the project. One pond is located on the east side of SR 241 at the bridge location and is tied to the flow in Nimisila Creek. The other ponds (one to the west and two further to the east of SR 241) are separated from the roadway by strips of land, with some of the ponds to the east of SR 241 also tied to the flow in Nimisila Creek.

The existing pavement within the project limits was generally in fair to good condition. Approach embankment slopes on either end of the bridge were heavily vegetated and were not observed to have evidence of slope instability. Away from the bridge, areas outside of the roadway shoulders range from relatively flat to slightly sloped with occasional ditches present. Evidence of multiple above or below ground utility lines (gas, electric, water, etc.) were observed across the site.



S&ME again visited the site on July 15, 2024, to mark the locations of Dynamic Cone Penetrometer (DCP) soundings. The soundings were performed in the cultivated field west of the intersection of SR 241 and Shuffel St. The existing grade was observed to be generally flat and filled with existing crops.

3.0 Exploration

3.1 Available Information

S&ME accessed the on-line ODOT Transportation Information Mapping System (TIMS) to determine if historic boring information was available for this area. No historic geotechnical information was available for this site.

In November 2023, ODOT OGE completed eight (8) Cone Penetrometer Test (CPT) soundings in the vicinity of the bridge to obtain additional information to assist with designing deep foundations for the replacement bridge and estimating settlement of the soft soils encountered in the soil borings.

3.2 Field Exploration

On August 16 through 18, 2023, and August 28 and 30, 2023, S&ME drilled Borings B-001-0-23 through B-008-0-23, which are hereafter referred to as B-001 through B-008. Roadway Borings B-001 and B-004 were drilled in the northbound lane of SR 241 from roughly 200 feet southwest to 400 feet northeast of the existing bridge, respectively. Bridge Borings B-002 and B-003 were drilled in the northbound lane of SR 241 roughly 15 feet behind (southwest and northeast) the existing bridge abutments, respectively. Subgrade Borings B-005 and B-006 were drilled in the northbound lane of SR 241 approximately 400 feet southwest of and at the southeast corner of the SR 241/Shuffel Street NW intersection, respectively. Subgrade Boring B-007 was performed in the southbound lane, roughly 640 feet northeast of the intersection, and Subgrade Boring B-008 was drilled in the westbound lane of Shuffel Street NE, approximately 450 feet east of the intersection.

S&ME obtained approximate latitude and longitude coordinates of the boring and DCP locations with a sub-meter accuracy GPS unit. Ground surface elevations and project stationing/offsets at the boring and DCP locations were provided by Michael Baker International (MB) based on the GPS coordinates obtained by S&ME.

The borings were advanced by a truck-mounted drill rig using a 3¼-inch hollow-stem auger. Disturbed, but representative, soil samples were attempted by lowering a 2-inch O.D. split-barrel sampler through the auger stem to the bottom of the boring and then driving the sampler into the soil with blows from a 140-pound hammer freely falling 30 inches (AASHTO T206 – Standard Penetration Test, SPT). In Boring B-003, more than 10 feet of heaving sand occurred at a depth of 65 feet, thus the original borehole had to be abandoned and an offset boring was drilled 4 feet northeast of the original boring to obtain samples between 65 and 100 feet. The offset boring for B-003 was performed with a bentonite slurry inside the augers to address the sand heave experienced in the original boring. On account of the drilling experience at Boring B-003, Boring B-002 was drilled with a bentonite slurry beginning at a depth of 5 feet after observing seepage at a depth of 3.1 feet.

Sulfate samples were collected from the first SPT interval at Borings B-001, and B-003 through B-008 using an oversized split spoon sampler driven after the initial SPT spoon was recovered. Minimally disturbed thin-walled samples (Shelby Tube samples) were collected at select depths at the bridge structure borings. In accordance with



ODOT specifications, the hammer system on the drilling rig was calibrated (ASTM D4633) on December 22, 2022, to determine the drill rod energy ratio (91.0%). The drill rig energy ratio used to compute N_{60} values has been limited to 90% per the ODOT *SGE*.

The SPT soil samples were examined in the field and representative portions were preserved in airtight glass jars and the Shelby Tube samples were sealed with wax. All recovered soil samples were transported to S&ME's lab for further examination and testing. Following the completion of drilling, the bridge structure borings were sealed with cement-bentonite grout and the pavement was repaired with quick-set concrete and topped with cold-patch asphalt. The roadway embankment and subgrade borings were backfilled with cuttings mixed with bentonite and the pavement surfaces were patched with cold-patch asphalt.

On July 19, 2024, S&ME performed two (2) Kessler DCP soundings (D-006-1-23 and D-006-2-23) and collected two (2) bulk hand auger samples (H-006-1-23 and H-006-2-23) adjacent to the DCPs in the private property area to the west of SR 241. The DCP's were performed to a depth of approximately 7.5 feet. Immediately adjacent to each DCP sounding, and below existing surficial materials (e.g., topsoil, cultivated soil), a bulk hand auger sample was obtained to depths of 1.8 to 2 feet.

The DCP soundings were completed using a Kessler K-100 series probe, by raising a 17.6-pound weight as much as 22.6 inches along a guide rod and allowing it to fall onto an anvil. Pairs of data including the penetration distance versus the number of blows are recorded as extension rods are added to achieve the planned termination depth (or refusal). After reaching the termination depth of the sounding, the penetration depth of the probe per blow may be correlated to the California Bearing Ratio (CBR) and the bearing capacity of the soils. The results of the DCP tests performed are provided as Plates 21 and 22 in Appendix I.

Approximate locations of the soil borings and DCPs/hand augers performed by S&ME and the CPT soundings performed by ODOT are shown on the Plans of Borings included as Plates 2A and 2B in Appendix I.

In the field, experienced personnel from S&ME observed the drilling procedures and performed the following specific duties: preserved all recovered samples; prepared a log of each boring; made seepage and groundwater observations in the borings; obtained hand-penetrometer measurements in soil samples exhibiting cohesion; and, provided liaison between the fieldwork and the Project Manager so that the program of exploration could be modified, if necessary, because of unanticipated conditions.

3.3 Laboratory Testing Program

In the laboratory, all soil samples were visually identified and tested for natural moisture content. Select, representative samples were tested for their Atterberg limits and particle size distributions. Samples collected for the purpose were tested for their sulfate content. Samples with a visual identification suggesting a higher organic content were tested for their organic content by Loss on Ignition (LOI). Results of the laboratory index tests are recorded numerically on the boring log. In addition, two (2) one-dimensional consolidation tests were performed on samples of soft soil obtained in Shelby tube samples recovered from Borings B-002 and B-003. The results of sulfate and consolidation tests performed on recovered soil samples are provided in Appendix II.

Based upon the results of the laboratory testing program, the field logs were modified, if necessary, and a copy of the laboratory corrected boring logs are submitted as Plates 4 through 19 of Appendix I. Shown on these logs are: descriptions of the soil stratigraphy encountered; depths from which samples were preserved; sampling efforts



(blow-counts) required to obtain the specimens in the borings; calculated N_{60} values; laboratory testing results; seepage and groundwater observations made at the time of drilling; and values of hand-penetrometer measurements made in soil samples exhibiting cohesion. For your reference, hand-penetrometer values are roughly equivalent to the unconfined compressive strength of the cohesive fraction of the soil sample.

Soils have been classified in general accordance with Section 603 of the ODOT *SGE* and described in general in accordance with Section 602. An explanation of the symbols and terms used on the boring logs, definitions of the special adjectives used to denote the minor soil components and information pertaining to sampling and identification are presented on Plate 3 of Appendix I. Group Indices determined from the results of the laboratory testing program are also provided on the boring logs.

4.0 Findings

Please refer to the boring logs (Plates 4 through 19 in Appendix I) and the Shelby Tube logs (Plate 20 in Appendix I) for the soil and groundwater/seepage conditions encountered at each boring location. Inferences should not be made regarding subsurface conditions in the areas away from or between the borings without performance of additional borings or other field verification.

4.1 Bridge Structure Borings (B-002 & B-003)

Beneath 11 to 13 inches of asphalt and 3 to 6 inches of granular base, bridge Borings B-002 and B-003 encountered the following general stratigraphy which is described in descending order;

- 1.9 to 2.1 feet of existing fill consisting of very-stiff brown and dark-brown SILT AND CLAY (A-6a) or very-stiff to hard dark-gray SANDY SILT (A-4a).
- 19.5 to 24.7 feet of an alluvial deposit containing very-soft to medium-stiff gray, dark-gray, dark-brown, and black SANDY SILT (A-4a), SILT (A-4b), SILT AND CLAY (A-6a), and SILTY CLAY (A-6b), and ORGANIC CLAY (A-8b), with organic content ranging from 0.4% to 2.7%. A layer of stiff SILT AND CLAY (A-6a) was encountered from 6.5 to 8.0 feet in Boring B-002. N_{60} values in these soils typically ranged from 0 to 2 blows per foot, with occasional samples achieving 3 to 5 blows per foot.
- 29 to 34 feet of very-loose to loose brown and gray GRAVEL WITH SAND (A-1-b), FINE SAND (A-3), SANDY SILT (A-4a), and SILT (A-4b) with an average N_{60} value of 5 blows per foot. At a depth of 32 feet in Boring B-002, an approximate 2.3-foot-thick zone of stiff brown SILT (A-4b) was encountered within this stratum.
- 5.5 feet of stiff gray SANDY SILT (A-4a) over 4.5 feet of loose to medium-dense gray GRAVEL WITH SAND (A-1-b) in Boring B-002, whereas Boring B-003 encountered 10.0 feet of loose to medium-dense brown COARSE AND FINE SAND (A-3a). N_{60} values in these materials ranged from 6 to 20 blows per foot.
- 21.9 to 32.8 feet of loose to medium-dense gray SILT (A-4b). N_{60} values in these materials ranged from 8 to 23 blows per foot.
- Boring B-002 was terminated after penetrating 0.2 feet into a deposit of medium-dense reddish-brown GRAVEL (A-1-a) with a few cobbles. Boring B-003 was terminated after encountering 3.1 feet of medium-



dense gray FINE SAND (A-3) over 8.0 feet of very-loose to medium-dense gray GRAVEL WITH SAND (A-1-b) with a few pockets of silt.

Seepage/groundwater was noted at the depths of 3.1 and 7.0 feet in Borings B-002 and B-003, respectively. Adjacent to the roadway, the water surface in Nimisila Creek was approximately 1.5 feet below the paved roadway surface.

4.2 Roadway Embankment Borings (B-001 & B-004)

Beneath 8 inches of asphalt over 6 inches of granular base, Boring B-001 encountered very-stiff brown SANDY SILT (A-4a) to a depth of 3 feet followed by medium-stiff reddish-brown SANDY SILT (A-4a) to a depth of 6 feet. Below these layers, 2 feet of loose reddish-brown COARSE AND FINE SAND (A-3a) was encountered over 2.5 feet of very-soft to medium-stiff reddish-brown SILT AND CLAY (A-6a). Boring B-001 was terminated at a depth of 15 feet after penetrating 4.5 feet into medium-stiff to stiff brown SANDY SILT (A-4a).

At Boring B-004, the existing pavement consisted of 14 inches of asphalt, 4 inches of brick with 2 inches of sand leveling course, over a 10-inch layer of COBBLES intermixed with clay. Beneath these surficial materials, Boring B-004 encountered existing fill and possible fill consisting of medium-dense brown GRAVEL WITH SAND (A-1-b) and medium-stiff dark-brown SILT AND CLAY (A-6a) to a depth of 4.5 feet. Beneath the fill, Boring B-004 encountered layers of very-soft to medium-stiff SILT AND CLAY (A-6a) and SILTY CLAY (A-6b) to the termination depth of 25 feet. A few stiff zones were encountered from 6.0 to 8.5 feet. N_{60} values in this very-soft to medium-stiff material ranged from 0 to 6 blows per foot.

During drilling, groundwater was not noted in Boring B-001, whereas Boring B-004 encountered seepage at 5.7 feet, and more significant quantities of groundwater at 18.9 feet. At completion, water was measured to have accumulated in Boring B-004 to a depth of 11.5 feet before removing the augers.

4.3 Roundabout Subgrade Borings (B-005 through B-008)

The existing SR 241 pavement section consisted of 14½ and 16 inches of asphalt over 5 to 6 inches of granular base at Borings B-005 and B-007, respectively. The pavement section on Shuffel Street NW consisted of 8 inches of asphalt with no discernable granular base layer at Boring B-008. Boring B-006 was performed outside the existing roadways and encountered 4 inches of topsoil.

Existing fill and probable/possible fill were encountered beneath the topsoil or pavement at Borings B-006 through B-008 to depths ranging from 5.0 to 6.2 feet below the existing grade. The fill consisted of medium-dense or stiff brown and/or red SANDY SILT (A-4a) or medium-dense brown GRAVEL WITH SAND (A-1-b), GRAVEL WITH SAND AND SILT (A-2-4) and COARSE AND FINE SAND (A-3a). Fill was not encountered at Boring B-005.

Below the surficial materials or existing fill, the roadway subgrade borings encountered natural soil consisting variably of loose to medium-dense brown COARSE AND FINE SAND (A-3a), medium-stiff brown SANDY SILT (A-4a), medium-stiff to stiff brown and/or gray SILT AND CLAY (A-6a) and SILTY CLAY (A-6b), and very-stiff brown CLAY (A-7-6). In Boring B-005, the consistency of the soil decreased with depth, becoming very-soft to soft below 6.0 feet to the termination depth of 7.5 feet.



4.4 Hand Auger Samples (D/H-006-1-23 and D/H-006-2-23)

Beneath 1 inch of topsoil, a bulk soil sample was obtained from Hand Auger H-006-1-23 and encountered SANDY SILT (A-4a) to a depth of 21 inches below the existing ground surface. Groundwater was not noted during augering.

Beneath 1 inch of topsoil, Hand Auger H-006-2-23 encountered SILT AND CLAY (A-6a) to a depth of 2 feet below the existing ground surface. Groundwater was not noted during augering.

4.5 Groundwater Observations

During drilling, seepage/groundwater was noted at depths of 3.1 and 7.0 feet in Borings B-002 and B-003, respectively. Adjacent to the roadway, the water surface in Nimisila Creek was approximately 1.5 feet below the paved roadway surface at the time of drilling.

Groundwater was not noted in Borings B-001, B-005, B-007 and DCP's D-006-1 and D-006-2. Seepage was encountered in Borings B-004, B-006, and B-008 at depths of 5.7, 3.3, and 5.3 feet, respectively, during drilling. At completion, water was measured to have accumulated in Boring B-004 to a depth of 11.5 feet before removing the augers.

4.6 Soil Sulfate Test Results

Sulfate content testing (ODOT Supplement 1122) was performed on soil samples obtained from the approximate proposed pavement subgrade level in all borings. The results of these tests indicated sulfate contents of non-detectable (less than 40 ppm) to 1947 ppm. These results are below the threshold value of 5,000 ppm identified by the ODOT *Geotechnical Design Manual (GDM)* as the sulfate content concentration above which chemical stabilization should not be performed. The results of these tests are provided on the log of each boring and on Plate 1A and Plate 1B in Appendix II.

4.7 Geotechnical Data for Scour Analysis

As directed in Section 1302 of the January 2025 ODOT *Geotechnical Design Manual (GDM)*, Table 4-1 presents laboratory determined D_{50} particle size, Critical Shear Stress (τ_c), $D_{50, \text{equivalent}}$, and Erosion Category (EC) based on the gradation testing of the soil samples recovered from the continuously sampled zone in Borings B-002 and B-003 performed at the site. This information is being provided for use by others to perform a scour analysis at the bridge foundations. This data was originally provided to ODOT D04 and Michael Baker International (lead designer) in Geotechnical Design Memorandum (GDM) #2 submitted on September 6, 2024, and a revised version of the GDM submitted on September 18, 2024.



Table 4-1: Scour Data

Boring Number	Sample ID	Sample Elevation	Lab D ₅₀ (mm)	Critical Shear τ_c (psf)	D _{50, equivalent} (mm)	Erosion Category (EC)
B-002-0-23	SS-2	1,069.3' - 1,067.8'	0.0188	0.1056	5.0552	3.48
	SS-3	1,067.8' - 1,066.3'	0.0211	0.0986	4.7195	3.55
	SS-4	1,066.3' - 1,064.8'	0.0160	0.1867	8.9396	3.07
	SS-5	1,064.8' - 1,063.3'	0.0375	0.0316	1.5113	3.07
	SS-6	1,063.3' - 1,061.8'	0.0139	0.0671	3.2112	3.07
B-003-0-23	SS-1	1,069.7' - 1,068.2'	0.0652	0.1118	5.3531	2.50
	SS-2	1,068.2' - 1,066.7'	0.0409	0.0991	4.7452	2.75
	SS-3	1,066.7' - 1,065.2'	0.0450	0.0029	0.1374	2.75
	ST-29	1,063.7' - 1,061.7'	0.0125	0.0610	2.9188	2.50

5.0 Analyses and Recommendations

5.1 Geotechnical Evaluation

This project involves the construction of a 3-legged roundabout at the intersection of SR 241 and Shuffel Street NW at the southwest corner of the Akron-Canton Airport. In addition to the roundabout, the project will also include the replacement of the existing single-span structure carrying SR 241 over Nimisila Creek in North Canton, Stark County, Ohio. This structure is located on SR 241 southwest of the proposed roundabout.

Based on Stage 1 plans prepared by MB and dated August 1, 2024, the proposed replacement structure currently being considered is a single span prestressed concrete box beam bridge on integral abutments and supported on deep foundations (driven piles). The bridge will have a 33-foot span (center-to-center of bearings) with a clear space/hydraulic opening of 30 feet between the face of each abutment. To address consistent flooding issues at this structure location, the profile of the roadway is proposed to be raised by 3 to 4 feet between approximately 250 feet south to 300 feet north of the bridge. The extended foundations for the proposed bridge structure will be designed by the ODOT Office of Geotechnical Engineering (OGE). Accordingly, S&ME has not included pile foundation recommendations in this report.

5.2 Roundabout Subgrade Analyses (~Sta. 591+50 to North End of Project)

5.2.1 ODOT Subgrade Analysis

Section 600 of the ODOT *GDM* provides a standard approach to performing explorations and assessing roadway subgrades. The associated spreadsheet (Ver. 14.7, updated 4/4/2024) created by the ODOT Office of Geotechnical Engineering (OGE) is used to estimate roadway subgrade support parameters and identify areas requiring

Subgrade and Structure Foundation Exploration Report – Final (Rev. #1)

STA-241-17.37 Roundabout and Bridge Replacement (PID 119585)

North Canton, Stark County, Ohio

S&ME Project No. 23170065A/B, Task Order No. 4-01-1



remediation. The spreadsheet (see Appendix IV) summarizes the soil type (by ODOT/HRB classification), group indices, depth, blow-counts, Atterberg Limit, and sulfate content values of the proposed subgrade soils encountered in the borings drilled for this project. Using this data, this spreadsheet computes an average of the estimated values of the California Bearing Ratio (CBR) for the soils encountered at or below the anticipated subgrade level of the proposed roadway profile.

The ODOT Subgrade Analysis spreadsheet also identifies subgrade soils which are “unsuitable” either by classification (A-4b, A-2-5, A-5, A-7-5, A-8a, A-8b) or if the Liquid Limit value is greater than 65%. Additionally, the spreadsheet determines if a subgrade soil may be potentially “unstable” and possibly require subgrade remediation by comparing the lab-measured moisture content to the estimated optimum moisture content of the subgrade soil, and/or by comparing the normalized blow-count (N_{60}) and the lowest N value (N_{60L}) from SPT sampling.

Based on these comparisons and correlations, the Subgrade Analysis spreadsheet provides alternative approaches to remediate and establish a stable soil subgrade using either “excavate and replace” (ODOT *Construction and Material Specifications (CMS)* Item 204) or chemical stabilization (CMS Item 206 and Supplement 1120). However, soils with a sulfate content above 5,000 ppm are generally prohibited from being chemically stabilized.

The subgrade remediation depths identified by the Subgrade Analysis spreadsheet presented in Appendix IV are based on the conditions encountered in the borings during this subsurface investigation. However, because the depth of remediation required is dependent on the moisture content of the subgrade soil at the time of construction, Section 600 of the ODOT *GDM* states that the ultimate decision on required remediation depths and limits should be based on observations during either proofrolling or test-rolling operations.

5.2.2 Subgrade Support Parameters

Based on a preliminarily assumed 15.25-inch-thick pavement section (including aggregate base course) and the proposed final grade profile provided in the Stage 1 plans, the following average California Bearing Ratio (CBR) is computed by the ODOT Subgrade Analysis spreadsheet for the anticipated subgrade soils encountered during this investigation:

CBR: 7%

Based on this average value, and Section 203.1 of the ODOT *Pavement Design Manual (PDM)*, the following value of Resilient Modulus (M_R) may be used during new pavement section design for this project.

M_R : 8,400 psi

These pavement subgrade support values may be used during complete pavement replacement design on this project provided that the entire proposed pavement subgrade is prepared in strict accordance with Item 204 of the ODOT *CMS*, and that all borrow soil placed as new fill within 3 feet of the final subgrade level of the new pavement can provide average subgrade support parameters which meet or exceed the above values. This subgrade evaluation also assumes that the subgrade for the new roadways is composed of the materials encountered in the borings. If, at the time of construction, it is determined that the subgrade consists of materials different than those encountered in the borings, the pavement design subgrade criteria should be reviewed and, if necessary, modified.



In addition to the soil borings, the results of the DCPs were assessed to compare their findings to the design CBR. Both DCPs encountered resistance correlated to a CBR of 10 or more in the upper 30 to 35 inches. In D-006-1-23, the correlated CBR dropped to as low as 4.5 between depths of 30 to 60 inches. In D-006-2-23, the correlated CBR dropped to as low as 5.5 between depths of 35 to 55 inches. Below 55 or 60 inches in both DCPs, a correlated CBR of 10 or higher was encountered to the termination depths.

5.2.3 Subgrade Remediation Assessment & Recommendations

Based on the results of the borings and the Subgrade Analysis spreadsheet (see Appendix IV), soil classified as being potentially unstable (low N_{60} and high moisture content) was encountered in Boring B-005 and would require remediation by the procedures recommended in the subgrade analysis. Additionally, no subgrade remediation is anticipated in the vicinity of the DCPs. As such, S&ME recommends that a subgrade remediation program consisting of excavate and replace (undercutting) be performed. Accordingly, Table 5-1 summarizes the estimated range and limits for subgrade remediation on the east leg of the intersection based on the results of the subgrade analysis.

Table 5-1: “Excavate and Replace” Subgrade Remediation Recommendations

Station Range	Recommended Remediation	Width Limits
SR 241 STA 594+50 to STA 598+25	Excavate and Replace – 12”	18 inches beyond the edge of the pavement, including under new curbs or shoulders

In accordance with Section 608 of ODOT *GDM*, Item 712.09 Geotextile Fabric Type D should be placed at the base of all undercuts, and be backfilled with Item 204 Granular Material, Type B or C. It is also recommended that all overexcavated subgrade areas that are backfilled with granular soil be drained to an underdrain, catch basin, or pipe, if possible.

All backfill and geotextile should be placed according to the specifications of ODOT Item 204. It should be noted, however, that ODOT *GDM* specifies that Item 204 Granular Material Type B without a geotextile fabric be utilized to backfill undercuts performed in the vicinity of any underdrains. As such, underdrains should be installed before placing geotextile and granular backfill. As “excavate and replace” is recommended to be used for remediation, Plan Note G121 from the ODOT *Location and Design Manual (L&D)*, Vol. 3 should be included in the plans.

Because of the variable nature of the wide spacing of the explorations, it is possible that other areas of unsuitable organic, elastic, or silt materials that were not encountered in any of the borings may be encountered during earthwork and proofrolling operations. Visual observation of the proofrolling procedures by the Geotechnical Engineer of Record may potentially result in a reduction of overexcavation of unsuitable soils in these areas.

S&ME recommends that construction traffic be minimized once the planned subgrade level has been attained. Construction traffic resulting from cyclical haul routes or limited access points may increase the quantity of soil identified by final proofrolling as requiring removal, particularly during periods of moist weather.



5.2.4 Embankment Foundation Preparation

Prior to commencing earthwork operations, it is recommended that all existing pavement, granular base, grass, topsoil, vegetation, and other miscellaneous materials be completely removed from the entire footprint of the proposed roadway embankment. Following the removal of these materials, it is recommended that the entire exposed subgrade and embankment foundation surface be examined by the Geotechnical Engineer of Record or their designated representative to identify any weak, wet, organic, or otherwise unsuitable soils that were not encountered during the subsurface exploration, especially in widened areas. Any such materials identified should be removed and replaced with suitable compacted fill (Item 203, or Item 204 when within 12 inches of the proposed subgrade).

Existing underground utility lines may be present beneath and adjacent to the existing roadway, and the type of material used and the relative compactness of backfill within any such utility trenches are unknown. S&ME recommends any planned utility relocation be performed prior to proofrolling. Some instability of utility trench backfill may occur during earthwork operations and/or proofrolling, and some recompaction of granular utility trench backfill may become necessary. Additionally, if water has accumulated within the utility backfill, the subgrade soil in the vicinity of any saturated utility trenches may have become sufficiently weak, soft, and/or wet that proofrolling may identify these additional areas as requiring overexcavation and replacement. In any case, care should be taken not to disturb any shallow utilities during proofrolling or overexcavation activities.

5.2.5 “At-Grade”/“Cut” Soil Subgrade Areas

Once the desired soil subgrade elevation has been attained in all “at-grade” and “cut” subgrade areas, the subgrade soil beneath the entire new roadway and shoulder pavement areas should be scarified and recompacted to a depth of 12 inches below the subgrade level in accordance with ODOT *CMS* Item 204.03. During recompaction, the moisture content of the subgrade soil should be maintained or adjusted in accordance with ODOT *CMS* Item 203.07.A.

Following the completion of the scarification and recompaction of the subgrade in these “cut”/“at-grade” areas, it is strongly recommended that construction traffic be restricted from traveling on the compacted subgrade until final acceptance proofrolling has been performed. Cohesive subgrade soils subjected to repeated moisture fluctuations resulting from exposure to rainfall and/or surface water runoff, may exhibit subgrade instability.

5.2.6 “Fill” Areas

Prior to commencing fill placement in embankment realignment or widening areas, S&ME recommends that consideration be given to performing Item 206.04 Test Rolling on all exposed embankment foundation areas beneath areas where new fill is required. Test rolling, performed in accordance with Item 204.06 of the ODOT *CMS* and Section 204 of the ODOT *Construction Administration Manual of Procedures*, would assist in identifying soft, wet, or weak zones, or areas of unsuitable, organic, or highly plastic soil that may be present in ditches, swales, or wetland areas. If any such zones of soft, wet, or weak soils are present, the materials contained in these zones should be scarified, dried, and thoroughly recompacted in place in accordance with ODOT *CMS* Item 203.07. If unsuitable or organic soils are encountered, these materials should be completely removed and the overexcavation filled in a controlled manner with compacted, suitable embankment material (*CMS* Item 203.02) which meet the additional requirements previously discussed in Section 5.2.2.



Soft, weak, wet, or unsuitable soils that are not removed from beneath a thin layer of fill may result in difficulties achieving the compaction percentages required for the new fill (ODOT *CMS* Items 203.07 or 204.03) such that final subgrade acceptance proofrolling may require overexcavation of the new fill where weak soils were “bridged” by a minimal thickness of new fill. Although ODOT *CMS* Item 203.05 permits the use of a “bridge lift” to aid in spanning soft or wet foundation areas, S&ME recommends that this practice not be permitted unless more than 3 feet of new embankment fill placement is required. Additionally, even if more than 3 feet of new fill is required in existing roadway ditches and the proposed widened areas, S&ME does not recommend that a bridge lift be permitted in these areas because of the potential for organic soil in these areas. Long term settlement within any organic soil left in the embankment areas may result in the development of a depression in the pavement surface.

5.2.7 *Benching*

After all unsuitable materials have been removed and prior to commencing fill placement, it is recommended that horizontal benches be cut into all existing sloping surfaces steeper than 8(H):1(V) to permit placement and compaction of new fill in horizontal lifts. Where new fill is to be placed on an existing ground surface with a slope between 8(H):1(V) and 4(H):1(V), S&ME recommends that benching of the existing ground be performed in accordance with Item 203.05 of the ODOT *CMS*. At locations where the existing ground surface is steeper than 4(H):1(V), S&ME recommends “Special Benching” procedures as outlined in the ODOT Geotechnical Bulletin GB2, “*Special Benching and Sidehill Embankment Fills*” and the ODOT *Construction Inspection Manual of Procedures (CIMP)* should be performed. Additionally, in accordance with ODOT GB2, wherever “Special Benching” is used, Plan Note G109 from the ODOT *L&D Manual, Vol. 3*, should be included in the General Notes.

During any required Special Benching procedures, S&ME also recommends the following: 1) only one bench be exposed at any given time and that excavation of the next bench should not be permitted until embankment fill placement and compaction has been completed to the top of the backslope of the previous bench; and, 2) the length of any given bench that is exposed should not exceed the quantity of embankment fill which may be properly placed and compacted in one day. Additionally, S&ME recommends that the final, completed side slopes of embankments be constructed no steeper than 2(H):1(V).

5.2.8 *Borrow Requirements and Compaction Criteria*

Soil used as backfill should consist of inorganic soil free of all miscellaneous materials, cobbles, and boulders, which is placed in uniform, thin layers and then compacted in accordance with either ODOT *CMS* Item 203 or when within 12 inches of the proposed subgrade level, *CMS* Item 204. Borrow materials should not be placed in a frozen condition or upon a frozen surface, and any sloping surfaces on which new fill is to be placed should first be benched in accordance with either ODOT *CMS* Item 203.05 or Section 800 of the ODOT *GDM*, depending on the slope of the existing ground surface at each location.

As previously discussed in Section 5.2.2 of this report, any borrow materials to be used as new fill or backfill within 3 feet of the proposed subgrade level should be tested in the laboratory to determine that the borrow materials are capable of exhibiting subgrade support characteristics that are no less than the CBR value used during the pavement design.

Compaction requirements for the construction of earthen embankments are based on ODOT *CMS* Item 203.07.B which specifies a minimum percent compaction based on the dry unit weight of the type of soil fill being placed



as borrow. S&ME recommends that once the source of borrow for this project is determined, sampling and testing of this borrow material be performed prior to construction to verify that the borrow soils are suitable for the planned construction.

5.2.9 Compaction/Moisture Conditioning Concerns

The cohesive soils encountered in the borings performed for this project, if exposed to inclement weather or rainfall, may rapidly absorb additional moisture and weaken. It is imperative that these soil types not be exposed to rainfall while in a loosened state (such as during discing and drying for moisture conditioning during fill placement). Should these materials become sufficiently saturated that additional moisture conditioning is impractical, the material should be wasted. Therefore, it is recommended that moisture conditioning only be performed when extended periods of suitable weather are anticipated, and that only the amount of borrow soil be exposed that may be moisture conditioned and properly compacted during suitable weather periods.

5.2.10 Groundwater Considerations for Roadway Subgrade

Based upon observations made at the time of this investigation, significant groundwater problems are not anticipated in connection with the proposed roundabout construction. The new roundabout subgrade should be graded to prevent surface runoff from pooling on the cohesive soils during construction as exposure of cohesive soils to moisture will result in a decrease in strength and an increase in compressibility. Soil softened by standing water or disturbed by construction activities should be removed before proceeding with construction.

In addition to proper subgrade preparation, we recommend that the pavement design and construction include surface and subsurface drainage measures. Water which infiltrates the pavement and remains trapped within the pavement components during traffic loading is one of the leading causes of premature pavement failure. Effective design measures include the use of perforated underdrain pipes or finger drains below pavements and/or the use of perimeter swales, perimeter edge drains, curbs, or a combination of these features to collect surface water runoff from areas adjacent to the pavement. Cohesive subgrade soils should be crowned or sloped to promote drainage of infiltrating water towards subsurface drainage collection systems.

5.3 Flexible Pavement Thickness Analysis

S&ME was requested by ODOT D04 to perform pavement section thickness analyses to recommend a flexible pavement section for all full-depth pavement replacement areas of the project (SR 241, the roundabout, and Shuffel Street). To perform the pavement thickness analysis, ODOT D04 provided S&ME with the traffic information presented in Table 5-2.

Table 5-2: Available Traffic Information

Roadway	2028 ADT	2048 ADT	T24
SR 241	9,980	10,770	3.4%
Shuffel St.	8,820	11,240	4.9%



Using this traffic information and an assumed design life of 20 years, S&ME used the design procedures in the ODOT Pavement Design Manual (PDM) to develop preliminary pavement thickness alternatives. These alternatives were discussed with ODOT D04 on July 31, 2024, at which time ODOT D04 indicated their preference for a flexible pavement section. Additional adjustments to the pavement section were requested by ODOT D04 in a January 22, 2025, email and the final flexible pavement section requested by ODOT D04 is provided in Table 5-3.

Table 5-3: Flexible Pavement Section Requested by ODOT D04

Pavement Material (ODOT CMS)	Layer Thickness (inches)	Application Rate (Gal./SY)
Item 442 Asphalt Concrete Surface Course, 12.5mm, Type A, PG76-22M	1¼	--
Item 407 Tack Coat	--	0.055
Item 442 Asphalt Concrete Intermediate Course, 12.5mm, Type A, (448), PG70-22M	1¾	--
Item 407 Tack Coat	---	0.055
Item 301 Asphalt Concrete Base, PG64-22, (449)	9	--
Item 304 Aggregate Base	6	--

5.4 SR 241 Embankment Modifications (~Sta. 586+50 to Sta. 591+50)

5.4.1 Summary of Embankment Foundation Conditions

As discussed previously, the project scope includes replacement of the bridge over Nimisila Creek and raising the roadway profile by as much as approximately 3 to 4 feet within an approximate 500-foot-long portion of SR 241. Based on the weak/soft/saturated soil conditions encountered in Borings B-001 through B-004 and within the CPT soundings, the addition of 3 to 4 feet of new fill is anticipated to induce settlement. Settlement within granular layers, such as those generally encountered below approximately 25 feet near the bridge abutments, is anticipated to occur during construction. However, settlement in cohesive soils typically requires significantly longer periods of time, often on the order of months to years, to occur. In addition, ODOT has indicated that the entire project (roundabout, embankment and bridge construction) is to be completed within one (1) construction season/year. This construction schedule will require the desired embankment settlement to be achieved within 3 months.

To meet the currently proposed construction schedule, minimize the magnitude of long-term settlement, and to achieve the proposed roadway profile, we recommend to preload the soft compressible soils with the thickness of suitable Item 203 embankment necessary to attain the new subgrade elevation plus an additional thickness of surcharge embankment equivalent to the combined thickness of the proposed pavement section plus an additional 1.0 feet of Item 203 embankment. This embankment "surcharge" shall be placed and allowed to settle prior to performing final grading of the embankment to the proposed subgrade level. Further, we recommend that wick drains along with a granular drainage layer (or other premanufactured drainage medium) be installed



beneath the planned embankment fill and the additional “surcharge” embankment to reduce the settlement waiting period.

Based on the conditions encountered in the embankment/bridge borings and CPT soundings, a summary of a potential construction sequence for revising the embankment profile near the bridge while minimizing the anticipated magnitude and duration of the primary settlement of the underlying weak and slightly organic soil is as follows:

- Remove the existing pavement and all other existing structures (except for the existing bridge).
- Prepare the embankment subgrade beneath the roadway and proposed embankment widening areas.
- Construct a construction platform and a drainage layer for the installation of prefabricated vertical (wick) drains (PVDs).
- Install the PVDs across the embankment footprint between approximate Sta. 586+00 and Sta. 588+35 (west/south of existing bridge) and between approximate Sta. 588+58 and Sta. 591+25 (east/north of existing bridge).
- Install settlement and pore water pressure measurement instrumentation.
- Construct the surcharge embankment using normal weight fill (CMS Item 203.02.R Suitable Materials) to an estimated 12 inches higher than the proposed final embankment elevation.
- Allow a waiting period for the primary settlement to occur within the existing soils beneath the surcharged embankment. See Section 5.4.3.2 for the recommended waiting period requirements.
- Remove the surcharge fill and wick drain materials necessary to attain the proposed pavement subgrade elevation.
- Perform final grading, proofrolling and install the proposed granular base and flexible pavement section.

Based on the results of our analyses, **it should be understood by D04 that this type of preloading of the compressible foundation soils will reduce, but not eliminate, the long term (secondary) settlement anticipated to occur during the proposed design life of the new pavement and bridge.** Ongoing maintenance to mitigate pavement cracking and the embankment profile, particularly adjacent to the new bridge, should be anticipated.

The following sections provide a summary of our settlement analyses results and geotechnical recommendations for both short- and long-term mitigation of the embankment profile near the bridge over Nimisila Creek.

5.4.2 Settlement Analyses – Bridge Approach Embankments

5.4.2.1 Geometry

Embankment settlement analyses were performed at four (4) cross-sections located at Sta. 587+00, Sta. 588+00 (rear abutment), Sta. 589+00 (forward abutment) and Sta. 590+50. The cross-sections were evaluated based on the proposed embankment geometry, the geometry of the existing ground surface, and the subsurface data obtained from the explorations, as well as CPT data provided by ODOT OGE. The surcharge and final embankments were evaluated with horizontal to vertical side slopes of 3H:1V to 4H:1V, as shown on the Stage 1 cross sections provided to S&ME.



5.4.2.2 Strength and Settlement Parameters

The strength and settlement parameters used in the stability and settlement analyses of the soils encountered beneath the embankment were estimated based on empirical correlations with laboratory index property testing, hand (pocket) penetrometer measurements, consolidation tests, published correlations to Standard Penetration Test (SPT) and Cone Penetrometer Test (CPT) results, literature values, and experience with similar soils. Based on these values/properties, the natural soils beneath the upper layer of fill were considered to range from an over-consolidation ratio (OCR) of 1.3 (beneath the existing roadway embankment) to normally consolidated, i.e., an OCR of 1.0 (outside the limits of the existing embankment). Summary tables presenting the soil parameters used in the analyses are included in Appendix IV.

5.4.2.3 Groundwater Modeling

At completion, groundwater depths reported for borings performed at the bridge ranged from 3.1 to 7.0 feet below the current pavement surface. Monitoring wells were not installed in the test borings, so there are no extended groundwater level measurements. Based on visual observation of water levels approximately the same as the bottom of the existing bridge deck, a groundwater depth of 3 feet below the existing roadway has been utilized to estimate the groundwater elevation during our stability and settlement analyses.

5.4.2.4 Settlement Analyses

S&ME estimated the settlement of the existing ground surface under the weight of the proposed roadway embankment and surcharge fill at the four (4) cross sections identified in Section 5.4.2.1. The magnitude of foundation soil settlement from the weight of the proposed roadway embankment and surcharge fill were estimated using the Settle3 (Ver. 5.025) computer program developed by Rocscience. At each of the four (4) cross-sections analyzed, the proposed embankment configuration (final roadway elevation and proposed embankment slopes) was obtained from the Stage 1 plans. Analyses were performed for the final embankment condition (using final grade information from the Stage 1 plans) and a construction condition which includes a 12-inch-thick surcharge fill across the embankment crest to accelerate the settlement. Settle3 analysis output for each station and scenario are presented in Appendix IV.

Due to the differing levels of consolidation that have and are anticipated to occur across the existing embankment and the widened embankment, localized differential settlement across the proposed embankment should be anticipated. Table 5-4 summarizes the estimated settlements at the centerline of the roadway (maximum settlement), the outside crest of the embankment, and at the left and right toe of the embankment slope for the proposed embankment configuration. As previously discussed, the installation of wick drains beneath the elevated approach embankments will be required for the anticipated settlement to occur within an approximate 3-month time frame (as requested by D04). Therefore, the settlement magnitudes and time frames discussed in the remainder of this report are based on wick drains being present beneath the new, widened SR 241 embankment.



Table 5-4: Estimated Total Settlement Across Cross Sections Analyzed

Station	Exist. Ground Surface Elev.	Proposed Final Embankment Elev.	Estimated Total Settlement (in)*		
			Centerline	Crest	Toe of Slope
587+00	1076.5	1077.6	1.8	1.5	0.4
588+00	1073.4	1076.3	5.4	4.6	0.7
589+00	1071.5	1075.1	5.7	4.8	0.6
590+50	1071.7	1073.4	2.5	1.9	0.5

*Total settlement includes immediate settlement, primary consolidation and secondary consolidation.

Since the maximum settlement will occur along the centerline of the embankment, the remainder of this report will only address the magnitude and the time for settlement to occur at the centerline of the embankment. Tables 5-5 and 5-6 provide a detailed breakdown of the three main types of settlement expected to occur which include elastic/immediate settlement (settlement occurring in granular soil shortly after/during construction), primary consolidation (settlement expected to occur during and within a few weeks to months after construction), and secondary consolidation (settlement expected to occur following the completion of the primary consolidation through the design life of the structure, assumed to be 75 years for the bridge).

Table 5-5: Estimated Settlement Magnitude at Centerline of Proposed Fill Embankment

Station	Exist. Ground Surface Elev.	Proposed Final Embankment Elev.	Estimated Settlement of Embankment (in)			
			Elastic (Immediate)	Primary	Secondary*	Total
587+00	1076.5	1077.6	0.4	0.9	0.5	1.8
588+00	1073.4	1076.3	0.9	3.0	1.5	5.4
589+00	1071.5	1075.1	0.7	3.4	1.6	5.7
590+50	1071.7	1073.4	0.3	1.4	0.8	2.5

*Secondary settlement estimates include settlement that is estimated to occur within 75 years after construction of the embankment.

Table 5-6: Estimated Settlement Magnitude for Proposed Embankment + Surcharge Fill

Station	Exist. Ground Surface Elev.	Surcharge Embankment Elev.	Estimated Settlement of Embankment (in)			
			Elastic (Immediate)	Primary	Secondary*	Total
587+00	1076.5	1078.6	0.6	2.1	1.0	3.7
588+00	1073.4	1077.3	1.1	4.3	1.8	7.2
589+00	1071.5	1076.1	0.9	4.7	1.9	7.5
590+50	1071.7	1074.4	0.5	2.6	1.3	4.4

*Secondary settlement estimates include settlement that is estimated to occur within 75 years after construction of the embankment.



These analyses have been performed for both the final embankment configuration as shown in the Stage 1 plans (Table 5-5) and for the temporary surcharge embankment condition (Table 5-6) which is implemented to accelerate the settlement process. Plots of the total settlement and primary consolidation by layer have been included in Appendix IV.

The settlement estimates shown in Tables 5-4 through 5-6 are based on the state of the practice 1-D theory of consolidation. However, because of the heterogeneous nature of the soils encountered in the borings, and inherent differences in behavior between laboratory specimens and in-situ soils, it is difficult to develop precise estimates for the magnitude of settlement. Consequently, the settlement estimates should be considered approximate.

5.4.3 *Embankment Design and Construction*

This section provides recommendations for the embankment design and construction near the bridge over Nimisila Creek.

5.4.3.1 Embankment Foundation Preparation

Prior to commencing earthwork operations, it is recommended that all existing pavement, structures (except the existing bridge), existing trees, vegetation, and other miscellaneous materials be removed from the entire footprint of the proposed roadway/embankment. In particular, the existing asphalt and concrete pavement should be completely removed as the wick drains cannot be penetrated through these materials. Where the new embankment footprint extends beyond the existing embankment, it is recommended that the surficial vegetation be removed.

5.4.3.2 Wick Drains Spacing and Waiting Period Duration

Based on discussions with ODOT D04, we understand the project is to be completed in a single construction season/year, requiring the primary consolidation settlement to occur within a maximum timeframe of 3 months. Based on our settlement analyses, settlement of the soft cohesive soils in the upper 28 feet of the soil profile would require more than 3 months (ranging from 4 months to 2 years) to achieve 90% of the estimated primary consolidation. Therefore, S&ME recommends that wick drains (a.k.a., prefabricated vertical drains, or PVD's) be installed into the natural soils beneath the existing and proposed portions of the widened roadway embankment to reduce the amount of time needed for the primary consolidation settlement to occur.

The spacing of the wick drains beneath the embankment will dictate the length of the waiting period. Using the results of dissipation tests performed by ODOT using the CPT rig, S&ME performed settlement analyses with wick drains spaced from 3 to 8 feet. Based on these analyses, wick drains spaced 3 feet apart are required to achieve a minimum of 90% of primary consolidation settlement within the allowable 3-month waiting period requested by ODOT.

A summary of these analyses are presented in Table 5-7. This table also includes the estimated time for 95% of primary consolidation to occur. The greater the percentage of primary consolidation allowed to occur before removing the surcharge material to meet the planned subgrade elevation will decrease the anticipated magnitude of secondary settlement. The estimated waiting period does not begin until the full height of embankment is completed.



Table 5-7: Estimated Time Required to Achieve 90% or 95% of Primary Consolidation With Wick Drains and Estimated Remaining Settlement

Station	Days to Achieve 90% Primary Consol.	Total Settlement Achieved (in.)	Estimated Remaining Settlement (in.)*	Days to Achieve 95% Primary Consol.	Total Settlement Achieved (in.)	Estimated Remaining Settlement (in.)*
587+00	70	2.4	0	90	2.5	0
588+00	60	4.9	0.5	80	5.2	0.3
589+00	30	5.1	0.6	50	5.3	0.4
590+50	50	2.8	0	90	2.9	0

*Estimate of remaining settlement represents the magnitude of settlement expected to occur with the final proposed embankment configuration, after consolidation has occurred beneath the surcharge loading, the surcharge fill has been removed and final paving has been performed.

As summarized in Table 5-7 above, additional settlement of approximately 0.3 to 0.6 inches is anticipated near each abutment from the completion of construction through an assumed 75-year design life of the new bridge. This additional settlement may result in “bridge bumps” at the bridge abutments or approach slabs, as the remaining primary consolidation and secondary consolidation of the embankment continues over time. Accordingly, the need for future maintenance to address these “bumps” should be anticipated. Additionally, the settlement over time at the abutments may also have impacts on the piles installed at each abutment. See Section 5.5.1 for discussion of these impacts.

For cost estimating purposes, the wick drains should extend below the bottom depth of the predominantly cohesive soils as encountered in the soil borings and CPT soundings. Within the south/west approach embankment (approximate Sta. 586+00 to Sta. 588+35) the wick drains should be estimated to have an average length of 29 feet. Within the north/east approach embankment (approximate Sta. 588+58 to Sta. 591+25) the wick drains should be estimated to have an average length of 24 feet. We note that near the north end of the wick drain area (near Boring B-004/CPT C-004-1-23) the clayey soils extend to a depth of approximately 43 feet. However, it is noted that very little fill (less than one foot) will be placed near Boring B-004. The lateral extents of the wick drains should cover the entire proposed embankment footprint.

We understand that due to utility conflicts, wick drains will not be able to be installed within a portion of the north side of the road east of the bridge. Wick drains should be installed, at a minimum, to the outside of the proposed roadway pavement and/or curb to prevent longitudinal cracks from forming prematurely as the embankment areas without wick drains settle over time. Additionally, we understand that wick drains will not be installed across a 20-foot-wide stretch of roadway near Sta. 591+00 due to an aerial utility crossing conflict. Areas without wick drains should be minimized to the extent possible as these areas will experience slower settlement timeframes compared to the remaining portions of the embankment, which may result in cracks, bumps, dips or sags in the roadway or embankment over time.

Because of the heterogeneous nature of the soil encountered in the borings, it is difficult to develop precise estimates for the length of time required for the majority of the primary consolidation settlement to be complete.



Due to two-dimensional flow and the potential presence of sand and silt lenses, the actual field time required to complete primary consolidation may vary from the calculated rate and should be revised based on a review of settlement data collected during the waiting period. For this reason, S&ME also recommends that settlement monitoring program be included as part of this project (see Section 5.4.3.4).

5.4.3.3 Embankment Fill Placement

As noted in the construction sequence discussion in Section 5.4.1, a drainage layer must be constructed to allow water from the wick drains to drain out/away from the embankment. This drainage layer is constructed at the base of the embankment, prior to placing any embankment fill. Following installation of the wick drains and construction of the drainage layer, the embankment should be constructed to the proposed surcharge elevation using normal weight fill materials. Since the majority of the embankment will be permanent, moisture control and compaction testing should be performed, and the fill should be constructed in strict accordance with ODOT Item 203.

5.4.3.4 Instrumentation and Monitoring

S&ME recommends that vibrating wire piezometers and settlement platforms be installed prior to surcharge fill placement to monitor the settlement and pore pressure during and after surcharge embankment construction. Recommended locations for these piezometers and settlement platforms are summarized in Table 5-8. Additionally, S&ME provided recommended plan notes to ODOT D04 and Michael Baker International on March 20, 2025, to be included in the Stage 2 plan to address the wick drain areas and recommended instrumentation.

Table 5-8: Recommended Locations for Embankment Monitoring Instrumentation

Approximate Station and Offset	Piezometer Tip Elevation (ft-MSL)	Settlement Platform
588+32, CL	El. 1057	Yes
588+60, CL	El. 1058	Yes
590+20, CL	N/A	Yes

Periodic review of piezometer and settlement platform instrumentation data during the waiting period will determine when the desire percentage of primary consolidation has actually been achieved and removal of the surcharge fill can commence.

5.4.3.5 General Embankment Construction Considerations

Recommendations and considerations provided in 5.2.6 through 5.2.9 of this report will generally apply to embankment construction and subgrade preparation. The following exceptions to the recommendations given in those sections do not apply to areas where wick drains are planned.

- Weak or wet materials do not need to be removed and replaced during embankment foundation preparation after existing pavement and topsoil has been removed and before wick drains are installed, provided the wick drain installer is able to safely operate equipment during installation.



- Bridge lifts during embankment construction should not be permitted for any reason.

5.4.3.6 Embankment Stability Analyses

In accordance with AASHTO and ODOT guidelines, the global stability of the proposed embankment section at Sta. 588+00 (near forward abutment) was analyzed for undrained (total stress) and drained (effective stress) conditions for the currently proposed final embankment. Stability at this embankment cross-section representing the greatest height of new fill was assessed using a two-dimensional limit equilibrium modeling technique which simplifies the failure or "slip" surfaces by dividing the slope into vertical "slices" and fitting line segments or arcs of various radii and centers, or plane slip surfaces. Various surfaces are then checked to determine the slip surface with the smallest ratio of resisting forces (soil strength) to driving forces (mass of the soil and water, and traffic loading). The ratio of the resisting forces divided by the summation of the driving forces acting on the slices is the factor of safety for the embankment section analyzed. The computer program SLIDE v6.0 was used to perform the analyses. We used the Spencer method to evaluate the stability of each cross-section analyzed. SLIDE output is presented in Appendix IV. In accordance with AASHTO specifications, a surcharge of 250 pounds per square foot (psf) was applied over the full width of the final (permanent) proposed roadway cross-section to represent traffic loading.

S&ME also analyzed the embankment for bearing capacity and lateral squeezing failure modes in accordance with FHWA recommendations. We have assumed minimum safety factors of 1.3, 1.5, and 1.5 for global embankment stability, embankment bearing resistance, and lateral squeezing, respectively, for temporary (i.e., construction) short term conditions associated with the surcharge embankment. A long-term minimum safety factor of 1.3 was used for the permanent fill embankment.

5.5 **Proposed Replacement Bridge Recommendations/Considerations**

5.5.1 *Downdrag Considerations*

ODOT OGE has performed analyses for the deep foundation system proposed to support the bridge. However, the magnitude of embankment settlement will also have an impact on the design of the proposed extended foundation system (currently, driven piles) for the proposed replacement bridge. It should be noted that depending on the degree of consolidation achieved during the waiting period, secondary (long-term) settlement occurring over the life of the structure may be sufficient (>0.4 inches) to result in downdrag loads acting on the driven piles. For this reason, S&ME previously recommended that consideration be given to isolating the piles from the surrounding soil undergoing settlement by installing "sleeves" to the bottom of the cohesive soils. However, based on direction from ODOT OGE combined with the estimated magnitude of settlement in excess of 0.4 inches and the relatively weak nature of the upper approximately 25 feet of cohesive soils, potential downdrag loading, if any, is anticipated to be negligible and pile sleeves are not necessary.

5.5.2 *Earth Pressures on Bridge Abutments*

The proposed abutments must be designed to withstand earth pressures, as well as hydrostatic pressures, that may develop behind the structures. The magnitude of the earth pressures varies based on soil type, permissible wall movement, and the configuration of the backfill.



To minimize earth pressures, the zone behind abutment walls should be backfilled with granular soil, and the backfill should be effectively drained. For effective drainage, a zone of free-draining gravel (ODOT *CMS* Item 518.03) should be used directly behind the structures for a minimum thickness of 24 inches in accordance with ODOT *CMS* Item 518.05. This granular zone should drain to either weepholes or a pipe, so that hydrostatic pressures do not develop against the walls.

The type of backfill beyond the free-draining granular zone, however, will govern the magnitude of the pressure to be used for structural design. Pressures of a relatively low magnitude will be developed by using granular backfill, whereas a cohesive (clay) backfill will result in the development of much higher pressures.

To minimize earth pressures, it is recommended that granular backfill be used behind the abutments and any wingwalls. The backfill should be placed in a wedge formed by the back of the structure and a line rising from the base of the wall abutment foundations at an angle no greater than 60 degrees from horizontal. Granular backfill behind the structures should be compacted in accordance with ODOT *CMS* Item 203, "Embankment Compaction". Over-compaction in areas directly behind the walls should be avoided, as this might cause damage to the structure.

If proper drainage is provided and compacted granular backfill is provided as described above, an equivalent fluid unit weight of 35 lb/ft³ (pcf) may be used if movement equivalent to 0.25 percent of the height of the abutment or wingwall (H) is allowed to occur. Such movement is considered sufficient to mobilize an active earth pressure condition, and the resultant lateral force should be taken as acting at 0.33H. If this movement is not anticipated or cannot occur, it is recommended that an "at-rest" equivalent fluid unit weight of 55 pcf be used.

Compacted cohesive materials tend alternatively to shrink, expand, and creep over periods of time and create significant lateral pressures on any adjacent structures. Cohesive materials also require a greater amount of movement to mobilize an active earth pressure condition. For these reasons, if proper drainage (ODOT *CMS* Item 518) is provided and a wall movement exceeding 1.0 percent of the height of the abutment or wingwall (H) is allowed to occur, an equivalent fluid unit weight of 65 pcf may be used for design of the abutment walls to resist the lateral loads imparted by drained cohesive backfill. If this amount of movement is not anticipated or cannot occur, it is recommended that an "at-rest" equivalent fluid unit weight of 95 pcf be used.

The structures must also be designed to withstand the surcharge effect of traffic in addition to the vertical load resulting from the weight of any fill and pavement to be placed over the structures. To estimate vertical loading, a total unit weight of 125 pcf and 135 pcf may be used for compacted cohesive and granular soil, respectively.

5.6 Groundwater Considerations for Embankment and Bridge Construction

Based upon observations made at the time of this investigation, groundwater may be encountered at shallow depths (within 3 feet below existing grade) or flooding may occur following periods of heavy rain. Installation of the wick drains and construction of the drainage layer may need to be performed during drier periods or when the water level in the pond adjacent to SR 241 is sufficiently low enough to perform the work. Once the wick drains have been installed and new embankment construction has commenced, surfaces should be graded to prevent surface runoff from pooling on cohesive soils used in construction. Additionally, excavations for proposed bridge abutments may be below the water elevation in the pond next to SR 241 and will likely require cofferdams and continuous dewatering during construction of the abutments.



5.7 Temporary Excavation Considerations

In Federal Register, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, part 1926, Subpart P". This document was issued to better ensure the safety of workers entering trenches or excavations. It is mandated by this federal regulation that excavations be constructed in accordance with the OSHA guidelines. It is our understanding that these regulations are being strictly enforced and if they are not closely followed, the owner and the contractor could be liable for substantial penalties.

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor's "responsible person", as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations. If an excavation, including a trench is extended to a depth of more than twenty (20) feet, it will be necessary to have the side slopes designed by a professional engineer registered in the state where the construction is occurring.

We are providing this information solely as a service to our client. S&ME does not assume responsibility for construction site safety or the contractor's or other parties' compliance with local, state, and federal safety or other regulations.

6.0 Considerations and Report Limitations

This draft report has been prepared in accordance with generally accepted geotechnical engineering practice for specific application to this project. The conclusions and recommendations contained in this report are based upon applicable standards of our practice in this geographic area at the time this report was prepared. No other representation or warranty, either expressed or implied, is made.

We relied on preliminary project information given to us to develop our conclusions and recommendations. If project information described in this report is not accurate, or if it changes during project development, we should be notified of the changes so that we can modify our recommendations based on this additional information, if necessary.

Our conclusions and recommendations are based on limited data from a field exploration program. Subsurface conditions can vary widely between explored areas. Some variations may not become evident until construction. If conditions are encountered which appear different than those described in our report, we should be notified. This report should not be construed to represent subsurface conditions for the entire site.

Unless specifically noted otherwise, our field exploration program did not include an assessment of regulatory compliance, environmental conditions or pollutants or presence of any biological materials (mold, fungi, bacteria). If there is a concern about these items, other studies should be performed. S&ME can provide a proposal and perform these services if requested.

Subgrade and Structure Foundation Exploration Report – Final (Rev. #1)

STA-241-17.37 Roundabout and Bridge Replacement (PID 119585)

North Canton, Stark County, Ohio

S&ME Project No. 23170065A/B, Task Order No. 4-01-1



S&ME should be retained to review the final plans and specifications to confirm that earthwork, foundation, and other recommendations are properly interpreted and implemented. The recommendations in this report are contingent on S&ME's review of final plans and specifications followed by our observation and monitoring of earthwork and foundation construction activities.

Subgrade and Structure Foundation Exploration Report – Final (Rev. #1)
STA-241-17.37 Roundabout and Bridge Replacement (PID 119585)
North Canton, Stark County, Ohio
S&ME Project No. 23170065A/B, Task Order No. 4-01-1



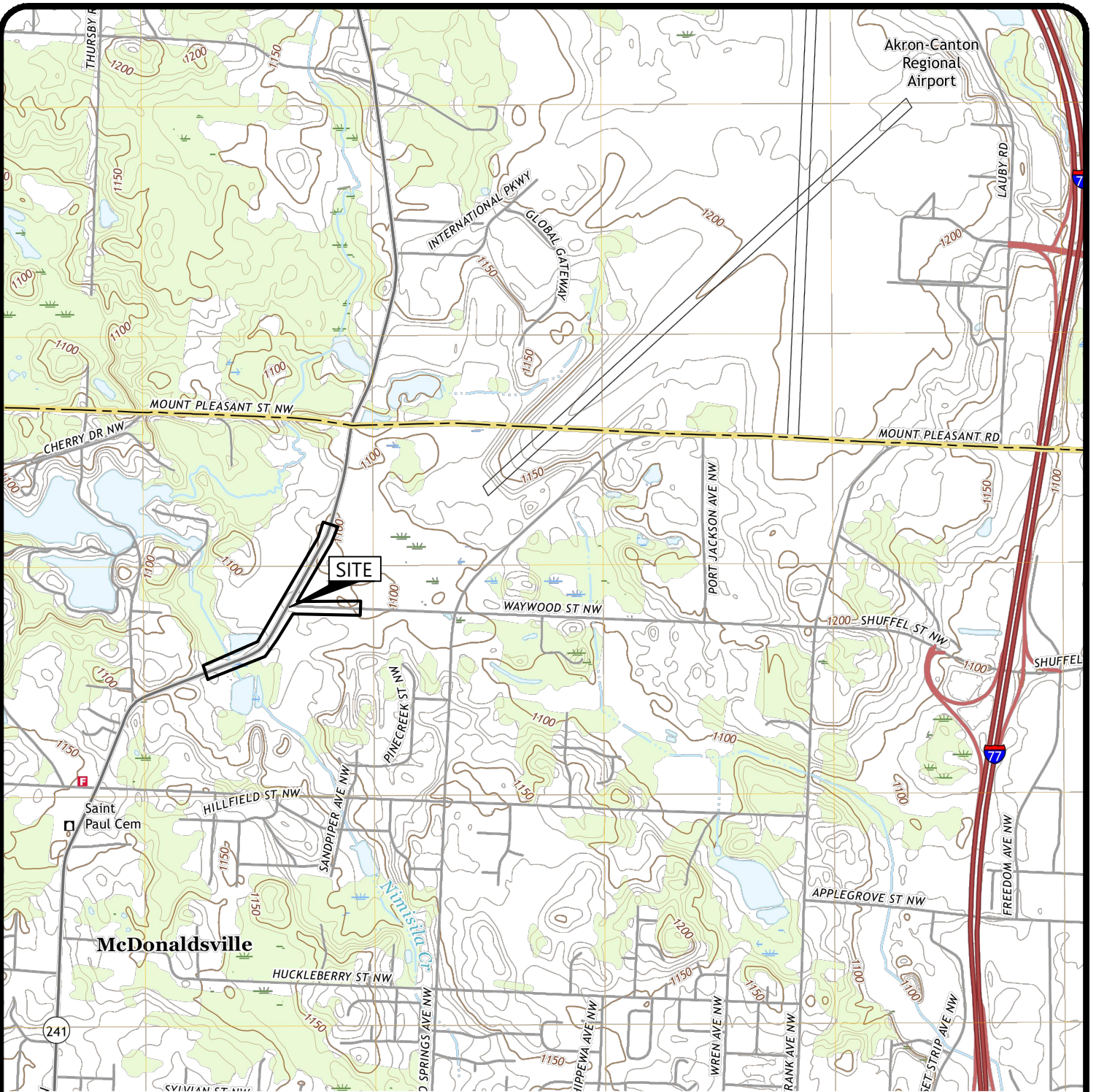
Appendices

Subgrade and Structure Foundation Exploration Report – Final (Rev. #1)
STA-241-17.37 Roundabout and Bridge Replacement (PID 119585)
North Canton, Stark County, Ohio
S&ME Project No. 23170065A/B, Task Order No. 4-01-1



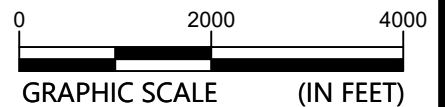
Appendix I – General Project Information

Drawing Path: T:\Columbus-1170\Projects\2023\23170065A_ODOT D4_STA-241-17.37 Roundabout & Bridge-North Canton\GEO\CAD\Construction\Plan of Borings & Vicinity Map.dwg



Project Location
Stark County, Ohio

USGS Mapping:
North Canton USGS Quad



Vicinity Map

Subgrade and Structure Foundation Exploration
STA-241-17.37 Roundabout & Bridge Replacement
North Canton, Stark County, Ohio

SCALE:	GRAPHIC
DATE:	08-14-2024
PROJECT NUMBER	23170065A

FIGURE NO.

1

Drawing Path: T:\Columbus-1170\Projects\2023\23170065A_ODOT D4_STA-241-17.37 Roundabout & Bridge\North Canton\GEO\CAD\Construction\Plan of Borings & Vicinity Map.dwg

MATCHLINE - SEE PLATE 2B



NOTE: AERIAL IMAGERY COURTESY OF OGRIP, OBTAINED SPRING 2017. CPT SOUNDINGS WERE PERFORMED BY ODOT. ALL REMAINING EXPLORATIONS WERE PERFORMED BY S&ME.

	<p>C-001-0-23 CPT SOUNDING NUMBER AND APPROXIMATE LOCATION (ODOT)</p> <p>D/H-001-0-23 DCP SOUNDING AND AUGER SAMPLE NUMBER AND APPROXIMATE LOCATION</p>	<p><u>LEGEND</u></p>	<p>B-001-0-23 BORING NUMBER AND APPROXIMATE LOCATION</p>	<p>0 100 200</p> <p>GRAPHIC SCALE (IN FEET)</p>
--	---	----------------------	---	--

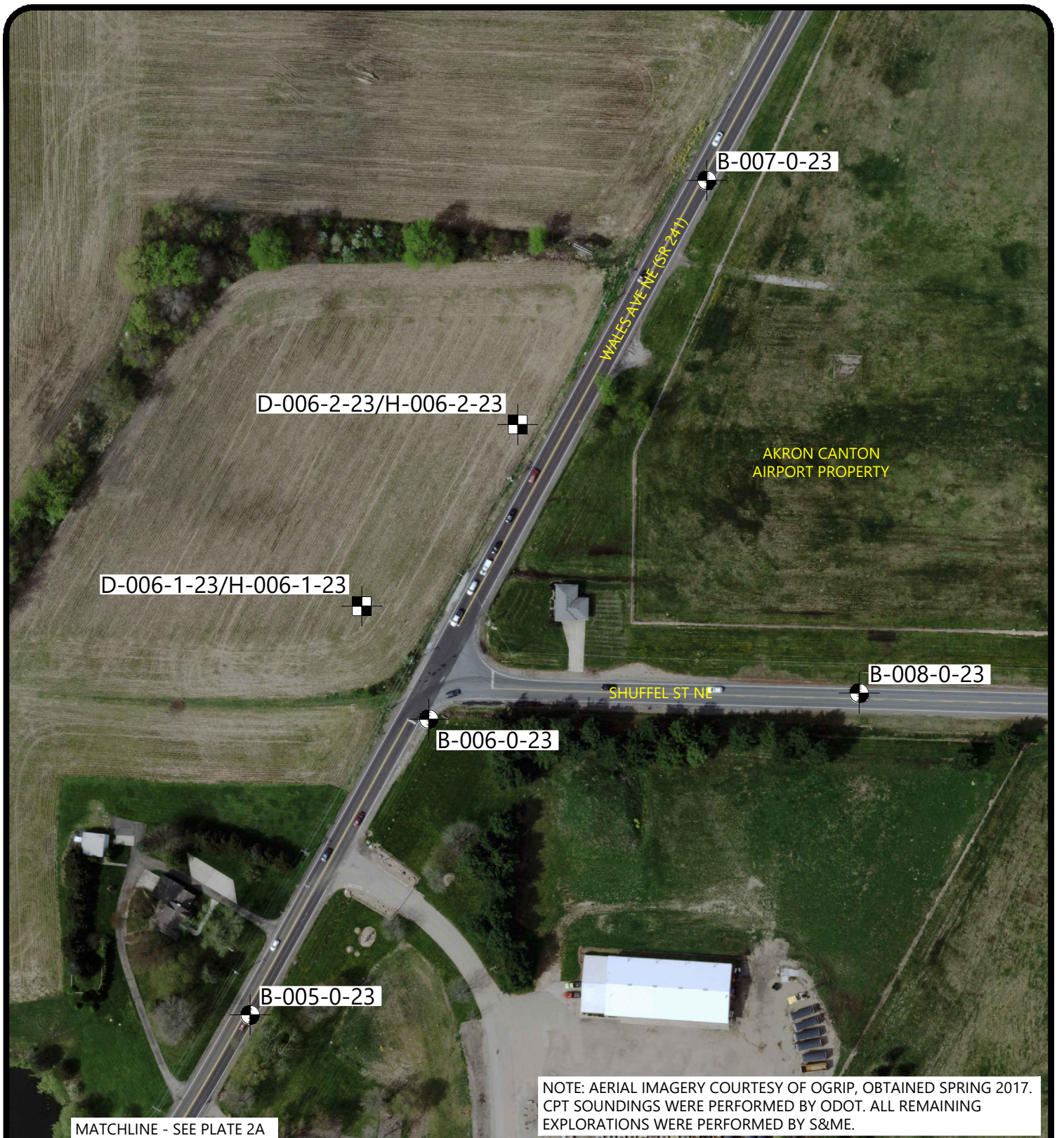


Plan of Borings

Subgrade and Structure Foundation Exploration
 STA-241-17.37 Roundabout & Bridge Replacement
 North Canton, Stark County, Ohio

SCALE:	GRAPHIC
DATE:	08-14-2024
PROJECT NUMBER	23170065A

FIGURE NO.	2A



MATCHLINE - SEE PLATE 2A



C-001-0-23 CPT SOUNDING NUMBER AND APPROXIMATE LOCATION (ODOT)



D/H-001-0-23 DCP SOUNDING AND AUGER SAMPLE NUMBER AND APPROXIMATE LOCATION

LEGEND



B-001-0-23 BORING NUMBER AND APPROXIMATE LOCATION

0 150 300



GRAPHIC SCALE (IN FEET)



Plan of Borings

Subgrade and Structure Foundation Exploration
 STA-241-17.37 Roundabout & Bridge Replacement
 North Canton, Stark County, Ohio

SCALE:

GRAPHIC

DATE:

08-14-2024

PROJECT NUMBER

23170065A

FIGURE NO.

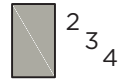
2B

ODOT SOIL LOG

LEGEND



The **STANDARD PENETRATION TEST (SPT)** as defined by AASHTO T206 (or ASTM D1586) is a method to obtain a disturbed soil sample for examination and testing and to obtain relative density and consistency information. A standard 1.4-inch I.D./2-inch O.D. split-barrel sampler is driven three 6-inch increments (see graphic at right) with a 140 lb. hammer freely falling 30 inches. The hammer can either be of a trip, free-fall design, or actuated by a rope and cathead. The SPT N Value is determined by adding the number of blows from the 2nd and 3rd 6-inch increments.



SPT BLOWCOUNT CORRECTION FOR HAMMER EFFICIENCY (N_{60}) is determined by the following equation: $N_{60} = N * [\text{Drill Rod Energy Ratio} (\%) / 60]$, and where the drill rod energy ratio is determined in accordance with ASTM D4633. If the drill rod energy ratio exceeds 90%, it is limited to 90% to determine the N_{60} value and is shown on the log as 90*.

SHELBY TUBE (ST) samples are obtained by hydraulically pushing a thin-walled tube (typically 3-inches in diameter) to obtain a relatively undisturbed sample for testing of fine-grained soils to determine engineering properties such as strength, compressibility, permeability, and density. Shelby tubes are sampled in general accordance with ASTM D1587 (AASHTO T207).



DESCRIPTIVE ORDER OF SOIL STRATA: Consistency/Density, color, ODOT soil classification description, minor soil constituents with percentage modifiers, organic content, miscellaneous constituents or descriptions, relative moisture condition.

ODOT SOIL CLASSIFICATION DESCRIPTION AND SYMBOL

	GRAVEL (A-1-a)		SILT (A-4b)		ORGANIC CLAY (A-8b)
	GRAVEL WITH SAND (A-1-B)		ELASTIC SILT AND CLAY (A-5)		PEAT
	FINE SAND (A-3)		SILT AND CLAY (A-6a)		UNCONTROLLED FILL
	COARSE AND FINE SAND (A-3a)		SILTY CLAY (A-6b)		BOULDERY ZONE
	GRAVEL WITH SAND AND SILT (A-2-4 OR A-2-5)		ELASTIC CLAY (A-7-5)		SOD/ROOTMAT/TOPSOIL
	GRAVEL WITH SAND, SILT AND CLAY (A-2-6 OR A-2-7)		CLAY (A-7-6)		PAVEMENT OR BASE
	SANDY SILT (A-4a)		ORGANIC SILT (A-8a)		CONCRETE

SOIL LOG SYMBOLS

SS - Split-Spoon Sample	Qu - Unconfined Compressive Strength	FS - Fine Sand Content, %
ST - Shelby Tube Sample	γ_d - Dry Unit Weight, pcf	SI - Silt Content, %
TR - Top of Rock	γ_m - Moist Unit Weight, pcf	CL - Clay Content, %
REC - Sample Recovery, %	GR - Gravel Content, %	LL - Liquid Limit
HP - Hand Penetrometer Value, tsf	CS - Coarse Sand Content, %	PL - Plastic Limit
LOI - Loss on Ignition Test, %		PI - Plasticity Index
		WC - Natural Water Content, %

NOTE: Particle size contents are expressed % by weight.

PARTICLE SIZE

Particle	Size	US Sieve Size
Boulder	>300 mm (12 in.)	12 in.
Cobble	75 - 300 mm (3 - 12 in.)	3 - 12 in.
Coarse gravel	19 - 75 mm (3/4 - 3 in.)	3/4 - 3 in.
Fine gravel	2 - 19 mm (0.08 - 3/4 in.)	#10 - 3/4 in.
Coarse sand	0.42 - 2.0 mm	#40 - #10
Fine sand	0.074 - 0.42 mm	#200 - #40
Silt	0.005 - 0.074 mm	NA
Clay	< 0.005 mm	NA

FINE-GRAINED SOIL (Relative Consistency)

	N_{60}	HP
Very soft	< 2 bpf	< 0.25 tsf
Soft	2 - 4 bpf	> 0.25 - 0.5 tsf
Medium stiff	5 - 8 bpf	> 0.5 - 1.0 tsf
Stiff	9 - 15 bpf	> 1.0 - 2.0 tsf
Very stiff	16 - 30 bpf	> 2.0 - 4.0 tsf
Hard	> 30 bpf	> 4.0 tsf

COARSE-GRAINED SOIL (Relative Density)

	N_{60}
Very loose	< 5 bpf
Loose	5 - 10 bpf
Medium dense	11 - 30 bpf
Dense	31 - 50 bpf
Very dense	> 50 bpf

MINOR CONSTITUENTS (% By Weight)

	Percentage
Trace	0% - 10%
Little	>10% - 20%
Some	>20% - 35%
"And"	\geq 35%

ORGANIC CONTENT OF SOIL (Determined by ASTM D2974 or AASHTO T267)

Classification	Percentage
Slightly organic	2% - 4%
Moderately organic	>4% - 10%
Highly organic	> 10%

RELATIVE MOISTURE CONDITION

Dry	Cohesive - Powdery, WC well below PL Granular - No moisture present
Damp	Cohesive - Leaves very little moisture when pressed, WC < PL Granular - Internal moisture, little to no surface moisture
Moist	Cohesive - Leaves moisture when pressed, PL < WC < LL - 3 Granular - Free water on surface, shiny appearance
Wet	Cohesive - Mushy, WC near or above LL Granular - Voids filled with free water

At Time of Drilling

At end of Drilling

24 hrs After Drilling

Free water (seepage or groundwater) observation made anytime during the drilling process. Depending on time of reading and drilling methodologies, this value may be influenced by the drilling process.

Free water measurement soon after the drilling processes are complete, and the borehole is at final depth. Drilling fluids, if introduced during drilling, may influence this measurement.

Free water measurements made in a borehole hours to days after drilling is complete including the time elapsed (i.e., "24 hrs" as shown at left). Depending on subsurface conditions, elapsed time, drilling process, etc. this observation may reflect a stabilized level.

REFERENCES:

Ohio Department of Transportation (ODOT), Specifications for Geotechnical Explorations (SGE)

S&ME ODOT SULFATE (8.5X11) - SGE 01/2019 - OH DOT GDT - 4/2/25 22:27 - R:\SERVICE LINES\CS-2557\CLEVELAND\01 - LABORATORY\02 - GINT\PROJECTS\23170065A.GPJ

S&ME JOB: 23170065A



PROJECT: STA-241-17.37	DRILLING FIRM / OPERATOR: OTB / C. STIVAK	DRILL RIG: OTB MOBILE B-57	STATION / OFFSET: 586+42, 3' RT	EXPLORATION ID B-001-0-23
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: S&ME / P. LEITER	HAMMER: CME AUTOMATIC	ALIGNMENT: SR 241 CL CONST.	
PID: 119585 BR ID: N/A	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 12/12/22	ELEVATION: 1078.0 (MSL) EOB: 15.0 ft.	PAGE 1 OF 1
START: 8/18/23 END: 8/18/23	SAMPLING METHOD: SPT	ENERGY RATIO (%): 90*	COORD: 40.898117 N, 81.471122 W	

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI				
ASPHALT - 8 INCHES	1077.3																		
GRANULAR BASE - 6 INCHES	1076.8	1																	
Very-stiff brown SANDY SILT , little clay, trace fine gravel, damp.	1075.0	2	7	14	78	SS-1	2.0-4.0	8	15	27	39	11	19	14	5	12	A-4a (3)	80	
Medium-stiff reddish-brown SANDY SILT , little fine gravel, trace clay, moist.	1072.0	3	2	5	50	SS-2	0.5	13	19	25	36	7	19	15	4	15	A-4a (2)	-	
Loose reddish-brown COARSE AND FINE SAND , trace to little fine gravel, trace to little clay, moist.	1070.0	4	2	1	5	SS-3	0.5-1.0	-	-	-	-	-	-	-	-	14	A-4a (V)	-	
Very-soft to medium-stiff reddish-brown becoming gray SILT AND CLAY , some fine to coarse sand, trace fine gravel, moist to wet.	1067.5	5	1	3	72	SS-5	0.0-1.0	-	-	-	-	-	-	-	-	29	A-6a (V)	-	
Medium-stiff to stiff brown SANDY SILT , some clay, trace fine gravel, moist to wet.	1063.0	6	2	1	5	SS-6	0.5-1.5	-	-	-	-	-	-	-	-	24	A-4a (V)	-	
- Becoming gray at 13.5'.		7	3	2	8	SS-4	-	-	-	-	-	-	-	-	-	13	A-3a (V)	-	
		8																	
		9	1	1	3	72	SS-5	0.0-1.0	-	-	-	-	-	-	-	29	A-6a (V)	-	
		10																	
		11	1	2	5	78	SS-6	0.5-1.5	-	-	-	-	-	-	-	24	A-4a (V)	-	
		12																	
		13																	
		14	1	2	5	83	SS-7	0.5-1.0	-	-	-	-	-	-	-	19	A-4a (V)	-	
		15																	

NOTES:
 - No seepage or groundwater were noted during drilling.
 - The borehole caved at 6.8' and was "dry" at completion.

NOTES: SEE ABOVE.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: ASPHALT PATCH; SOIL CUTTINGS MIXED WITH BENTONITE



PROJECT: STA-241-17.37	DRILLING FIRM / OPERATOR: OTB / C. STIVAK	DRILL RIG: OTB MOBILE B-57	STATION / OFFSET: 588+17, 5' RT	EXPLORATION ID B-002-0-23
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: S&ME / P. LEITER	HAMMER: CME AUTOMATIC	ALIGNMENT: SR 241 CL CONST.	
PID: 119585 BR ID: 7606273	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 12/12/22	ELEVATION: 1072.8 (MSL) EOB: 100.0 ft.	PAGE 1 OF 4
START: 8/17/23 END: 8/17/23	SAMPLING METHOD: SPT / ST	ENERGY RATIO (%): 90*	COORD: 40.898289 N, 81.470523 W	

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	SO4 ppm	HOLE SEALED
								GR	CS	FS	SI	CL	LL	PL	PI				
ASPHALT - 11 INCHES	1072.8																		
GRANULAR BASE - 3 INCHES	1071.9 1071.6	1																	
FILL: Very-stiff brown and dark-brown SILT AND CLAY , little to some fine to coarse sand, trace fine gravel, damp.	1069.5	2	5	14	78	SS-1	3.5-4.0	-	-	-	-	-	-	-	8	A-6a (V)	-		
Soft to medium-stiff dark-brown, gray, and black SILTY CLAY , little fine to coarse sand, trace fine gravel, slightly organic, few wood fragments, damp to moist. @ SS-3 (5.0' - 6.5'); LOI = 1.1%; ODLL / LL = 0.86	1066.3	3	2	5	61	SS-2	0.5-1.0	4	4	7	66	19	40	24	16	34	A-6b (10)	-	
Stiff gray and brown SILT AND CLAY , trace fine to coarse sand, few wood fibers, damp.	1064.8	4	1	5	78	SS-3	0.25-0.5	2	4	15	60	19	37	20	17	30	A-6b (11)	-	
Very-soft dark-brown and gray SILT AND CLAY , some becoming trace fine to coarse sand, few wood fibers, few sand seams, slightly organic, wet. @ SS-5 (8.0' - 9.5'); LOI = 1.1% @ SS-6 (9.5' - 11.0'); LOI = 1.1%; ODLL / LL = 1.00	1061.8	5	0	2	67	SS-4	1.0-2.0	0	1	4	76	19	33	22	11	26	A-6a (8)	-	
Very-soft gray becoming dark-gray SILT , some clay, trace fine to coarse sand, slightly organic, wet.		6	0	2	67	SS-5	0.0-0.25	5	10	22	48	15	28	17	11	29	A-6a (6)	-	
		7	0	3	94	SS-6	0.0-0.25	1	1	3	71	24	31	20	11	30	A-6a (8)	-	
		8	0	2	100	ST		-	-	-	-	-	-	-	-	-	-	-	
		9	0	1	2	100	SS-7	0.0-0.25	-	-	-	-	-	-	-	-	39	A-4b (V)	-
@ SS-8 (16.0' - 17.5'); LOI = 1.2%		10	0	0	100	SS-8	0.0-0.25	-	-	-	-	-	-	-	-	-	33	A-4b (V)	-
@ ST-9 (17.5' - 19.5'); LOI = 2.7%; ODLL / LL = 0.94		11	0	0	83	ST-9	0.0	0	1	3	64	32	31	21	10	35	A-4b (8)	-	
		12	0	0	100	SS-10	0.0-0.25	-	-	-	-	-	-	-	-	-	33	A-4b (V)	-
Very-soft dark-gray SILT AND CLAY , trace fine sand, slightly organic, moist to wet.	1051.8	13	0	0	100	SS-11	0.0	-	-	-	-	-	-	-	-	-	37	A-6a (V)	-
@ SS-12 (23.5' - 25.0'); LOI = 0.4%		14	0	0	100	SS-12	0.0-0.25	0	0	1	78	21	30	18	12	37	A-6a (9)	-	
		15	0	0	100	SS-13	0.0-0.25	-	-	-	-	-	-	-	-	-	28	A-6a (V)	-
	1044.8	16	0	0	100	SS-14	0.0-0.25	-	-	-	-	-	-	-	-	-	26	A-1-b (V)	-
Very-loose brown and gray GRAVEL WITH SAND , trace to little silt, trace clay, wet.		17	1	3	56			-	-	-	-	-	-	-	-	-			

S&ME ODOT SULFATE (8.5X1.1) - SGE 01/2019 - OH DOT GDT - 4/2/25 22:27 - R:\SERVICE LINES\CS-2557\CLEVELAND\01 - LABORATORY\02 - GINT\PROJECTS\23170065A.GPJ

PLATE 5



PID: 119585		BR ID: 7606273		PROJECT: STA-241-17.37		STATION / OFFSET: 588+17, 5' RT		START: 8/17/23		END: 8/17/23		PG 2 OF 4		B-002-0-23										
MATERIAL DESCRIPTION AND NOTES			ELEV.	DEPTHS	SPT/RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	SO4 ppm	HOLE SEALED			
										GR	CS	FS	SI	CL	LL	PL	PI							
Very-loose brown and gray GRAVEL WITH SAND , trace to little silt, trace clay, wet. (continued)			1042.8	31																				
Stiff brown SILT , some fine to coarse sand, trace clay, trace fine gravel, moist to wet.			1040.8	32																				
Loose brown and gray FINE SAND , some coarse sand, little fine gravel, trace silt, trace clay, wet.			1038.5	33																				
				34	4	11	67	SS-15A	1.0-2.0	10	13	21	52	4	18	15	3	16	A-4b (4)	-				
				35	3			SS-15B	-	-	-	-	-	-	-	-	15	A-3 (V)	-					
				36																				
				37																				
				38																				
				39	2	6	56	SS-16	-	14	21	57	8	0	NP	NP	NP	21	A-3 (0)	-				
				40	2	2																		
				41																				
				42																				
				43																				
				44	1	5	61	SS-17	-	-	-	-	-	-	-	-	-	-	28	A-3 (V)	-			
				45	2																			
				46																				
				47																				
				48																				
				49	2	3	83	SS-18	-	-	-	-	-	-	-	-	-	24	A-3 (V)	-				
				50	1	1																		
				51																				
				52																				
				53																				
				54	3	9	67	SS-19	-	-	-	-	-	-	-	-	-	-	21	A-3 (V)	-			
				55	3																			
				56																				
Stiff gray SANDY SILT , little clay, trace fine gravel, moist.			1015.8	57																				
				58																				
				59	3	20	50	SS-20	1.0-2.0	-	-	-	-	-	-	-	-	-	24	A-4a (V)	-			
				60	5																			
				61	8																			



PID: 119585		BR ID: 7606273		PROJECT: STA-241-17.37		STATION / OFFSET: 588+17, 5' RT		START: 8/17/23		END: 8/17/23		PG 3 OF 4		B-002-0-23													
MATERIAL DESCRIPTION AND NOTES			ELEV.	DEPTHS	SPT/RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	SO4 ppm	HOLE SEALED						
										GR	CS	FS	SI	CL	LL	PL	PI										
Medium-dense gray GRAVEL WITH SAND , little silt, trace clay, wet.			1010.3	63																							
			64	5	14	89	SS-21	-	40	13	27	19	1	NP	NP	NP	13	A-1-b (0)	-								
			65	4																							
			66																								
			67																								
Loose to medium-dense gray SILT , little clay, wet.			1005.8	68																							
			69	3	8	72	SS-22	-	-	-	-	-	-	-	-	-	23	A-4b (V)	-								
			70	2																							
			71																								
			72																								
			73																								
			74	4	12	100	SS-23	-	-	-	-	-	-	-	-	-	31	A-4b (V)	-								
			75	4																							
			76																								
			77																								
Medium-dense gray SILT , trace clay, wet.			985.8	78																							
			79	4	12	100	SS-24	-	0	0	0	85	15	20	17	3	26	A-4b (8)	-								
			80	4																							
			81																								
			82																								
			83																								
			84	3	9	100	SS-25	-	-	-	-	-	-	-	-	-	25	A-4b (V)	-								
			85	2																							
			86	4																							
			87																								
				88																							
			89	4	20	100	SS-26	-	0	0	0	93	7	25	18	7	28	A-4b (8)	-								
			90	7																							
			91	6																							
			92																								
			93																								
			94	4																							



PID: 119585	BR ID: 7606273	PROJECT: STA-241-17.37	STATION / OFFSET: 588+17, 5' RT	START: 8/17/23	END: 8/17/23	PG 4 OF 4	B-002-0-23
-------------	----------------	------------------------	---------------------------------	----------------	--------------	-----------	-------------------

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	SO4 ppm	HOLE SEALED		
								GR	CS	FS	SI	CL	LL	PL	PI						
Medium-dense gray SILT , trace clay, wet. <i>(continued)</i>	978.6		4	15	100	SS-27	-	-	-	-	-	-	-	-	-	-	25	A-4b (V)	-		
		95																			
		96																			
		97																			
		98																			
	973.0		3	7	29	100	SS-28A	-	0	1	8	87	4	21	15	6	24	A-4b (8)	-		
Medium-dense reddish-brown GRAVEL WITH SAND , little silt, trace clay, few cobbles, damp to moist.	972.8	EOB	12			SS-28B	-	-	-	-	-	-	-	-	-	-	13	A-1-b (V)	-		

NOTES:
 - Seepage noted at 3.1'.
 - Introduced bentonite into HSA beginning at 5.0'. Maintained column of bentonite slurry inside HSA throughout entire boring.
 - Encountered 18" of sand heave at 43.5'.
 - Boring advanced to 75.5' on 8/17/23. Boring completed on 8/18/23.

NOTES: SEE ABOVE.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: ASPHALT PATCH; BENTONITE AND CEMENT GROUT MIXTURE

S&ME ODOT SULFATE (8.5X11) - SGE 01/2019 - OH DOT GDT - 4/2/25 22:28 - R:\SERVICE LINES\CS-2557\CLEVELAND\01 - LABORATORY\02 - GINT\PROJECTS\23170065A.GPJ

S&ME JOB: 23170065A



PROJECT: STA-241-17.37	DRILLING FIRM / OPERATOR: OTB / C. STIVAK	DRILL RIG: OTB MOBILE B-57	STATION / OFFSET: 588+70, 6' RT	EXPLORATION ID
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: S&ME / P. LEITER	HAMMER: CME AUTOMATIC	ALIGNMENT: SR 241 CL CONST.	B-003-0-23
PID: 119585 BR ID: 7606273	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 12/12/22	ELEVATION: 1071.7 (MSL) EOB: 100.0 ft.	PAGE
START: 8/16/23 END: 8/30/23	SAMPLING METHOD: SPT	ENERGY RATIO (%): 90*	COORD: 40.898339 N, 81.470347 W	1 OF 4

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	SO4 ppm	HOLE SEALED
								GR	CS	FS	SI	CL	LL	PL	PI				
ASPHALT - 13 INCHES	1071.7																		
GRANULAR BASE - 6 INCHES	1070.6	1																	
FILL: Very-stiff to hard dark-gray SANDY SILT , trace fine gravel, trace clay, damp.	1070.1	2																	
Very-soft gray and black SANDY SILT , little clay, trace fine gravel, few wood fibers, slightly organic, moist. @ SS-3 (3.5' - 5.0'); LOI = 1.2%	1068.2	3	7	5	12	83	SS-1	2.0-4.5	9	13	26	42	10	21	15	6	13	A-4a (3)	260
Very-soft gray and black SANDY SILT , little clay, trace fine gravel, few wood fibers, slightly organic, moist. @ SS-3 (3.5' - 5.0'); LOI = 1.2%	1066.7	4	0	1	3	78	SS-2	0.0-0.25	2	11	26	48	13	23	15	8	17	A-4a (5)	-
Very-soft to soft dark-gray and black ORGANIC CLAY , "and" silt, "and" fine to coarse sand, trace fine gravel, few wood fibers, moist to wet. @ SS-3 (5.0' - 6.5'); LOI = 2.0%; ODLL / LL = 0.72	1065.2	5	0	1	3	67	SS-3	0.0-0.5	2	13	27	42	16	47	39	8	72	A-8b (5)	-
Very-soft gray SILT , some clay, trace fine to coarse sand, wet. - ST-29 from offset boring from 8.0' to 10.0' with 100% recovery. LOI = 2.1%; ODLL / LL = 0.89.	1060.7	6	0	2	3	67	SS-3	0.0-0.5	2	13	27	42	16	47	39	8	72	A-8b (5)	-
Very-soft gray SILT AND CLAY , slightly organic, wet.	1060.7	7	5	1	3	83	SS-4	0.0-0.25	-	-	-	-	-	-	-	-	29	A-4b (V)	-
Very-soft gray SILT , some clay, trace fine to coarse sand, wet. - ST-29 from offset boring from 8.0' to 10.0' with 100% recovery. LOI = 2.1%; ODLL / LL = 0.89.	1060.7	8	1	1	3	83	SS-4	0.0-0.25	-	-	-	-	-	-	-	-	29	A-4b (V)	-
Very-soft gray SILT AND CLAY , slightly organic, wet.	1060.7	9	0	0	0	78	SS-5	0.0	-	-	-	-	-	-	-	-	29	A-4b (V)	-
Very-soft gray SILT AND CLAY , slightly organic, wet.	1060.7	10	0	0	0	100	ST-29	0.0-0.25	0	2	3	67	28	28	22	6	28	A-4b (8)	-
Very-soft gray SILT AND CLAY , slightly organic, wet.	1060.7	11	0	0	0	100	SS-6	0.0	-	-	-	-	-	-	-	-	26	A-4b (V)	-
Very-soft gray SILT AND CLAY , slightly organic, wet.	1060.7	12	0	0	0	100	SS-7	0.0	-	-	-	-	-	-	-	-	36	A-6a (V)	-
Very-soft gray SILT AND CLAY , slightly organic, wet.	1060.7	13	0	0	0	100	SS-7	0.0	-	-	-	-	-	-	-	-	36	A-6a (V)	-
@ SS-8 (13.5' - 15.0'); LOI = 0.9%	1056.2	14	0	0	0	100	SS-8	0.0	0	0	0	71	29	35	23	12	37	A-6a (9)	-
Very-soft gray SILT , some clay, trace fine to coarse sand, wet. - ST-30 from offset boring (16.0' to 18.0'); LOI = 1.6%; ODLL / LL = 0.81	1053.7	15	0	0	0	100	SS-8	0.0	0	0	0	71	29	35	23	12	37	A-6a (9)	-
Very-soft gray SILT AND CLAY , trace fine to coarse sand, slightly organic, wet.	1053.7	16	0	0	0	100	SS-9	0.0-0.25	-	-	-	-	-	-	-	-	33	A-4b (V)	-
Very-soft gray SILT AND CLAY , trace fine to coarse sand, slightly organic, wet.	1053.7	17	0	0	0	100	SS-9	0.0-0.25	-	-	-	-	-	-	-	-	33	A-4b (V)	-
Very-soft gray SILT AND CLAY , trace fine to coarse sand, slightly organic, wet.	1053.7	18	0	0	0	100	ST-30	0.0-0.25	0	2	3	66	29	31	21	10	34	A-4b (8)	-
Very-soft gray SILT AND CLAY , trace fine to coarse sand, slightly organic, wet.	1053.7	19	0	0	0	100	SS-10	0.0	-	-	-	-	-	-	-	-	32	A-6a (V)	-
Very-soft gray SILT AND CLAY , trace fine to coarse sand, slightly organic, wet.	1053.7	20	0	0	0	100	SS-10	0.0	-	-	-	-	-	-	-	-	32	A-6a (V)	-
@ SS-11 (21.0' - 22.5'); LOI = 0.9%	1048.7	21	0	0	0	100	SS-11	0.0	0	0	1	71	28	33	20	13	33	A-6a (9)	-
Very-loose to loose brown and gray GRAVEL WITH SAND , little silt, trace clay, wet.	1048.7	22	0	0	0	100	SS-11	0.0	0	0	1	71	28	33	20	13	33	A-6a (9)	-
Very-loose to loose brown and gray GRAVEL WITH SAND , little silt, trace clay, wet.	1048.7	23	1	1	5	56	SS-12	-	-	-	-	-	-	-	-	-	16	A-1-b (V)	-
Very-loose to loose brown and gray GRAVEL WITH SAND , little silt, trace clay, wet.	1048.7	24	1	2	5	56	SS-12	-	-	-	-	-	-	-	-	-	16	A-1-b (V)	-
Very-loose to loose brown and gray GRAVEL WITH SAND , little silt, trace clay, wet.	1048.7	25	2	1	3	39	SS-13	-	-	-	-	-	-	-	-	-	20	A-1-b (V)	-
Very-loose to loose brown and gray GRAVEL WITH SAND , little silt, trace clay, wet.	1048.7	26	2	1	3	39	SS-13	-	-	-	-	-	-	-	-	-	20	A-1-b (V)	-
Very-loose to loose brown and gray GRAVEL WITH SAND , little silt, trace clay, wet.	1048.7	27	1	1	5	61	SS-14	-	19	34	33	13	1	NP	NP	NP	17	A-1-b (0)	-
Very-loose to loose brown and gray GRAVEL WITH SAND , little silt, trace clay, wet.	1048.7	28	1	1	5	61	SS-14	-	19	34	33	13	1	NP	NP	NP	17	A-1-b (0)	-
Very-loose to loose brown and gray GRAVEL WITH SAND , little silt, trace clay, wet.	1048.7	29	1	2	5	61	SS-14	-	19	34	33	13	1	NP	NP	NP	17	A-1-b (0)	-

PLATE 9



PID: 119585		BR ID: 7606273		PROJECT: STA-241-17.37		STATION / OFFSET: 588+70, 6' RT		START: 8/16/23		END: 8/30/23		PG 2 OF 4		B-003-0-23													
MATERIAL DESCRIPTION AND NOTES			ELEV.	DEPTHS	SPT/RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	SO ₄ ppm	HOLE SEALED						
										GR	CS	FS	SI	CL	LL	PL	PI										
Very-loose to loose brown and gray GRAVEL WITH SAND , little silt, trace clay, wet. (continued)			1041.7	31																							
			32																								
			33																								
			34	1	2	5	72	SS-15	-	-	-	-	-	-	-	-	-	-	-	22	A-1-b (V)	-					
			35		1																						
			36																								
			37																								
			38																								
			39	1	2	6	72	SS-16	-	-	-	-	-	-	-	-	-	-	-	22	A-1-b (V)	-					
			40		2																						
Loose brown and gray SANDY SILT , trace fine gravel, trace clay, wet.			1029.7	41																							
			42																								
			43																								
			44	1	1	5	100	SS-17	-	-	-	-	-	-	-	-	-	-	-	26	A-4a (V)	-					
			45		2																						
			46																								
			47																								
			48																								
			49	2	2	6	100	SS-18	-	8	8	45	37	2	NP	NP	NP		20	A-4a (1)	-						
			50		2																						
Loose to medium-dense brown COARSE AND FINE SAND , little silt, trace fine gravel, trace clay, wet.			1014.7	51																							
			52																								
			53																								
			54	4	2	6	100	SS-19	-	-	-	-	-	-	-	-	-	-	-	20	A-4a (V)	-					
			55		2																						
			56																								
			57																								
			58																								
			59	3	5	14	78	SS-20	-	-	-	-	-	-	-	-	-	-	-	17	A-3a (V)	-					
			60		4																						
61																											

S&ME ODOT SULFATE (8.5X11) - SGE 01/2019 - OH DOT GDT - 4/2/25 22:28 - R:\SERVICE LINES\CS-2557\CLEVELAND\01 - LABORATORY\02 - GINT\PROJECTS\23170065A.GPJ

PLATE 10



PID: 119585		BR ID: 7606273		PROJECT: STA-241-17.37		STATION / OFFSET: 588+70, 6' RT		START: 8/16/23		END: 8/30/23		PG 3 OF 4		B-003-0-23									
MATERIAL DESCRIPTION AND NOTES			ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	SO4 ppm	HOLE SEALED		
										GR	CS	FS	SI	CL	LL	PL	PI						
Loose to medium-dense brown COARSE AND FINE SAND , little silt, trace fine gravel, trace clay, wet. <i>(continued)</i>			1009.6	63	2	6	100	SS-21	-	5	26	56	12	1	NP	NP	NP	18	A-3a (0)	-			
																						2	2
Medium-dense gray SILT , little clay, wet.			1004.7	64	4	23	78	SS-22	-	-	-	-	-	-	-	-	-	23	A-4b (V)	-			
																						7	8
																						65	
																						66	
																						67	
																						68	
																						69	
																						70	
																						71	
																						72	
Medium-dense gray FINE SAND , little coarse sand, trace silt, trace clay, wet.			982.8	74	3	15	78	SS-23	-	0	0	0	82	18	24	22	2	27	A-4b (8)	-			
																						5	5
																						73	
																						74	
																						75	
																						76	
																						77	
																						78	
																						79	
																						80	
Very-loose to medium-dense gray GRAVEL WITH SAND , trace silt, trace clay, few pockets of silt, wet.			979.7	79	4	15	78	SS-24	-	-	-	-	-	-	-	-	-	36	A-4b (V)	-			
																						4	6
																						81	
																						82	
																						83	
																						84	
																						85	
																						86	
																						87	
																						88	
			982.8	89	4	14	78	SS-26A	-	-	-	-	-	-	-	-	23	A-4b (V)	-				
								SS-26B										28			A-3 (V)		
			979.7	90	5																		
			979.7	91																			
			979.7	92																			
			979.7	93																			
			979.7	94	2																		



PID: 119585	BR ID: 7606273	PROJECT: STA-241-17.37	STATION / OFFSET: 588+70, 6' RT	START: 8/16/23	END: 8/30/23	PG 4 OF 4	B-003-0-23													
MATERIAL DESCRIPTION AND NOTES		ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (G)	SO4 ppm	HOLE SEALED
									GR	CS	FS	SI	CL	LL	PL	PI				
Very-loose to medium-dense gray GRAVEL WITH SAND , trace silt, trace clay, few pockets of silt, wet. <i>(continued)</i>		977.5		1	3	89	SS-27	-	41	40	9	9	1	NP	NP	NP	12	A-1-b (0)	-	
		971.7	EOB	3	11	72	SS-28	-	-	-	-	-	-	-	-	-	11	A-1-b (V)	-	

NOTES:
 - Groundwater noted at 7.0'.
 - Approx. 8' of sand heave occurred at 23.5'. Introduced bentonite into HSA.
 - Original borehole abandoned due to excessive heave in augers below 65'.
 - Completed boring from offset boring located 4 feet east of original borehole on 8/28/23.
 - Obtained Shelby tubes from depths of 8'-10' (ST-29) and 16'-18' (ST-30) in the offset boring.

NOTES: SEE ABOVE.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: ASPHALT PATCH; BENTONITE AND CEMENT GROUT MIXTURE



PROJECT: STA-241-17.37	DRILLING FIRM / OPERATOR: OTB / C. STIVAK	DRILL RIG: OTB MOBILE B-57	STATION / OFFSET: 592+49, 5' RT	EXPLORATION ID: B-004-0-23
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: S&ME / S. SMITH	HAMMER: CME AUTOMATIC	ALIGNMENT: SR 241 CL CONST.	
PID: 119585 BR ID: N/A	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 12/12/22	ELEVATION: 1074.6 (MSL) EOB: 25.0 ft.	PAGE: 1 OF 1
START: 8/30/23 END: 8/30/23	SAMPLING METHOD: SPT	ENERGY RATIO (%): 90*	COORD: 40.898751 N, 81.469092 W	

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI				
ASPHALT - 14 INCHES	1074.6																		
BRICK - 4 INCHES	1073.4	1																	
SANDY UNDERLAYMENT - 2 INCHES	1073.1																		
COBBLES INTERMIXED WITH CLAY - 10 INCHES	1072.9	2																	
FILL: Medium-dense brown GRAVEL WITH SAND, some silt, trace clay, damp.	1072.1																		
POSSIBLE FILL: Medium-stiff dark-brown SILT AND CLAY, little fine to coarse sand, trace fine gravel, damp.	1070.6	3	7	11	29	67	SS-1	-	30	26	21	21	2	NP	NP	NP	8	A-1-b (0)	320
Soft to medium-stiff dark-gray to black SILTY CLAY, "and" fine to coarse sand, trace fine gravel, moist	1070.1	4	2	1	5	67	SS-2A	0.5-1.0	-	-	-	-	-	-	-	-	21	A-6a (V)	-
Stiff becoming medium-stiff brown and gray SILT AND CLAY, trace fine to coarse sand, few wood fibers and roots, moist.	1068.6	5	0	2	100	100	SS-2B	0.5	3	9	27	43	18	40	22	18	31	A-6b (8)	-
		6	0	1	2	100	SS-3A	0.5-0.8	-	-	-	-	-	-	-	-	18	A-6b (V)	-
		7	1	1	2	100	SS-3B	1.0-1.3	-	-	-	-	-	-	-	-	20	A-6a (V)	-
		8	2	2	6	78	SS-4	1.0-1.5	-	-	-	-	-	-	-	-	24	A-6a (V)	-
		9	1	1	3	100	SS-5A	0.5-0.8	-	-	-	-	-	-	-	-	25	A-6a (V)	-
		10	1	1	3	100	SS-5B	0.5-0.8	-	-	-	-	-	-	-	-	25	A-6a (V)	-
Very-soft to soft gray mottled with brown SILT AND CLAY, trace fine to coarse sand, moist to wet.	1064.1	11	0	0	0	89	SS-6	0.0-0.3	-	-	-	-	-	-	-	-	34	A-6a (V)	-
		12	0	0	0	89	SS-6	0.0-0.3	-	-	-	-	-	-	-	-	34	A-6a (V)	-
		13	0	0	0	89	SS-6	0.0-0.3	-	-	-	-	-	-	-	-	34	A-6a (V)	-
		14	0	0	2	28	SS-7	0.0-0.3	-	-	-	-	-	-	-	-	34	A-6a (V)	-
		15	0	1	2	28	SS-7	0.0-0.3	-	-	-	-	-	-	-	-	34	A-6a (V)	-
		16	0	1	2	28	SS-7	0.0-0.3	-	-	-	-	-	-	-	-	34	A-6a (V)	-
		17	0	1	2	28	SS-7	0.0-0.3	-	-	-	-	-	-	-	-	34	A-6a (V)	-
		18	0	1	2	28	SS-7	0.0-0.3	-	-	-	-	-	-	-	-	34	A-6a (V)	-
- Becoming gray at 18.5'. - Fine sand seam at 18.9'.		19	0	1	5	100	SS-8	0.0-0.3	-	-	-	-	-	-	-	-	28	A-6a (V)	-
		20	1	2	5	100	SS-8	0.0-0.3	-	-	-	-	-	-	-	-	28	A-6a (V)	-
		21	1	2	5	100	SS-8	0.0-0.3	-	-	-	-	-	-	-	-	28	A-6a (V)	-
		22	1	2	5	100	SS-8	0.0-0.3	-	-	-	-	-	-	-	-	28	A-6a (V)	-
		23	1	2	5	100	SS-8	0.0-0.3	-	-	-	-	-	-	-	-	28	A-6a (V)	-
		24	0	1	3	100	SS-9	0.0-0.5	-	-	-	-	-	-	-	-	29	A-6a (V)	-
		25	1	1	3	100	SS-9	0.0-0.5	-	-	-	-	-	-	-	-	29	A-6a (V)	-

NOTES:

- Seepage was noted at 5.7'.
- Groundwater was encountered at 18.9'.
- Groundwater was measured at 11.5' inside the augers after drilling.
- The borehole remained open after augers were removed.

NOTES: SEE ABOVE.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: ASPHALT PATCH; PLASTIC HOLE PLUG DEVICE; SOIL CUTTINGS MIXED WITH BENTONITE

S&ME ODOT SULFATE (8.5X11) - SGE 01/2019 - OH DOT GDT - 4/22/25 22:28 - R:\SERVICE LINES\CS-2557\CLEVELAND\01 - LABORATORY\02 - GINT\PROJECTS\23170065A.GPJ



PROJECT: <u>STA-241-17.37</u>	DRILLING FIRM / OPERATOR: <u>OTB / C. STIVAK</u>	DRILL RIG: <u>OTB MOBILE B-57</u>	STATION / OFFSET: <u>596+47, 2' RT</u>	EXPLORATION ID B-005-0-23
TYPE: <u>ROADWAY</u>	SAMPLING FIRM / LOGGER: <u>S&ME / S. SMITH</u>	HAMMER: <u>CME AUTOMATIC</u>	ALIGNMENT: <u>SR 241 CL CONST.</u>	
PID: <u>119585</u> BR ID: <u>N/A</u>	DRILLING METHOD: <u>3.25" HSA</u>	CALIBRATION DATE: <u>12/12/22</u>	ELEVATION: <u>1086.9 (MSL)</u> EOB: <u>7.5 ft.</u>	PAGE 1 OF 1
START: <u>8/30/23</u> END: <u>8/30/23</u>	SAMPLING METHOD: <u>SPT</u>	ENERGY RATIO (%): <u>90*</u>	COORD: <u>40.899587 N, 81.468180 W</u>	

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI				
ASPHALT - 14½ INCHES	1086.9																		
GRANULAR BASE - 5 INCHES	1085.7	1																	
Very-stiff brown CLAY , "and" silt, trace fine to coarse sand, damp.	1085.3	2	2																
	1083.9	3	3	9	78	SS-1	2.2-4.0	0	1	2	61	36	45	21	24	22	A-7-6 (15)	140	
Medium-stiff to stiff brown and reddish-brown SILTY CLAY , trace fine to coarse sand, damp to moist.		4	2																
		5	2	8	72	SS-2	1.2-2.0	0	1	1	62	36	38	19	19	25	A-6b (12)	-	
- Becoming very soft to soft, brown and red at 6.0'.		6	2																
		7	2	6	78	SS-3	0.7-1.5	-	-	-	-	-	-	-	-	24	A-6b (V)	-	
	1079.4	7	1	3	100	SS-4	0.0-0.5	-	-	-	-	-	-	-	-	24	A-6b (V)	-	

NOTES:
 - No seepage or groundwater noted.
 - The borehole remained open and was "dry" after removing augers.

NOTES: SEE ABOVE.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: ASPHALT PATCH; SOIL CUTTINGS MIXED WITH BENTONITE



PROJECT: STA-241-17.37	DRILLING FIRM / OPERATOR: OTB / C. STIVAK	DRILL RIG: OTB MOBILE B-57	STATION / OFFSET: 200+01, 16' RT	EXPLORATION ID B-006-0-23
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: S&ME / S. SMITH	HAMMER: CME AUTOMATIC	ALIGNMENT: SR 241 CL R/W	
PID: 119585 BR ID: N/A	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 12/12/22	ELEVATION: 1091.6 (MSL) EOB: 7.5 ft.	PAGE 1 OF 1
START: 8/30/23 END: 8/30/23	SAMPLING METHOD: SPT	ENERGY RATIO (%): 90*	COORD: 40.900411 N, 81.467500 W	

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI				
TOPSOIL - 4 INCHES	1091.3																		
FILL: Medium-dense brown SANDY SILT, trace fine gravel, trace clay, damp to wet.			2																
			3	12	56	SS-1	-	9	18	37	28	8	21	17	4	14	A-4a (0)	480	
	1088.3	W 1088.3	5																
POSSIBLE FILL: Medium-dense brown and red GRAVEL WITH SAND AND SILT, trace clay, damp to wet.			2			SS-2A	-	-	-	-	-	-	-	-	-	23	A-4a (V)	-	
			3	11	67	SS-2B	-	-	-	-	-	-	-	-	-	10	A-2-4 (V)	-	
			4																
			2																
	1085.1		6	12	56	SS-3	-	33	20	15	28	4	17	16	1	12	A-2-4 (0)	-	
			5																
	1084.1	EOB	5	17	94	SS-4A	-	-	-	-	-	-	-	-	-	21	A-2-4 (V)	-	
Medium-dense brown COARSE AND FINE SAND, little to some silt, trace to little fine gravel, trace clay, damp.			6			SS-4B	-	-	-	-	-	-	-	-	-	8	A-3a (V)	-	
			6																

NOTES:
 - Seepage was noted at 3.3'.
 - The borehole remained open and was "dry" after removing the augers.

NOTES: SEE ABOVE.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: ASPHALT PATCH; SOIL CUTTINGS MIXED WITH BENTONITE



PROJECT: <u>STA-241-17.37</u>	DRILLING FIRM / OPERATOR: <u>S&ME / P. LEITER</u>	DRILL RIG: <u>HAND AUGER</u>	STATION / OFFSET: <u>200+97, 137' LT</u>	EXPLORATION ID H-006-1-23
TYPE: <u>ROADWAY</u>	SAMPLING FIRM / LOGGER: <u>S&ME / P. LEITER</u>	HAMMER: <u>HAND SAMPLING</u>	ALIGNMENT: <u>SR 241 CL R/W</u>	
PID: <u>119585</u> BR ID: <u>N/A</u>	DRILLING METHOD: <u>3.25" HSA</u>	CALIBRATION DATE: <u>N/A</u>	ELEVATION: <u>1089.4 (MSL)</u> EOB: <u>1.8 ft.</u>	PAGE 1 OF 1
START: <u>7/19/24</u> END: <u>7/19/24</u>	SAMPLING METHOD: <u>HAND AUGER</u>	ENERGY RATIO (%): <u>9</u>	COORD: <u>40.900845 N, 81.467811 W</u>	

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI				
TOPSOIL - 1 INCH Orangish-brown SANDY SILT , little clay, trace fine gravel, damp.	1089.4 1087.6	1 EOB				HA-1		3	2	43	35	17	19	14	5	8	A-4a (3)	40	< 7 4 7 7

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: SOIL CUTTINGS



PROJECT: <u>STA-241-17.37</u>	DRILLING FIRM / OPERATOR: <u>S&ME / P. LEITER</u>	DRILL RIG: <u>HAND AUGER</u>	STATION / OFFSET: <u>203+68, 54' LT</u>	EXPLORATION ID H-006-2-23
TYPE: <u>ROADWAY</u>	SAMPLING FIRM / LOGGER: <u>S&ME / P. LEITER</u>	HAMMER: <u>HAND SAMPLING</u>	ALIGNMENT: <u>SR 241 CL R/W</u>	
PID: <u>119585</u> BR ID: <u>N/A</u>	DRILLING METHOD: <u>3.25" HSA</u>	CALIBRATION DATE: <u>N/A</u>	ELEVATION: <u>1090.9 (MSL)</u> EOB: <u>2.0 ft.</u>	PAGE 1 OF 1
START: <u>7/19/24</u> END: <u>7/19/24</u>	SAMPLING METHOD: <u>HAND AUGER</u>	ENERGY RATIO (%): <u>9</u>	COORD: <u>40.901385 N, 81.467073 W</u>	

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI				
TOPSOIL - 1 INCH Orangish-brown SILT AND CLAY , some fine to coarse sand, trace fine gravel, damp.	1090.8	1																<7 >7 <7 >7	
	1088.9	EOB - 2				HA-1		1	2	25	44	28	32	18	14	15	A-6a (9)	40	<7 >7 <7 >7

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: SOIL CUTTINGS



PROJECT: STA-241-17.37	DRILLING FIRM / OPERATOR: OTB / C. STIVAK	DRILL RIG: OTB MOBILE B-57	STATION / OFFSET: 207+14, 11' LT	EXPLORATION ID B-007-0-23
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: S&ME / S. SMITH	HAMMER: CME AUTOMATIC	ALIGNMENT: SR 241 CL R/W	
PID: 119585 BR ID: N/A	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 12/12/22	ELEVATION: 1096.2 (MSL) EOB: 8.0 ft.	PAGE 1 OF 1
START: 8/30/23 END: 8/30/23	SAMPLING METHOD: SPT	ENERGY RATIO (%): 90*	COORD: 40.902155 N, 81.466327 W	

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI				
ASPHALT - 16 INCHES	1096.2																		
GRANULAR BASE - 6 INCHES	1094.8	1																	
	1094.3																		
FILL: Stiff brown and red SANDY SILT , little fine gravel, little clay, few concrete fragments, damp.	1093.6	2	4	5	29	67	SS-1A	-	13	25	15	33	14	27	18	9	15	A-4a (2)	300
PROBABLE FILL: Medium-dense brown COARSE AND FINE SAND , some silt, trace fine gravel, trace clay, damp.		3	4	10	26	100	SS-1B	-	-	-	-	-	-	-	-	-	7	A-3a (V)	-
	1091.2	4	4	10	26	100	SS-2	-	7	16	43	33	1	NP	NP	NP	6	A-3a (0)	-
Medium-stiff brown SANDY SILT , little to some clay, trace fine gravel, damp.	1090.6	5	1	1	5	72	SS-3A	0.7-1.5	-	-	-	-	-	-	-	-	14	A-4a (V)	-
Loose brown COARSE AND FINE SAND , little silt, trace fine gravel, trace clay, damp.		6	1	1	5	72	SS-3B	-	-	-	-	-	-	-	-	-	13	A-3a (V)	-
	1088.2	7	3	3	9	94	SS-4	-	-	-	-	-	-	-	-	-	14	A-3a (V)	-
		8	3	3															

NOTES:

- Seepage and groundwater were not noted during drilling.
- The borehole caved at 5.0' and was "dry" after removing the augers.

NOTES: SEE ABOVE.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: ASPHALT PATCH; SOIL CUTTINGS MIXED WITH BENTONITE

S&ME ODOT SULFATE (8.5X11) - SGE 01/2019 - OH DOT GDT - 4/2/25 22:28 - R:\SERVICE LINES\CS-2557\CLEVELAND\01 - LABORATORY\02 - GINT\PROJECTS\23170065A.GPJ

S&ME JOB: 23170065A



PROJECT: STA-241-17.37	DRILLING FIRM / OPERATOR: OTB / C. STIVAK	DRILL RIG: OTB MOBILE B-57	STATION / OFFSET: 24+80, 27' LT	EXPLORATION ID B-008-0-23
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: S&ME / S. SMITH	HAMMER: CME AUTOMATIC	ALIGNMENT: CR 230 CL R/W	
PID: 119585 BR ID: N/A	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 12/12/22	ELEVATION: 1097.6 (MSL) EOB: 7.5 ft.	PAGE 1 OF 1
START: 8/30/23 END: 8/30/23	SAMPLING METHOD: SPT	ENERGY RATIO (%): 90*	COORD: 40.900572 N, 81.465724 W	

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	SO4 ppm	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI				
ASPHALT - 8 INCHES	1097.6																		
FILL: Medium-dense brown GRAVEL WITH SAND , little silt, trace clay, few slag and asphalt fragments, damp.	1096.9	1	9																
		2	18	47	100	SS-1	-	51	19	13	14	3	19	16	3	6	A-1-b (0)	260	
		3	5																
		4	11	29	100	SS-2	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	-	
	1092.4	5	17	9	89	SS-3A	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	-	
Medium-stiff to stiff brown and gray SILT AND CLAY , some fine to coarse sand, little fine gravel, few very-soft zones, damp.	1090.9	6	4	2		SS-3B	0.0-1.0	11	14	19	41	15	29	16	13	17	A-6a (5)	-	
	1090.9	7	1			SS-4A	0.7-2.0	-	-	-	-	-	-	-	-	16	A-6a (V)	-	
Loose brown COARSE AND FINE SAND , little silt, little clay, trace fine gravel, moist.	1090.1	7	3	9	78	SS-4B	-	-	-	-	-	-	-	-	-	14	A-3a (V)	-	

NOTES:

- Seepage observed at 5.3'.
- The borehole caved at 4.8' and was "dry" after removing the augers.

NOTES: SEE ABOVE.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: ASPHALT PATCH; SOIL CUTTINGS MIXED WITH BENTONITE

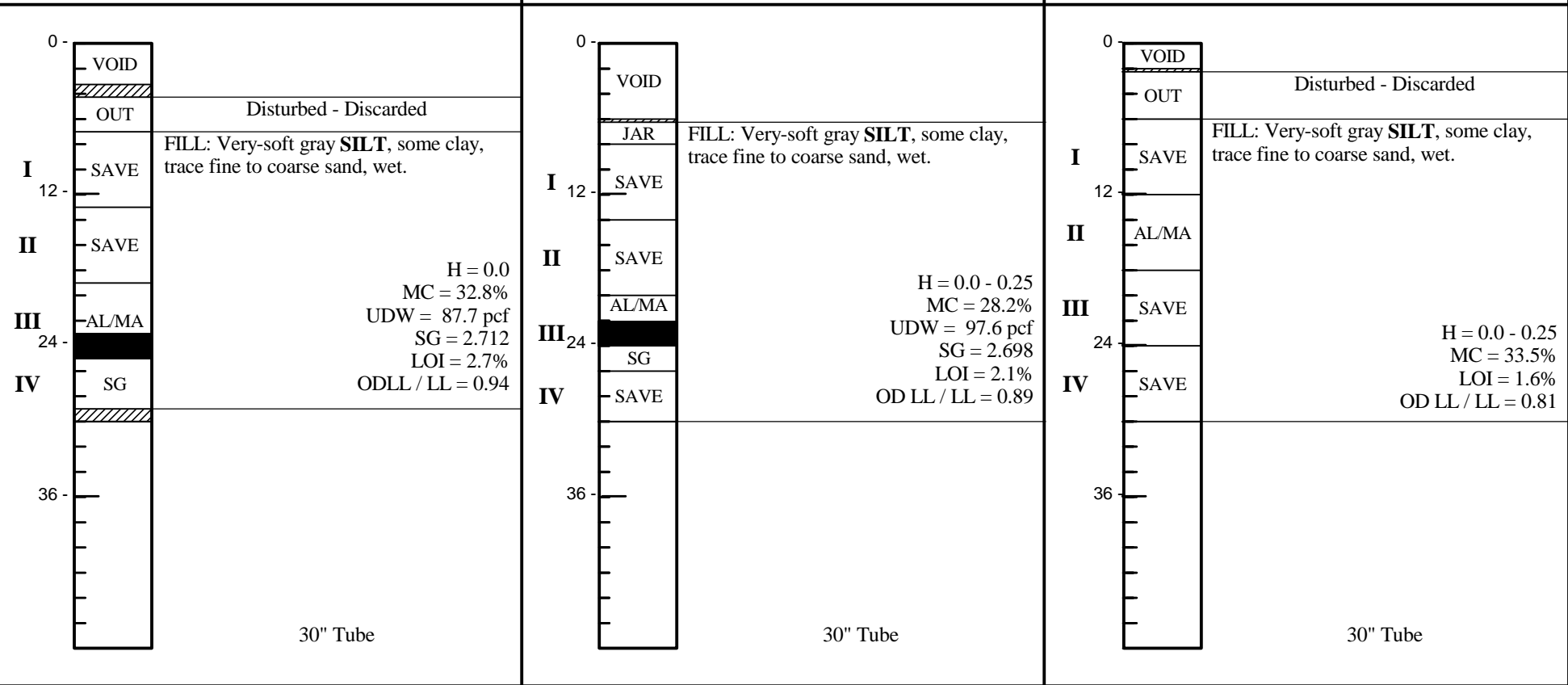
JOB NUMBER : 23170065A
 PROJECT : STA-241-17.37
 PID : 119585



LABORATORY LOG OF SHELBY TUBES

Boring : B-002-0-23	Sample : ST-9	Boring : B-003-0-23	Sample : ST-29	Boring : B-003-0-23	Sample : ST-30
----------------------------	----------------------	----------------------------	-----------------------	----------------------------	-----------------------

Depth : 17.5' - 19.5'	Recovery : 20"	Depth : 8.0' - 10.0'	Recovery : 24"	Depth : 16.0' - 18.0'	Recovery : 24"
------------------------------	-----------------------	-----------------------------	-----------------------	------------------------------	-----------------------



LEGEND

- Consolidation, Incremental	- Swelling, Test	- Wax
- Consolidation, C R S	- Unconfined Compression Test	- Triaxial Compression Test
- Permeability, Vertical / Horizontal		

H - Hand Penetrometer (tsf)	TV - Torvane (tsf)
Ds - Direct Shear	POR - Porosity
LOI - Loss on Ignition	UDW - Unit Dry Weight
AL - Atterberg Limits	MC - Moisture Content
MA - Sieve/Hydrometer	D _R - Relative Density
SG - Specific Gravity	S - Sieve

SAME SHELBY TUBE LOG - OH_DOT.GDT - 8/17/24 16:41 - R:\SERVICE LINES\CS-2557\CLEVELAND\01 - LABORATORY\02 - GINT\PROJECTS\23170065A.GPJ

DCP TEST DATA

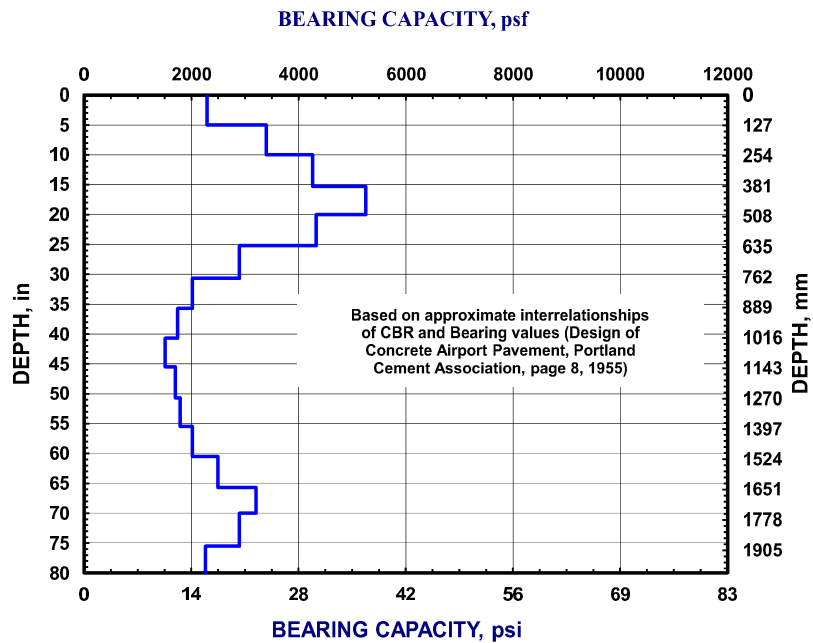
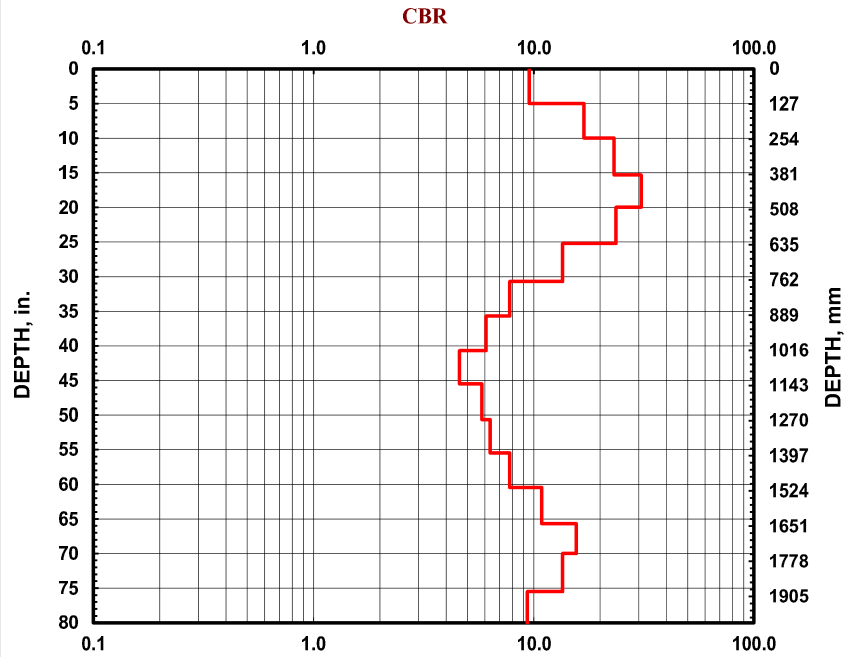
Project: 23170065A STA-241-17.37
Exploration ID: D-006-1-23
Surface Elev: 1089.35
Lat / Long: 40.900845; -81.467811

PID: 119585
Date: 7/19/2024
Surface Materials: Topsoil (1-inch)
Test Starting Depth: 0.0

Hammer
 10.1 lbs.
 17.6 lbs.
 Both hammers used

Soil Type
 GH
 CL
 All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	0	1
6	127	1
10	254	1
14	388.62	1
16	508	1
14	640.08	1
9	779.78	1
5	906.78	1
4	1033.78	1
3	1155.7	1
4	1287.78	1
4	1409.7	1
5	1536.7	1
7	1668.78	1
8	1778	1
9	1917.7	1
6	2047.24	1
7	2171.7	1
10	2296.16	2



DCP TEST DATA

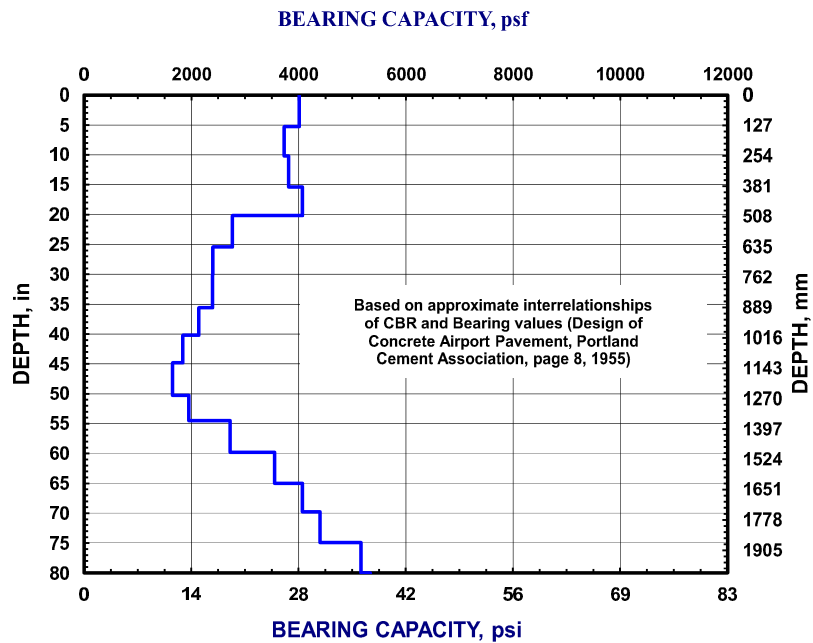
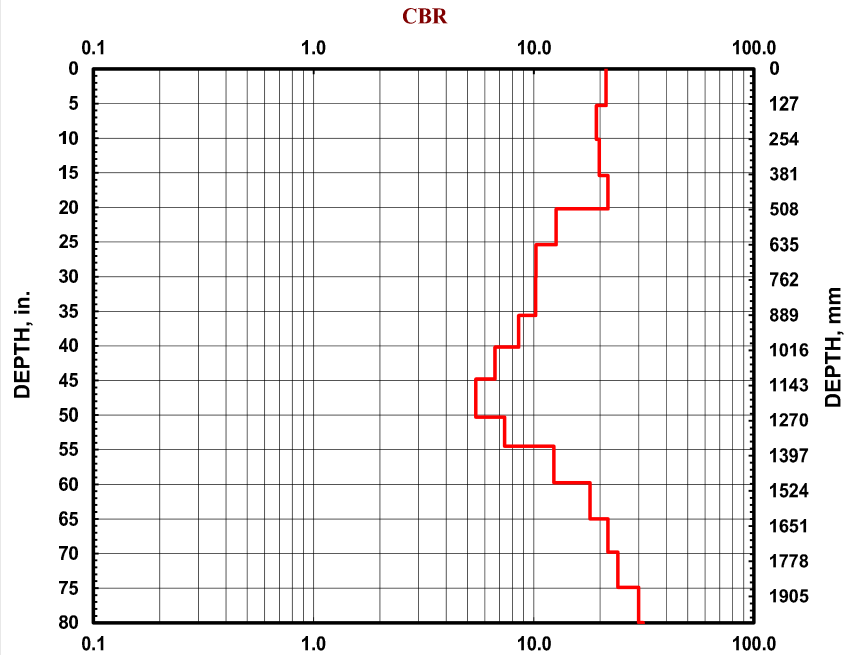
Project: 23170065A STA-241-17.37
Exploration ID: D-006-2-23
Surface Elev: 1090.9
Lat / Long: 40.901385; -81.467073

PID: 119585
Date: 7/19/2024
Surface Materials: Topsoil (1-inch)
Test Starting Depth: 0.0

Hammer
 10.1 lbs.
 17.6 lbs.
 Both hammers used

Soil Type
 GH
 CL
 All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	0	1
13	134.62	1
11	259.08	1
12	391.16	1
12	513.08	1
8	645.16	1
6	764.54	1
7	904.24	1
5	1021.08	1
4	1137.92	1
4	1277.62	1
4	1384.3	1
8	1518.92	1
11	1651	1
12	1772.92	1
14	1902.46	1
17	2032	1
17	2156.46	1
19	2288.54	1





Important Information About Your Geotechnical Engineering Report

Variations in subsurface conditions can be a principal cause of construction delays, cost overruns and claims. The following information is provided to assist you in understanding and managing the risk of these variations.

Geotechnical Findings Are Professional Opinions

Geotechnical engineers cannot specify material properties as other design engineers do. Geotechnical material properties have a far broader range on a given site than any manufactured construction material, and some geotechnical material properties may change over time because of exposure to air and water, or human activity.

Site exploration identifies subsurface conditions at the time of exploration and only at the points where subsurface tests are performed or samples obtained. Geotechnical engineers review field and laboratory data and then apply their judgment to render professional opinions about site subsurface conditions. Their recommendations rely upon these professional opinions. Variations in the vertical and lateral extent of subsurface materials may be encountered during construction that significantly impact construction schedules, methods and material volumes. While higher levels of subsurface exploration can mitigate the risk of encountering unanticipated subsurface conditions, no level of subsurface exploration can eliminate this risk.

Scope of Geotechnical Services

Professional geotechnical engineering judgment is required to develop a geotechnical exploration scope to obtain information necessary to support design and construction. A number of unique project factors are considered in developing the scope of geotechnical services, such as the exploration objective; the location, type, size and weight of the proposed structure; proposed site grades and improvements; the construction schedule and sequence; and the site geology.

Geotechnical engineers apply their experience with construction methods, subsurface conditions and exploration methods to develop the exploration scope. The scope of each exploration is unique based on available project and site information. Incomplete project information or constraints on the scope of exploration increases the risk of variations in subsurface conditions not being identified and addressed in the geotechnical report.

Services Are Performed for Specific Projects

Because the scope of each geotechnical exploration is unique, each geotechnical report is unique. Subsurface conditions are explored and recommendations are made for a specific project.

Subsurface information and recommendations may not be adequate for other uses. Changes in a proposed structure location, foundation loads, grades, schedule, etc. may require additional geotechnical exploration, analyses, and consultation. The geotechnical engineer should be consulted to determine if additional services are required in response to changes in proposed construction, location, loads, grades, schedule, etc.

Geo-Environmental Issues

The equipment, techniques, and personnel used to perform a geo-environmental study differ significantly from those used for a geotechnical exploration. Indications of environmental contamination may be encountered incidental to performance of a geotechnical exploration but go unrecognized. Determination of the presence, type or extent of environmental contamination is beyond the scope of a geotechnical exploration.

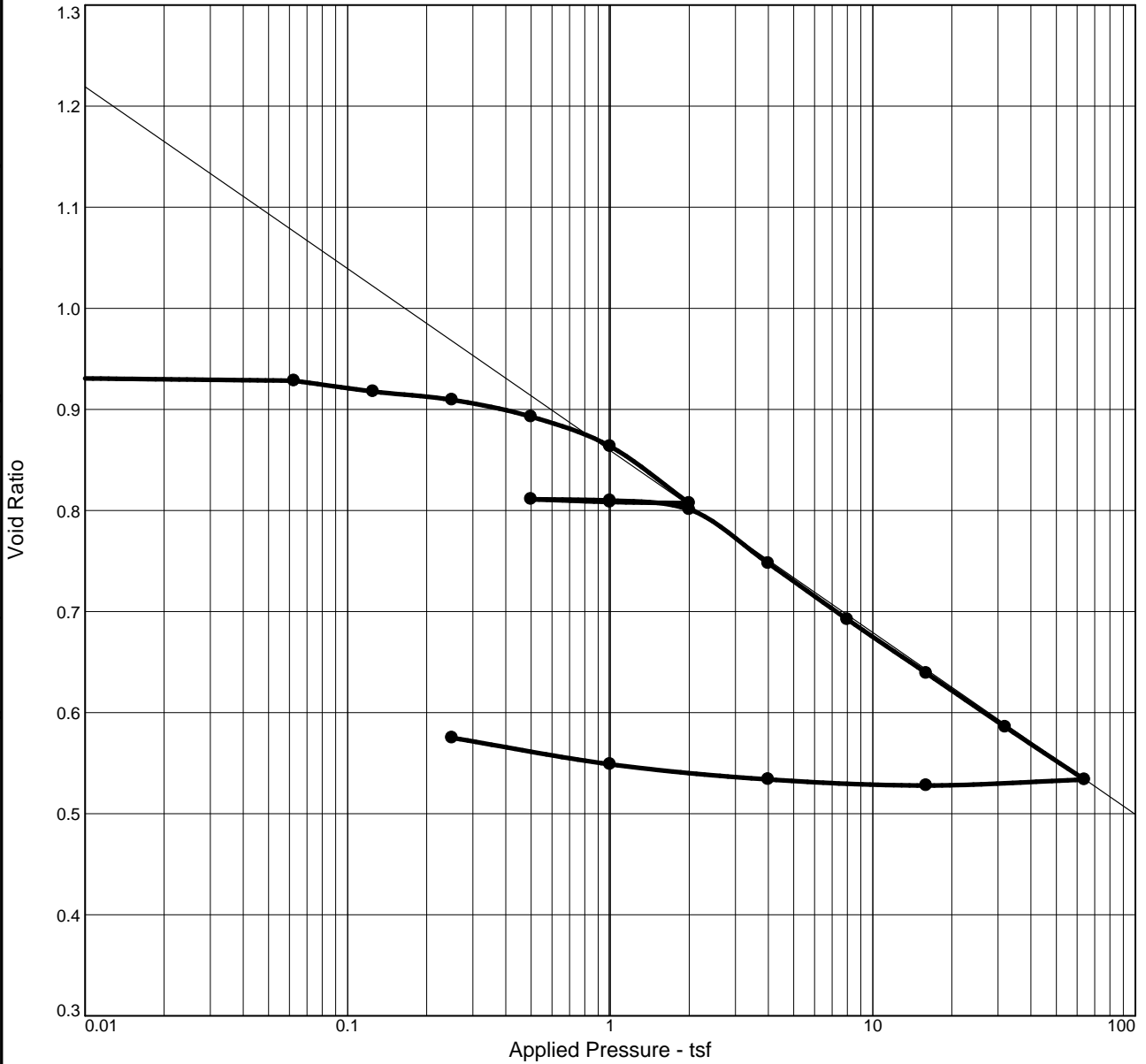
Geotechnical Recommendations Are Not Final

Recommendations are developed based on the geotechnical engineer's understanding of the proposed construction and professional opinion of site subsurface conditions. Observations and tests must be performed during construction to confirm subsurface conditions exposed by construction excavations are consistent with those assumed in development of recommendations. It is advisable to retain the geotechnical engineer that performed the exploration and developed the geotechnical recommendations to conduct tests and observations during construction. This may reduce the risk that variations in subsurface conditions will not be addressed as recommended in the geotechnical report.



Appendix II – Laboratory Test Results

CONSOLIDATION TEST REPORT



C and phi are not test results but an interpretation of the test results. The designer is responsible for interpreting test data as provided by S&ME.

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
95.6 %	32.8 %	87.7	31	10	2.712	CL	A-4b	0.931

MATERIAL DESCRIPTION

SILT (A-4b), some clay, trace fine to coarse sand, slightly organic

Project No. 23170065A Client: ODOT District 4 Project: STA-241-17.37 Roundabout & Bridge North Canton, Ohio Location: B-002-0-23 Depth: 17.5'-19.5' Sample Number: ST-9 (II)	Remarks: ASTM D2435
S&ME, Inc. Dublin, Ohio	

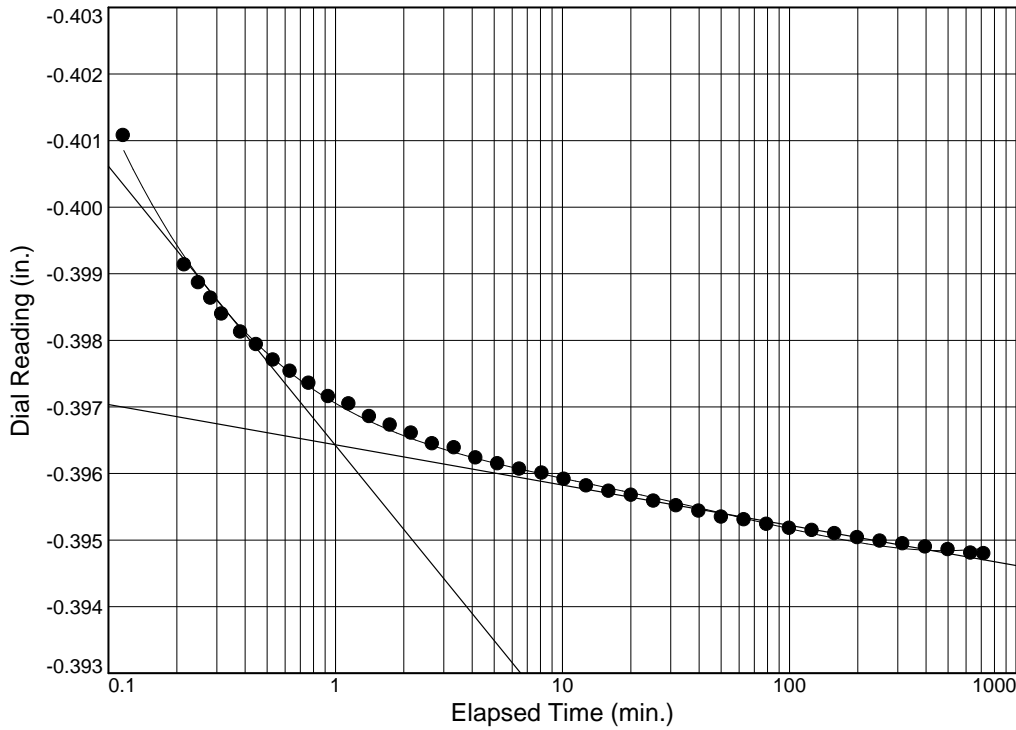
Figure 1

Tested By: PJM

Dial Reading vs. Time

Project No.: 23170065A
 Project: STA-241-17.37 Roundabout & Bridge

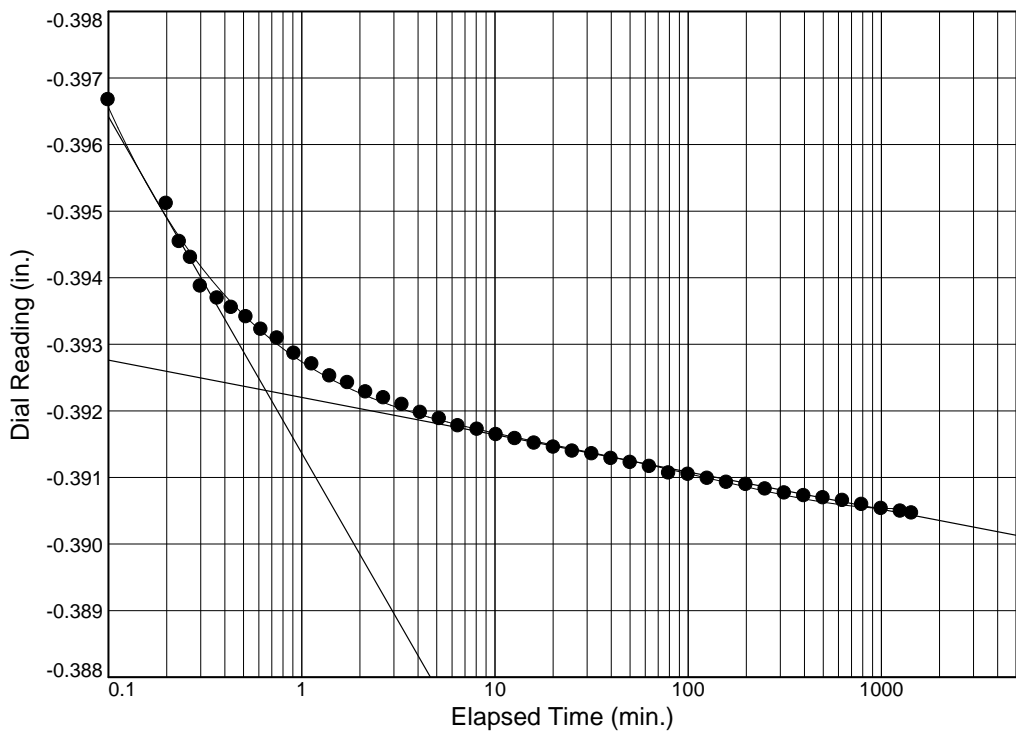
Location: B-002-0-23 Depth: 17.5'-19.5' Sample Number: ST-9 (II)



Load No.= 2
 Load= 0.13 tsf
 $D_0 = -0.4021$
 $D_{50} = -0.3993$
 $D_{100} = -0.3964$
 $T_{50} = 0.21 \text{ min.}$

$C_v @ T_{50}$
 2.293 ft.2/day

$C_\alpha = 0.001$



Load No.= 3
 Load= 0.25 tsf
 $D_0 = -0.3972$
 $D_{50} = -0.3948$
 $D_{100} = -0.3923$
 $T_{50} = 0.21 \text{ min.}$

$C_v @ T_{50}$
 2.277 ft.2/day

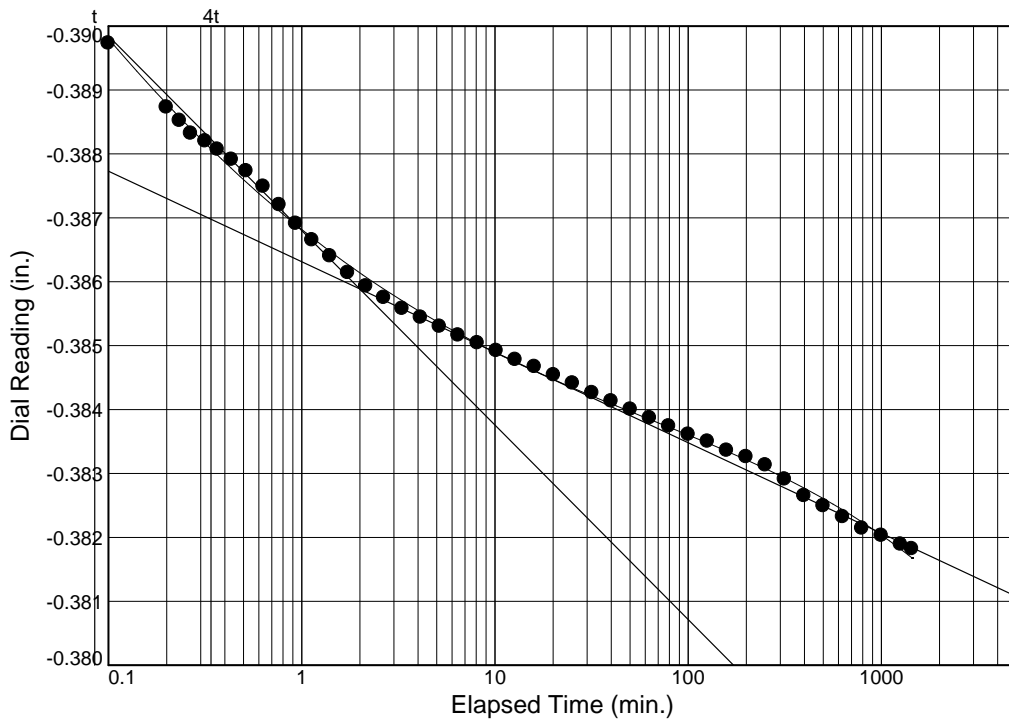
$C_\alpha = 0.001$

Figure 2

Dial Reading vs. Time

Project No.: 23170065A
 Project: STA-241-17.37 Roundabout & Bridge

Location: B-002-0-23 Depth: 17.5'-19.5' Sample Number: ST-9 (II)

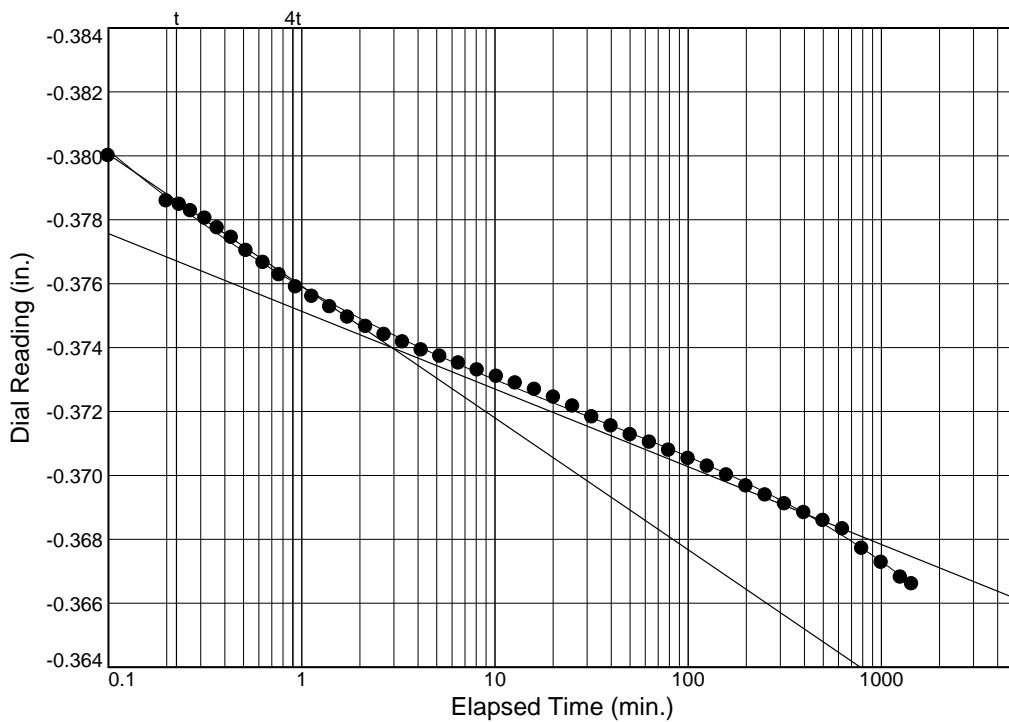


Load No.= 4
 Load= 0.50 tsf

$D_0 = -0.3920$
 $D_{50} = -0.3890$
 $D_{100} = -0.3859$
 $T_{50} = 0.18 \text{ min.}$

$C_v @ T_{50}$
 2.732 ft.²/day

$C_\alpha = 0.003$



Load No.= 5
 Load= 1.00 tsf

$D_0 = -0.3809$
 $D_{50} = -0.3774$
 $D_{100} = -0.3740$
 $T_{50} = 0.39 \text{ min.}$

$C_v @ T_{50}$
 1.203 ft.²/day

$C_\alpha = 0.005$

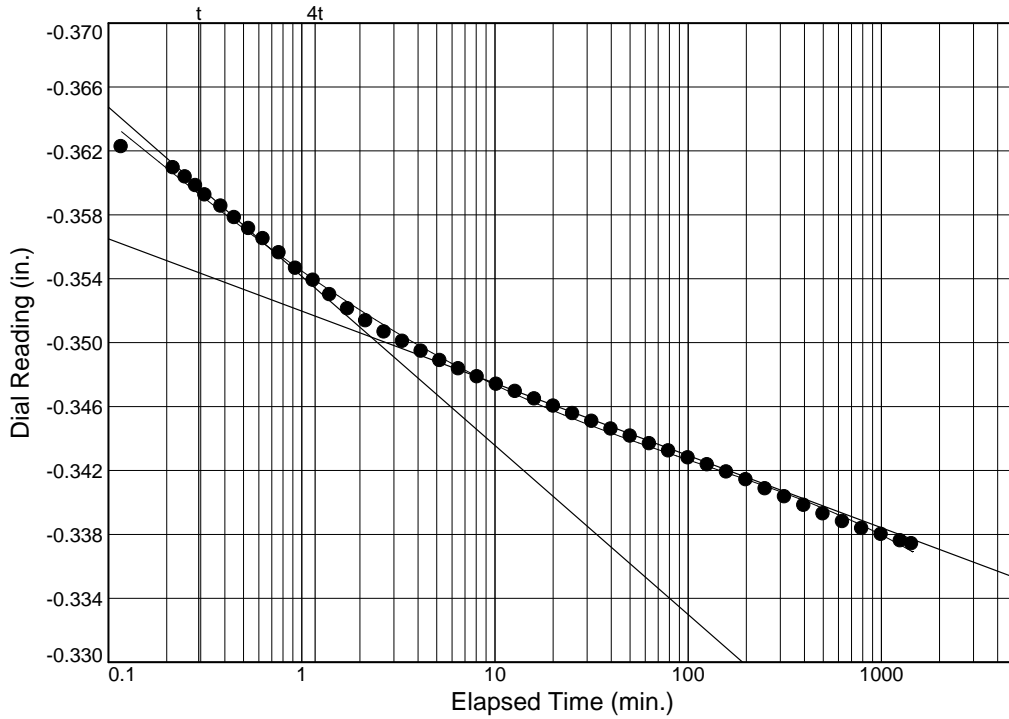
S&ME, Inc.

Figure 3

Dial Reading vs. Time

Project No.: 23170065A
 Project: STA-241-17.37 Roundabout & Bridge

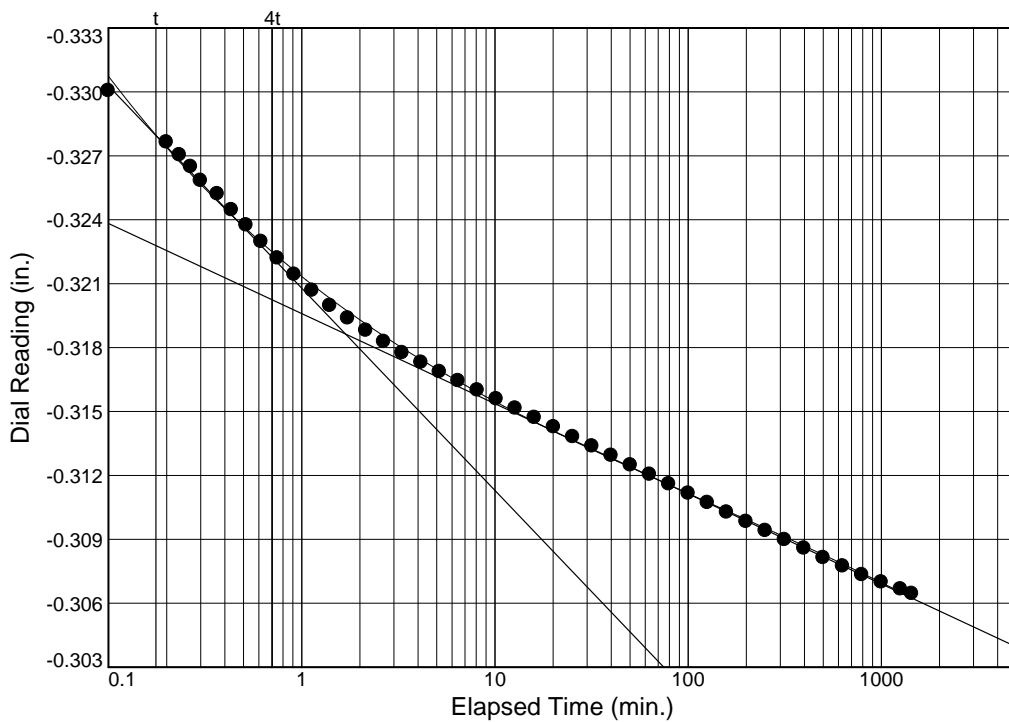
Location: B-002-0-23 Depth: 17.5'-19.5' Sample Number: ST-9 (II)



Load No.= 6
 Load= 2.00 tsf
 $D_0 = -0.3647$
 $D_{50} = -0.3575$
 $D_{100} = -0.3504$
 $T_{50} = 0.45 \text{ min.}$

$C_v @ T_{50}$
 0.990 ft.²/day

$C_\alpha = 0.009$



Load No.= 11
 Load= 4.00 tsf
 $D_0 = -0.3335$
 $D_{50} = -0.3260$
 $D_{100} = -0.3186$
 $T_{50} = 0.27 \text{ min.}$

$C_v @ T_{50}$
 1.538 ft.²/day

$C_\alpha = 0.008$

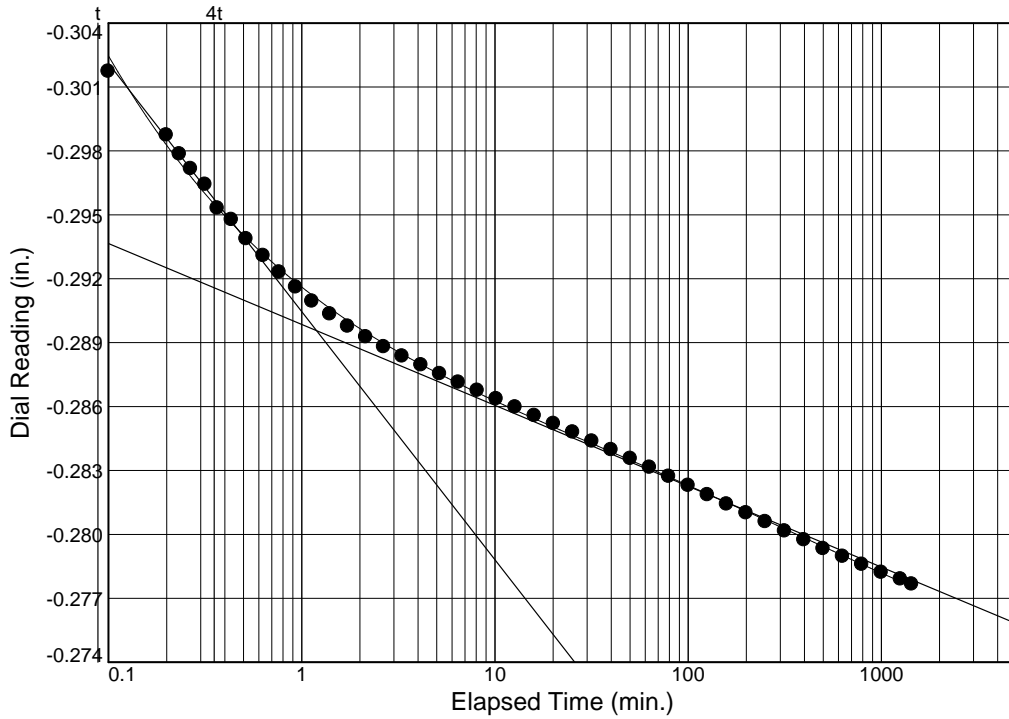
S&ME, Inc.

Figure 4

Dial Reading vs. Time

Project No.: 23170065A
 Project: STA-241-17.37 Roundabout & Bridge

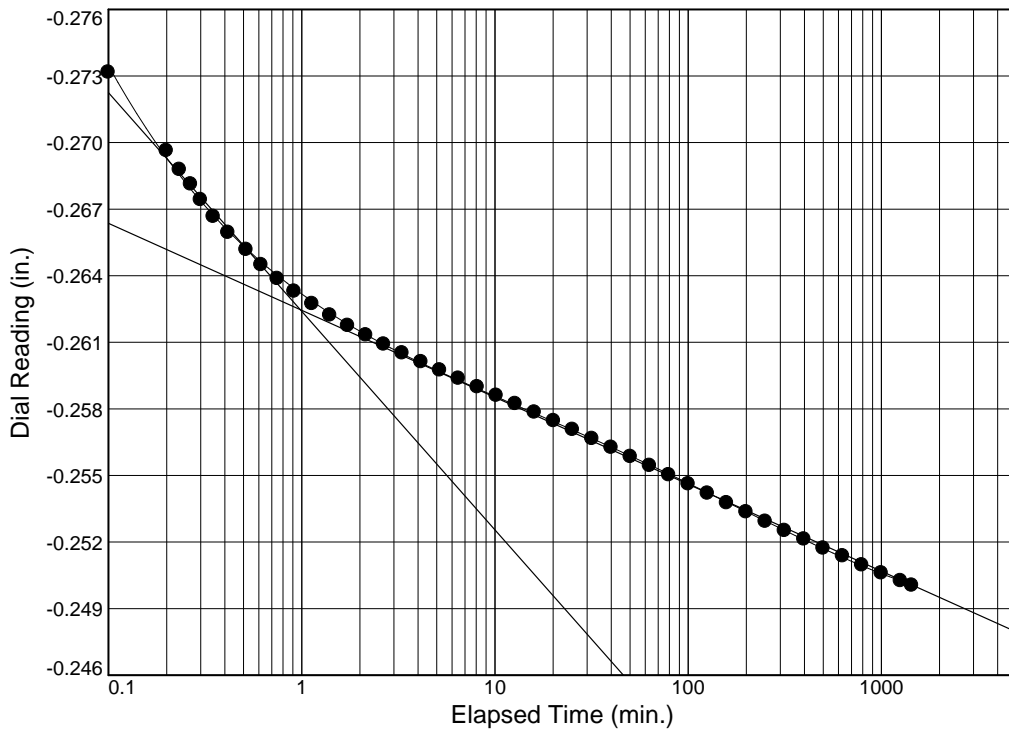
Location: B-002-0-23 Depth: 17.5'-19.5' Sample Number: ST-9 (II)



Load No.= 12
 Load= 8.00 tsf
 $D_0 = -0.3112$
 $D_{50} = -0.3004$
 $D_{100} = -0.2896$
 $T_{50} = 0.14 \text{ min.}$

$C_v @ T_{50}$
 2.855 ft.²/day

$C_\alpha = 0.007$



Load No.= 13
 Load= 16.00 tsf
 $D_0 = -0.2835$
 $D_{50} = -0.2730$
 $D_{100} = -0.2625$
 $T_{50} = 0.11 \text{ min.}$

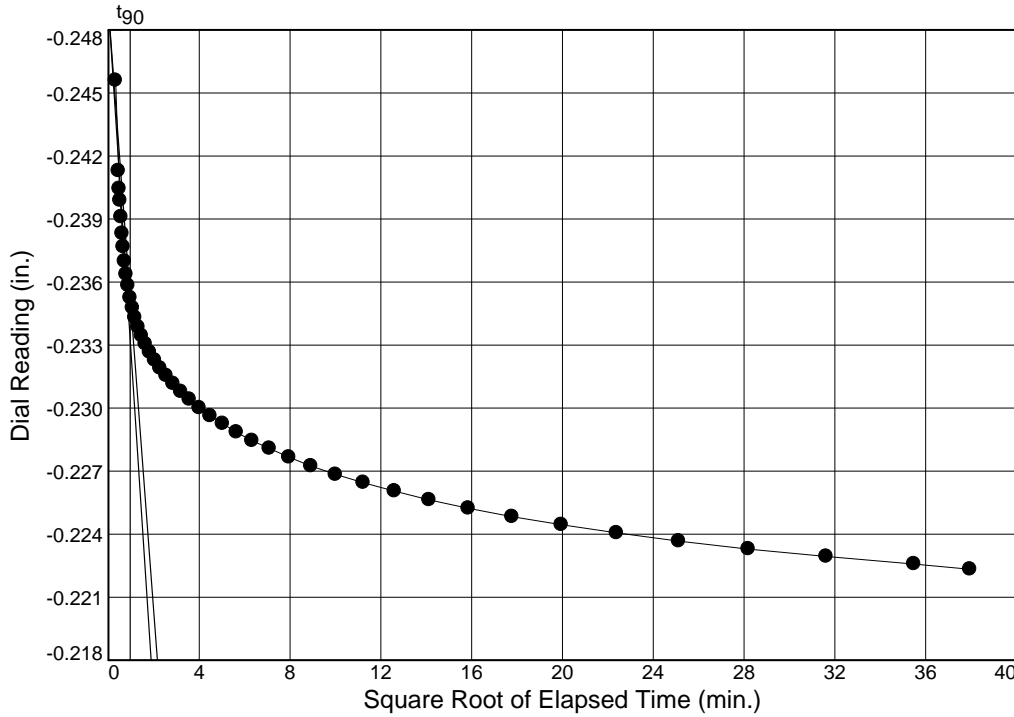
$C_v @ T_{50}$
 3.447 ft.²/day

$C_\alpha = 0.008$

Dial Reading vs. Time

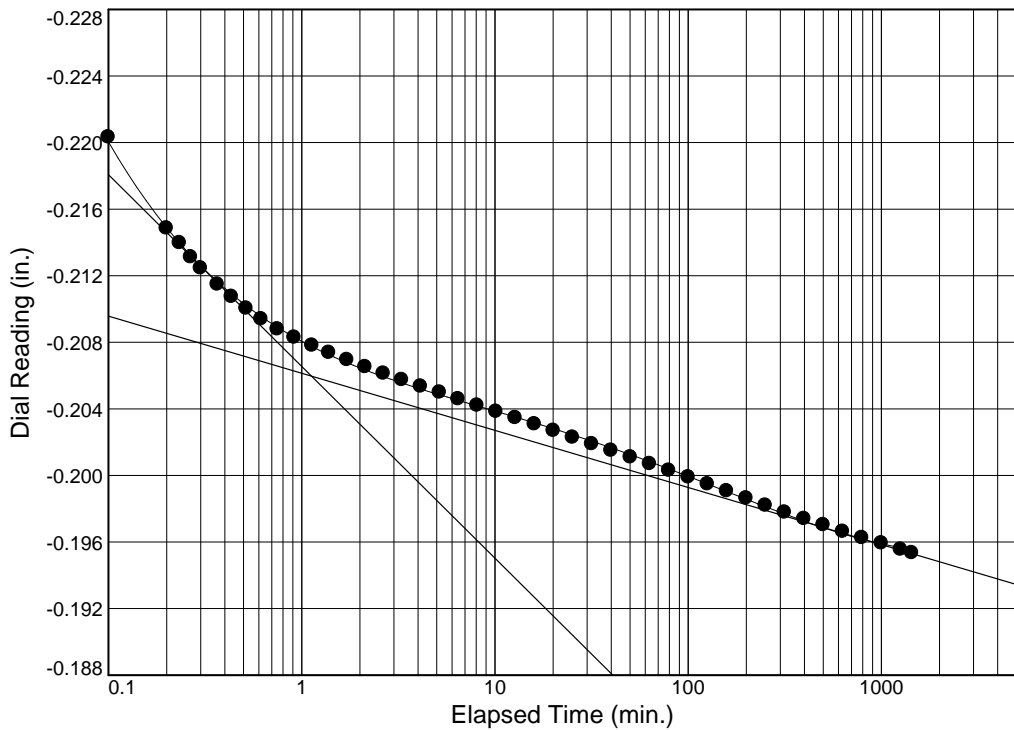
Project No.: 23170065A
 Project: STA-241-17.37 Roundabout & Bridge

Location: B-002-0-23 Depth: 17.5'-19.5' Sample Number: ST-9 (II)



Load No.= 14
 Load= 32.00 tsf
 $D_0 = -0.2490$
 $D_{90} = -0.2353$
 $D_{100} = -0.2337$
 $T_{90} = 0.91 \text{ min.}$

$C_v @ T_{90}$
 1.644 ft.²/day



Load No.= 15
 Load= 64.00 tsf
 $D_0 = -0.2339$
 $D_{50} = -0.2199$
 $D_{100} = -0.2060$
 $T_{50} = 0.10 \text{ min.}$

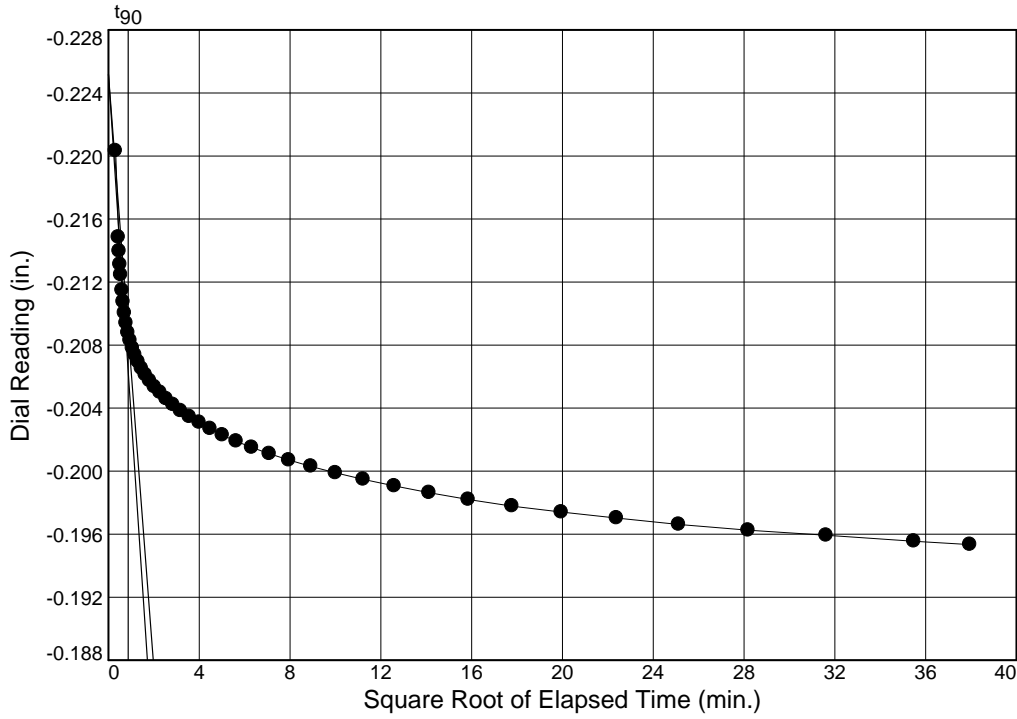
$C_v @ T_{50}$
 3.240 ft.²/day

$C_\alpha = 0.007$

Dial Reading vs. Time

Project No.: 23170065A
Project: STA-241-17.37 Roundabout & Bridge

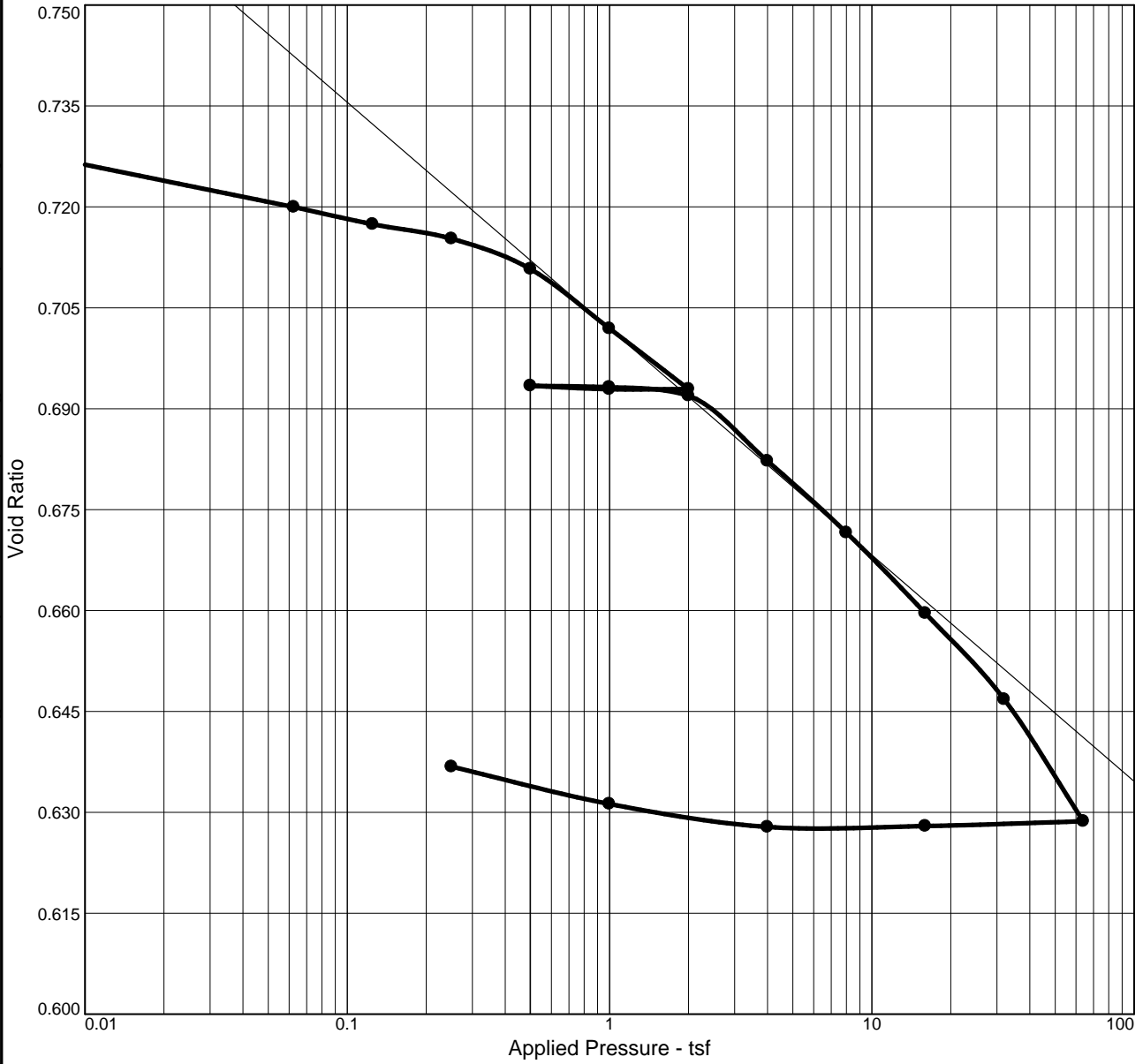
Location: B-002-0-23 Depth: 17.5'-19.5' Sample Number: ST-9 (II)



Load No.= 15
Load= 64.00 tsf
 $D_0 = -0.2252$
 $D_{90} = -0.2088$
 $D_{100} = -0.2070$
 $T_{90} = 0.76 \text{ min.}$

$C_v @ T_{90}$
1.869 ft.²/day

CONSOLIDATION TEST REPORT



C and phi are not test results but an interpretation of the test results. The designer is responsible for interpreting test data as provided by S&ME.

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
104.6 %	28.2 %	97.6	28	6	2.698	CL-ML	A-4b	0.726

MATERIAL DESCRIPTION

SILT (A-4b), some clay, trace fine to coarse sand

Project No. 23170065A	Client: ODOT District 4
Project: STA-241-17.37 Roundabout & Bridge North Canton, Ohio	
Location: B-003-0-23	Depth: 8.0'-10.0'
Sample Number: ST-29 (III)	
S&ME, Inc.	
Dublin, Ohio	

Remarks:
ASTM D2435

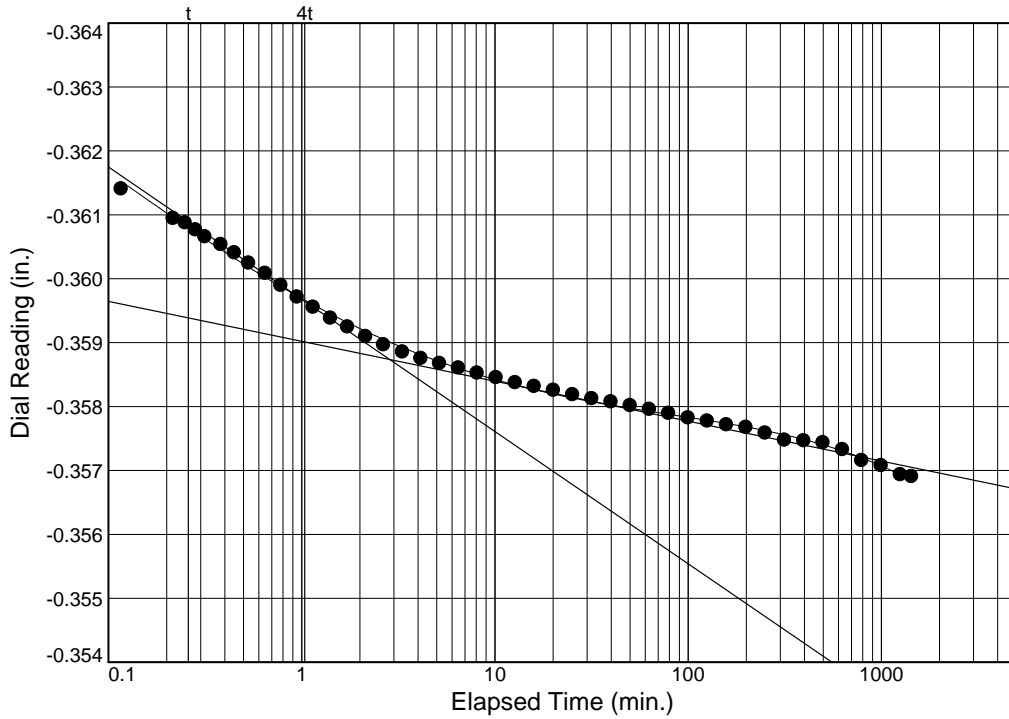
Figure 1

Tested By: PJM

Dial Reading vs. Time

Project No.: 23170065A
 Project: STA-241-17.37 Roundabout & Bridge

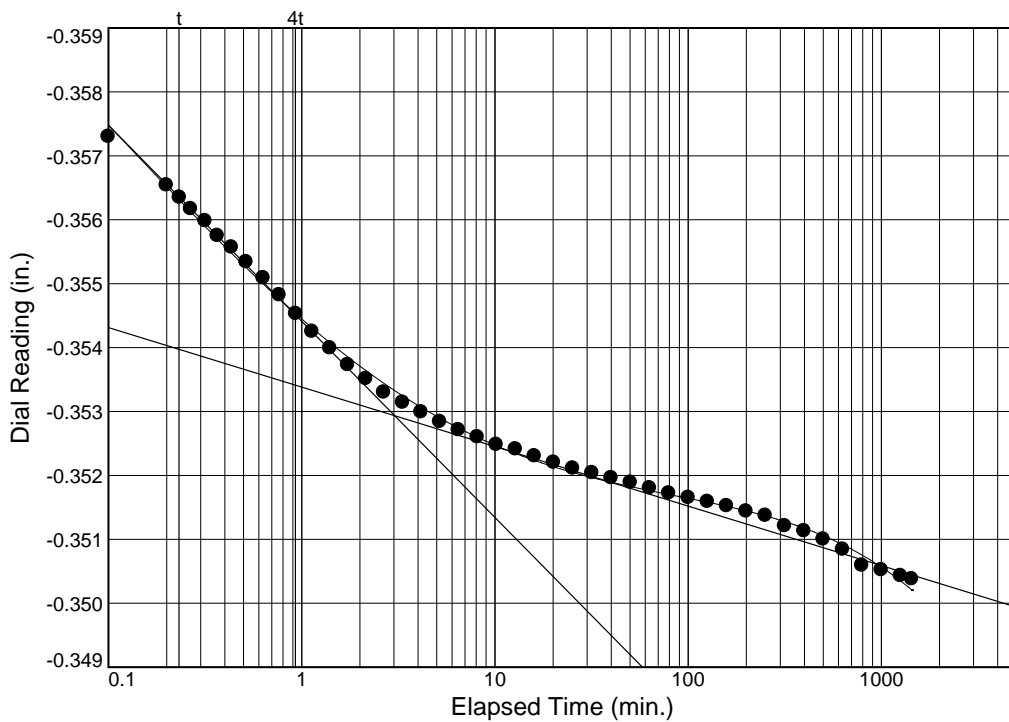
Location: B-003-1-23 Depth: 8.0'-10.0' Sample Number: ST-29 (III)



Load No.= 3
 Load= 0.25 tsf
 $D_0 = -0.3619$
 $D_{50} = -0.3603$
 $D_{100} = -0.3587$
 $T_{50} = 0.44 \text{ min.}$

$C_v @ T_{50}$
 6.851 ft.²/day

$C_\alpha = 0.000$



Load No.= 4
 Load= 0.50 tsf
 $D_0 = -0.3581$
 $D_{50} = -0.3555$
 $D_{100} = -0.3529$
 $T_{50} = 0.42 \text{ min.}$

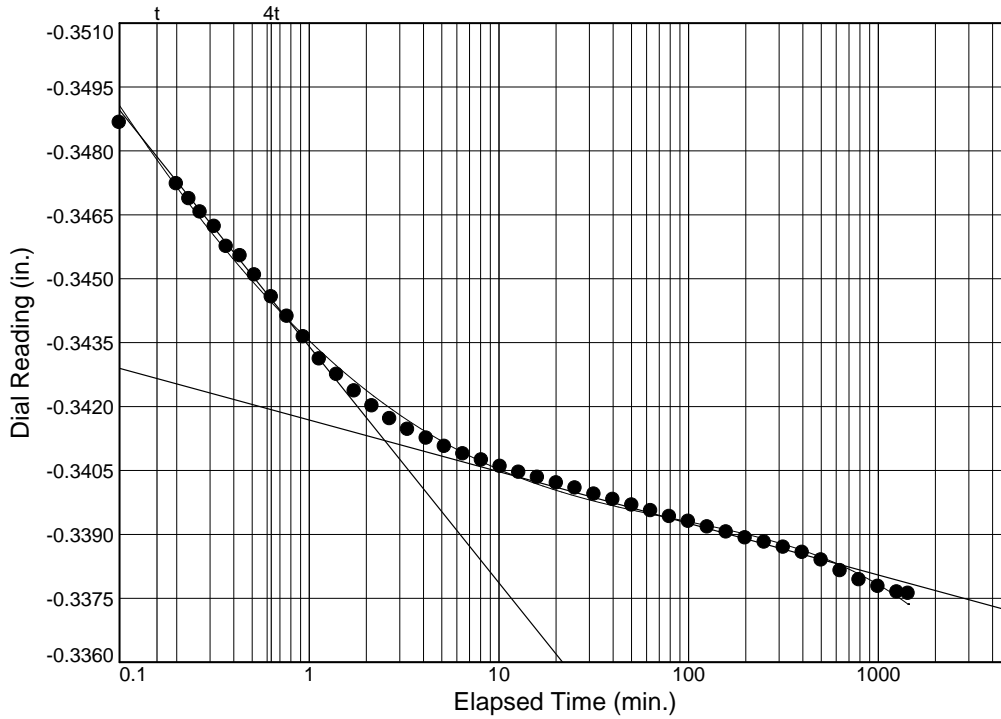
$C_v @ T_{50}$
 7.190 ft.²/day

$C_\alpha = 0.001$

Dial Reading vs. Time

Project No.: 23170065A
 Project: STA-241-17.37 Roundabout & Bridge

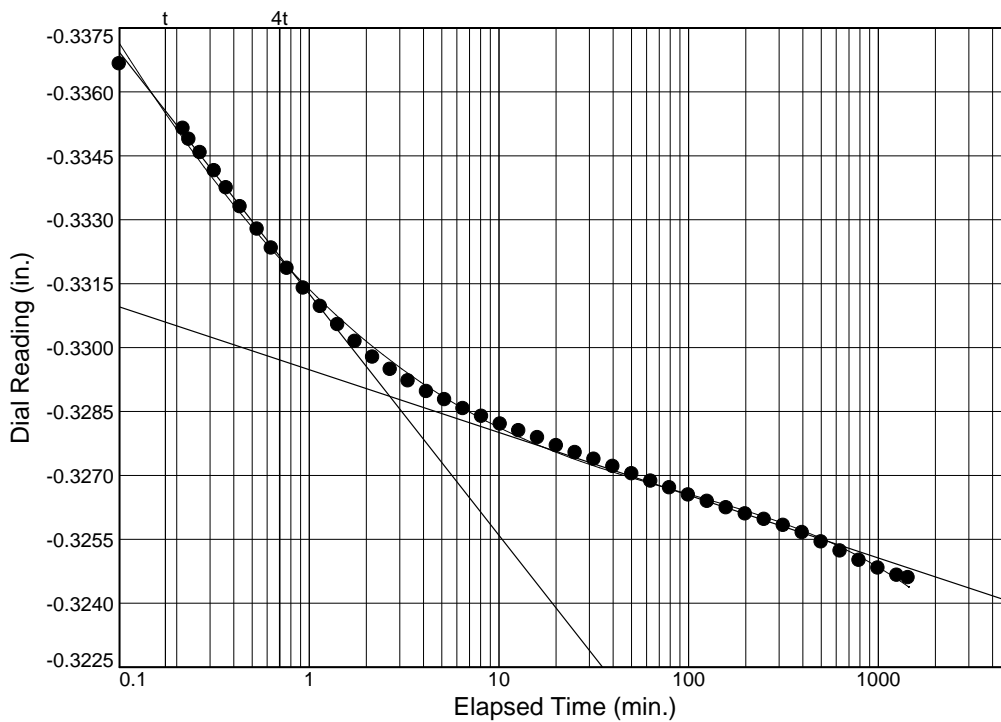
Location: B-003-1-23 Depth: 8.0'-10.0' Sample Number: ST-29 (III)



Load No.= 5
 Load= 1.00 tsf
 $D_0 = -0.3511$
 $D_{50} = -0.3462$
 $D_{100} = -0.3412$
 $T_{50} = 0.30 \text{ min.}$

$C_v @ T_{50}$
 10.150 ft.²/day

$C_\alpha = 0.001$



Load No.= 6
 Load= 2.00 tsf
 $D_0 = -0.3389$
 $D_{50} = -0.3339$
 $D_{100} = -0.3289$
 $T_{50} = 0.32 \text{ min.}$

$C_v @ T_{50}$
 9.248 ft.²/day

$C_\alpha = 0.001$

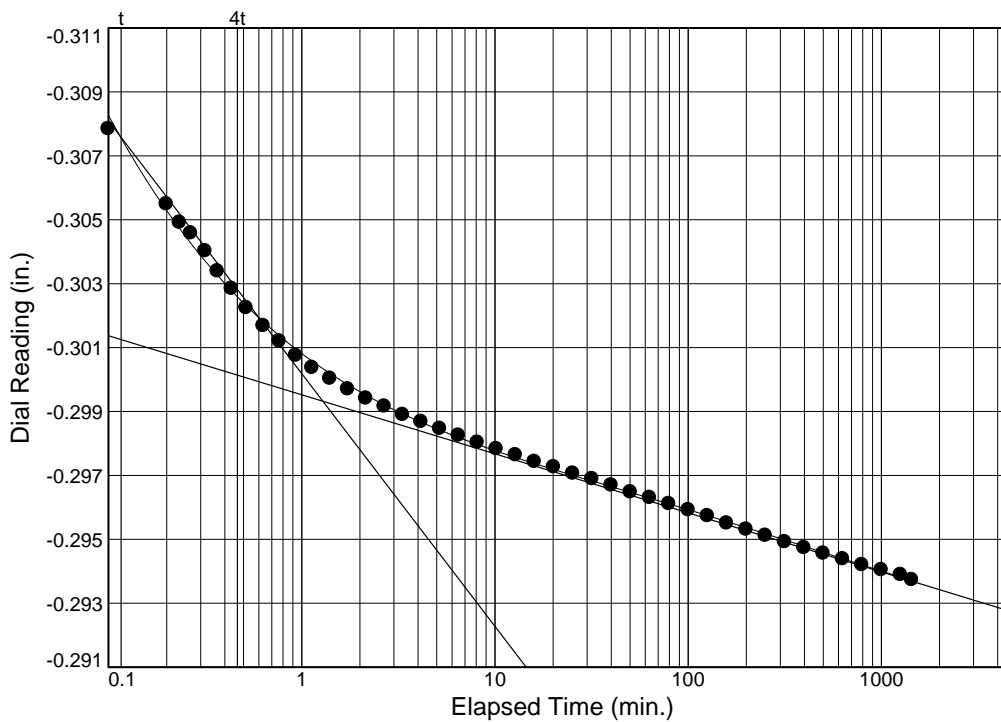
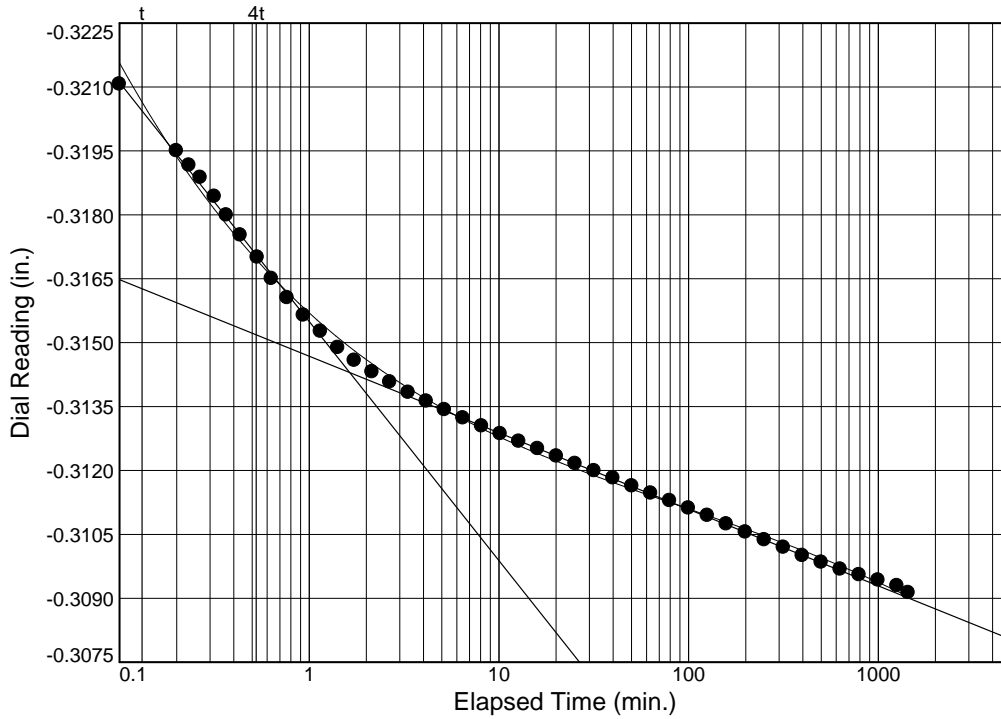
S&ME, Inc.

Figure 3

Dial Reading vs. Time

Project No.: 23170065A
 Project: STA-241-17.37 Roundabout & Bridge

Location: B-003-1-23 Depth: 8.0'-10.0' Sample Number: ST-29 (III)



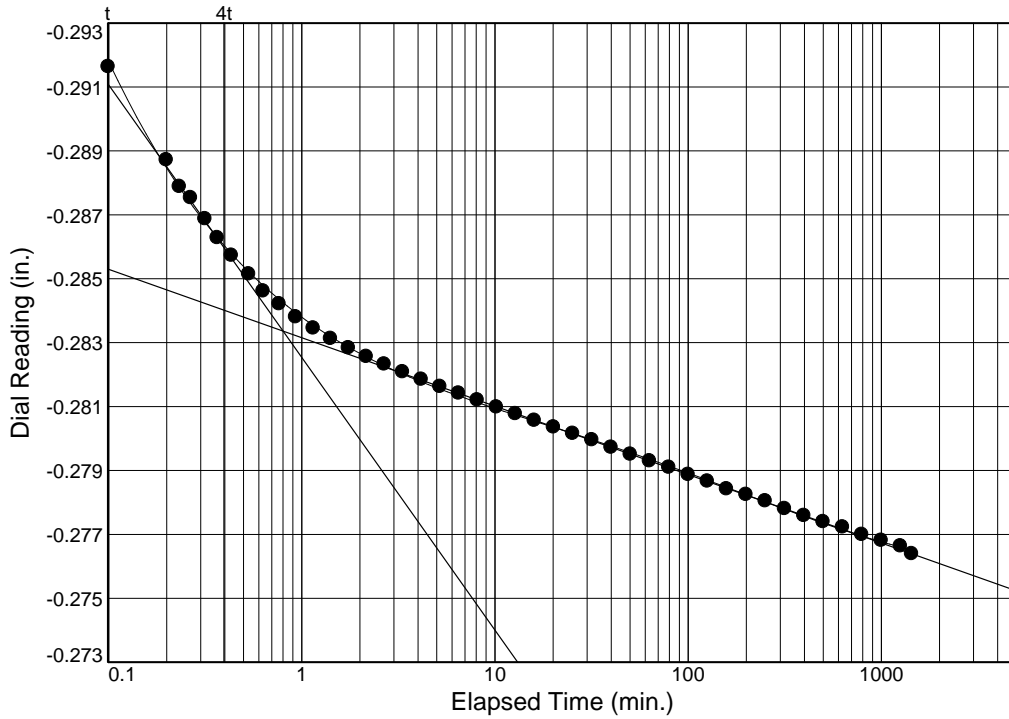
S&ME, Inc.

Figure 4

Dial Reading vs. Time

Project No.: 23170065A
 Project: STA-241-17.37 Roundabout & Bridge

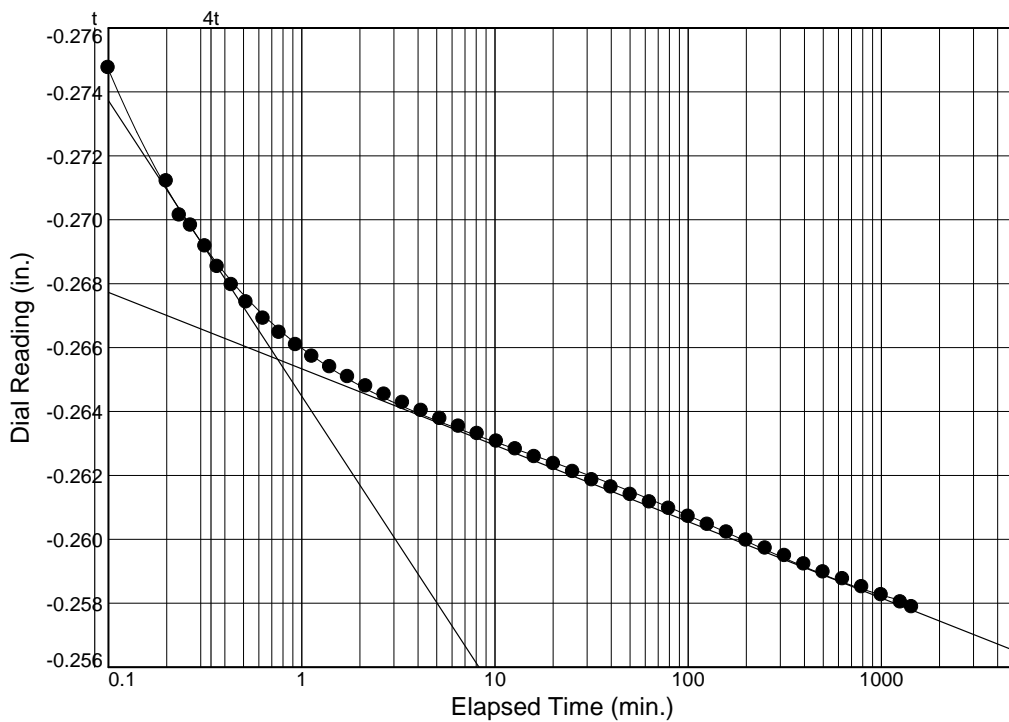
Location: B-003-1-23 Depth: 8.0'-10.0' Sample Number: ST-29 (III)



Load No.= 13
 Load= 16.00 tsf
 $D_0 = -0.2977$
 $D_{50} = -0.2905$
 $D_{100} = -0.2834$
 $T_{50} = 0.13 \text{ min.}$

$C_v @ T_{50}$
 22.513 ft.²/day

$C_\alpha = 0.001$



Load No.= 14
 Load= 32.00 tsf
 $D_0 = -0.2827$
 $D_{50} = -0.2741$
 $D_{100} = -0.2656$
 $T_{50} = 0.11 \text{ min.}$

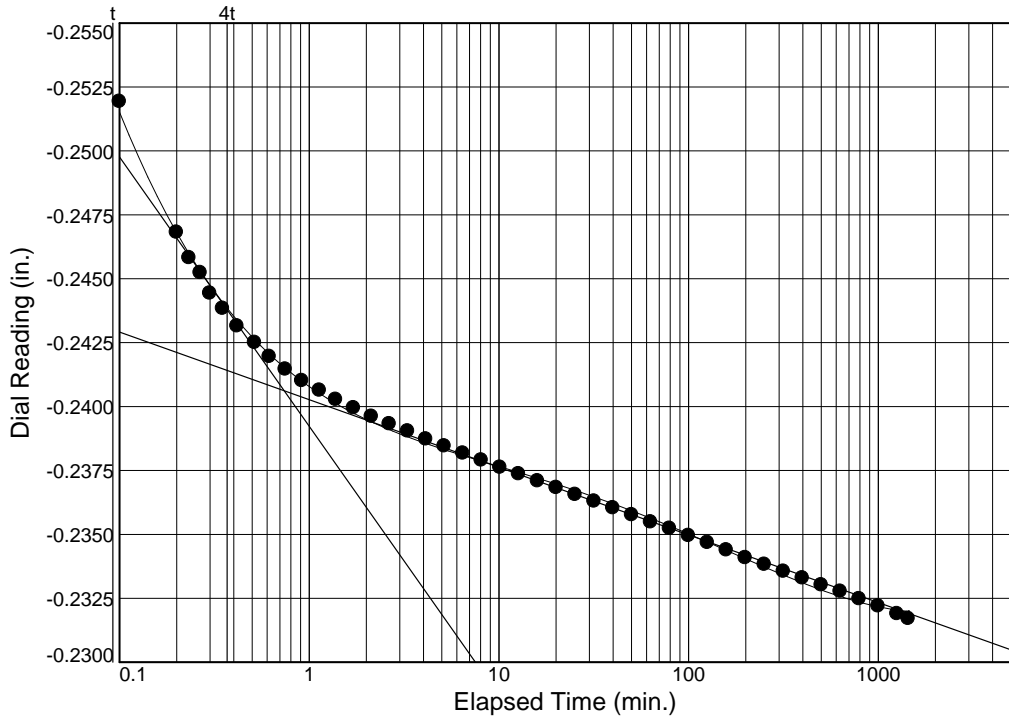
$C_v @ T_{50}$
 25.815 ft.²/day

$C_\alpha = 0.002$

Dial Reading vs. Time

Project No.: 23170065A
 Project: STA-241-17.37 Roundabout & Bridge

Location: B-003-1-23 Depth: 8.0'-10.0' Sample Number: ST-29 (III)



Load No.= 15
 Load= 64.00 tsf
 $D_0 = -0.2605$
 $D_{50} = -0.2506$
 $D_{100} = -0.2406$
 $T_{50} = 0.11 \text{ min.}$

$C_v @ T_{50}$
 24.530 ft.²/day

$C_\alpha = 0.002$

Subgrade and Structure Foundation Exploration Report – Final (Rev. #1)
STA-241-17.37 Roundabout and Bridge Replacement (PID 119585)
North Canton, Stark County, Ohio
S&ME Project No. 23170065A/B, Task Order No. 4-01-1



Appendix III – ODOT CPT Report

CONE PENETRATION TEST SOUNDINGS REPORT

Office of Geotechnical Engineering Division of Engineering

Project: STA-241-17.37

PID: 119585

Date: December 11, 2023

Number of Soundings: 8

Equipment: A.P. van den Berg, 23 Ton Crawler, Hyson 200kN

Sounding ID	Completion Date	Probe SN	Calibration Date	Elevation	Latitude	Longitude	Surface Material	Depth (ft.)
C-001-1-23	11/14/2023	201039	8/11/2023	1074.5	40.898226	-81.470733	Asphalt (14.5")	76.18
C-002-1-23	11/14/2023	090304	8/9/2023	1072.4	40.898295	-81.470503	Asphalt (12")	104.68
C-002-2-23	11/14/2023	201039	8/11/2023	1071.4	40.898337	-81.470357	Asphalt (12.5")	127.11
C-003-1-23	11/15/2023	090304	8/9/2023	1070.3	40.898488	-81.470458	Topsoil (4")	19.65
C-003-2-23	11/15/2023	090304	8/9/2023	1070.9	40.898459	-81.469945	Asphalt (14.5")	46.01
C-003-3-23	11/15/2023	201039	8/11/2023	1071.8	40.898597	-81.469487	Asphalt (15.5")	41.63
C-004-1-23	11/15/2023	090304	8/9/2023	1074.7	40.898759	-81.469077	Asphalt (13.5") Brick (4")	49.21
C-004-2-23	11/15/2023	201039	8/11/2023	1077.6	40.898985	-81.46871	Asphalt (11") Concrete (4")	44.87

Project Information

Eight soundings were completed for this project. Sounding C-003-1-23 was completed off road to run overnight dissipation tests. All other soundings were completed within the road through pre-cored holes. A pavement core report is attached. The static water levels reported on the attached logs were determined by pore pressure response and dissipation test results. Soundings C-003-2-23, C-003-3-23, C-004-1-23, and C-004-2-23 were terminated due to excessive tip resistance and hydraulic down pressure. All other soundings reached the target depths. The latitude, longitude, and elevation values are from district survey grade instruments. The sounding locations are shown on the attached exploration plan.

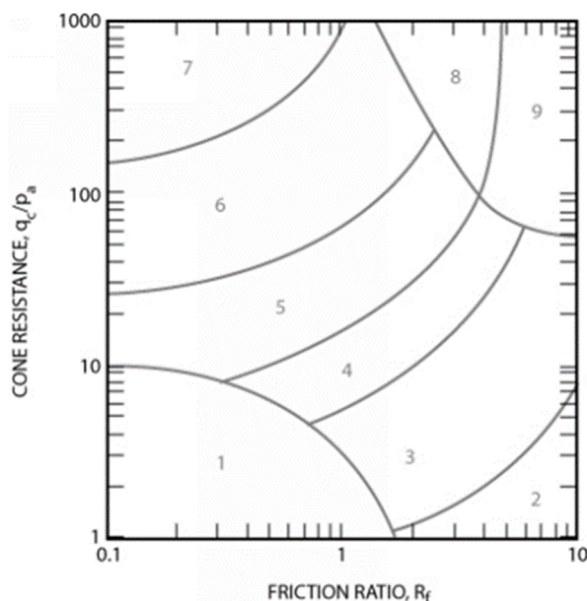
The raw CPT data is available upon request. The included CPT logs are for informational purposes only. The CPT logs have been filtered for negative values, corrected for inclination at depth, and filtered for data spikes. Additionally, for each sounding, the measured values of q_c and f_s were shifted relative to one another with a cross correlation function.

Cone Penetration Test Data and Interpretation

These Cone Penetration Test (CPT) Soundings follow ASTM D 5778 and were made by ordinary and conventional methods and with care deemed adequate for the Department's design purposes. Since subsurface conditions outside each CPT sounding are unknown, and soil, rock, and water conditions cannot be relied upon to be consistent or uniform, no warrant is made that conditions adjacent to this sounding will necessarily be the same as or similar to those shown in this report.

The CPT data collected are presented as graphical plots in the report, generated by CPeT-IT software. The plots include interpreted Soil Behavior Type (SBT) based on the method described by Robertson (2010). The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed.

The department does not warrant the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software. Furthermore, the Department will not be responsible for an interpretations, assumptions, projections, or interpolations made by the contractor, or other users of this report. While the Department believes that the information as to the condition and materials reported is accurate, it does not warrant that the information is necessarily complete. Water pressure measurements and subsequent interpreted water levels shown in this report should be used with discretion since they represent dynamic conditions. Dynamic pore water pressure measurements may deviate substantially from hydrostatic conditions, especially in cohesive soils.



Zone	Soil Behavior Type
1	Sensitive, fine grained
2	Organic soils - clay
3	Clay - silty clay to clay
4	Silt mixtures - clayey silt to silty clay
5	Sand mixtures - silty sand to sandy silt
6	Sands - clean sand to silty sand
7	Gravelly sand to dense sand
8	Very stiff sand to clayey sand*
9	Very stiff fine grained*

* Heavily overconsolidated or cemented

Non-normalized CPT Soil Behavior Type (SBT) chart

Robertson, P.K. and Cabal, K.L., 2016. *Guide to Cone Penetration Testing for Geotechnical Engineering*, 6th Edition. Signal Hill, California: 34.

<http://www.greggdrilling.com/wp-content/uploads/2017/07/CPT-Guide-6th-Edition-2016.pdf>

Accessed May 21, 2019

 OGE CPT Soundings

 S&ME Borings

C-004-2-23

C-004-1-23

B-004-0-23

C-003-3-23

SR 241

C-003-2-23

C-002-2-23

B-003-0-23

C-003-1-23

C-002-1-23

B-002-0-23

C-001-1-23

B-001-0-23



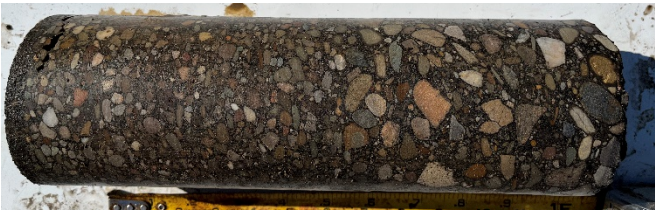



STA-241-17.37
Exploration Plan

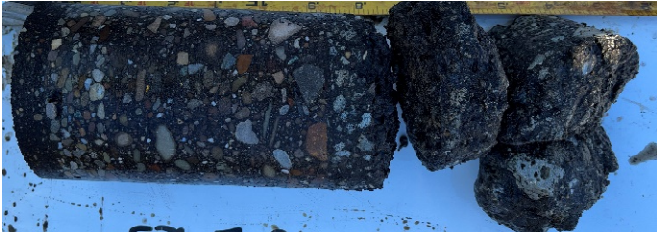





PLATE 3

STA-241-17.37 PID 119585
PAVEMENT CORE REPORT

Location	Position	Lat/Long	Pavement Thickness (in)	Core Photo	Comments
C-001-1-23	North Bound Lane	40.898226, -81.470733	Asphalt (14.5")		
C-002-1-23	North Bound Lane	40.898295, -81.470503	Asphalt (12")		
C-002-2-23	North Bound Lane	40.898337, -81.470357	Asphalt (12.5")		

STA-241-17.37 PID 119585
PAVEMENT CORE REPORT

Location	Position	Lat/Long	Pavement Thickness (in)	Core Photo	Comments
C-003-2-23	North Bound Lane	40.898459, -81.469945	Asphalt (14.5")		
C-003-3-23	North Bound Lane	40.898597, -81.469487	Asphalt (15.5")		Cored through large diameter rock below asphalt, possible maccadam base.
C-004-1-23	North Bound Lane	40.898759, -81.469077	Asphalt (13.5") Brick (4")		
C-004-2-23	North Bound Lane	40.898985, -81.46871	Asphalt (11") Concrete (4")		



Project: STA-241-17.37

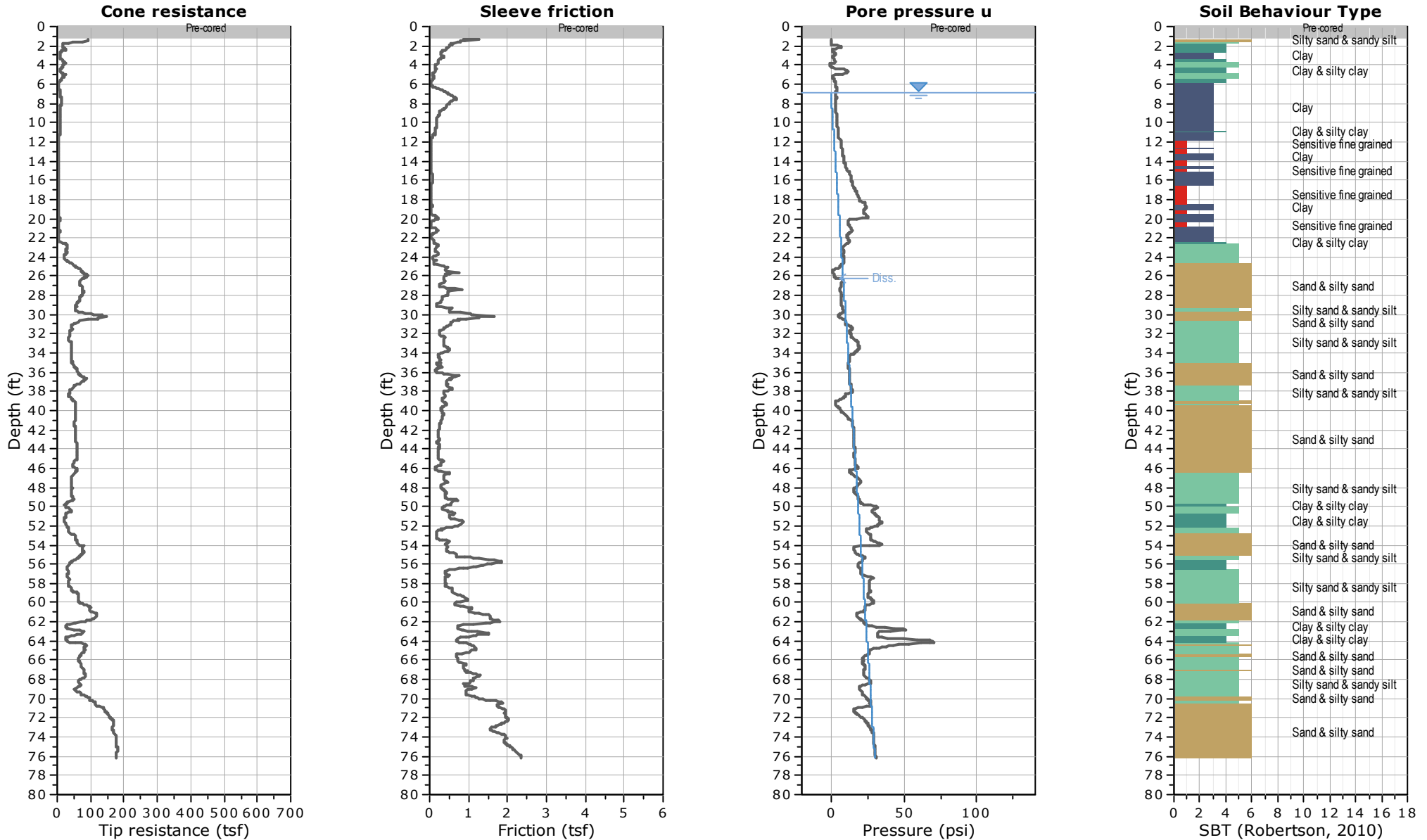
Location: Stark County

CPT: C-001-1-23

Total depth: 76.18 ft, Date: 11/14/2023

Surface Elevation: 1074.55 ft

Coords: lat 40.898226° lon -81.470733°



Dissipation Tests Results

Dissipation tests

Dissipation tests consists of stopping the piezocone penetration and observing porepressures (u) with elapsed time (t). The data are automatic recorded by the field computer and should take place until a minimum of 50% dissipation.

The porepressures are plotted as a function of square root of (t). The graphical technique suggested by Robertson and Campanella (1989), yields a value for t_{50} , which corresponds to the time for 50% consolidation.

The value of the coefficient of consolidation in the radial or horizontal direction c_h was then calculated by Houlsby and Teh's (1988) theory using the following equation:

$$c_h = \frac{T \times r^2 \times I_r^{0.5}}{t_{50}}$$

where:

T: time factor given by Houlsby and Teh's (1988) theory corresponding to the porepressure position

r: piezocone radius

I_r : stiffness index, equal to shear modulus G divided by the undrained strength of clay (S_u).

t_{50} : time corresponding to 50% consolidation

Permeability estimates based on dissipation test

The dissipation of pore pressures during a CPTu dissipation test is controlled by the coefficient of consolidation in the horizontal direction (c_h) which is influenced by a combination of the soil permeability (k_h) and compressibility (M), as defined by the following:

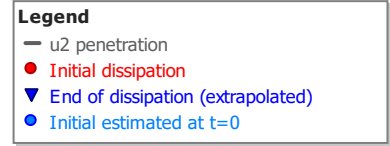
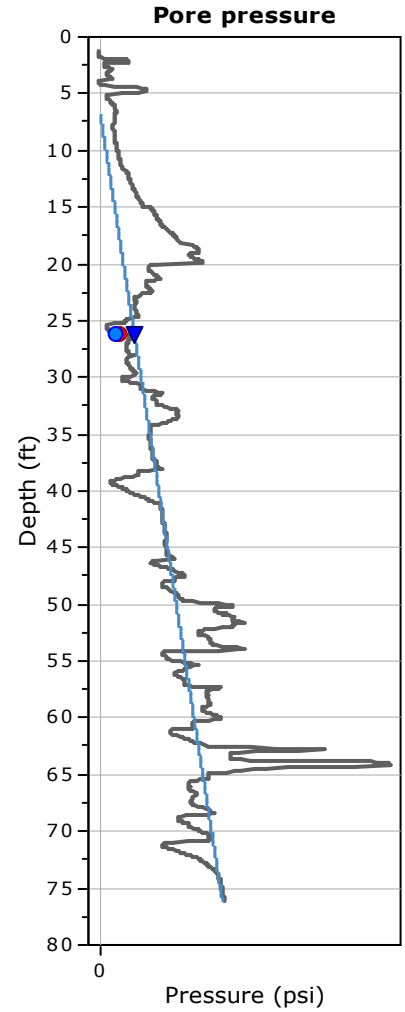
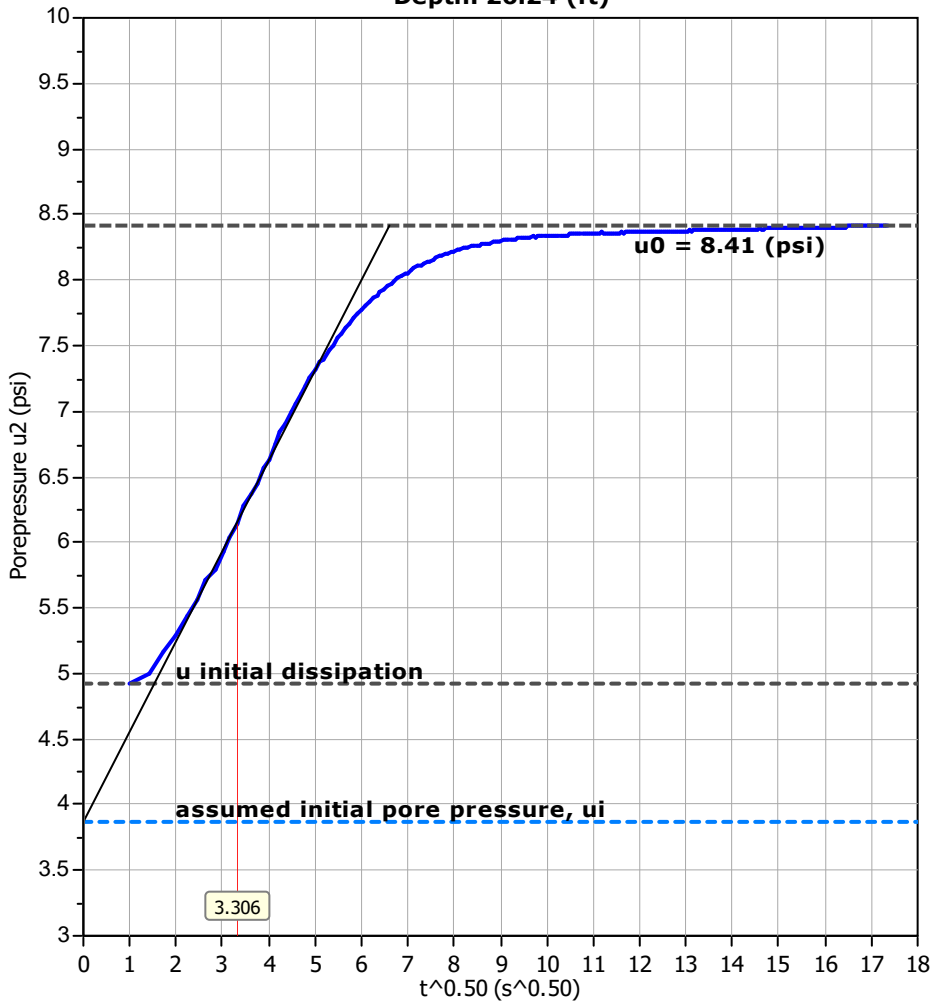
$$k_h = c_h \times \gamma_w / M$$

where: M is the 1-D constrained modulus and γ_w is the unit weight of water, in compatible units.

Tabular results

CPTU Borehole	Depth (ft)	$(t_{50})^{0.50}$	t_{50} (s)	t_{50} (years)	G/ S_u	c_h (ft ² /s)	c_h (ft ² /year)	M (tsf)	k_h (ft/s)
C-001-1-23	26.24	3.3	11	3.47E-007	100.00	1.15E-003	36335	595.72	6.04E-008

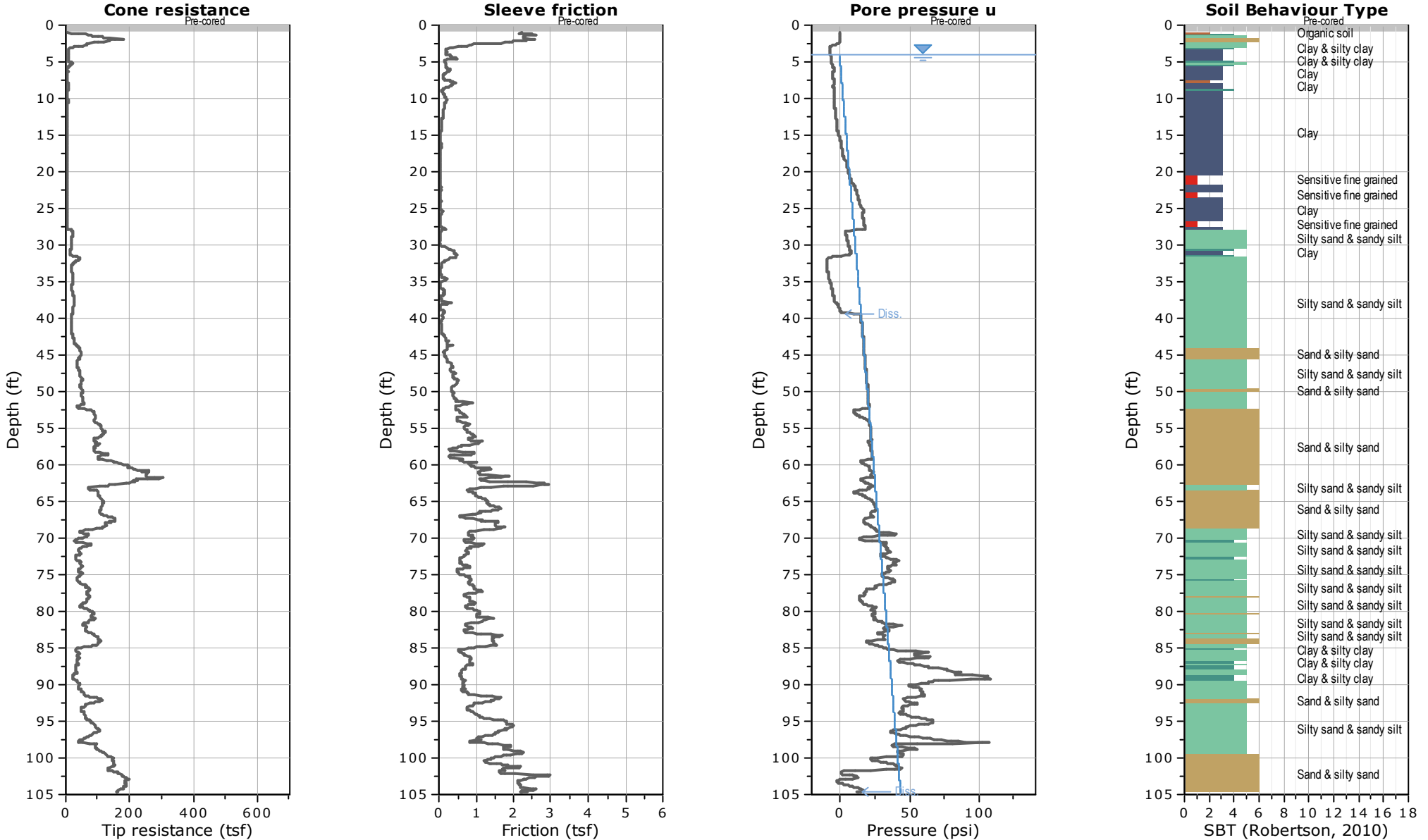
Piezocone Dissipation Test: C-001-1-23
Depth: 26.24 (ft)





Project: STA-241-17.37

Location: Stark County



Project: STA-241-17.37

Location: Stark County

Dissipation Tests Results

Dissipation tests

Dissipation tests consists of stopping the piezocone penetration and observing porepressures (u) with elapsed time (t). The data are automatic recorded by the field computer and should take place until a minimum of 50% dissipation.

The porepressures are plotted as a function of square root of (t). The graphical technique suggested by Robertson and Campanella (1989), yields a value for t_{50} , which corresponds to the time for 50% consolidation.

The value of the coefficient of consolidation in the radial or horizontal direction c_h was then calculated by Houlsby and Teh's (1988) theory using the following equation:

$$c_h = \frac{T \times r^2 \times I_r^{0.5}}{t_{50}}$$

where:

T: time factor given by Houlsby and Teh's (1988) theory corresponding to the porepressure position

r: piezocone radius

I_r : stiffness index, equal to shear modulus G divided by the undrained strength of clay (S_u).

t_{50} : time corresponding to 50% consolidation

Permeability estimates based on dissipation test

The dissipation of pore pressures during a CPTu dissipation test is controlled by the coefficient of consolidation in the horizontal direction (c_h) which is influenced by a combination of the soil permeability (k_h) and compressibility (M), as defined by the following:

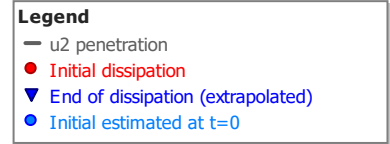
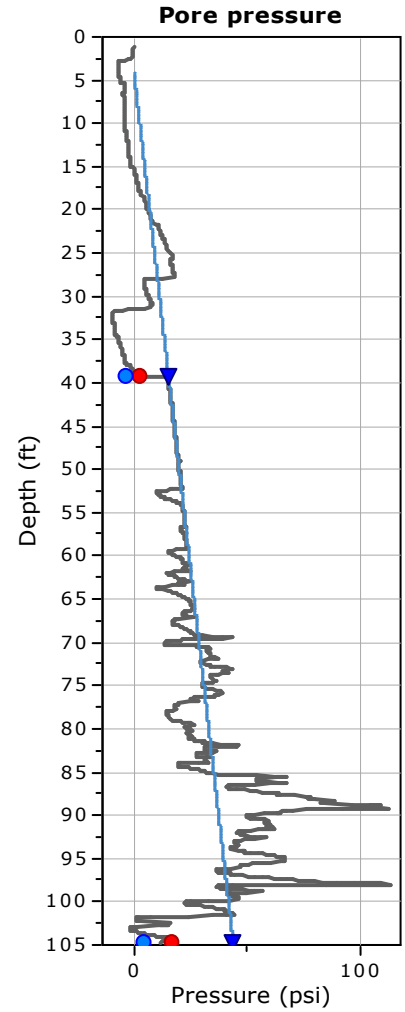
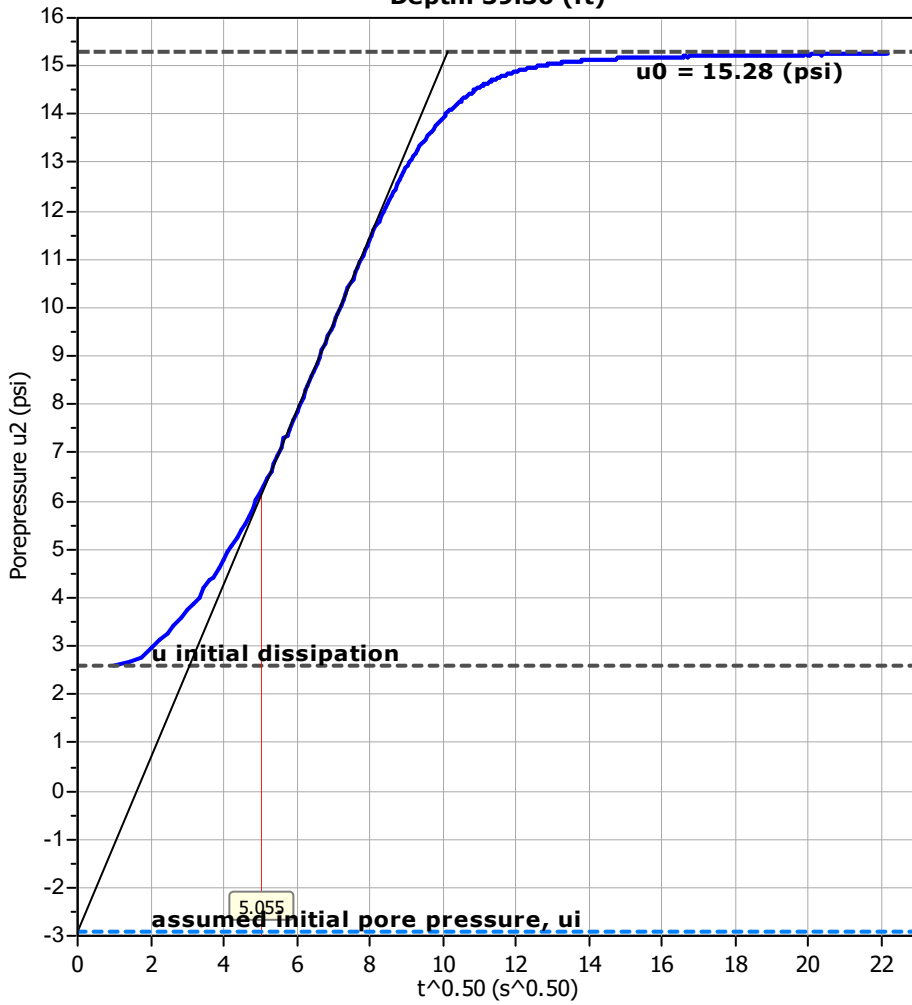
$$k_h = c_h \times \gamma_w / M$$

where: M is the 1-D constrained modulus and γ_w is the unit weight of water, in compatible units.

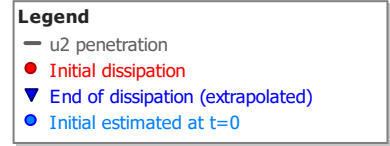
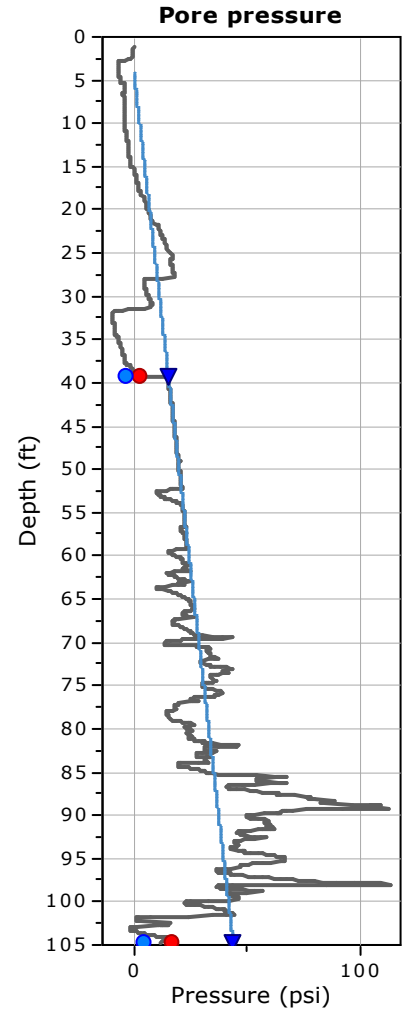
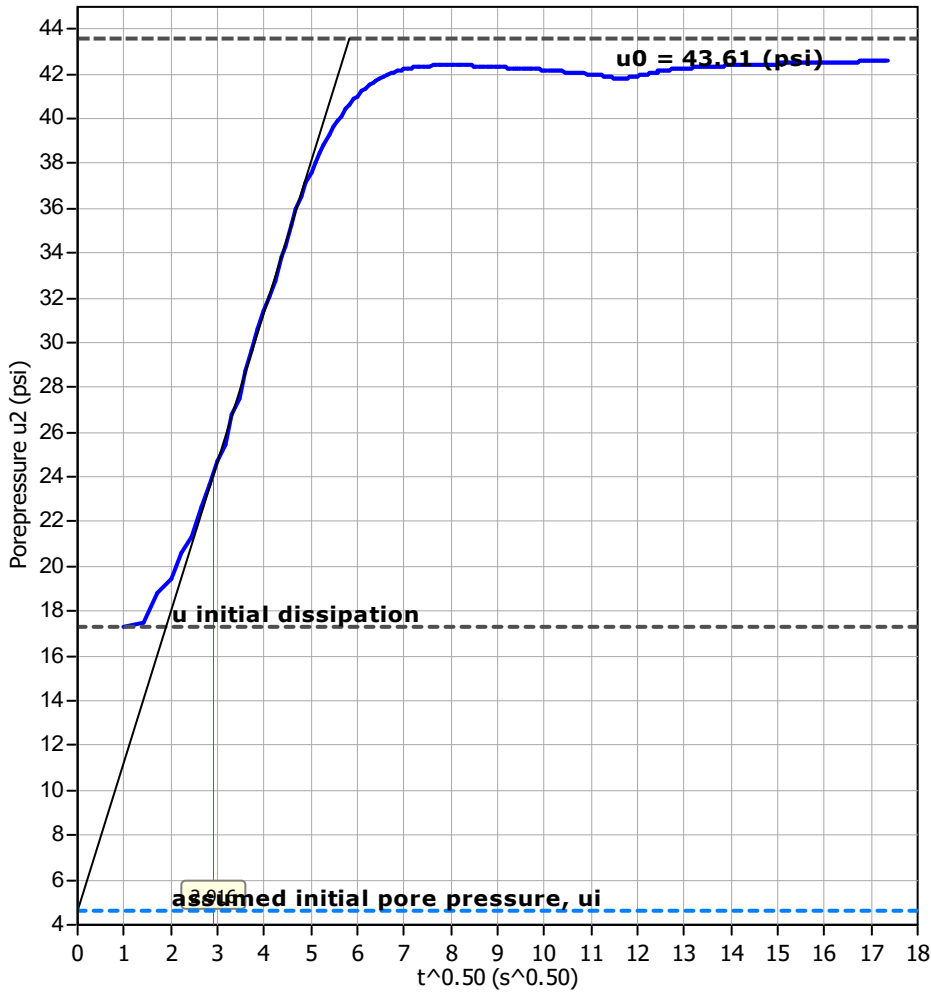
Tabular results

CPTU Borehole	Depth (ft)	$(t_{50})^{0.50}$	t_{50} (s)	t_{50} (years)	G/ S_u	c_h (ft ² /s)	c_h (ft ² /year)	M (tsf)	k_h (ft/s)
C-002-1-23	39.36	5.1	26	8.10E-007	100.00	4.93E-004	15541	259.47	5.93E-008
C-002-1-23	104.68	2.9	9	2.70E-007	100.00	1.48E-003	46713	2035.45	2.27E-008

Piezocone Dissipation Test: C-002-1-23
Depth: 39.36 (ft)



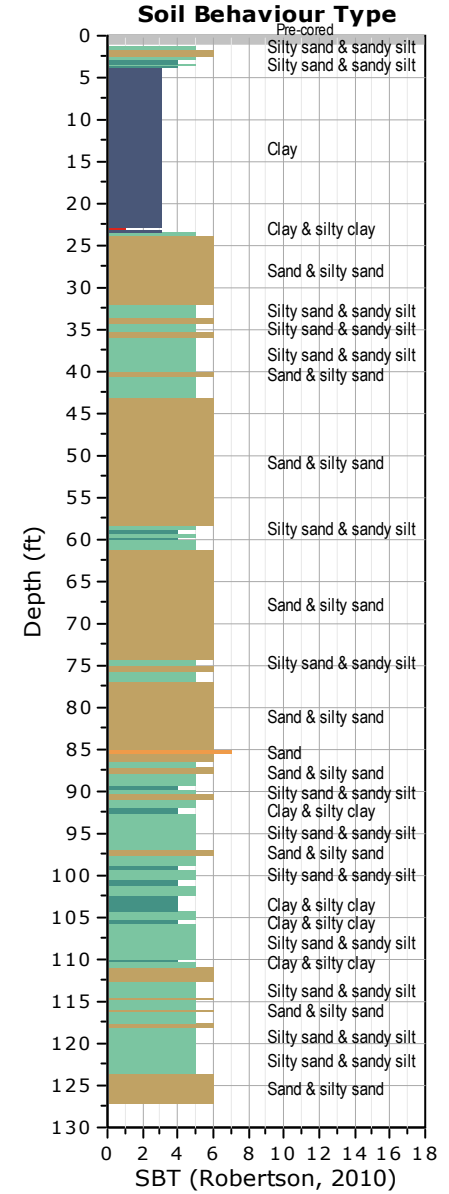
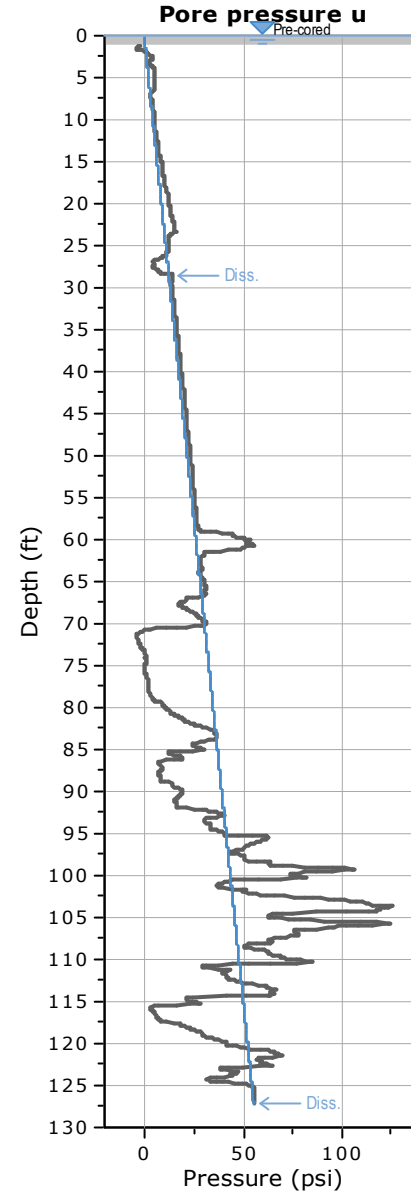
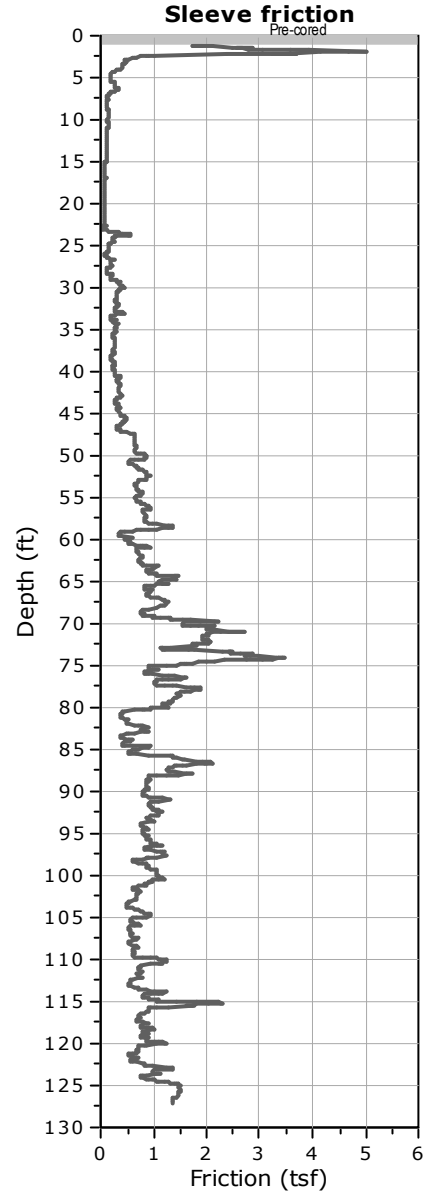
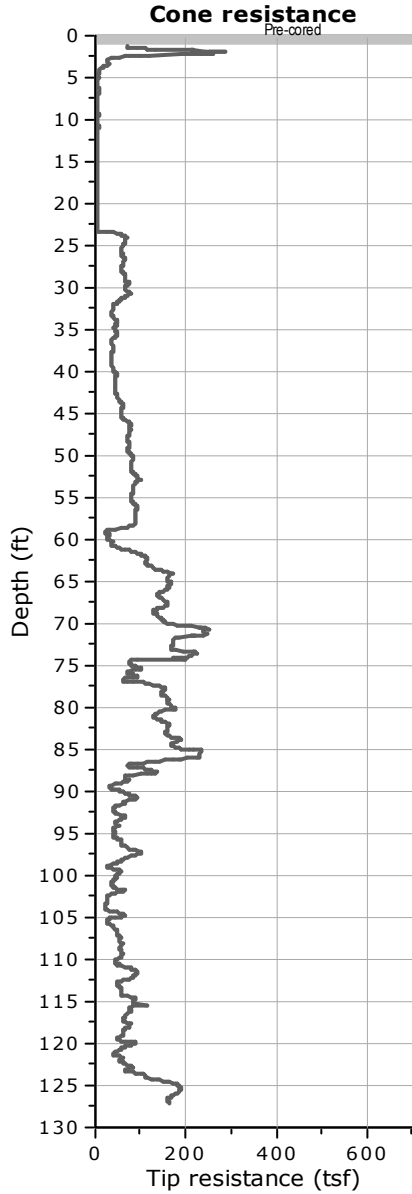
Piezocone Dissipation Test: C-002-1-23
Depth: 104.68 (ft)





Project: STA-241-17.37

Location: Stark County



Dissipation Tests Results

Dissipation tests

Dissipation tests consists of stopping the piezocone penetration and observing porepressures (u) with elapsed time (t). The data are automatic recorded by the field computer and should take place until a minimum of 50% dissipation.

The porepressures are plotted as a function of square root of (t). The graphical technique suggested by Robertson and Campanella (1989), yields a value for t_{50} , which corresponds to the time for 50% consolidation.

The value of the coefficient of consolidation in the radial or horizontal direction c_h was then calculated by Houlsby and Teh's (1988) theory using the following equation:

$$c_h = \frac{T \times r^2 \times I_r^{0.5}}{t_{50}}$$

where:

- T: time factor given by Houlsby and Teh's (1988) theory corresponding to the porepressure position
- r: piezocone radius
- I_r : stiffness index, equal to shear modulus G divided by the undrained strength of clay (S_u).
- t_{50} : time corresponding to 50% consolidation

Permeability estimates based on dissipation test

The dissipation of pore pressures during a CPTu dissipation test is controlled by the coefficient of consolidation in the horizontal direction (c_h) which is influenced by a combination of the soil permeability (k_h) and compressibility (M), as defined by the following:

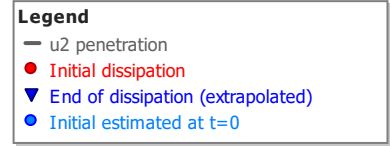
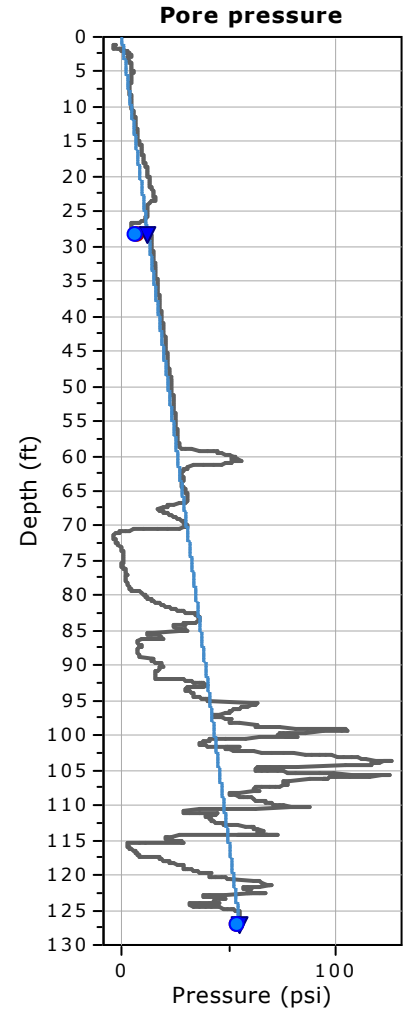
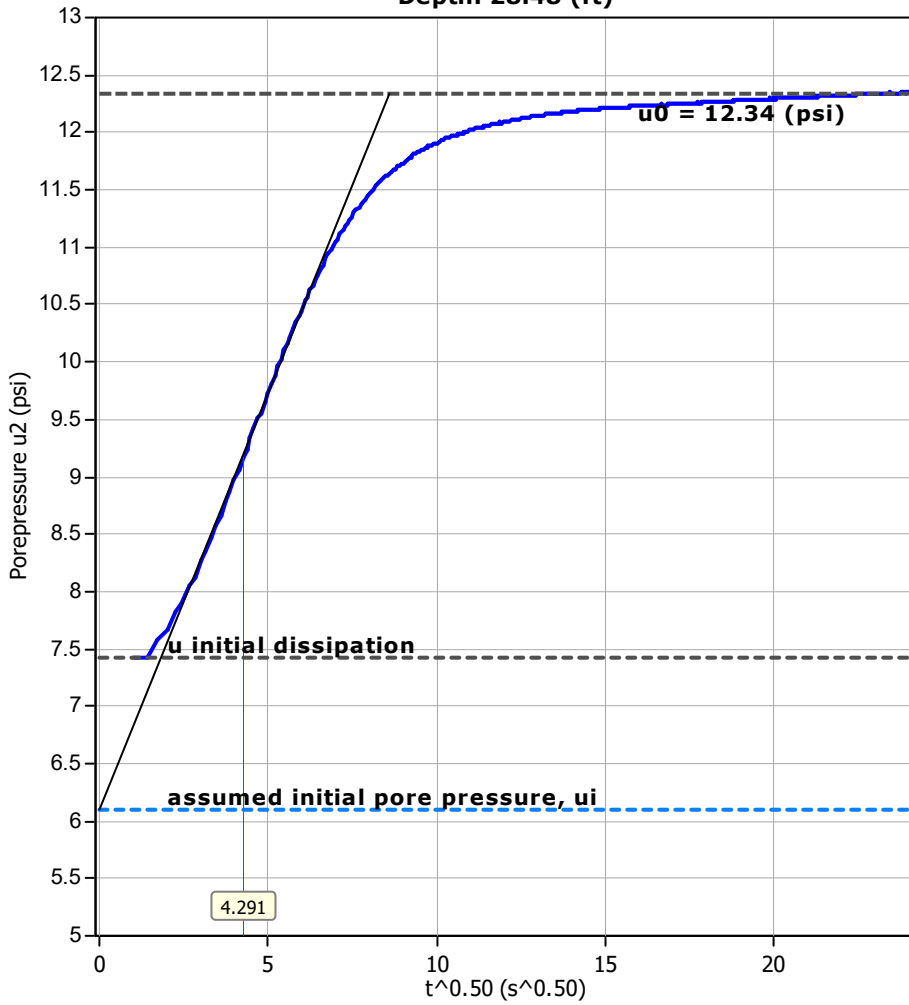
$$k_h = c_h \times \gamma_w / M$$

where: M is the 1-D constrained modulus and γ_w is the unit weight of water, in compatible units.

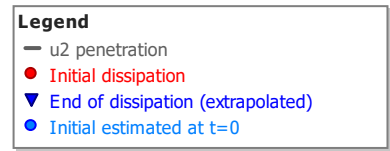
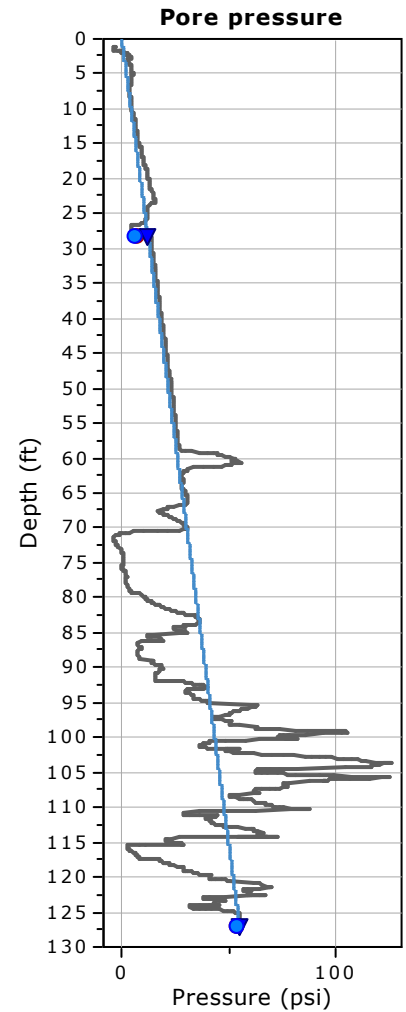
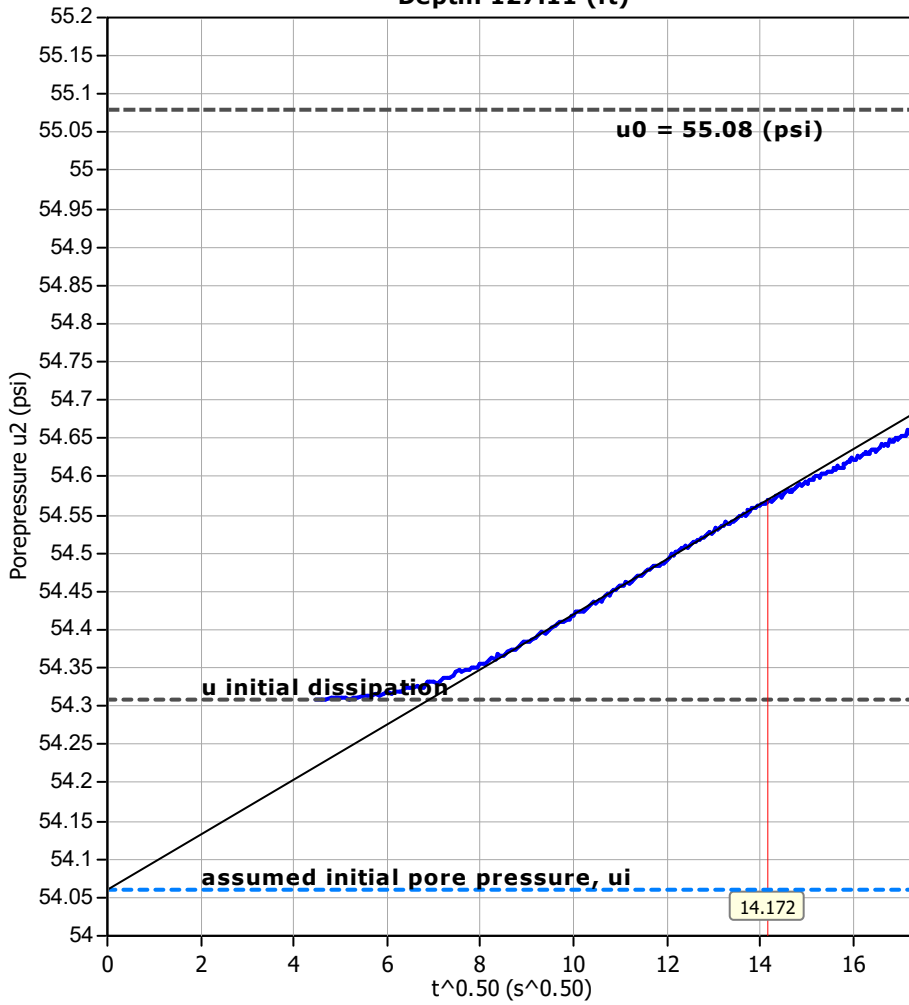
Tabular results

CPTU Borehole	Depth (ft)	$(t_{50})^{0.50}$	t_{50} (s)	t_{50} (years)	G/ S_u	c_h (ft ² /s)	c_h (ft ² /year)	M (tsf)	k_h (ft/s)
C-002-2-23	28.48	4.3	18	5.84E-007	100.00	6.84E-004	21577	504.67	4.23E-008
C-002-2-23	127.11	14.2	201	6.37E-006	100.00	6.27E-005	1978	1891.38	1.04E-009

Piezcone Dissipation Test: C-002-2-23
Depth: 28.48 (ft)



Piezocone Dissipation Test: C-002-2-23
Depth: 127.11 (ft)





Project: STA-241-17.37

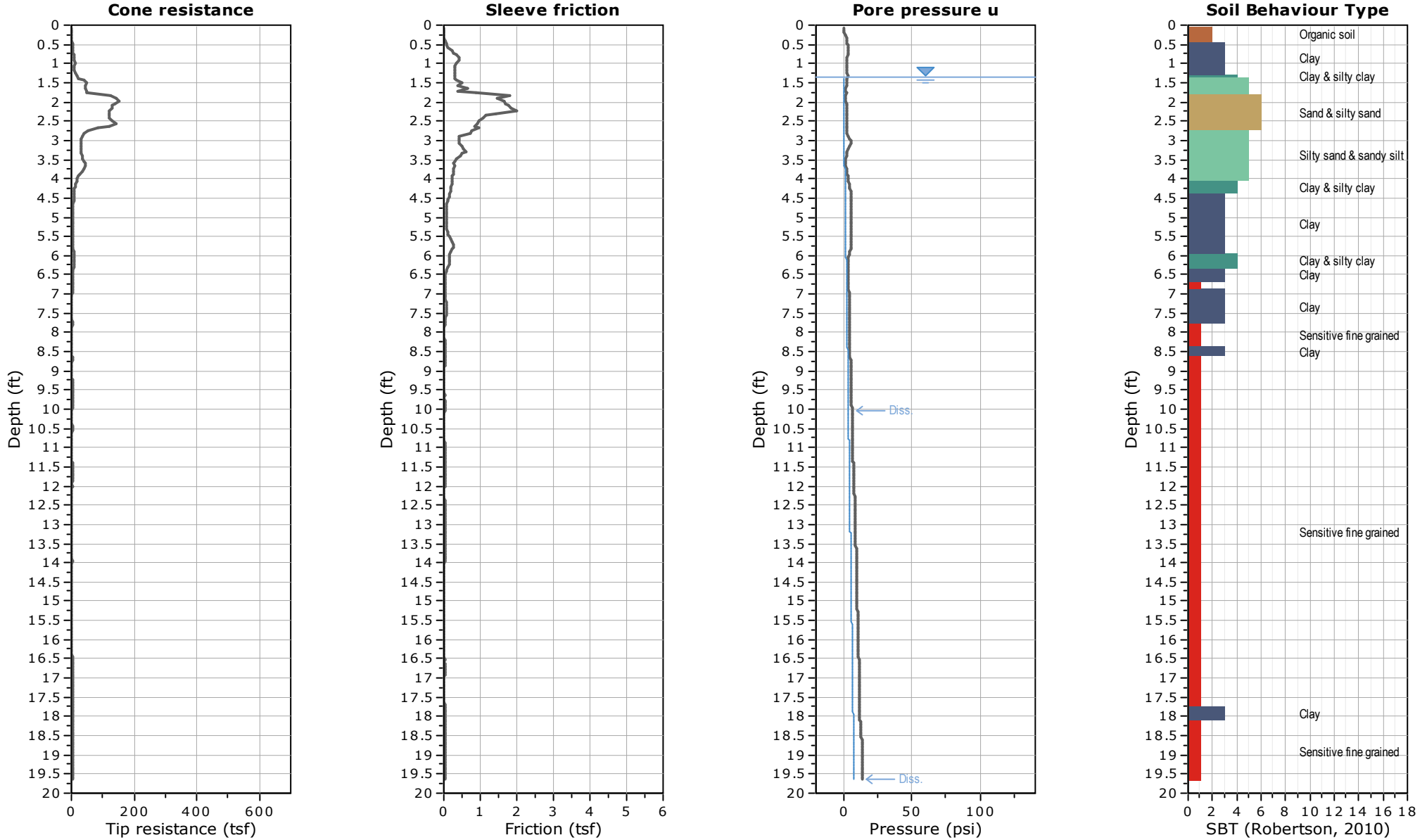
Location: Stark County

CPT: C-003-1-23

Total depth: 19.65 ft, Date: 11/14/2023

Surface Elevation: 1070.34 ft

Coords: lat 40.898488° lon -81.470458°



Project: STA-241-17.37

Location: Stark County

Dissipation Tests Results

Dissipation tests

Dissipation tests consists of stopping the piezocone penetration and observing porepressures (u) with elapsed time (t). The data are automatic recorded by the field computer and should take place until a minimum of 50% dissipation.

The porepressures are plotted as a function of square root of (t). The graphical technique suggested by Robertson and Campanella (1989), yields a value for t_{50} , which corresponds to the time for 50% consolidation.

The value of the coefficient of consolidation in the radial or horizontal direction c_h was then calculated by Houlsby and Teh's (1988) theory using the following equation:

$$c_h = \frac{T \times r^2 \times I_r^{0.5}}{t_{50}}$$

where:

T: time factor given by Houlsby and Teh's (1988) theory corresponding to the porepressure position

r: piezocone radius

I_r : stiffness index, equal to shear modulus G divided by the undrained strength of clay (S_u).

t_{50} : time corresponding to 50% consolidation

Permeability estimates based on dissipation test

The dissipation of pore pressures during a CPTu dissipation test is controlled by the coefficient of consolidation in the horizontal direction (c_h) which is influenced by a combination of the soil permeability (k_h) and compressibility (M), as defined by the following:

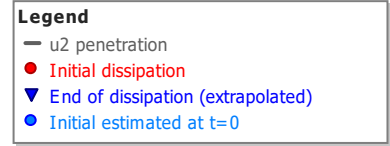
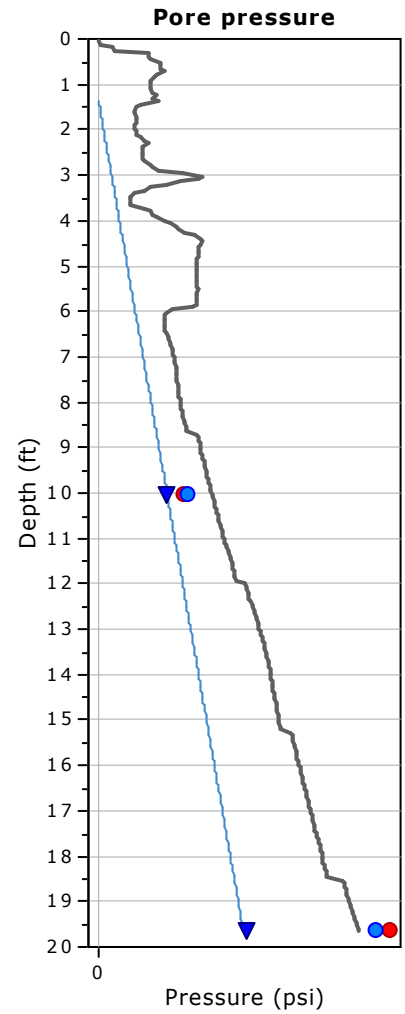
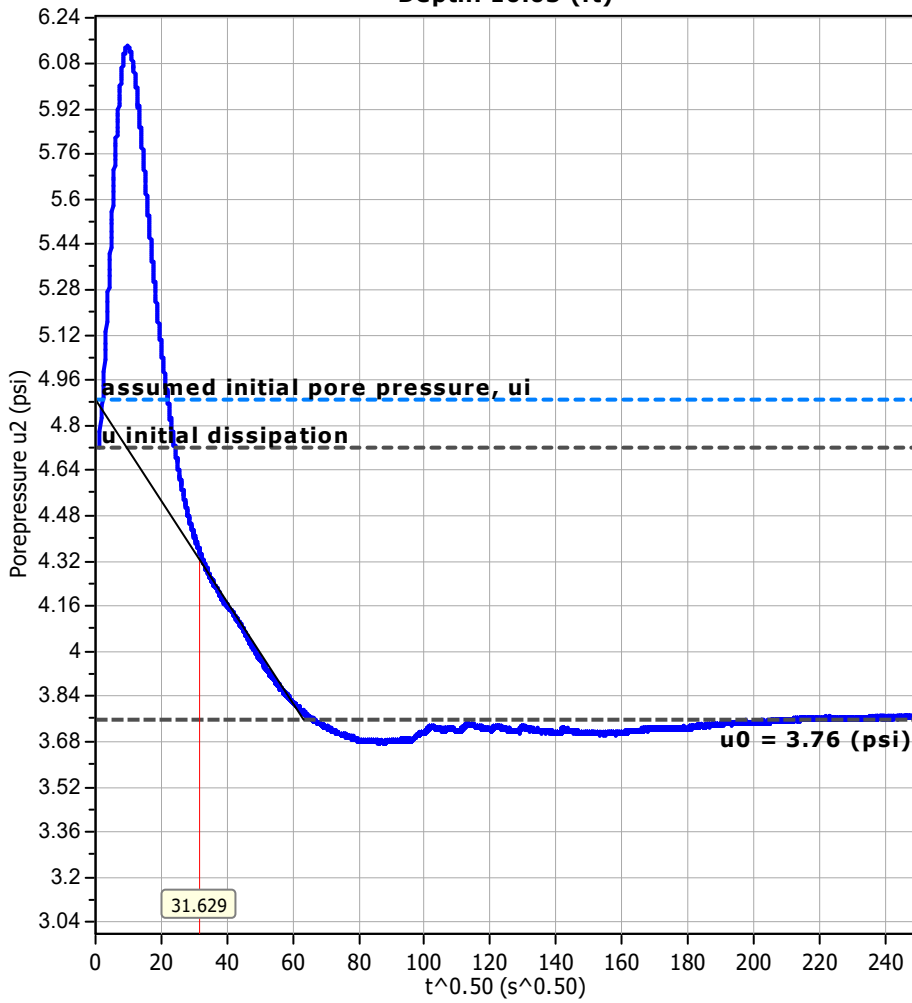
$$k_h = c_h \times \gamma_w / M$$

where: M is the 1-D constrained modulus and γ_w is the unit weight of water, in compatible units.

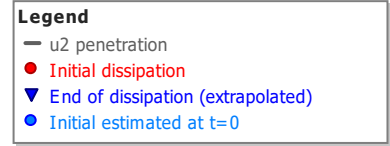
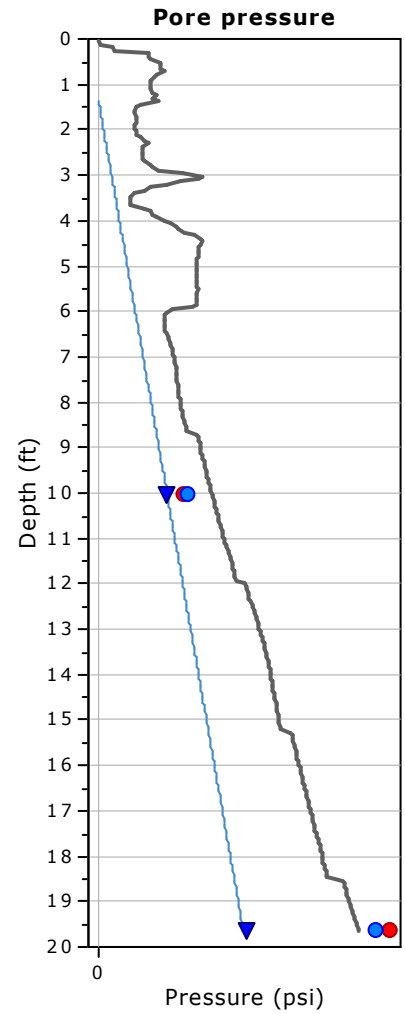
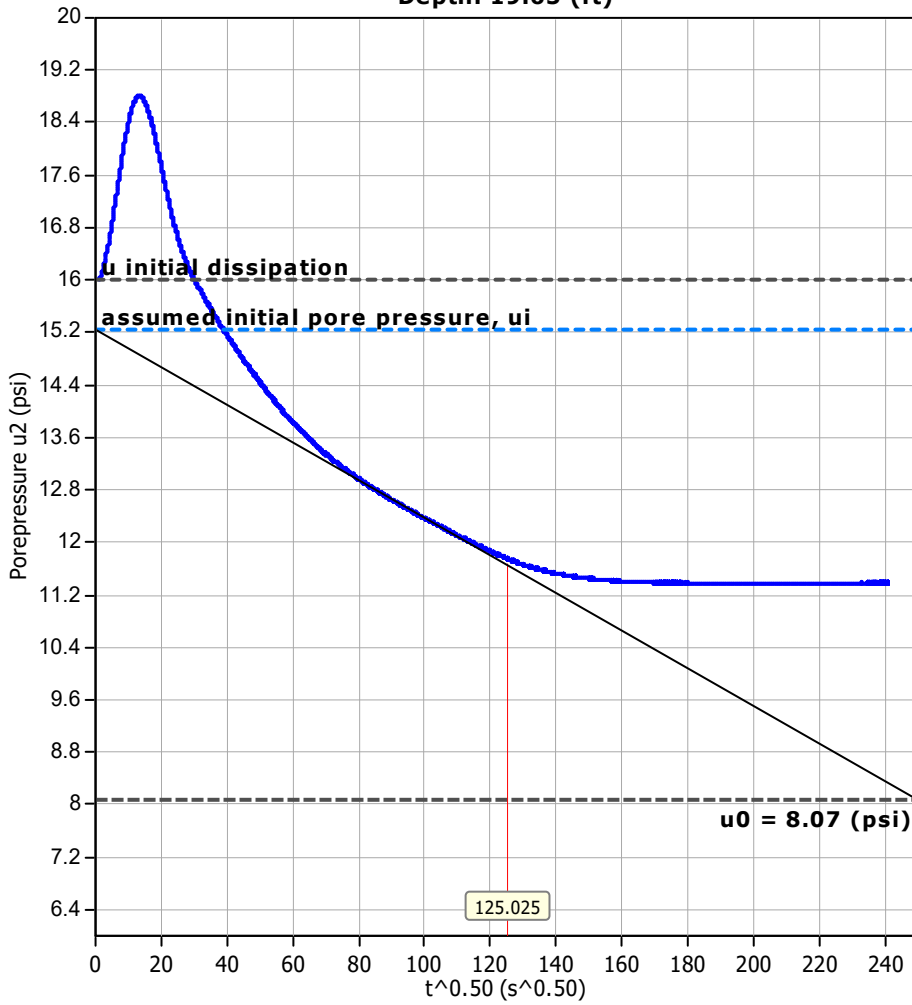
Tabular results

CPTU Borehole	Depth (ft)	$(t_{50})^{0.50}$	t_{50} (s)	t_{50} (years)	G/ S_u	c_h (ft ² /s)	c_h (ft ² /year)	M (tsf)	k_h (ft/s)
C-003-1-23	19.65	125.0	15632	4.96E-004	391519.53	5.04E-005	1590	19.20	8.20E-008
C-003-1-23	10.03	31.6	1001	3.17E-005	98.76	1.25E-005	395	17.61	2.22E-008

Piezocone Dissipation Test: C-003-1-23
Depth: 10.03 (ft)



Piezocone Dissipation Test: C-003-1-23
Depth: 19.65 (ft)





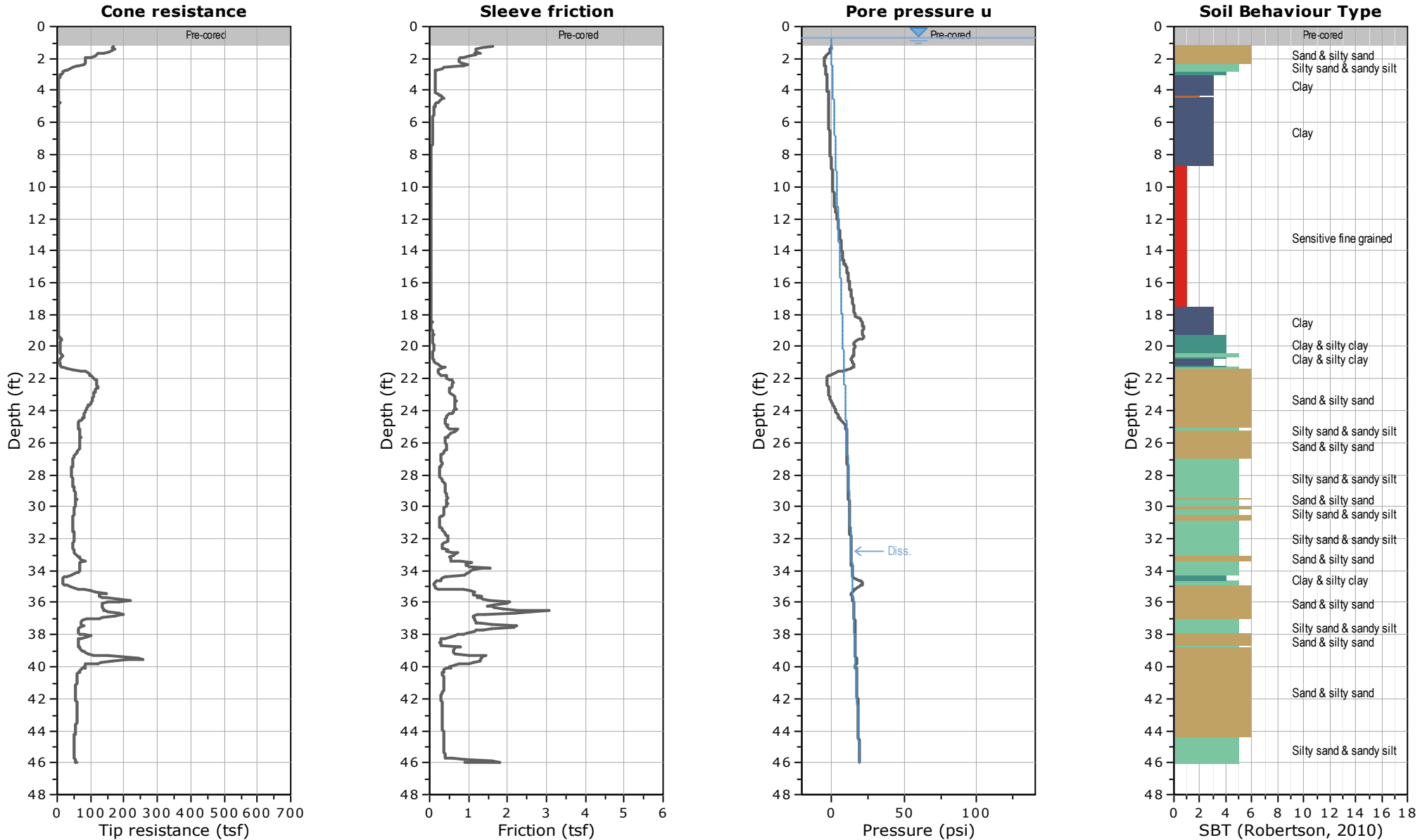
Project: STA-241-17.37

Location: Stark County

Total depth: 46.01 ft, Date: 11/15/2023

Surface Elevation: 1070.96 ft

Coords: lat 40.898459° lon -81.469945°



Project: STA-241-17.37

Location: Stark County

Dissipation Tests Results

Dissipation tests

Dissipation tests consists of stopping the piezocone penetration and observing porepressures (u) with elapsed time (t). The data are automatic recorded by the field computer and should take place until a minimum of 50% dissipation.

The porepressures are plotted as a function of square root of (t). The graphical technique suggested by Robertson and Campanella (1989), yields a value for t_{50} , which corresponds to the time for 50% consolidation.

The value of the coefficient of consolidation in the radial or horizontal direction c_h was then calculated by Houlsby and Teh's (1988) theory using the following equation:

$$c_h = \frac{T \times r^2 \times I_r^{0.5}}{t_{50}}$$

where:

T: time factor given by Houlsby and Teh's (1988) theory corresponding to the porepressure position

r: piezocone radius

I_r : stiffness index, equal to shear modulus G divided by the undrained strength of clay (S_u).

t_{50} : time corresponding to 50% consolidation

Permeability estimates based on dissipation test

The dissipation of pore pressures during a CPTu dissipation test is controlled by the coefficient of consolidation in the horizontal direction (c_h) which is influenced by a combination of the soil permeability (k_h) and compressibility (M), as defined by the following:

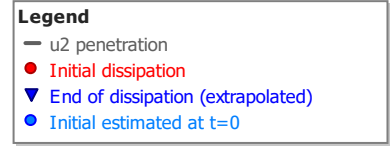
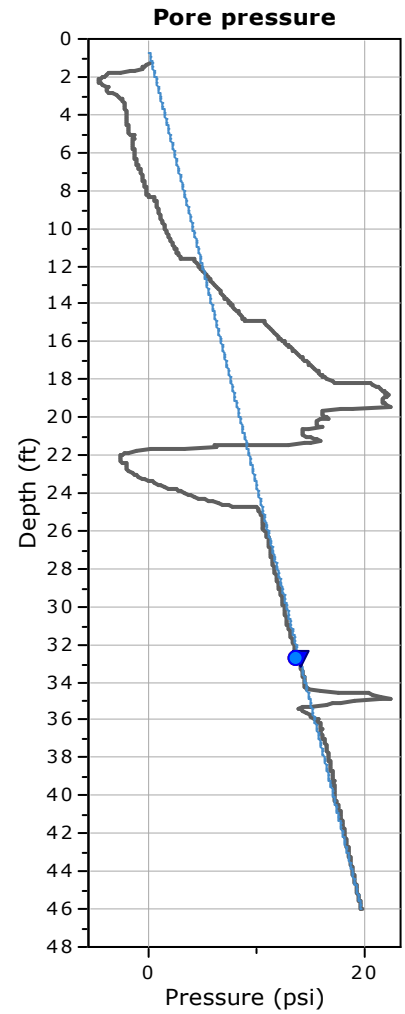
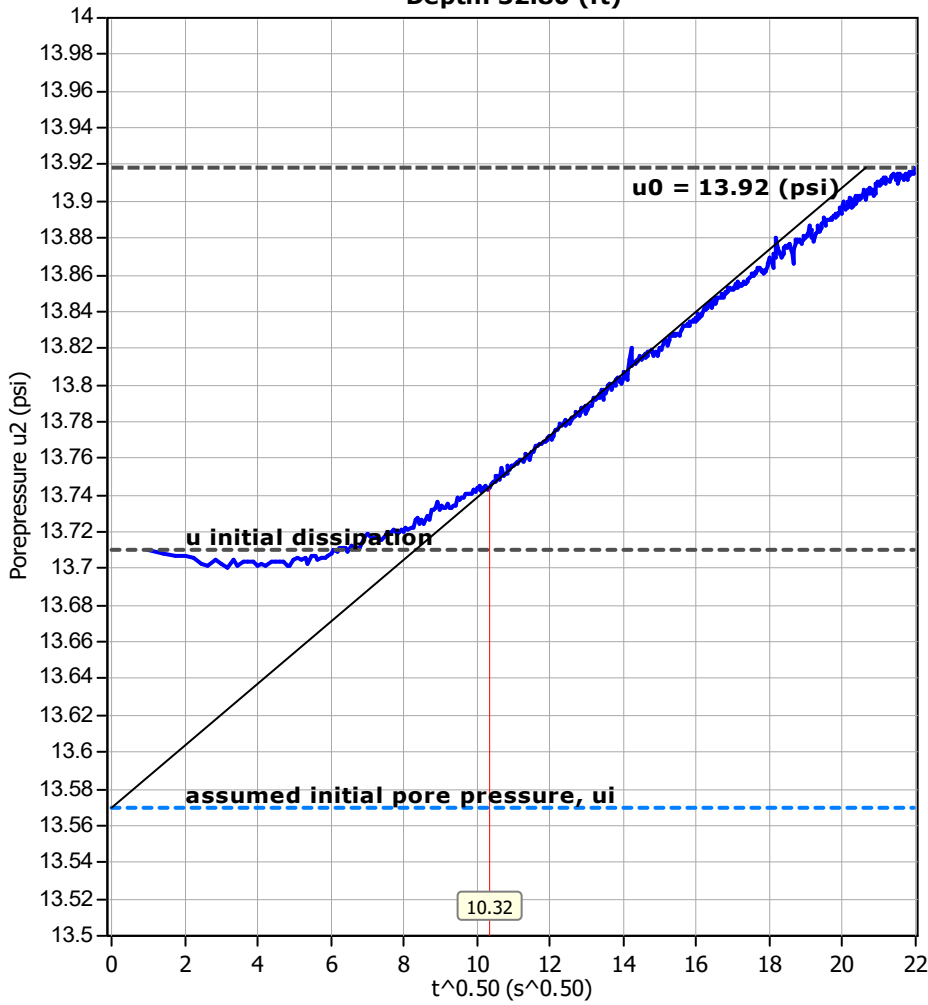
$$k_h = c_h \times \gamma_w / M$$

where: M is the 1-D constrained modulus and γ_w is the unit weight of water, in compatible units.

Tabular results

CPTU Borehole	Depth (ft)	$(t_{50})^{0.50}$	t_{50} (s)	t_{50} (years)	G/ S_u	c_h (ft ² /s)	c_h (ft ² /year)	M (tsf)	k_h (ft/s)
C-003-2-23	32.80	10.3	106	3.38E-006	100.00	1.18E-004	3730	602.81	6.13E-009

Piezocone Dissipation Test: C-003-2-23
Depth: 32.80 (ft)





Project: STA-241-17.37

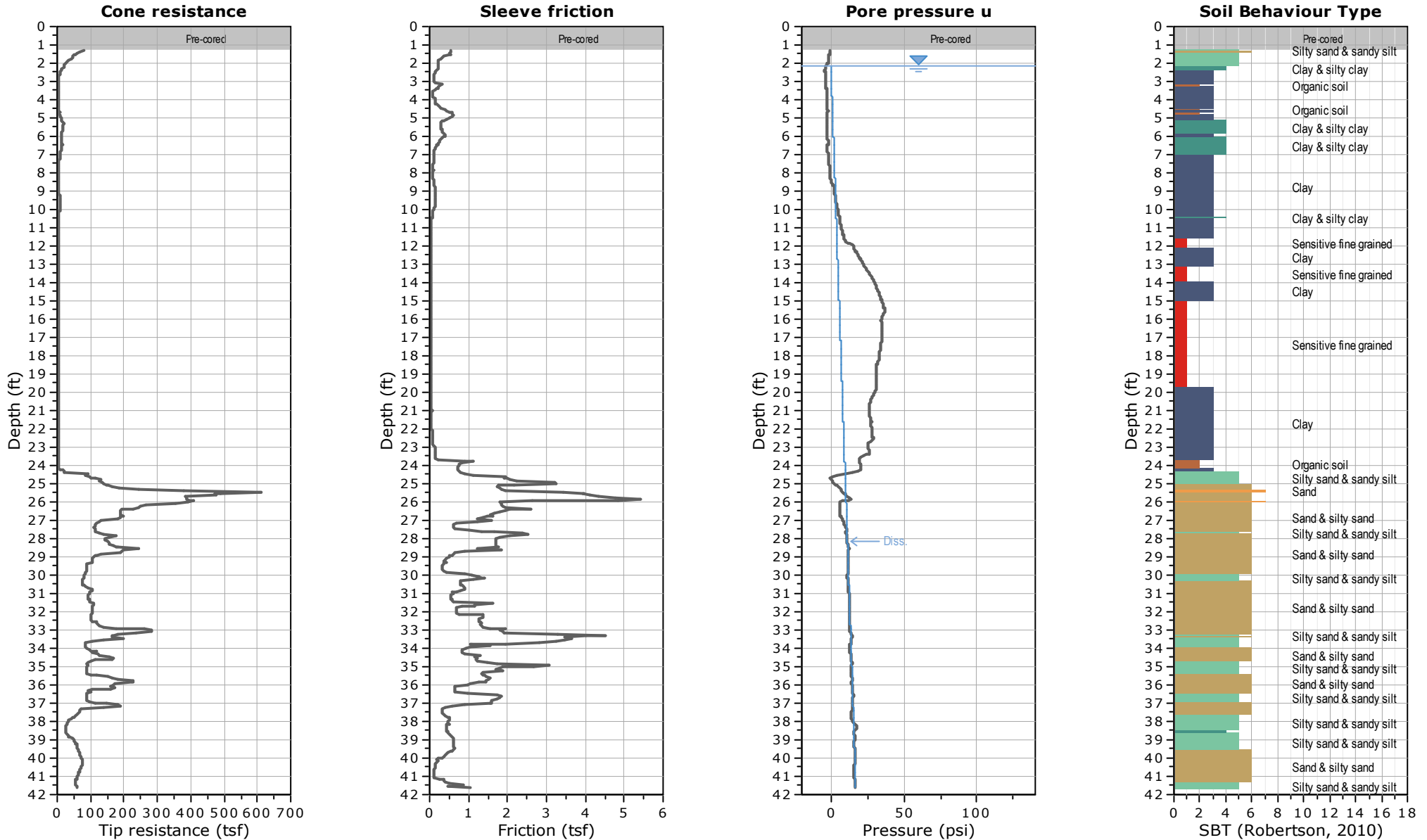
Location: Stark County

CPT: C-003-3-23

Total depth: 41.63 ft, Date: 11/15/2023

Surface Elevation: 1071.83 ft

Coords: lat 40.898597° lon -81.469487°



Dissipation Tests Results

Dissipation tests

Dissipation tests consists of stopping the piezocone penetration and observing porepressures (u) with elapsed time (t). The data are automatic recorded by the field computer and should take place until a minimum of 50% dissipation.

The porepressures are plotted as a function of square root of (t). The graphical technique suggested by Robertson and Campanella (1989), yields a value for t_{50} , which corresponds to the time for 50% consolidation.

The value of the coefficient of consolidation in the radial or horizontal direction c_h was then calculated by Houlsby and Teh's (1988) theory using the following equation:

$$c_h = \frac{T \times r^2 \times I_r^{0.5}}{t_{50}}$$

where:

- T: time factor given by Houlsby and Teh's (1988) theory corresponding to the porepressure position
- r: piezocone radius
- I_r : stiffness index, equal to shear modulus G divided by the undrained strength of clay (S_u).
- t_{50} : time corresponding to 50% consolidation

Permeability estimates based on dissipation test

The dissipation of pore pressures during a CPTu dissipation test is controlled by the coefficient of consolidation in the horizontal direction (c_h) which is influenced by a combination of the soil permeability (k_h) and compressibility (M), as defined by the following:

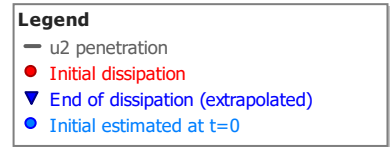
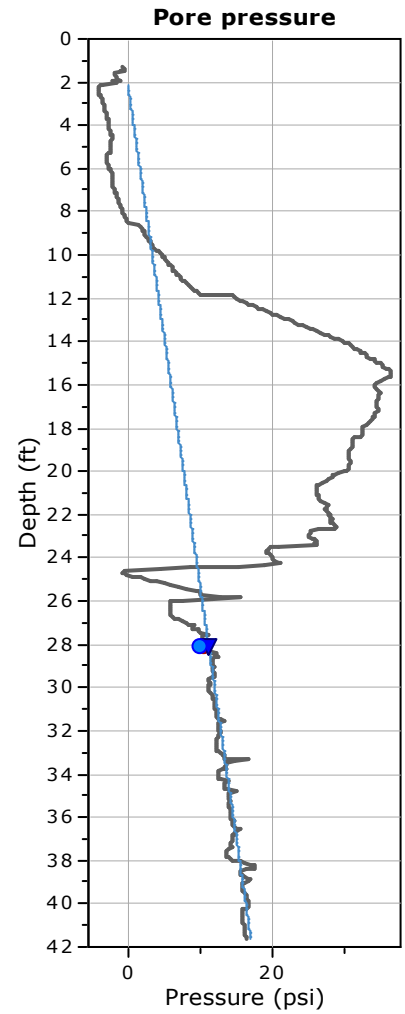
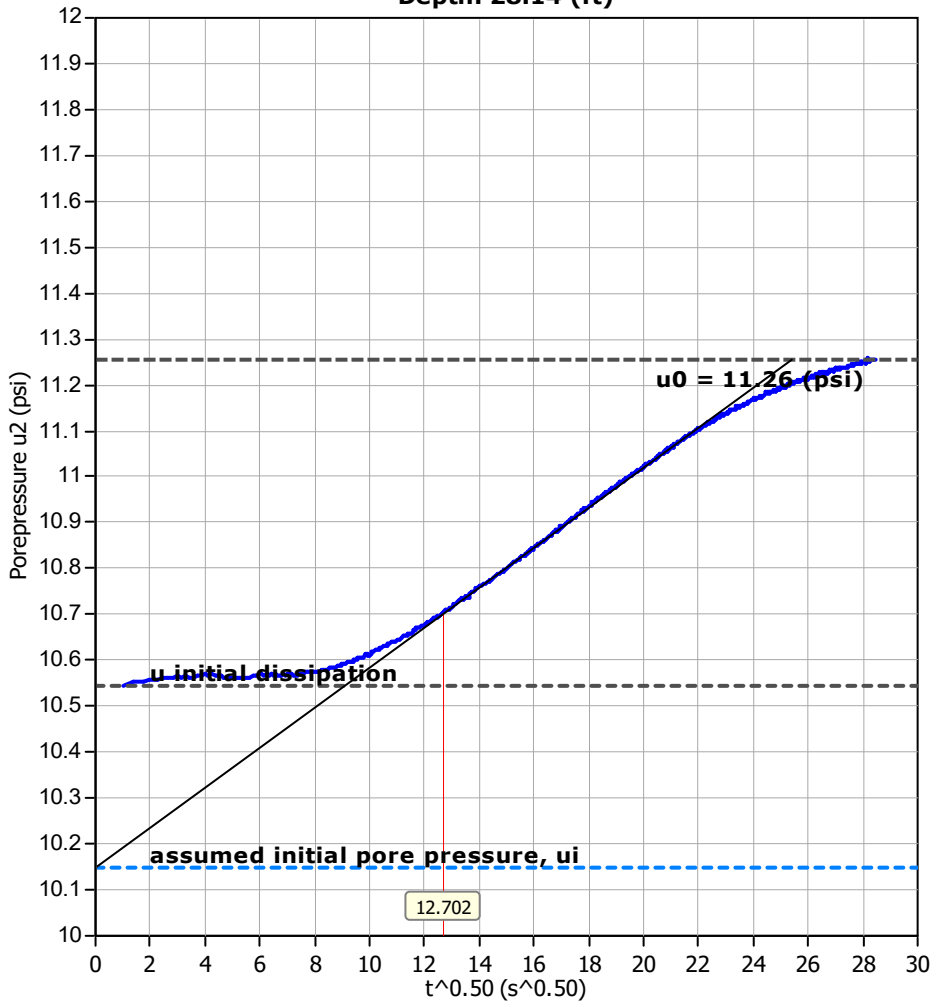
$$k_h = c_h \times \gamma_w / M$$

where: M is the 1-D constrained modulus and γ_w is the unit weight of water, in compatible units.

Tabular results

CPTU Borehole	Depth (ft)	$(t_{50})^{0.50}$	t_{50} (s)	t_{50} (years)	G/ S_u	c_h (ft ² /s)	c_h (ft ² /year)	M (tsf)	k_h (ft/s)
C-003-3-23	28.14	12.7	161	5.12E-006	100.00	7.81E-005	2462	1187.32	2.05E-009

Piezocone Dissipation Test: C-003-3-23
Depth: 28.14 (ft)





Project: STA-241-17.37

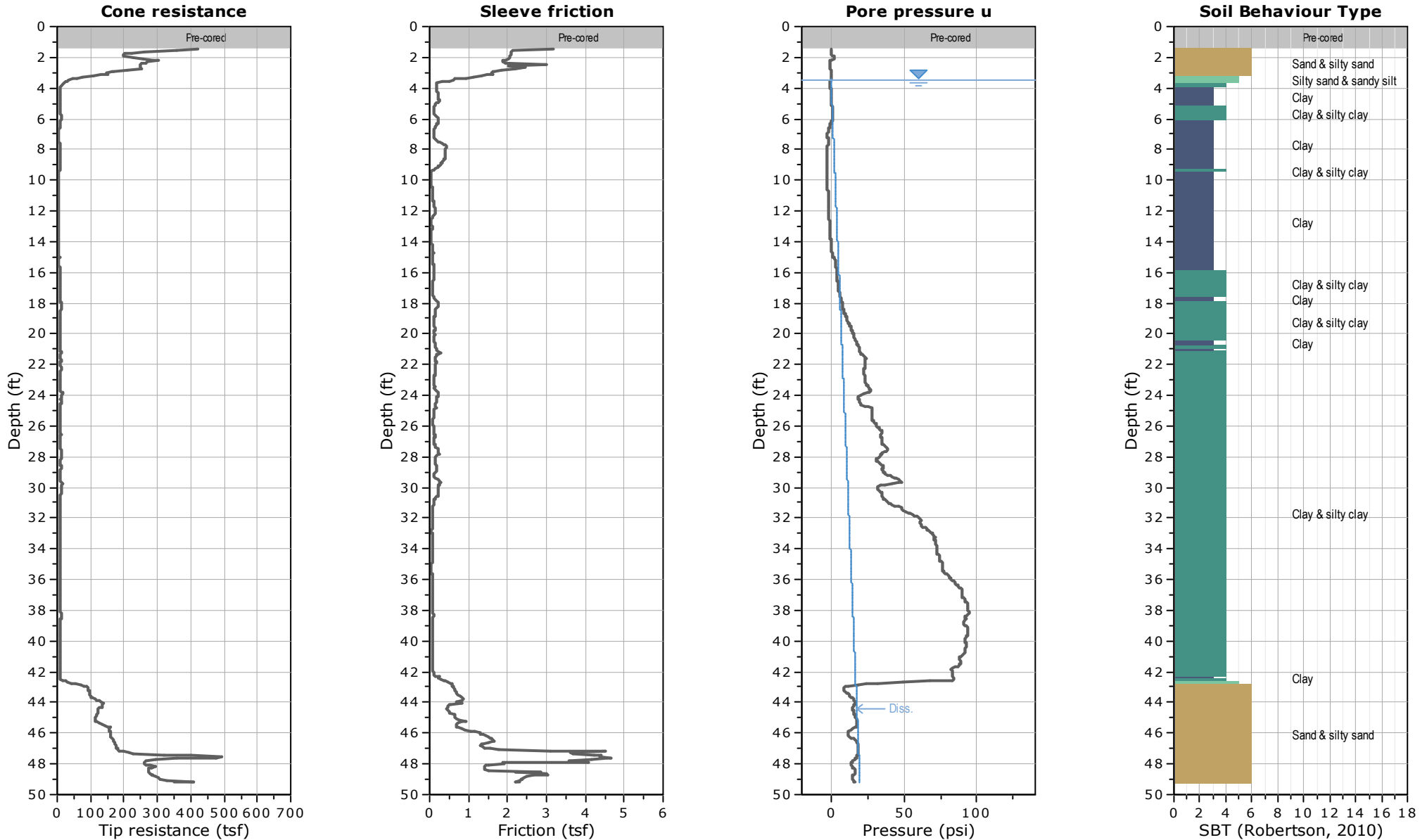
Location: Stark County

CPT: C-004-1-23

Total depth: 49.21 ft, Date: 11/15/2023

Surface Elevation: 1074.73 ft

Coords: lat 40.898759° lon -81.469077°



Project: STA-241-17.37

Location: Stark County

Dissipation Tests Results

Dissipation tests

Dissipation tests consists of stopping the piezocone penetration and observing porepressures (u) with elapsed time (t). The data are automatic recorded by the field computer and should take place until a minimum of 50% dissipation.

The porepressures are plotted as a function of square root of (t). The graphical technique suggested by Robertson and Campanella (1989), yields a value for t_{50} , which corresponds to the time for 50% consolidation.

The value of the coefficient of consolidation in the radial or horizontal direction c_h was then calculated by Houlsby and Teh's (1988) theory using the following equation:

$$c_h = \frac{T \times r^2 \times I_r^{0.5}}{t_{50}}$$

where:

T: time factor given by Houlsby and Teh's (1988) theory corresponding to the porepressure position

r: piezocone radius

I_r : stiffness index, equal to shear modulus G divided by the undrained strength of clay (S_u).

t_{50} : time corresponding to 50% consolidation

Permeability estimates based on dissipation test

The dissipation of pore pressures during a CPTu dissipation test is controlled by the coefficient of consolidation in the horizontal direction (c_h) which is influenced by a combination of the soil permeability (k_h) and compressibility (M), as defined by the following:

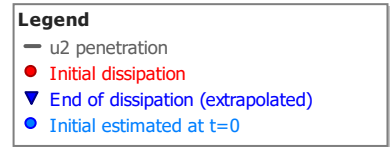
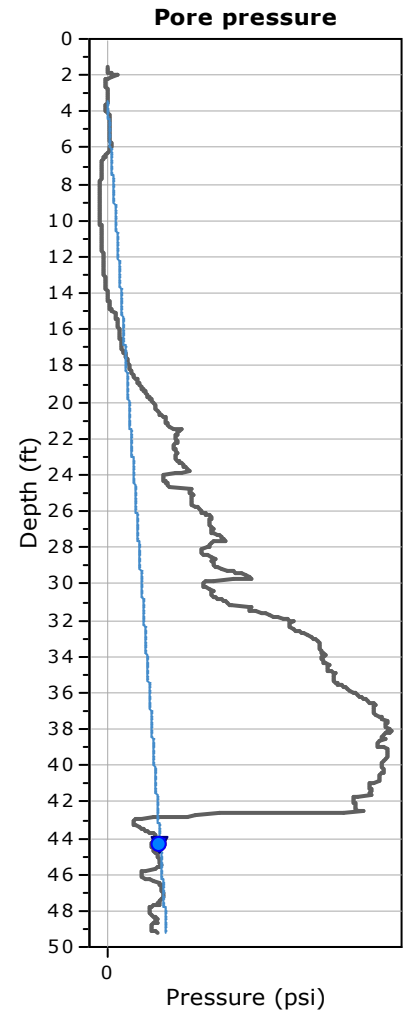
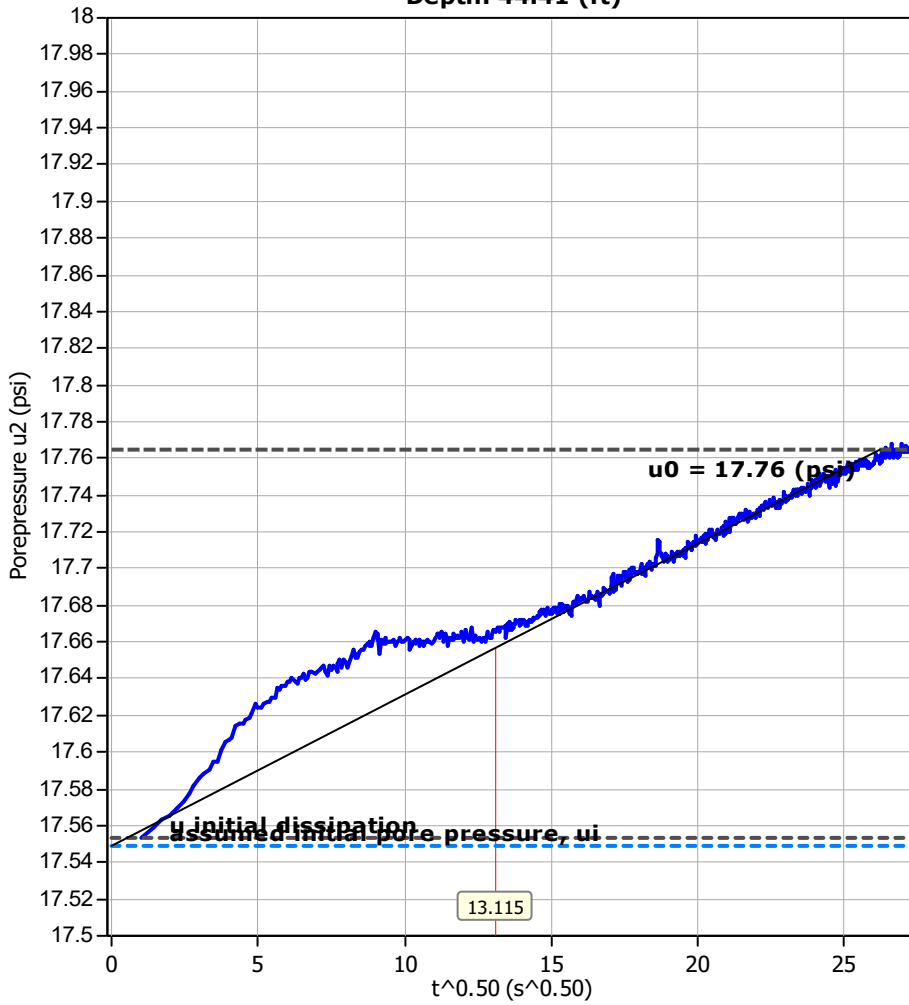
$$k_h = c_h \times \gamma_w / M$$

where: M is the 1-D constrained modulus and γ_w is the unit weight of water, in compatible units.

Tabular results

CPTU Borehole	Depth (ft)	$(t_{50})^{0.50}$	t_{50} (s)	t_{50} (years)	G/ S_u	c_h (ft ² /s)	c_h (ft ² /year)	M (tsf)	k_h (ft/s)
C-004-1-23	44.41	13.1	172	5.45E-006	100.00	7.32E-005	2310	818.69	2.79E-009

Piezocene Dissipation Test: C-004-1-23
Depth: 44.41 (ft)





Project: STA-241-17.37

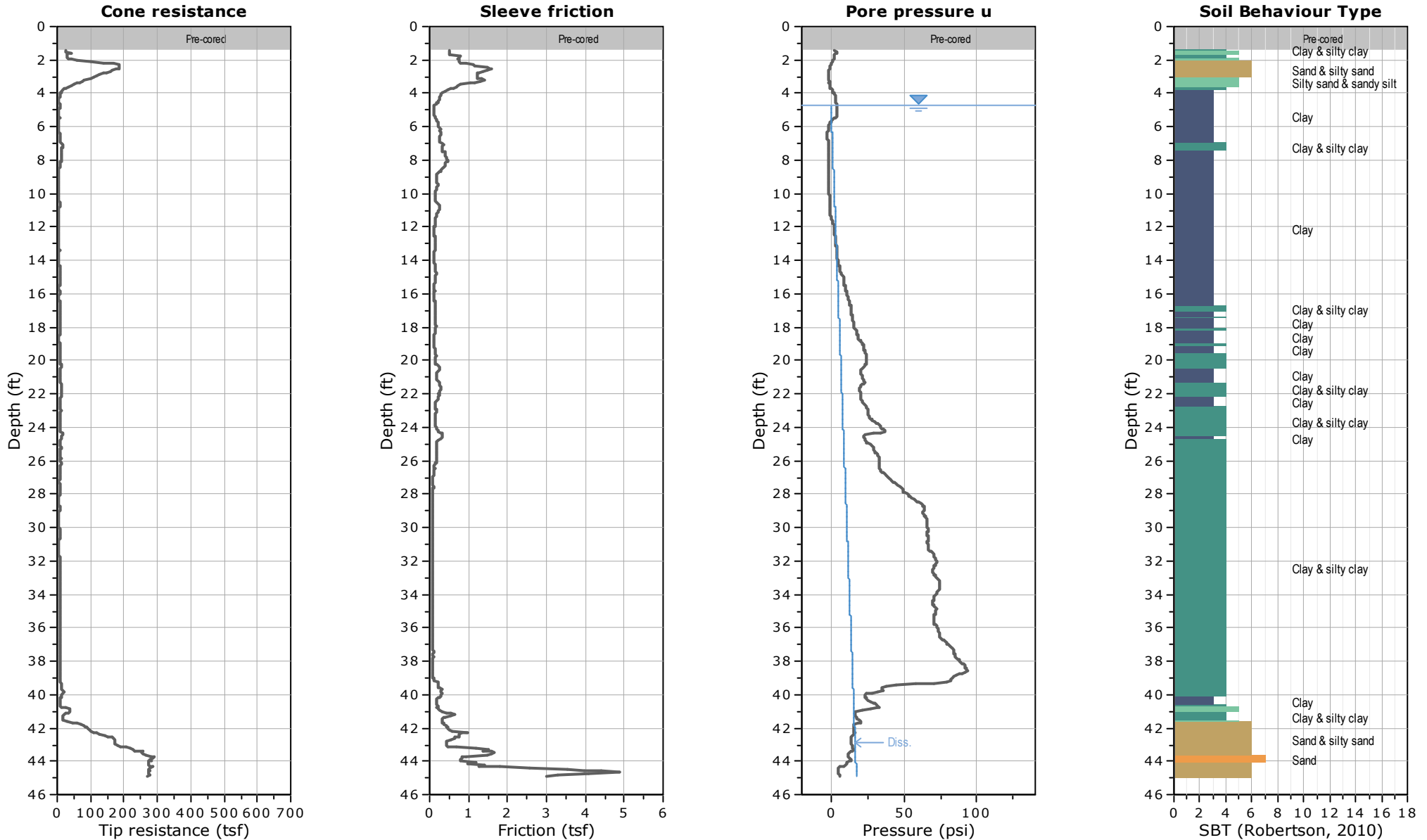
Location: Stark County

CPT: C-004-2-23

Total depth: 44.87 ft, Date: 11/15/2023

Surface Elevation: 1077.61 ft

Coords: lat 40.898985° lon -81.46871°



Project: STA-241-17.37

Location: Stark County

Dissipation Tests Results

Dissipation tests

Dissipation tests consists of stopping the piezocone penetration and observing porepressures (u) with elapsed time (t). The data are automatic recorded by the field computer and should take place until a minimum of 50% dissipation.

The porepressures are plotted as a function of square root of (t). The graphical technique suggested by Robertson and Campanella (1989), yields a value for t_{50} , which corresponds to the time for 50% consolidation.

The value of the coefficient of consolidation in the radial or horizontal direction c_h was then calculated by Houlsby and Teh's (1988) theory using the following equation:

$$c_h = \frac{T \times r^2 \times I_r^{0.5}}{t_{50}}$$

where:

T: time factor given by Houlsby and Teh's (1988) theory corresponding to the porepressure position

r: piezocone radius

I_r : stiffness index, equal to shear modulus G divided by the undrained strength of clay (S_u).

t_{50} : time corresponding to 50% consolidation

Permeability estimates based on dissipation test

The dissipation of pore pressures during a CPTu dissipation test is controlled by the coefficient of consolidation in the horizontal direction (c_h) which is influenced by a combination of the soil permeability (k_h) and compressibility (M), as defined by the following:

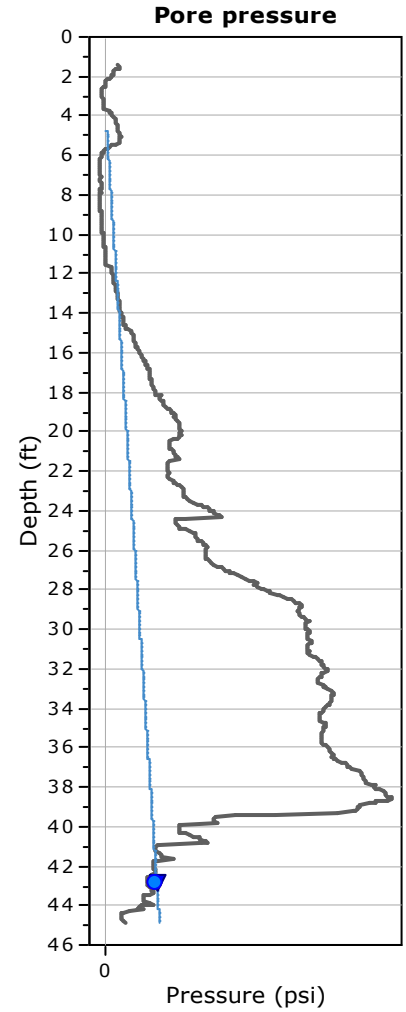
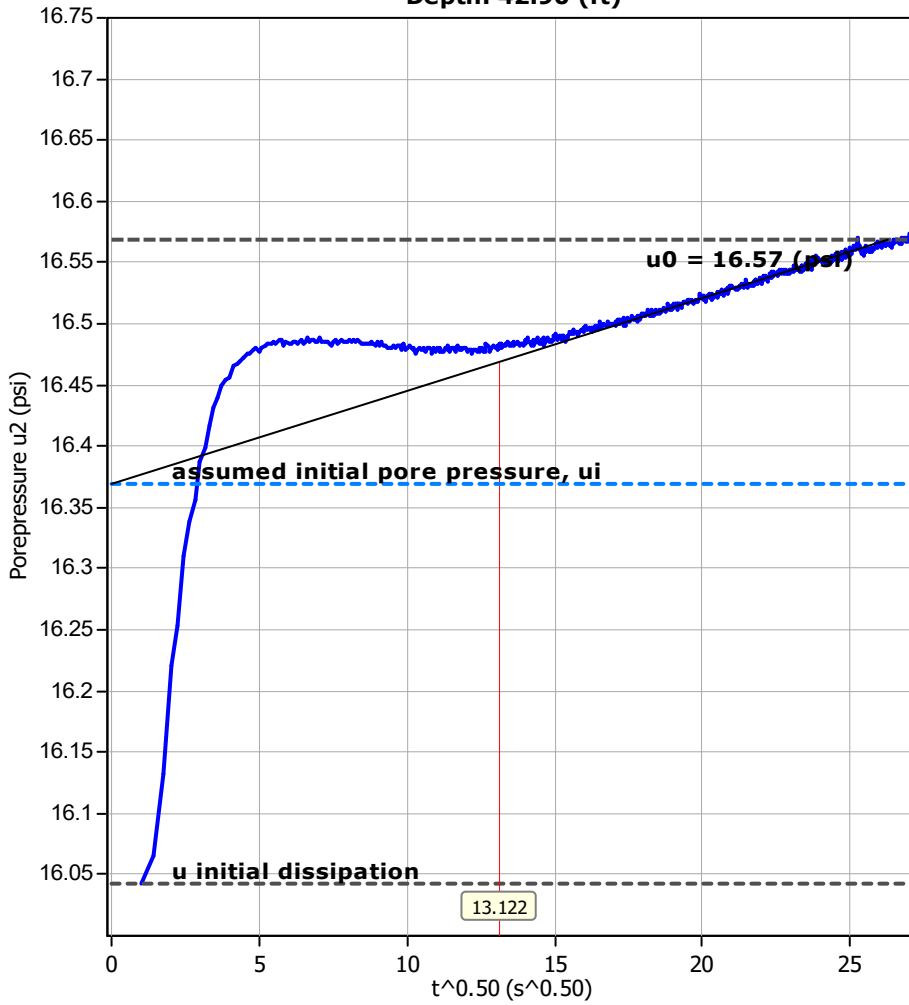
$$k_h = c_h \times \gamma_w / M$$

where: M is the 1-D constrained modulus and γ_w is the unit weight of water, in compatible units.

Tabular results

CPTU Borehole	Depth (ft)	$(t_{50})^{0.50}$	t_{50} (s)	t_{50} (years)	G/ S_u	c_h (ft ² /s)	c_h (ft ² /year)	M (tsf)	k_h (ft/s)
C-004-2-23	42.90	13.1	172	5.46E-006	100.00	7.32E-005	2308	879.61	2.60E-009

Piezocone Dissipation Test: C-004-2-23
Depth: 42.90 (ft)



Legend

- u2 penetration
- Initial dissipation
- ▼ End of dissipation (extrapolated)
- Initial estimated at t=0



Appendix IV – Analysis Output

SUBGRADE ANALYSIS

OHIO DEPARTMENT OF TRANSPORTATION**OFFICE OF GEOTECHNICAL ENGINEERING****PLAN SUBGRADES****Geotechnical Design Manual Section 600****STA-241-17.37
119585****PROJECT DESCRIPTION - Roundabout to be constructed at intersection of SR 241 and Shuffel St,
along with the Sk e a k e t a replacement of existing bridge over Nimisila Creek****S&ME, Inc.****Prepared By: Brian K. Sears, PE
Date prepared: Wednesday, April 2, 2025****S&ME, Inc.
6190 Enterprise Court
Dublin, OH 43016****614-793-2226
bsears@smeinc.com****NO. OF BORINGS: 10**

#	Boring ID	Alignment	Station	Offset	Dir	Drill Rig	ER	Boring EL.	Proposed Subgrade EL	Cut Fill
1	B-001-0-23	SR 241 CL Const.	586+42	3	Right	OTB MOBILE B-57	90	1078.0	1077.3	0.7 C
2	B-002-0-23	SR 241 CL Const.	588+17	5	Right	OTB MOBILE B-57	90	1072.8	1074.7	1.9 F
3	B-003-0-23	SR 241 CL Const.	588+70	6	Right	OTB MOBILE B-57	90	1071.7	1074.1	2.4 F
4	B-004-0-23	SR 241 CL Const.	592+49	5	Right	OTB MOBILE B-57	90	1074.6	1072.9	1.7 C
5	B-005-0-23	SR 241 CL Const.	596+47	2	Right	OTB MOBILE B-57	90	1086.9	1084.9	2.0 C
6	B-006-0-23	SR 241 CL R/W	200+01	16	Right	OTB MOBILE B-57	90	1091.6	1093.1	1.5 F
7	H-006-1-23	SR 241 CL R/W	200+97	137	Left	Hand Auger	0	1089.4	1089.9	0.5 F
8	H-006-2-23	SR 241 CL R/W	203+68	54	Left	Hand Auger	0	1090.9	1091.1	0.2 F
9	B-007-0-23	SR 241 CL R/W	207+14	11	Left	OTB MOBILE B-57	90	1096.2	1094.9	1.3 C
10	B-008-0-23	CR 230 CL R/W	24+80	27	Left	OTB MOBILE B-57	90	1097.6	1096.3	1.3 C

#	Boring	Sample	Sample Depth		Subgrade Depth		Standard Penetration		HP (tsf)	Physical Characteristics					Moisture		Ohio DOT		Sulfate Content (ppm)	Problem		Excavate and Replace (Item 204)		Recommendation (Enter depth in inches)	
			From	To	From	To	N ₆₀	N _{60L}		LL	PL	PI	% Silt	% Clay	P200	M _c	M _{OPT}	Class		GI	Unsuitable	Unstable	Unsuitable		Unstable
1	B 001-0 23	SS-1	1.5	3.0	0.8	2.3	14	5	2	19	14	5	39	11	50	12	10	A-4a	3	80					WICK DRAIN AREA
		SS-2	3.0	4.5	2.3	3.8	5		0.5	19	15	4	36	7	43	15	10	A-4a	2						
		SS-3	4.5	6.0	3.8	5.3	5		0.7							14	10	A-4a	8						
		SS-4	6.0	7.5	5.3	6.8	8									13	8	A-3a							
2	B 002-0 23	SS-2	1.5	3.0	3.4	4.9	14	5	3.5							8	14	A-6a	10						WICK DRAIN AREA
		SS-3	3.5	5.0	5.4	6.9	5		0.7	40	24	16	66	19	85	34	19	A-6b							
		SS-4	5.0	6.5	6.9	8.4	0		0.3	37	20	17	60	19	79	30	16	A-6b							
		SS-5	6.5	8.0	8.4	9.9	5		1	33	22	11	76	19	95	26	17	A-6a							
3	B 003-0 23	SS-1	2.0	3.5	4.4	5.9	12	3	2	21	15	6	42	10	52	13	10	A-4a	3	260					WICK DRAIN AREA
		SS-2	3.5	5.0	5.9	7.4	3		0.1	23	15	8	48	13	61	17	10	A-4a							
		SS-3	5.0	6.5	7.4	8.9	3		0.2	47	39	8	42	16	58	72		A-8b							
		SS-4	6.5	8.0	8.9	10.4	3		0.1							29	10	A-4b							
4	B 004-0 23	SS-1	2.5	4.0	0.8	2.3	29	2		NP	NP	NP	21	2	23	8	6	A-1-b	0	320					WICK DRAIN AREA
		SS-2A	4.0	4.5	2.3	2.8	5		0.7							21	14	A-6a	10			HP & Mc			
		SS-2B/3A	4.5	6.0	2.8	4.3	2		0.5	40	22	18	43	18	61	31	17	A-6b	8						
		SS-3A	6.0	7.0	4.3	5.3	6		1.1							20	14	A-6a	10						
5	B 005-0 23	SS-1	1.5	3.0	-0.5	1.0	9	3	2.5	45	21	24	61	36	97	22	18	A-7-6	15	140			N ₆₀ & Mc	12"	Exc. & Repl. 12" 204 Geotextile
		SS-2	3.0	4.5	1.0	2.5	8		1.5	38	19	19	62	36	98	25	16	A-6b	12			HP & Mc	12"		
		SS-3	4.5	6.0	2.5	4.0	6		1							24	16	A-6b	16						
		SS-4	6.0	7.5	4.0	5.5	3		0.3							24	16	A-6b	16						
6	B 006-0 23	SS-1/2A	1.5	3.3	3.0	4.8	12	11		21	17	4	28	8	36	14	12	A-4a	0	480					
		SS-2B	3.3	4.5	4.8	6.0	11									10	10	A-2-4	0						
		SS-3/4A	4.5	6.5	6.0	8.0	12									12	10	A-2-4							
		SS-4B	6.5	7.5	8.0	9.0	17			17	16	1	28	4	32	8	8	A-3a							
7	H 006-1 23	HA-1	1.3	1.8	1.8	2.3				19	14	5	35	17	52	8	10	A-4a	3	40			N ₆₀		
8	H 006-2 23	HA-1	1.5	2.0	1.7	2.2				32	18	14	44	28	72	15	14	A-6a	9	40			N ₆₀		
9	B 007-0 23	SS-1A	2.0	2.4	0.7	1.1	29	5		27	18	9	33	14	47	15	13	A-4a	2	300					
		SS-1B/2	2.4	5.0	1.1	3.7	26			NP	NP	NP	33	1	34	7	8	A-3a	0						
		SS-3A	5.0	5.6	3.7	4.3	5		1							14	10	A-4a	8						
		SS-3B/4	5.6	8.0	4.3	6.7	9									14	8	A-3a	0						

#	Boring	Sample	Sample Depth		Subgrade Depth		Standard Penetration		HP (tsf)	Physical Characteristics					Moisture		Ohio DOT		Sulfate Content (ppm)	Problem		Excavate and Replace (Item 204)		Recommendation (Enter depth in inches)	
			From	To	From	To	N ₆₀	N _{60L}		LL	PL	PI	% Silt	% Clay	P200	M _c	M _{OPT}	Class		GI	Unsuitable	Unstable	Unsuitable		Unstable
10	B 008-0 23	SS-1	1.5	3.0	0.2	1.7	47			19	16	3	14	3	17	6	6	A-1-b	0	260					
		SS-2/3A	3.0	5.2	1.7	3.9	29									9	6	A-1-b	0						
		SS-3B/4A	5.2	6.7	3.9	5.4	9		0.7	29	16	13	41	15	56	17	14	A-6a	5						
		SS-4B	6.7	7.5	5.4	6.2	9	9								14	8	A-3a							

PID: 119585

County-Route-Section: STA-241-17.37

No. of Borings: 10

Geotechnical Consultant: S&ME, Inc.

Prepared By: Brian K. Sears, PE

Date prepared: 4/2/2025

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

Excavate and Replace Stabilization Options	
Global Geotextile Average(N60L):	21"
Average(HP):	12"
Global Geogrid Average(N60L):	15"
Average(HP):	0"

Design CBR	7
---------------	---

% Samples within 3 feet of subgrade			
N ₆₀ ≤ 5	10%	HP ≤ 0.5	7%
N ₆₀ < 12	21%	0.5 < HP ≤ 1	7%
12 ≤ N ₆₀ < 15	7%	1 < HP ≤ 2	7%
N ₆₀ ≥ 20	17%	HP > 2	3%
M+	10%		
Rock	0%		
Unsuitable Soil	0%		

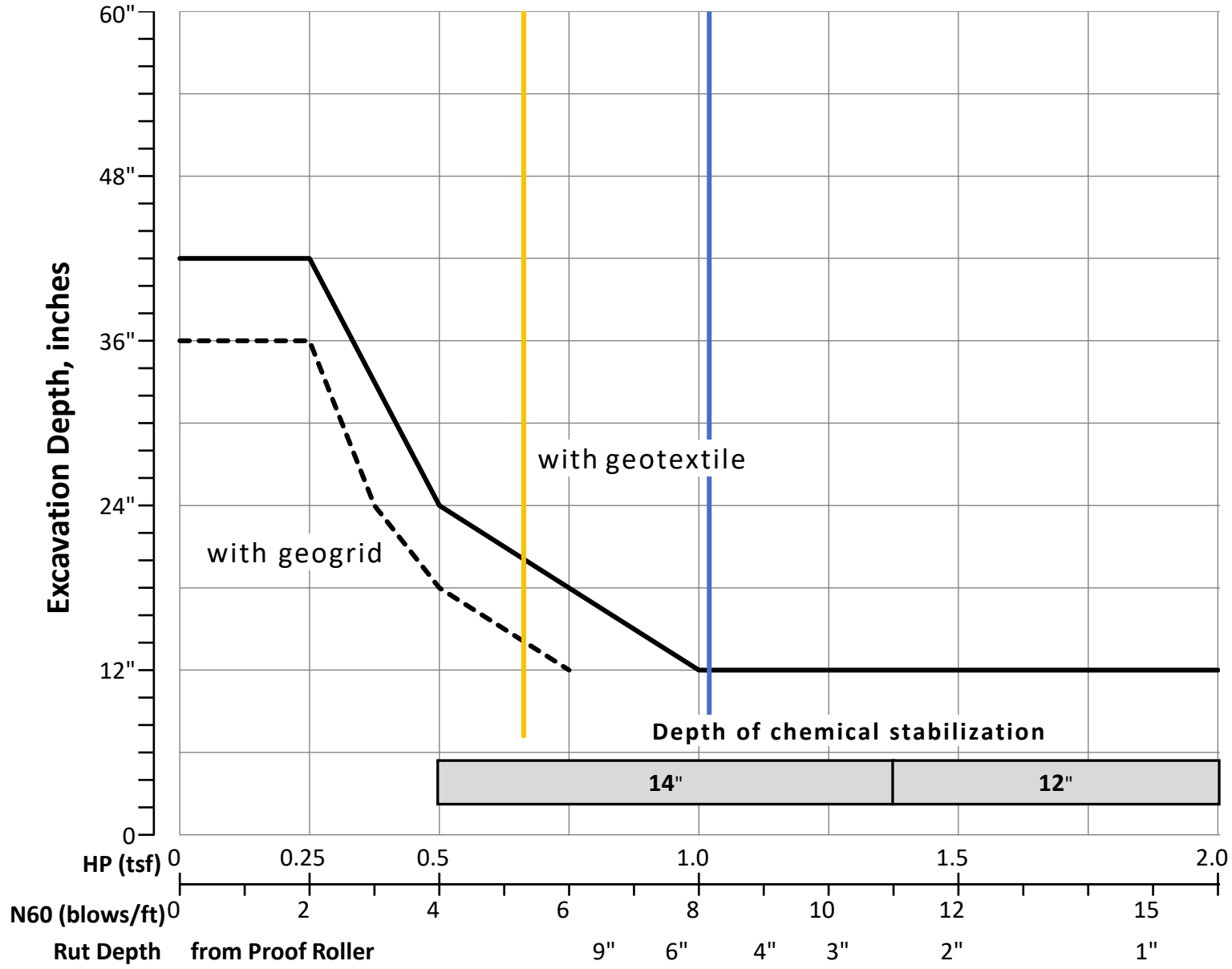
Excavate and Replace at Surface	
Average	0"
Maximum	0"
Minimum	0"

% Proposed Subgrade Surface	
Unstable & Unsuitable	38%
Unstable	38%
Unsuitable (Soil & Rock)	0%

	N ₆₀	N _{60L}	HP	LL	PL	PI	Silt	Clay	P 200	M _C	M _{OPT}	GI
Average	11	5	1.02	29	19	10	43	15	57	18	12	6
Maximum	47	11	3.50	47	39	24	76	36	98	72	19	16
Minimum	0	2	0.10	17	14	1	14	1	17	6	6	0

Classification Counts by Sample																					
ODOT Class	UCF	Rock	A-1-a	A-1-b	A-2-4	A-2-5	A-2-6	A-2-7	A-3	A-3a	A-4a	A-4b	A-5	A-6a	A-6b	A-7-5	A-7-6	A-8a	A-8b	Totals	
Count	0	0	0	3	2	0	0	0	0	5	9	1	0	6	6	0	1	0	1	34	
Percent	0%	0%	0%	9%	6%	0%	0%	0%	0%	15%	26%	3%	0%	18%	18%	0%	3%	0%	3%	100%	
% Rock Granular Cohesive	0%	0%	56%									44%									100%
Surface Class Count	0	0	0	3	0	0	0	0	0	1	4	0	0	2	2	0	1	0	0	13	
Surface Class Percent	0%	0%	0%	23%	0%	0%	0%	0%	0%	8%	31%	0%	0%	15%	15%	0%	8%	0%	0%	100%	

Fig. 600-1 – Subgrade Stabilization



OVERRIDE TABLE

Calculated Average	New Values	Check to Override
1.02		<input type="checkbox"/> HP
5.38		<input type="checkbox"/> N60L

Average HP



Average N₆₀L



PLATE 6

PARAMETER ESTIMATION FOR
SETTLEMENT ANALYSES



= User Entry
Required

SOIL PARAMETER SUMMARY

S&ME Project No. 23170065A
 Client ODOT District 4
 Project STA-241-17.37
 Analysis Desc. Emb. Settlement - Cohesive Parameter Est.

Calculated By BKS Date 4/22/24
 Check By RSW Date 4/22/24

REAR ABUTMENT														
Boring No:		B-002-0-23		Ground El:		1073 MSL		D _{GWT} :		5		ft		
Soil Layer	Depth (ft)	Description	N (bpf)	γ _m (pcf)	w _n (%)	%CL (%)	LL	PL	PI	LI	e ₀ (3)	C _c (4)	Cr (5)	Comments
1	3.3	A-6a	14	125	8	-	-	-	-	-	0.6	0.06	0.006	
2	6.5	A-6b	3	115	34	19	40	16	24	0.75	0.9	0.26	0.026	
3	11	A-6a	3	110	30	24	31	11	20	0.95	0.9	0.22	0.022	
4	21	A-4b	0	105	35	32	31	21	10	1.4	0.93	0.18	0.018	From laboratory tested sample
5	28	A-6a	0	105	37	21	30	18	12	1.583	0.9	0.24	0.024	
6														
7														
8														
9														
10														
11														

Estimation of C _c by various correlations accompanied by basic statistical results of the methods. Equations for each method is provided below.			(Assumed or Known values)	USER DEFINED (overwrite option)	Terzaghi and Peck (1967)	Assouz et.al (1976)	Kulhawy and Mayne (1990)	Nakase et al. (1988)	Nagaraj and Srinivasa Murthy (1985, 1986)	Skempton (1944)	Azzouz et al. (1976)	Hough (1957)	Sowers (1970)	Peck and Reed (1954)	Azzouz et al. (1976)	Azzouz et al. (1976)	Al-Khafaji and Andersland (1992)	AVERAGE	MINIMUM	MAXIMUM	STANDARD DEV.
Soil Layer	Depth (ft)	Description	SG	Cc																	
1	3.3	A-6a	2.7				0.08					0.1	0.08	-0.08	0.03	0.14		0.06	-0.08	0.14	0.08
2	6.5	A-6b	2.7		0.27	0.25	0.34	0.3	0.25	0.23	0.19	0.19	0.3	0.27	0.29	0.26	0.24	0.26	0.19	0.34	0.04
3	11	A-6a	2.7		0.19	0.24	0.3	0.25	0.2	0.17	0.13	0.19	0.3	0.2	0.25	0.26	0.23	0.22	0.13	0.3	0.05
4	21	A-4b	2.7	0.18	0.19	0.25	0.35	0.15	0.2	0.17	0.13	0.2	0.32	0.29	0.3	0.27	0.24	0.24	0.13	0.35	0.07
5	28	A-6a	2.7		0.18	0.24	0.37	0.17	0.19	0.16	0.13	0.19	0.3	0.33	0.32	0.26	0.23	0.24	0.13	0.37	0.07
6																					
7																					
8																					
9																					
10																					
11																					



= User Entry
Required

SOIL PARAMETER SUMMARY

S&ME Project No. 23170065A
 Client ODOT District 4
 Project STA-241-17.37
 Analysis Desc. Emb. Settlement - Cohesive Parameter Est.

Calculated By BKS Date 4/22/24
 Check By RSW Date 4/22/24

FORWARD ABUTMENT

Boring No: B-003-0-23 Ground El: 1072 MSL D_{GWT} : 5 ft

Soil Layer	Depth (ft)	Description	N (bpf)	γ_m (pcf)	w_n (%)	%CL (%)	LL	PL	PI	LI	e_0 (3)	Cc (4)	Cr (5)	Comments
1	3.5	A-4a	12	125	13	-	-	-	-	-	0.6	0.08	0.008	
2	5	A-4a	3	110	17	19	40	16	24	0.042	0.9	0.21	0.021	
3	6.5	A-4b	3	105	72	24	31	11	20	3.05	1.4	0.45	0.045	
4	11	A-4b	1	105	29	32	31	21	10	0.8	0.8	0.20	0.020	
5	15.5	A-6a	0	105	36	21	30	18	12	1.5	0.9	0.23	0.023	
6	18	A-4b	0	105	33	-	-	-	-	-	0.73	0.18	0.018	From laboratory tested sample
7	23	A-6a	0	105	33	4	18	15	3	6	0.9	0.19	0.019	
8				-	-	-	-	-	-	-	-	-	-	
9				-	-	-	-	-	-	-	-	-	-	
10				-	-	-	-	-	-	-	-	-	-	
11				-	-	-	-	-	-	-	-	-	-	

Soil Layer			Estimation of Cc by various correlations accompanied by basic statistical results of the methods. Equations for each method is provided below.															AVERAGE	MINIMUM	MAXIMUM	STANDARD DEV.
Soil Layer	Depth (ft)	Description	SG	CC	Terzaghi and Peck (1967)	Assouz et.al (1976)	Kulhawy and Mayne (1990)	Nakase et al. (1988)	Nagaraj and Srinivasa Murthy (1985, 1986)	Skempton (1944)	Azzouz et al. (1976)	Hough (1957)	Sowers (1970)	Peck and Reed (1954)	Azzouz et al. (1976)	Azzouz et al. (1976)	Al-Khafaji and Andersland (1992)				
1	3.5	A-4a	2.7				0.13					0.1	0.08	-0.03	0.08	0.14		0.08	-0.03	0.14	0.06
2	5	A-4a	2.7		0.27	0.25	0.17	0.3	0.25	0.23	0.19	0.19	0.3	0.02	0.12	0.26	0.24	0.21	0.02	0.3	0.08
3	6.5	A-4b	2.7		0.19	0.43	0.72	0.25	0.2	0.17	0.13	0.34	0.68	1.21	0.67	0.46	0.44	0.45	0.13	1.21	0.3
4	11	A-4b	2.7		0.19	0.21	0.29	0.15	0.2	0.17	0.13	0.16	0.23	0.19	0.24	0.22	0.19	0.20	0.13	0.29	0.04
5	15.5	A-6a	2.7		0.18	0.24	0.36	0.17	0.19	0.16	0.13	0.19	0.3	0.31	0.31	0.26	0.23	0.23	0.13	0.36	0.07
6	18	A-4b	2.7	0.18			0.33					0.14	0.17	0.25	0.28	0.19		0.23	0.14	0.33	0.07
7	23	A-6a	2.7		0.07	0.23	0.33	0.08	0.11	0.08	0.05	0.19	0.3	0.25	0.28	0.26	0.22	0.19	0.05	0.33	0.1
8																					
9																					
10																					
11																					



= User Entry
Required

SOIL PARAMETER SUMMARY

S&ME Project No. 23170065A
 Client ODOT District 4
 Project STA-241-17.37
 Analysis Desc. Emb. Settlement - Cohesive Parameter Est.

Calculated By BKS Date 4/22/24
 Check By RSW Date 4/22/24

REFERENCE INFORMATION

Sheet 3 of 3

- e_o estimated from one or both of the following references:

$$e_o = \frac{w_n G_s}{S} \quad \text{where } G_s = 2.7 \text{ and Saturation } (S) = 100\%$$

FHWA-HI-88-099 Soils and Foundations Workshop Manual - 2nd Edition, 1994

Reproduction of Table 1.4 from Principles of Foundation Engineering, 5th Edition (Das, 2004)	
Material Type	Void Ratio, e_o
Loose uniform sand	0.8
Dense uniform sand	0.45
Loose ang.-grained silty sand	0.65
Dense ang.-grained silty sand	0.4
Stiff clay	0.6
Soft clay	0.9 - 1.4
Loess	0.9
Soft organic clay	2.5 - 3.2
Glacial till	0.3

- C_c determined as average value from the following methods (or by determination from laboratory consolidation testing).

Equation	Source	Applicable Soil Types
$C_c = 0.009(LL - 10)$	Terzaghi and Peck (1967)	Inorganic soils, sensitivity < 4
$C_c = 0.37(e_o + 0.003LL + 0.0004W_n - 0.34)$	Azzouz et al. (1976)	Lean to fat clays
$C_c = 0.046 + 0.0104PI$	Nakase et al. (1988)	Soils with plasticity index < 50
$C_c = 0.00234(LL)(G_s)$	Nagaraj and Srinivasa Murthy (1985,1986)	Inorganic clays
$C_c = 0.01W_n$	Kulhawy and Mayne (1990)	Chicago clays
$C_c = 0.007(LL - 7)$	Skempton (1944)	Remolded clays
$C_c = 0.006(LL - 9)$	Azzouz et al. (1976)	Lean to fat clays
$C_c = 0.30(e_o - 0.27)$	Hough (1957)	Inorganic soil (silt, silty clay, some clay)
$C_c = 0.75(e_o - 0.50)$	Sowers (1970)	Very low plasticity soils
$C_c = (17.66 \times 10^{-5})(w_n^2) + (5.93 \times 10^{-3})(w_n - 0.135)$	Peak and Reed (1954)	Chicago clays
$C_c = 0.01w_n - 0.05$	Azzouz et al. (1976)	Lean to fat clays
$C_c = 0.4(e_o - 0.25)$	Azzouz et al. (1976)	Lean to fat clays

- C_r determined as: $C_r = 0.1C_c$

Note: The ratio of C_r to C_c generally ranges from 0.05 to 0.1. Based on previous experience with Ohio soils, a ratio of 0.1 is believed to most closely approximate the value of C_r .



S&ME Project No. 23170065A
 Client ODOT District 4
 Project Name STA-241-17.37 Roundabout & Bridge
 Analysis Desc. Embankment Settlement - Elastic

Calculated By BKS
 Date 4/22/2024
 Check By RSW
 Date 4/22/2024

= User Entry Req'd

Sheet 1 of 4

ESTIMATING ELASTIC MODULUS IN GRANULAR SOILS - Boring B-002-0-23

Based on AASHTO LRFD 10.4.6.2.4 and Table C10.4.6.3-1 and Additional Reference Manuals

Reference Information Provided on Sheets 2 through 4

	Boring ID	Sample ID	Depth ft	N ₆₀ bpf	σ' _v ksf	C _N (a)	N ₍₁₎₆₀ (b)	Low ksf	Average ksf	High ksf	1 St Dev ksf	Design ksf
	SAND											

	Boring ID	Sample ID	Depth ft	N ₆₀ bpf	σ' _v ksf	C _N (a)	N ₍₁₎₆₀ (b)	Low ksf	Average ksf	High ksf	1 St Dev ksf	Design ksf
	FINE or SUBMERGED SAND	B-002-0-23	SS-16	39	6	4.68	0.72	4	56	141	240	78
B-002-0-23		SS-17	44	5	5.28	0.68	3	42	137	240	84	100
B-002-0-23		SS-18	49	3	5.88	0.64	2	28	131	240	91	100
B-002-0-23		SS-19	54	9	6.48	0.61	5	70	149	240	71	100



S&ME Project No. 23170065A
 Client ODOT District 4
 Project Name STA-241-17.37 Roundabout & Bridge
 Analysis Desc. Embankment Settlement - Elastic

Calculated By BKS
 Date 4/22/2024
 Check By RSW
 Date 4/22/2024

= User Entry Req'd

Sheet 2 of 4

ESTIMATING ELASTIC MODULUS IN GRANULAR SOILS - Boring B-002-0-23

Based on AASHTO LRFD 10.4.6.2.4 and Table C10.4.6.3-1 and Additional Reference Manuals

Reference Information Provided on Sheets 2 through 4

	Boring ID	Sample ID	Depth ft	N ₆₀ bpf	σ' _v ksf	C _N (a)	N ₍₁₎₆₀ (b)	Low ksf	Average ksf	High ksf	1 St Dev ksf	Design ksf
	SILT, SILTY SAND or CLAYEY SAND	B-002-0-23	SS-15A	34	11	4.08	0.76	8	40	167	450	166
B-002-0-23		SS-20	59	20	7.08	0.58	12	40	197	450	159	175
B-002-0-23		SS-22	69	8	8.28	0.53	4	32	153	450	173	150
B-002-0-23		SS-23	74	12	8.88	0.50	6	40	166	450	168	150
B-002-0-23		SS-24	79	12	9.48	0.48	6	40	166	450	168	150
B-002-0-23		SS-25	84	9	10.08	0.46	4	32	155	450	173	150
B-002-0-23		SS-26	89	20	10.68	0.44	9	40	192	450	163	200
B-002-0-23		SS-27	94	15	11.28	0.42	6	40	174	450	168	200
B-002-0-23		SS-28A	99	29	11.88	0.41	12	40	220	450	163	200

	Boring ID	Sample ID	Depth ft	N ₆₀ bpf	σ' _v ksf	C _N (a)	N ₍₁₎₆₀ (b)	Low ksf	Average ksf	High ksf	1 St Dev ksf	Design ksf
	GRAVEL, SANDY GRAVEL	B-002-0-23	SS-14	29	3	3.48	0.82	2	48	1226	3028	1585
B-002-0-23		SS-21	64	14	7.68	0.55	8	192	1264	2000	949	1000
B-002-0-23		SS-28B	100	29	12	0.40	12	289	1296	2000	895	1600



S&ME Project No. 23170065A
Client ODOT District 4
Project Name STA-241-17.37 Roundabout & Bridge
Analysis Desc. Embankment Settlement - Elastic

Calculated By BKS
Date 4/22/2024
Check By RSW
Date 4/22/2024

ESTIMATING ELASTIC MODULUS IN GRANULAR SOILS

Reference Equations and Sources

<u>Ref.</u>	<u>Applicable Material Type/Equation Reference</u>	<u>Equation</u>	<u>Source</u>
(a)	Overburden coefficient (Eqn. 10.4.6.2.4-1)	$C_N = 0.77 \log(40/\sigma'_v); C_N \leq 2.0$	AASHTO
(b)	Corrected SPT Blowcount (Eqn. 10.4.6.2.4-3)	$N_{(1)60} = C_N N_{60}$	AASHTO
(1)	Coarse sands and sands with little gravel	$= 0.139N_{(1)60}$	AASHTO
(2)	Sand (normally consolidated)	$= 500(N_{60} + 15)$	Bowles
(3)	Sand (normally consolidated)	$= 18000 + 750N_{60}$	Bowles
(4)	Sand (normally consolidated)	$= 15200 \ln(N_{60})$	Bowles
(5)	Sand (normally consolidated)	$= 18500 \ln(N_{60})$	Bowles
(6)	Sand (normally consolidated)	$= 22000 \ln(N_{60})$	Bowles
(7)	Sand (normally consolidated)	$= 7000 * \text{sqrt}(N_{60})$	Bowles
(8)	Gravelly sand	$= 1200(N_{60} + 6)$	Bowles
(9)	Loose, Medium-dense or Dense Sand	Given Ranges	AASHTO, Das, McCarthy, Bowles
(10)	Clean fine to medium sands and slightly silty sands	$= 0.097N_{(1)60}$	AASHTO
(11)	Sand (saturated)	$= 250(N_{60} + 15)$	Bowles
(12)	Loose, Medium-dense or Dense Sand	Given Ranges	AASHTO, Bowles
(13)	Silts, sandy silts, slightly cohesive mixtures	$= 0.056N_{(1)60}$	AASHTO
(14)	Clayey sand	$= 320(N_{60} + 15)$	Bowles
(15)	Silty sand	$= 300(N_{60} + 6)$	Bowles
(16)	Silt and Silty sand	Given Ranges	Das, McCarthy, Bowles
(17)	Sandy gravel and gravels	$= 0.167N_{(1)60}$	AASHTO
(18)	Loose, Medium-dense or Dense Gravel	Given Ranges	AASHTO, Das, McCarthy, Bowles

Estimated Ranges from Reference Manuals (Source Provided)

Silty Sand						
Low	High		Low	High		Reference
1.50	2.50	ksi	216.0	360.0	ksf	(Das, <i>Principles of Foundation Engineering</i> , 4th Edition)
1.02	3.05	ksi	146.20	438.59	ksf	(McCarthy, 1998; from Settle3 Program)
1.04	3.13	ksi	150.00	450.00	ksf	(Bowles, <i>Foundation Analysis and Design</i> , 4th Edition)
0.35	2.90	ksi	50.13	417.71	ksf	(McCarthy, 1998; from Settle3 Program) - Silt
0.28	2.78	ksi	40.00	400.00	ksf	(Bowles, <i>Foundation Analysis and Design</i> , 4th Edition) - Silt



S&ME Project No. 23170065A
 Client ODOT District 4
 Project Name STA-241-17.37 Roundabout & Bridge
 Analysis Desc. Embankment Settlement - Elastic

Calculated By BKS
 Date 4/22/2024
 Check By RSW
 Date 4/22/2024

ESTIMATING ELASTIC MODULUS IN GRANULAR SOILS

Estimated Ranges from Reference Manuals (Source Provided) - Continued

Loose Sand						
Low	High		Low	High		Reference
1.39	4.17	ksi	200.16	600.48	ksf	(AASHTO LRFD Table C10.4.6.3-1)
1.50	3.50	ksi	216.0	504.0	ksf	(Das, <i>Principles of Foundation Engineering</i> , 4th Edition)
1.45	4.06	ksi	208.85	584.79	ksf	(Das, 2002; from Settle3 Program)
1.31	3.63	ksi	187.97	522.14	ksf	(McCarthy, 1998; from Settle3 Program)
1.39	3.47	ksi	200.00	500.00	ksf	(Bowles, <i>Foundation Analysis and Design</i> , 4th Edition)
1.11	1.67	ksi	159.84	240.48	ksf	(AASHTO LRFD Table C10.4.6.3-1) - fine sand
Medium-dense Sand						
Low	High		Low	High		Reference
2.50	4.00	ksi	360.0	576.0	ksf	(Das, <i>Principles of Foundation Engineering</i> , 4th Edition)
4.17	6.94	ksi	600.48	999.36	ksf	(AASHTO LRFD Table C10.4.6.3-1)
1.67	2.78	ksi	240.48	400.32	ksf	(AASHTO LRFD Table C10.4.6.3-1) - fine sand
Dense Sand						
Low	High		Low	High		Reference
6.94	11.11	ksi	999.36	1599.84	ksf	(AASHTO LRFD Table C10.4.6.3-1)
5.00	8.00	ksi	720.0	1152.0	ksf	(Das, <i>Principles of Foundation Engineering</i> , 4th Edition)
5.08	10.15	ksi	730.99	1461.98	ksf	(Das, 2002; from Settle3 Program)
6.53	11.60	ksi	939.84	1670.83	ksf	(McCarthy, 1998; from Settle3 Program)
6.94	11.81	ksi	1000.00	1700.00	ksf	(Bowles, <i>Foundation Analysis and Design</i> , 4th Edition)
2.78	4.17	ksi	400.32	600.48	ksf	(AASHTO LRFD Table C10.4.6.3-1) - fine sand
Loose Gravel						
Low	High		Low	High		Reference
4.17	11.11	ksi	600.48	1599.84	ksf	(AASHTO LRFD Table C10.4.6.3-1)
6.53	21.03	ksi	939.84	3028.39	ksf	(McCarthy, 1998; from Settle3 Program)
6.94	20.83	ksi	1000.00	3000.00	ksf	(Bowles, <i>Foundation Analysis and Design</i> , 4th Edition)
Medium-dense Gravel						
Low	High		Low	High		Reference
11.11	13.89	ksi	1599.84	2000.16	ksf	(AASHTO LRFD Table C10.4.6.3-1)
Dense Gravel						
Low	High		Low	High		Reference
13.89	27.78	ksi	2000.16	4000.32	ksf	(AASHTO LRFD Table C10.4.6.3-1)
13.05	26.11	ksi	1879.69	3759.38	ksf	(McCarthy, 1998; from Settle3 Program)
10.00	25.00	ksi	1440.0	3600.0	ksf	(Das, <i>Principles of Foundation Engineering</i> , 4th Edition)
13.89	27.78	ksi	2000.00	4000.00	ksf	(Bowles, <i>Foundation Analysis and Design</i> , 4th Edition)



Version 1.0 (4/22/2024)

= User Entry Req'd

S&ME Project No. 23170065A
 Client ODOT District 4
 Project Name STA-241-17.37 Roundabout & Bridge
 Analysis Desc. Embankment Settlement - Elastic

Calculated By BKS
 Date 4/22/2024
 Check By RSW
 Date 4/22/2024

Sheet 1 of 4

ESTIMATING ELASTIC MODULUS IN GRANULAR SOILS - Boring B-003-0-23

Based on AASHTO LRFD 10.4.6.2.4 and Table C10.4.6.3-1 and Additional Reference Manuals

Reference Information Provided on Sheets 3 through 4

	Boring ID	Sample ID	Depth ft	N ₆₀ bpf	σ' _v ksf	C _N (a)	N ₍₁₎₆₀ (b)	Low ksf	Average ksf	High ksf	1 St Dev ksf	Design ksf
	SAND	B-003-0-23	SS-20	59	14	7.08	0.58	8	160	648	1213	354
B-003-0-23		SS-21	64	6	7.68	0.55	3	60	429	823	243	550

	Boring ID	Sample ID	Depth ft	N ₆₀ bpf	σ' _v ksf	C _N (a)	N ₍₁₎₆₀ (b)	Low ksf	Average ksf	High ksf	1 St Dev ksf	Design ksf
	FINE or SUBMERGED SAND	B-003-0-23	SS-16	39	6	4.68	0.72	4	56	141	240	78
B-003-0-23		SS-26B	89.5	14	10.74	0.44	6	84	219	400	137	200



Version 1.0 (4/22/2024)

= User Entry Req'd

S&ME Project No. 23170065A
 Client ODOT District 4
 Project Name STA-241-17.37 Roundabout & Bridge
 Analysis Desc. Embankment Settlement - Elastic

Calculated By BKS
 Date 4/22/2024
 Check By RSW
 Date 4/22/2024

Sheet 2 of 4

ESTIMATING ELASTIC MODULUS IN GRANULAR SOILS - Boring B-003-0-23

Based on AASHTO LRFD 10.4.6.2.4 and Table C10.4.6.3-1 and Additional Reference Manuals

Reference Information Provided on Sheets 3 through 4

	Boring ID	Sample ID	Depth ft	N ₆₀ bpf	σ' _v ksf	C _N (a)	N ₍₁₎₆₀ (b)	Low ksf	Average ksf	High ksf	1 St Dev ksf	Design ksf
	SILT, SILTY SAND or CLAYEY SAND	B-003-0-23	SS-17	44	5	5.28	0.68	3	24	143	450	176
B-003-0-23		SS-18	49	6	5.88	0.64	4	32	148	450	174	100
B-003-0-23		SS-19	54	6	6.48	0.61	4	32	148	450	174	100
B-003-0-23		SS-22	69	23	8.28	0.53	12	40	204	450	160	175
B-003-0-23		SS-23	74	15	8.88	0.50	8	40	177	450	165	175
B-003-0-23		SS-24	79	15	9.48	0.48	7	40	176	450	166	175
B-003-0-23		SS-25	84	15	10.08	0.46	7	40	176	450	166	175
B-003-0-23		SS-26A	88.5	14	10.62	0.44	6	40	172	450	168	175

	Boring ID	Sample ID	Depth ft	N ₆₀ bpf	σ' _v ksf	C _N (a)	N ₍₁₎₆₀ (b)	Low ksf	Average ksf	High ksf	1 St Dev ksf	Design ksf
	GRAVEL, SANDY GRAVEL	B-003-0-23	SS-12	24	5	2.88	0.88	4	96	1242	3028	1568
B-003-0-23		SS-13	26.5	3	3.18	0.85	3	72	1234	3028	1577	500
B-003-0-23		SS-14	29	5	3.48	0.82	4	96	1242	3028	1568	500
B-003-0-23		SS-15	34	5	4.08	0.76	4	96	1242	3028	1568	500
B-003-0-23		SS-16	39	6	4.68	0.72	4	96	1242	3028	1568	500
B-003-0-23		SS-27	94	3	11.28	0.42	1	24	1218	3028	1594	500
B-003-0-23		SS-28	99	11	11.88	0.41	4	96	1232	2000	1004	500



S&ME Project No. 23170065A
 Client ODOT District 4
 Project Name STA-241-17.37 Roundabout & Bridge
 Analysis Desc. Embankment Settlement - Elastic

Calculated By BKS
 Date 4/22/2024
 Check By RSW
 Date 4/22/2024

ESTIMATING ELASTIC MODULUS IN GRANULAR SOILS

Reference Equations and Sources

<u>Ref.</u>	<u>Applicable Material Type/Equation Reference</u>	<u>Equation</u>	<u>Source</u>
(a)	Overburden coefficient (Eqn. 10.4.6.2.4-1)	$C_N = 0.77 \log(40/\sigma'_v); C_N \leq 2.0$	AASHTO
(b)	Corrected SPT Blowcount (Eqn. 10.4.6.2.4-3)	$N_{(1)60} = C_N N_{60}$	AASHTO
(1)	Coarse sands and sands with little gravel	$= 0.139N_{(1)60}$	AASHTO
(2)	Sand (normally consolidated)	$= 500(N_{60} + 15)$	Bowles
(3)	Sand (normally consolidated)	$= 18000 + 750N_{60}$	Bowles
(4)	Sand (normally consolidated)	$= 15200 \ln(N_{60})$	Bowles
(5)	Sand (normally consolidated)	$= 18500 \ln(N_{60})$	Bowles
(6)	Sand (normally consolidated)	$= 22000 \ln(N_{60})$	Bowles
(7)	Sand (normally consolidated)	$= 7000 * \text{sqrt}(N_{60})$	Bowles
(8)	Gravelly sand	$= 1200(N_{60} + 6)$	Bowles
(9)	Loose, Medium-dense or Dense Sand	Given Ranges	AASHTO, Das, McCarthy, Bowles
(10)	Clean fine to medium sands and slightly silty sands	$= 0.097N_{(1)60}$	AASHTO
(11)	Sand (saturated)	$= 250(N_{60} + 15)$	Bowles
(12)	Loose, Medium-dense or Dense Sand	Given Ranges	AASHTO, Bowles
(13)	Silts, sandy silts, slightly cohesive mixtures	$= 0.056N_{(1)60}$	AASHTO
(14)	Clayey sand	$= 320(N_{60} + 15)$	Bowles
(15)	Silty sand	$= 300(N_{60} + 6)$	Bowles
(16)	Silt and Silty sand	Given Ranges	Das, McCarthy, Bowles
(17)	Sandy gravel and gravels	$= 0.167N_{(1)60}$	AASHTO
(18)	Loose, Medium-dense or Dense Gravel	Given Ranges	AASHTO, Das, McCarthy, Bowles

Estimated Ranges from Reference Manuals (Source Provided)

Silty Sand						
Low	High		Low	High		Reference
1.50	2.50	ksi	216.0	360.0	ksf	(Das, <i>Principles of Foundation Engineering</i> , 4th Edition)
1.02	3.05	ksi	146.20	438.59	ksf	(McCarthy, 1998; from Settle3 Program)
1.04	3.13	ksi	150.00	450.00	ksf	(Bowles, <i>Foundation Analysis and Design</i> , 4th Edition)
0.35	2.90	ksi	50.13	417.71	ksf	(McCarthy, 1998; from Settle3 Program) - Silt
0.28	2.78	ksi	40.00	400.00	ksf	(Bowles, <i>Foundation Analysis and Design</i> , 4th Edition) - Silt



S&ME Project No. 23170065A
 Client ODOT District 4
 Project Name STA-241-17.37 Roundabout & Bridge
 Analysis Desc. Embankment Settlement - Elastic

Calculated By BKS
 Date 4/22/2024
 Check By RSW
 Date 4/22/2024

ESTIMATING ELASTIC MODULUS IN GRANULAR SOILS

Estimated Ranges from Reference Manuals (Source Provided) - Continued

Loose Sand						
Low	High		Low	High		Reference
1.39	4.17	ksi	200.16	600.48	ksf	(AASHTO LRFD Table C10.4.6.3-1)
1.50	3.50	ksi	216.0	504.0	ksf	(Das, <i>Principles of Foundation Engineering</i> , 4th Edition)
1.45	4.06	ksi	208.85	584.79	ksf	(Das, 2002; from Settle3 Program)
1.31	3.63	ksi	187.97	522.14	ksf	(McCarthy, 1998; from Settle3 Program)
1.39	3.47	ksi	200.00	500.00	ksf	(Bowles, <i>Foundation Analysis and Design</i> , 4th Edition)
1.11	1.67	ksi	159.84	240.48	ksf	(AASHTO LRFD Table C10.4.6.3-1) - fine sand

Medium-dense Sand						
Low	High		Low	High		Reference
2.50	4.00	ksi	360.0	576.0	ksf	(Das, <i>Principles of Foundation Engineering</i> , 4th Edition)
4.17	6.94	ksi	600.48	999.36	ksf	(AASHTO LRFD Table C10.4.6.3-1)
1.67	2.78	ksi	240.48	400.32	ksf	(AASHTO LRFD Table C10.4.6.3-1) - fine sand














Dense Sand						
Low	High		Low	High		Reference
6.94	11.11	ksi	999.36	1599.84	ksf	(AASHTO LRFD Table C10.4.6.3-1)
5.00	8.00	ksi	720.0	1152.0	ksf	(Das, <i>Principles of Foundation Engineering</i> , 4th Edition)
5.08	10.15	ksi	730.99	1461.98	ksf	(Das, 2002; from Settle3 Program)
6.53	11.60	ksi	939.84	1670.83	ksf	(McCarthy, 1998; from Settle3 Program)
6.94	11.81	ksi	1000.00	1700.00	ksf	(Bowles, <i>Foundation Analysis and Design</i> , 4th Edition)
2.78	4.17	ksi	400.32	600.48	ksf	(AASHTO LRFD Table C10.4.6.3-1) - fine sand

Loose Gravel						
Low	High		Low	High		Reference
4.17	11.11	ksi	600.48	1599.84	ksf	(AASHTO LRFD Table C10.4.6.3-1)
6.53	21.03	ksi	939.84	3028.39	ksf	(McCarthy, 1998; from Settle3 Program)
6.94	20.83	ksi	1000.00	3000.00	ksf	(Bowles, <i>Foundation Analysis and Design</i> , 4th Edition)














Medium-dense Gravel						
Low	High		Low	High		Reference
11.11	13.89	ksi	1599.84	2000.16	ksf	(AASHTO LRFD Table C10.4.6.3-1)

Dense Gravel						
Low	High		Low	High		Reference
13.89	27.78	ksi	2000.16	4000.32	ksf	(AASHTO LRFD Table C10.4.6.3-1)
13.05	26.11	ksi	1879.69	3759.38	ksf	(McCarthy, 1998; from Settle3 Program)
10.00	25.00	ksi	1440.0	3600.0	ksf	(Das, <i>Principles of Foundation Engineering</i> , 4th Edition)
13.89	27.78	ksi	2000.00	4000.00	ksf	(Bowles, <i>Foundation Analysis and Design</i> , 4th Edition)

**SOIL LAYER PARAMETERS USED FOR SETTLEMENT ANALYSES AT STA. 587+00 AND STA. 588+00
(REAR ABUTMENT)**

Material Name	Color	Unit Weight (kips/ft3)	Sat. Unit Weight (kips/ft3)	Es (ksf)	Eur (ksf)	Material Type	Cc	Cr	OCR	e0	Cv (ft2/d)	Cvr (ft2/d)	Ca	Car
Fill: Silt & Clay with Sand		0.125	0.125	-	-	Non-Linear	0.2	0.02	3	0.6	0.05	0.005	0.007	0.0007
Silty Clay		0.115	0.125	-	-	Non-Linear	0.26	0.026	1.3	0.9	0.05	0.005	0.006	0.0006
Silt & Clay_1		0.11	0.125	-	-	Non-Linear	0.22	0.022	1.3	0.9	0.13	0.013	0.005	0.0005
Silt_1		0.105	0.125	-	-	Non-Linear	0.18	0.018	1.3	0.93	0.13	0.013	0.006	0.0006
Silt & Clay_2		0.105	0.125	-	-	Non-Linear	0.24	0.024	1.3	0.9	0.23	0.023	0.005	0.0005
Gravel with Sand_1		0.11	0.125	600	600	-	-	-	-	-	-	-	-	-
Silt_2		0.115	0.125	150	150	-	-	-	-	-	-	-	-	-
Fine Sand		0.11	0.125	100	100	-	-	-	-	-	-	-	-	-
Sandy Silt		0.12	0.125	175	175	-	-	-	-	-	-	-	-	-
Gravel with Sand_2		0.12	0.125	1000	1000	-	-	-	-	-	-	-	-	-
Silt_3		0.115	0.125	150	150	-	-	-	-	-	-	-	-	-
Silt_4		0.12	0.125	200	200	-	-	-	-	-	-	-	-	-
Gravel with Sand_3		0.125	0.125	1600	1600	-	-	-	-	-	-	-	-	-

SOIL LAYER PARAMETERS USED FOR SETTLEMENT ANALYSES AT STA. 589+00 (FORWARD ABUTMENT) AND STA. 590+50

Material Name	Color	Unit Weight (kips/ft3)	Sat. Unit Weight (kips/ft3)	Es (ksf)	Eur (ksf)	Material Type	Cc	Cr	OCR	e0	Cv (ft2/d)	Cvr (ft2/d)	Ca	Car
Fill: Silt & Clay with Sand		0.125	0.125	-	-	Non-Linear	0.2	0.02	3	0.6	0.04	0.004	0.003	0.0003
Silty Clay		0.11	0.125	-	-	Non-Linear	0.21	0.021	1.3	0.9	0.04	0.004	0.006	0.0006
Silt & Clay_1		0.105	0.125	-	-	Non-Linear	0.45	0.045	1.3	1.4	0.04	0.004	0.009	0.0009
Silt_1		0.105	0.125	-	-	Non-Linear	0.2	0.02	1.3	0.8	0.08	0.008	0.005	0.0005
Silt & Clay_2		0.105	0.125	-	-	Non-Linear	0.23	0.023	1.3	0.9	0.12	0.012	0.006	0.0006
Silt_2		0.105	0.125	-	-	Non-Linear	0.18	0.018	1.3	0.73	0.12	0.012	0.005	0.0005
Silt & Clay_3		0.105	0.125	-	-	Non-Linear	0.19	0.019	1.3	0.9	0.16	0.016	0.004	0.0004
Gravel with Sand_1		0.11	0.125	500	500	-	-	-	-	-	-	-	-	-
Sandy Silt		0.105	0.125	100	100	-	-	-	-	-	-	-	-	-
Coarse & Fine Sand		0.115	0.125	550	550	-	-	-	-	-	-	-	-	-
Silt_3		0.12	0.125	175	175	-	-	-	-	-	-	-	-	-
Fine Sand		0.12	0.125	200	200	-	-	-	-	-	-	-	-	-
Gravel with Sand_2		0.125	0.125	500	500	-	-	-	-	-	-	-	-	-

SETTLEMENT ANALYSES

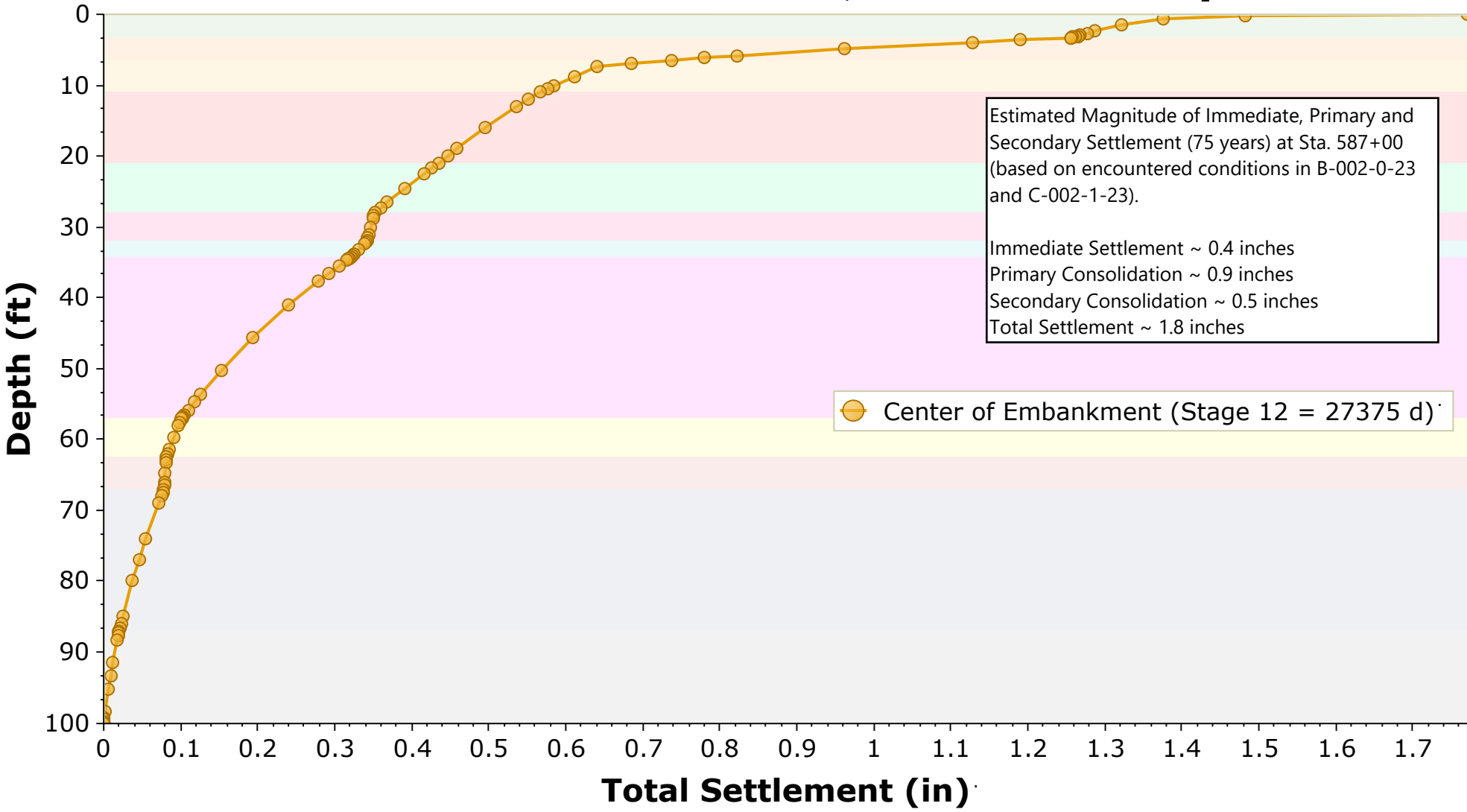
STA. 587+00

PROPOSED EMBANKMENT

CONFIGURATION

(NO SURCHARGE)

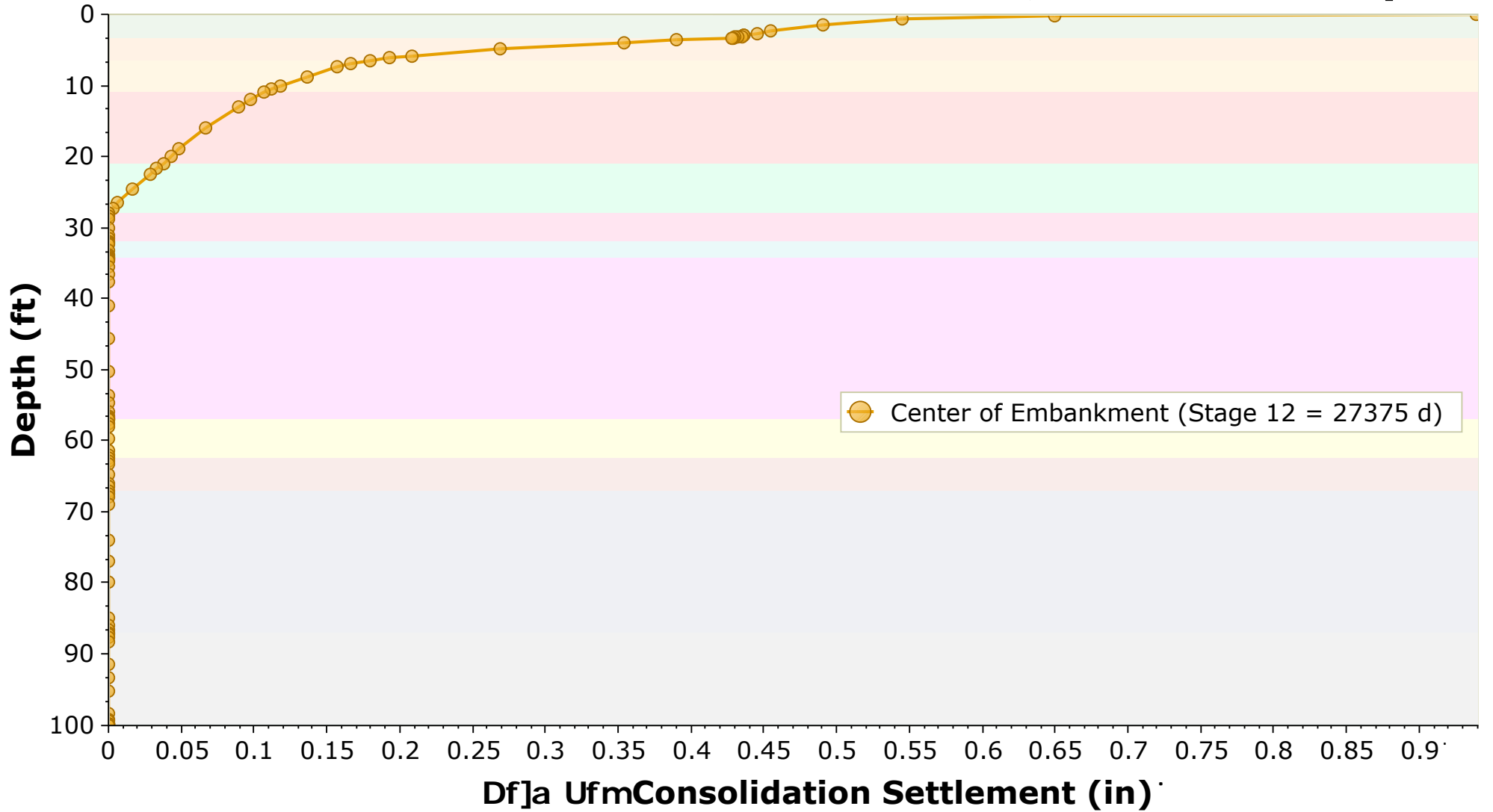
Total Settlement (in) vs. Depth (ft)



SETTLE3 5.025

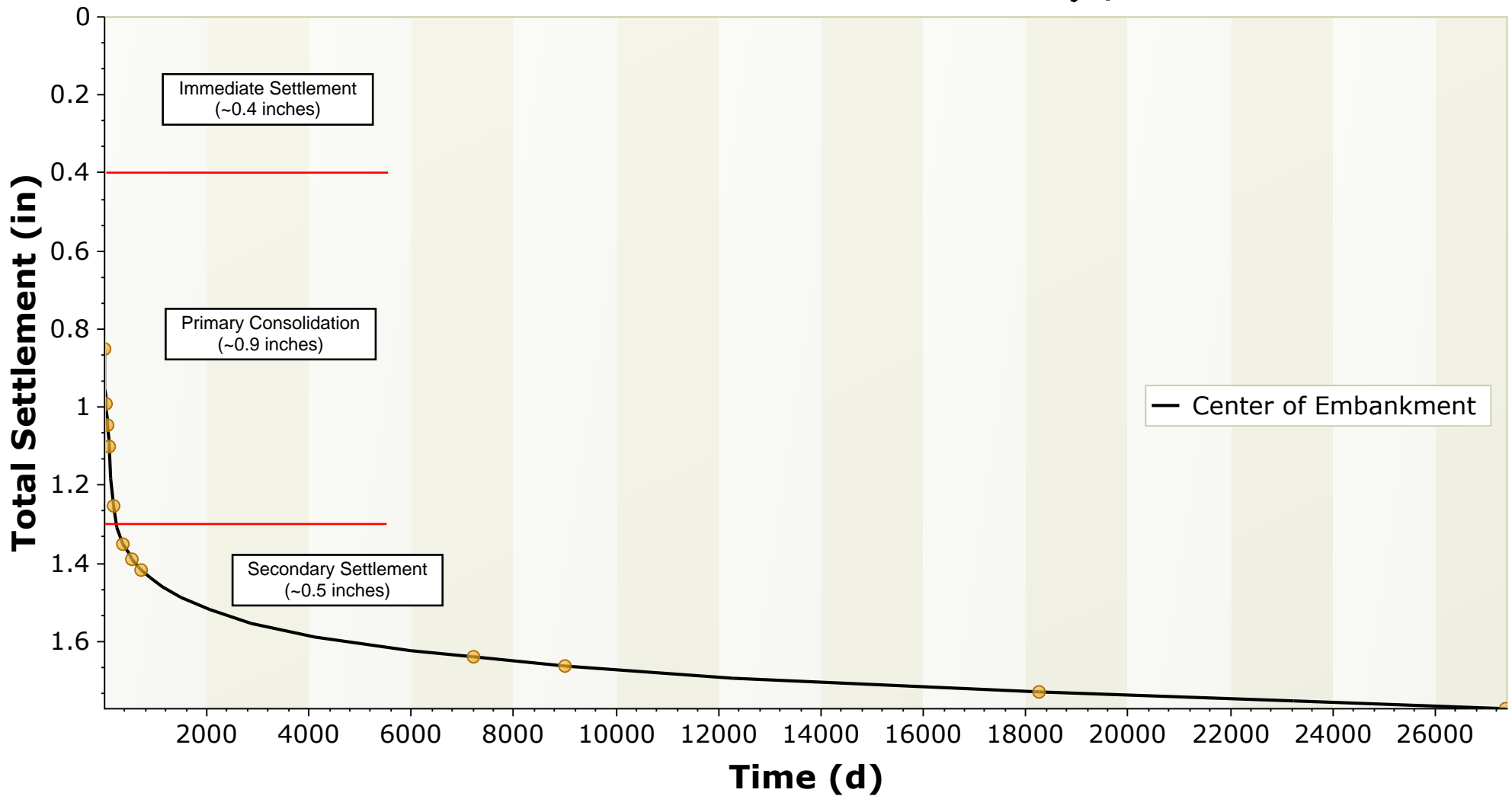
<i>Project</i>	STA-241-17.37 Roundabout & Bridge		
<i>Analysis Description</i>	Sta. 587+00 - Final Embankment Configuration - Wick Drains Spaced at 3 Feet		
<i>Drawn By</i>	DRS	<i>Company</i>	S&ME
<i>Date</i>	3/12/2025	<i>File Name</i>	Sta 587+00 _B-002_ALT-2_Wick Drain_S-3_D-28.s3z

Df]a UfmConsolidation Settlement (in) vs. Depth (ft)



<i>Project</i>	STA-241-17.37 Roundabout & Bridge		
<i>Analysis Description</i>	Sta. 587+00 - Final Embankment Configuration - Wick Drains Spaced at 3 Feet		
<i>Drawn By</i>	DRS	<i>Company</i>	S&ME
<i>Date</i>	3/12/2025	<i>File Name</i>	Sta 587+00 _B-002_ALT-2_Wick Drain_S-3_D-28.s3z

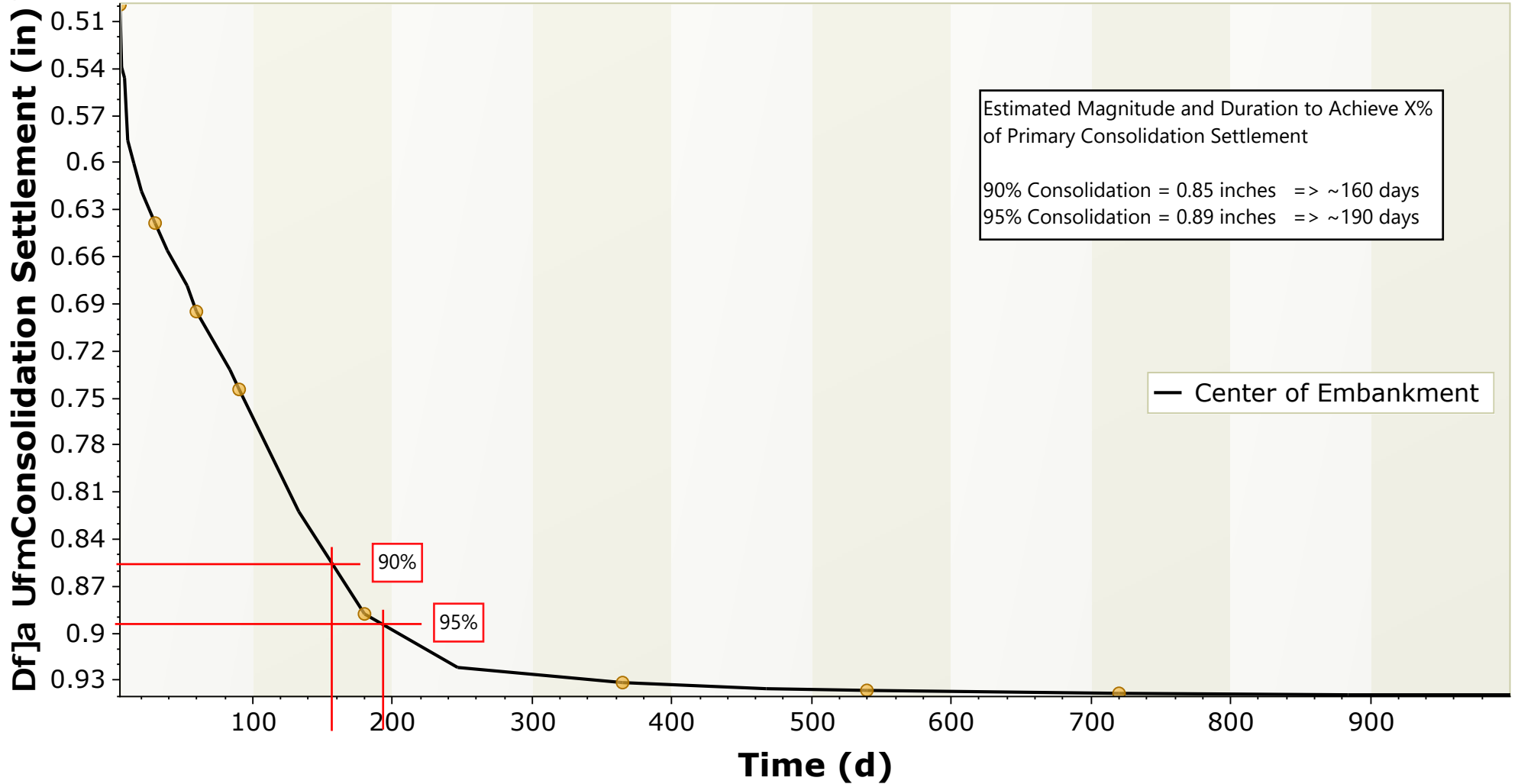
Time vs. Total Settlement (in)



SETTLE3 5.025

Project	STA-241-17.37 Roundabout & Bridge		
Analysis Description	Sta. 587+00 - Final Embankment Configuration - Wick Drains Spaced at 3 Feet		
Drawn By	DRS	Company	S&ME
Date	3/12/2025	File Name	Sta 587+00 _B-002_ALT-2_Wick Drain_S-3_D-28.s3z

Time vs. Deflection Uf Consolidation Settlement (in) , +Z \$\$\$



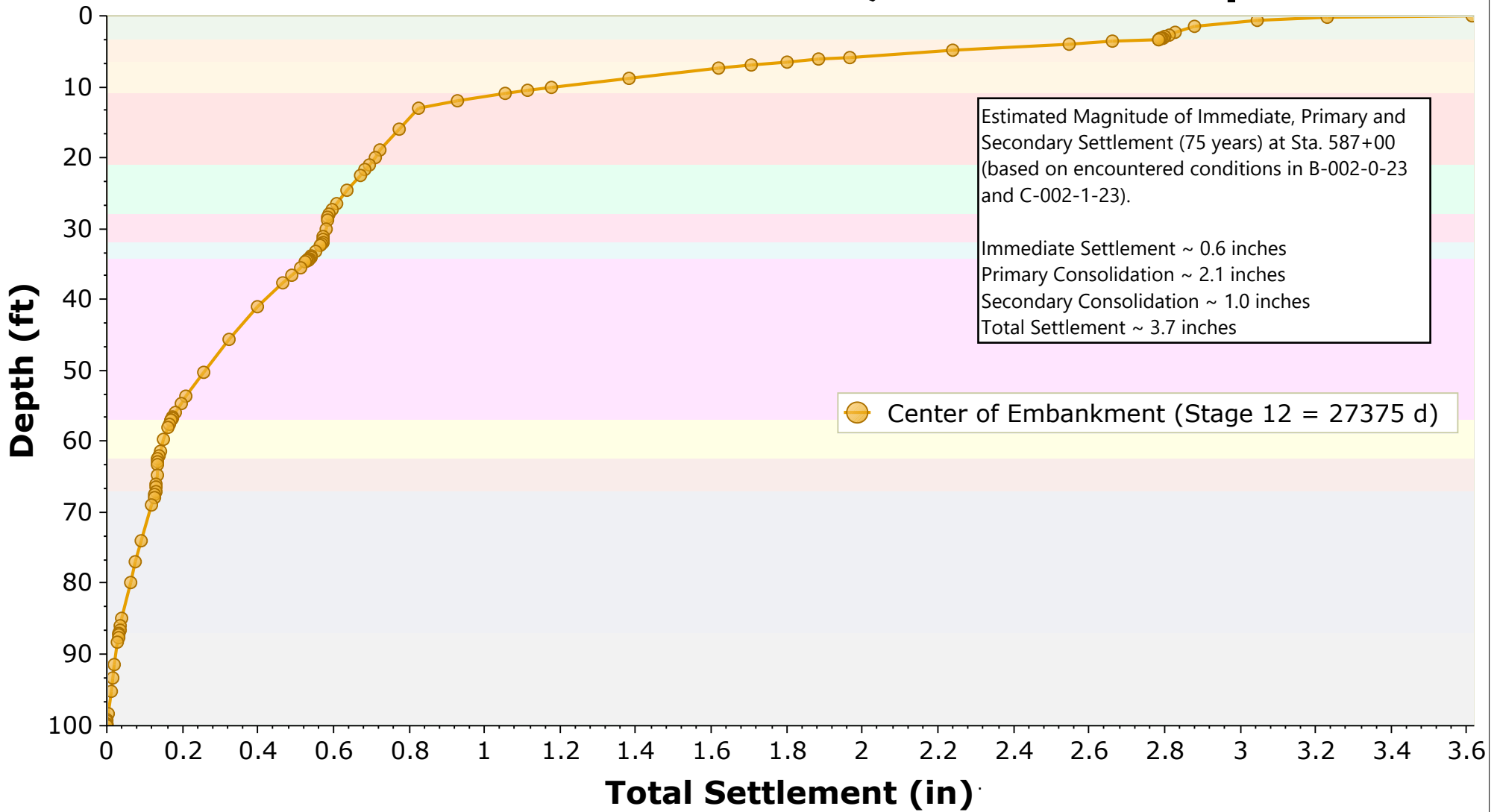
<i>Project</i>	STA-241-17.37 Roundabout & Bridge		
<i>Analysis Description</i>	Sta. 587+00 - Final Embankment Configuration - Wick Drains Spaced at 3 Feet		
<i>Drawn By</i>	DRS	<i>Company</i>	S&ME
<i>Date</i>	3/12/2025	<i>File Name</i>	Sta 587+00 _B-002_ALT-2_Wick Drain_S-3_D-28.s3z

SETTLEMENT ANALYSES

STA. 587+00

SURCHARGE EMBANKMENT
CONFIGURATION

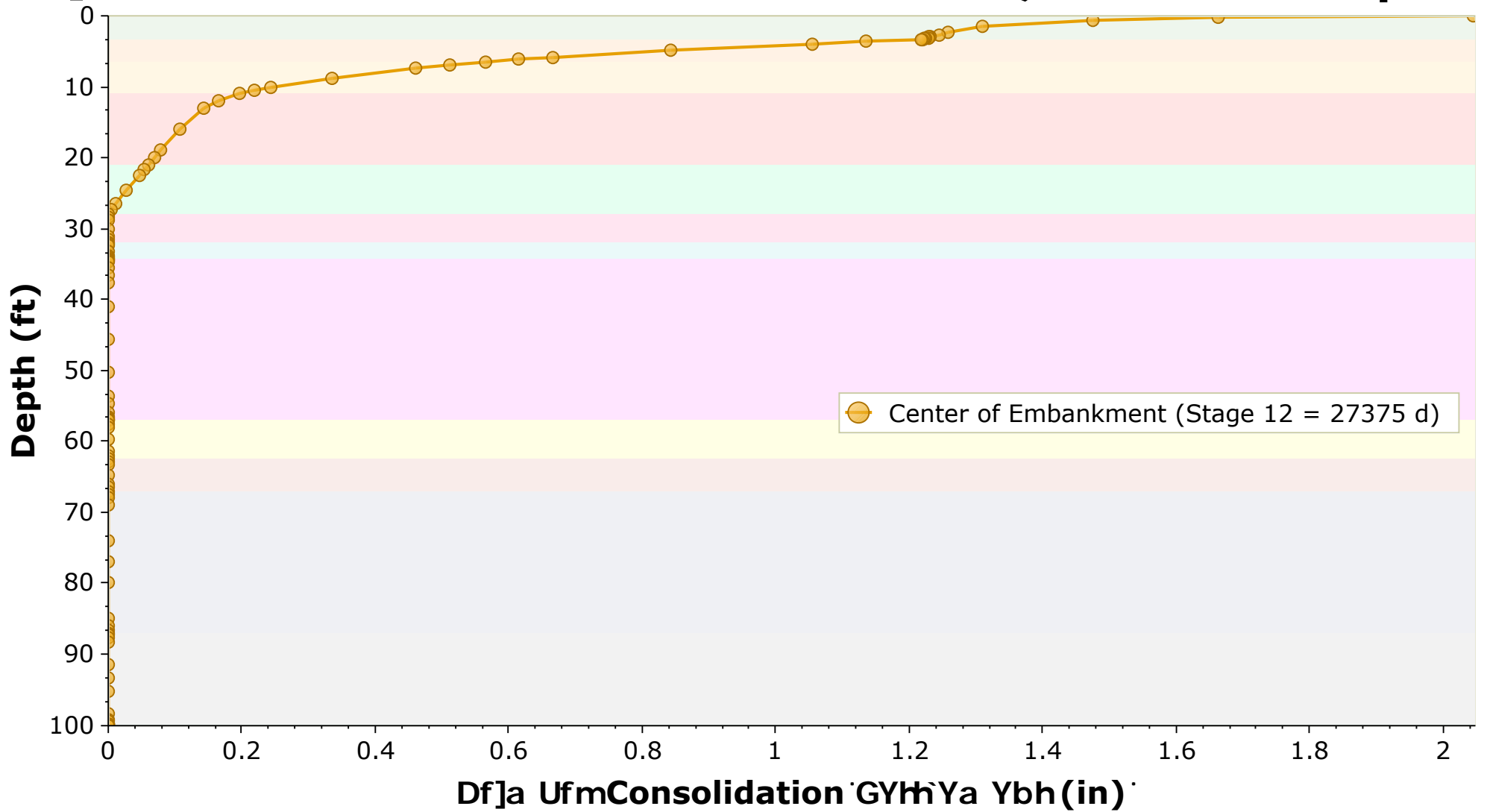
Total Settlement (in) vs. Depth (ft)



SETTLE3 5.025

<i>Project</i>	STA-241-17.37 Roundabout & Bridge		
<i>Analysis Description</i>	Sta. 587+00 - Surcharge Embankment Configuration - Wick Drains Spaced at 3 Feet		
<i>Drawn By</i>	DRS	<i>Company</i>	S&ME
<i>Date</i>	3/12/2025	<i>File Name</i>	Sta 587+00 _B-002_ALT-2_Wick Drain_S-3_D-28_Embk+1.s3z

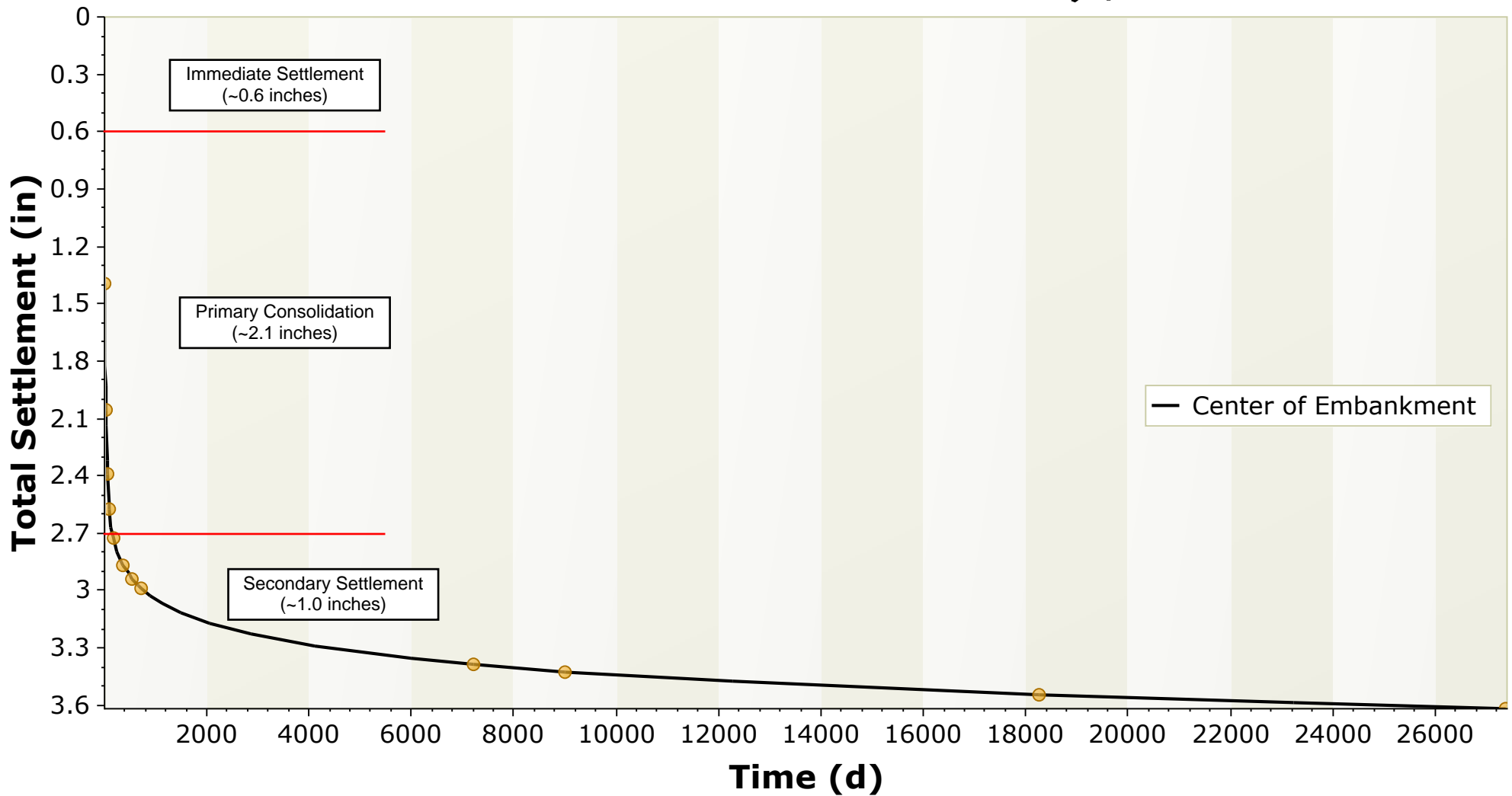
Df]a UfmConsolidation Settlement 'Uh`GhU"') , +Ž \$\$\$ vs. Depth



SETTLE3 5.025

<i>Project</i>	STA-241-17.37 Roundabout & Bridge		
<i>Analysis Description</i>	Sta. 587+00 - Surcharge Embankment Configuration - Wick Drains Spaced at 3 Feet		
<i>Drawn By</i>	DRS	<i>Company</i>	S&ME
<i>Date</i>	3/12/2025	<i>File Name</i>	Sta 587+00 _B-002_ALT-2_Wick Drain_S-3_D-28_Embk+1.s3z

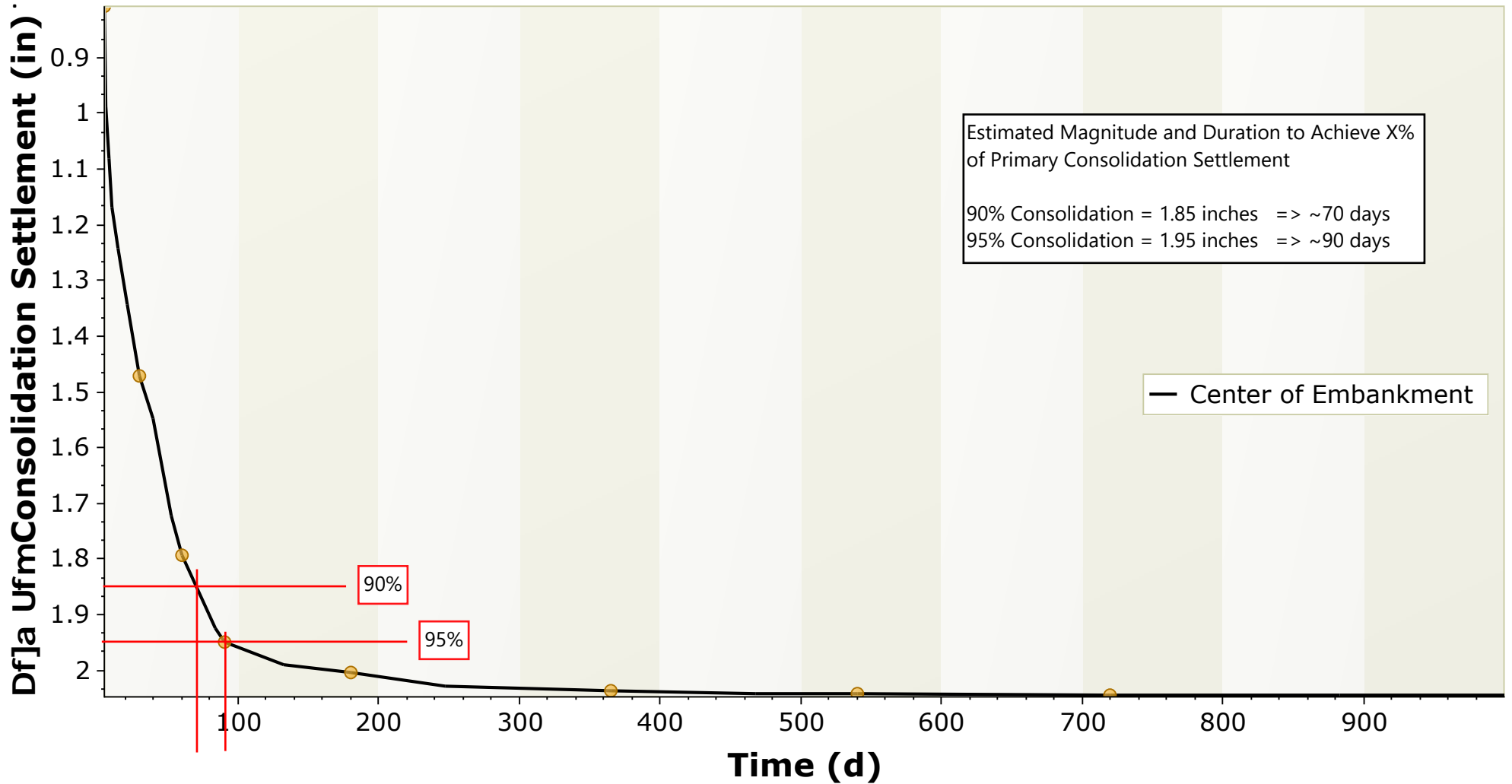
Time vs. Total Settlement (in)



SETTLE3 5.025

Project	STA-241-17.37 Roundabout & Bridge		
Analysis Description	Sta. 587+00 - Surcharge Embankment Configuration - Wick Drains Spaced at 3 Feet		
Drawn By	DRS	Company	S&ME
Date	3/12/2025	File Name	Sta 587+00 _B-002_ALT-2_Wick Drain_S-3_D-28_Embk+1.s3z

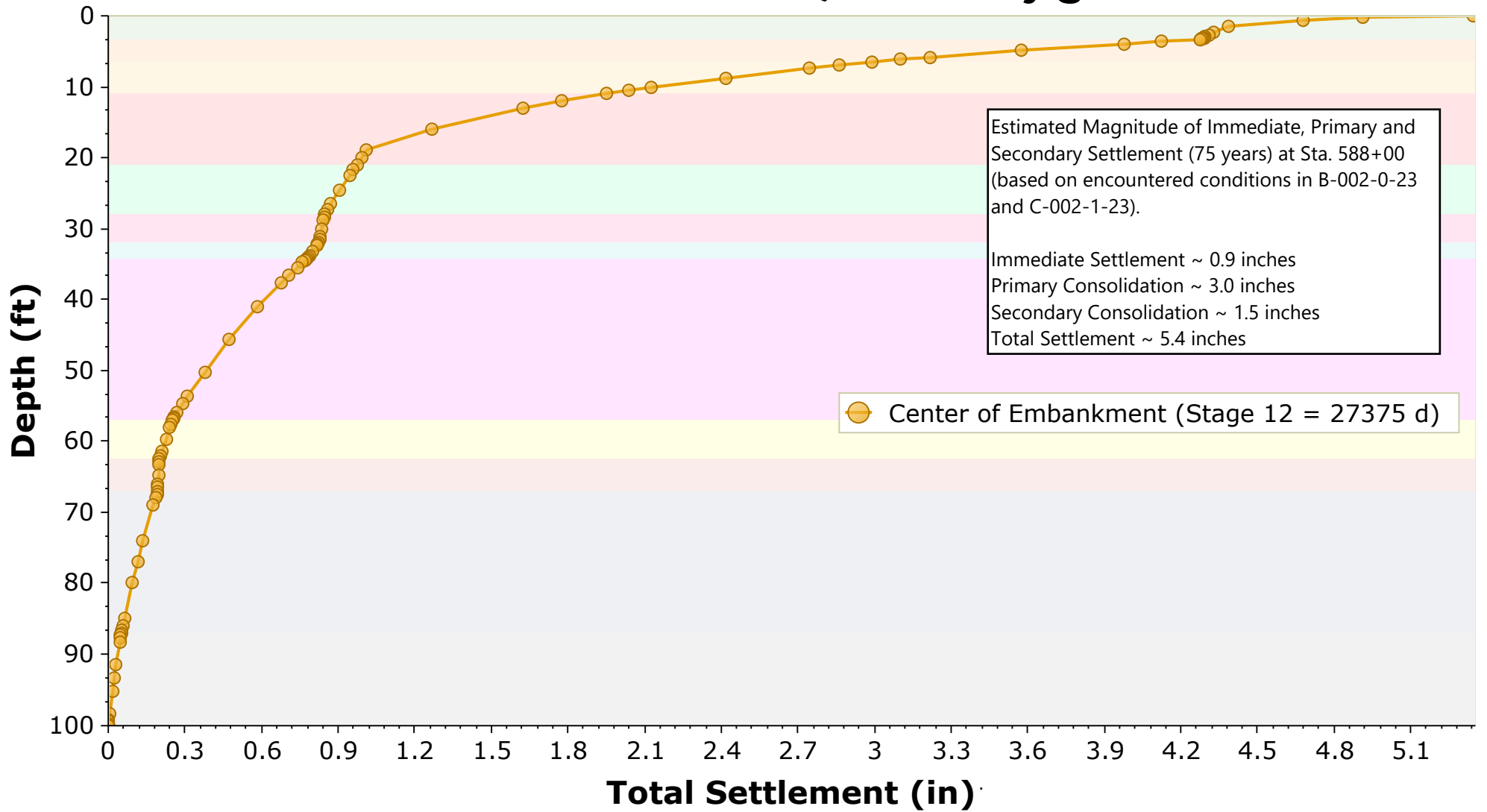
Time vs. Deflection of Primary Consolidation Settlement (in) , +Z \$\$\$



Project	STA-241-17.37 Roundabout & Bridge		
Analysis Description	Sta. 587+00 - Surcharge Embankment Configuration - Wick Drains Spaced at 3 Feet		
Drawn By	DRS	Company	S&ME
Date	3/12/2025	File Name	Sta 587+00 _B-002_ALT-2_Wick Drain_S-3_D-28_Embk+1.s3z

SETTLEMENT ANALYSES
STA. 588+00 (REAR ABUTMENT)
PROPOSED EMBANKMENT
CONFIGURATION
(NO SURCHARGE)

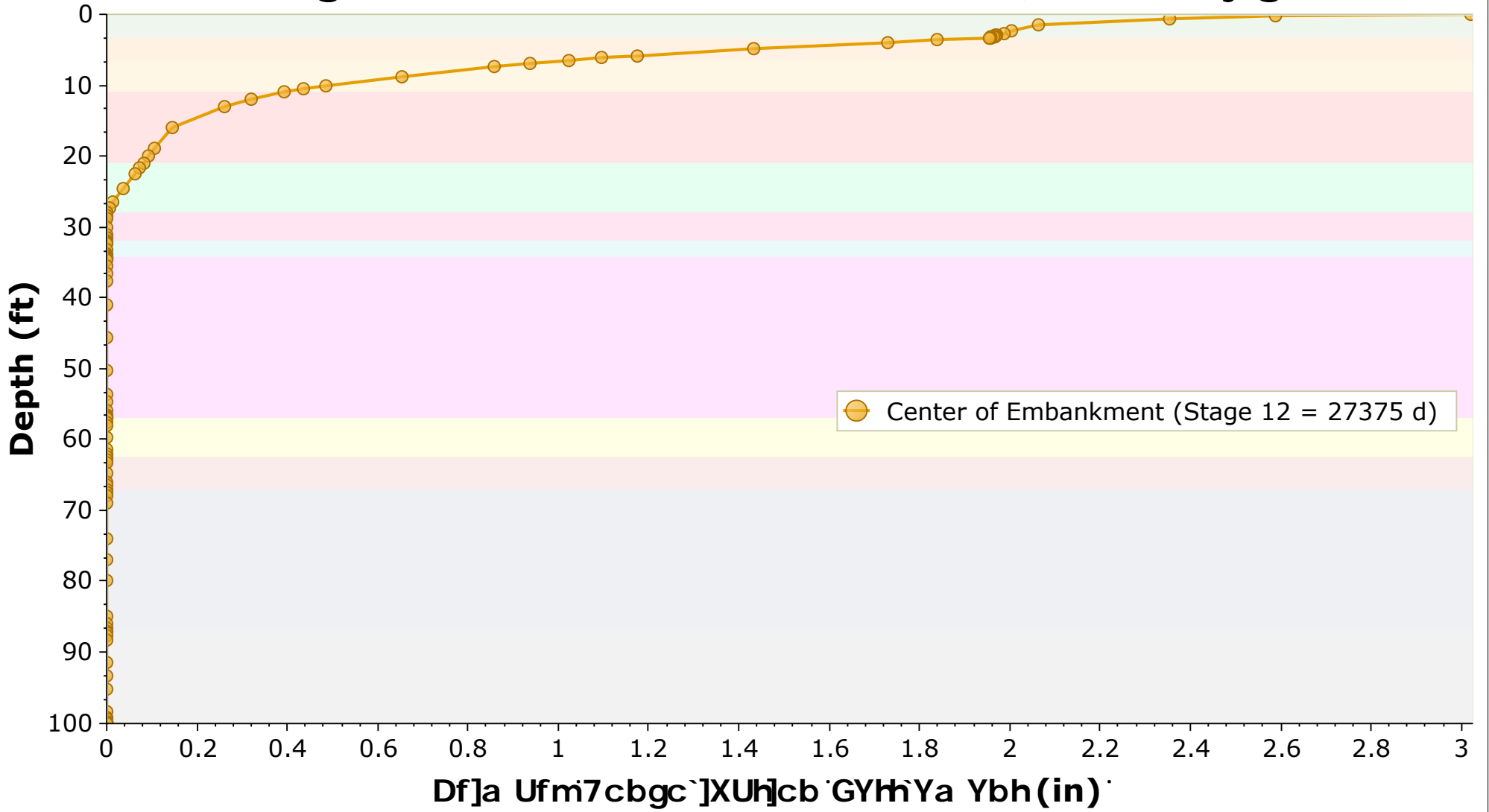
HchU`GYh`Ya Ybh`Uh`GHU"') , , Ž \$\$\$ j g" 8YdhΛ`



<i>Project</i>	STA-241-17.37 Roundabout & Bridge		
<i>Analysis Description</i>	Sta. 588+00 - Final Embankment Configuration - Wick Drains Spaced at 3 Feet		
<i>Drawn By</i>	DRS	<i>Company</i>	S&ME
<i>Date</i>	3/12/2025	<i>File Name</i>	Sta 588+00_B-002_ALT-2_Wick Drain_S-3_D-28.s3z

SETTLE3 5.025

Df]a Ufm7cbgc`]XUh]cb`GYh`Ya Ybh`Uh`GHU"') , , Ž \$\$\$ j g"8Ydh`

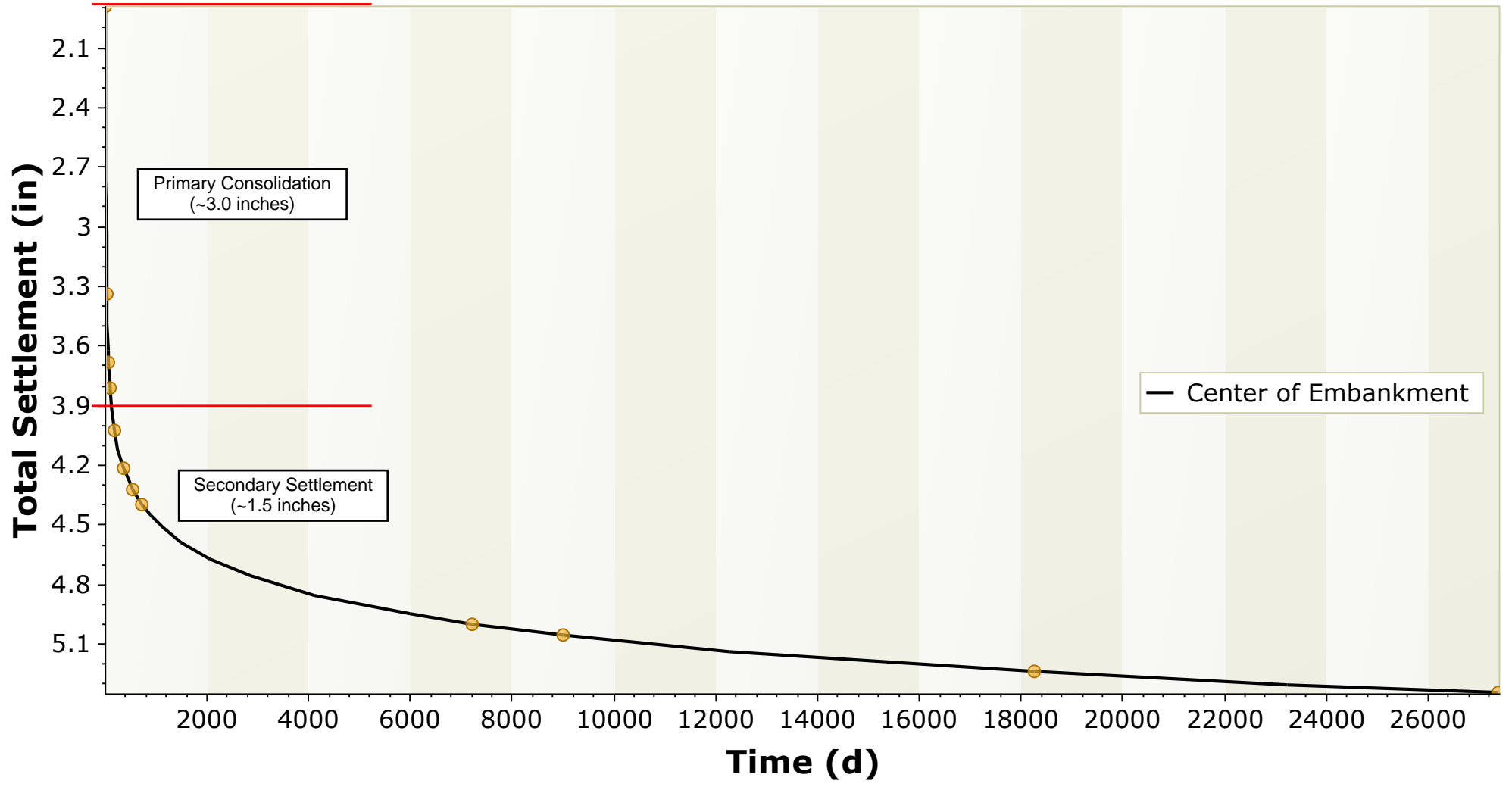


SETTLE3 5.025

<i>Project</i>	STA-241-17.37 Roundabout & Bridge		
<i>Analysis Description</i>	Sta. 588+00 - Final Embankment Configuration - Wick Drains Spaced at 3 Feet! 'Bc`G`fWUf[Y`		
<i>Drawn By</i>	DRS	<i>Company</i>	S&ME
<i>Date</i>	3/12/2025	<i>File Name</i>	Sta 588+00_B-002_ALT-2_Wick Drain_S-3_D-28.s3z

Immediate Settlement
(~0.9 inches)

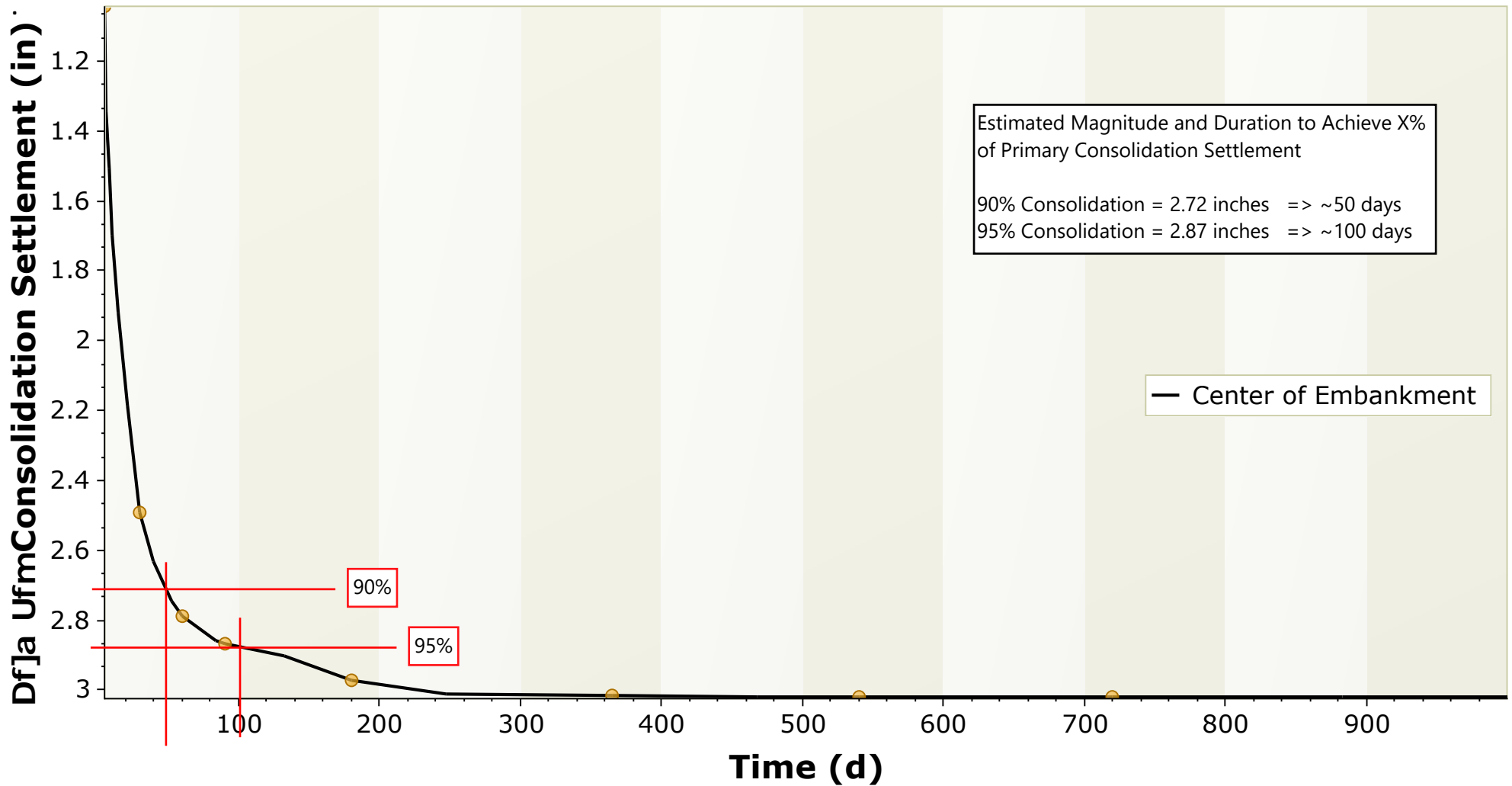
H]a Y'j g''HchU`GYh`Ya Ybh'Uh'GhU'' , , Ž \$\$\$`



SETTLE3 5.025

Project	STA-241-17.37 Roundabout & Bridge		
Analysis Description	Sta. 588+00 - Final Embankment Configuration - Wick Drains Spaced at 3 Feet! 'Bc`G`fWUf[Y`		
Drawn By	DRS	Company	S&ME
Date	3/12/2025	File Name	Sta 588+00_B-002_ALT-2_Wick Drain_S-3_D-28.s3z

H]a Y j g" Df]a Ufm7cbgc`]XUh]cb`GYh`Ya Ybh`Uh`GhU"`) , , Ž \$\$\$`

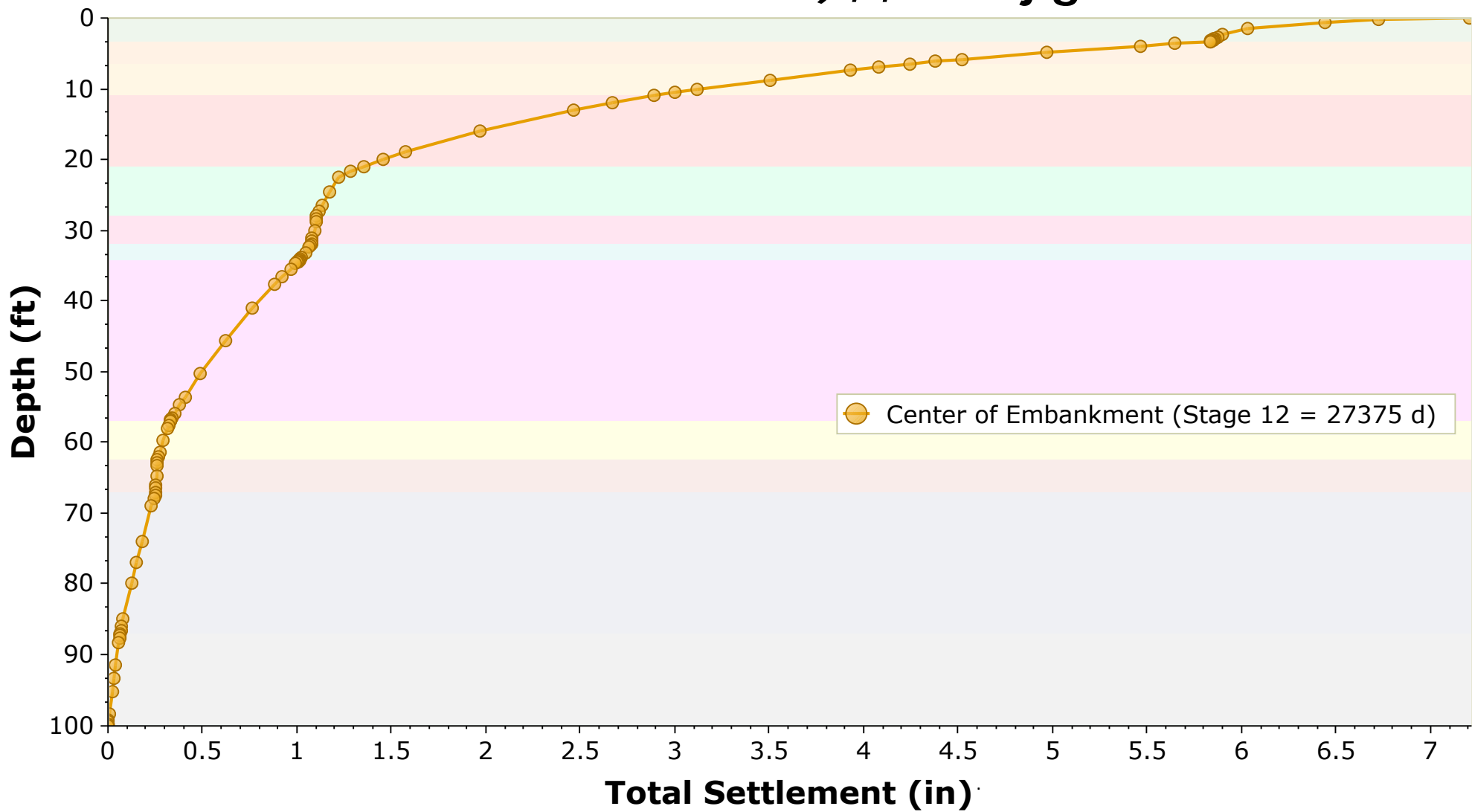


SETTLE3 5.025

<i>Project</i>	STA-241-17.37 Roundabout & Bridge		
<i>Analysis Description</i>	Sta. 588+00 - Final Embankment Configuration - Wick Drains Spaced at 3 Feet!`Bc`G`fWUf[Y`		
<i>Drawn By</i>	DRS	<i>Company</i>	S&ME
<i>Date</i>	3/12/2025	<i>File Name</i>	Sta 588+00_B-002_ALT-2_Wick Drain_S-3_D-28.s3z

SETTLEMENT ANALYSES
STA. 588+00 (REAR ABUTMENT)
SURCHARGE EMBANKMENT
CONFIGURATION

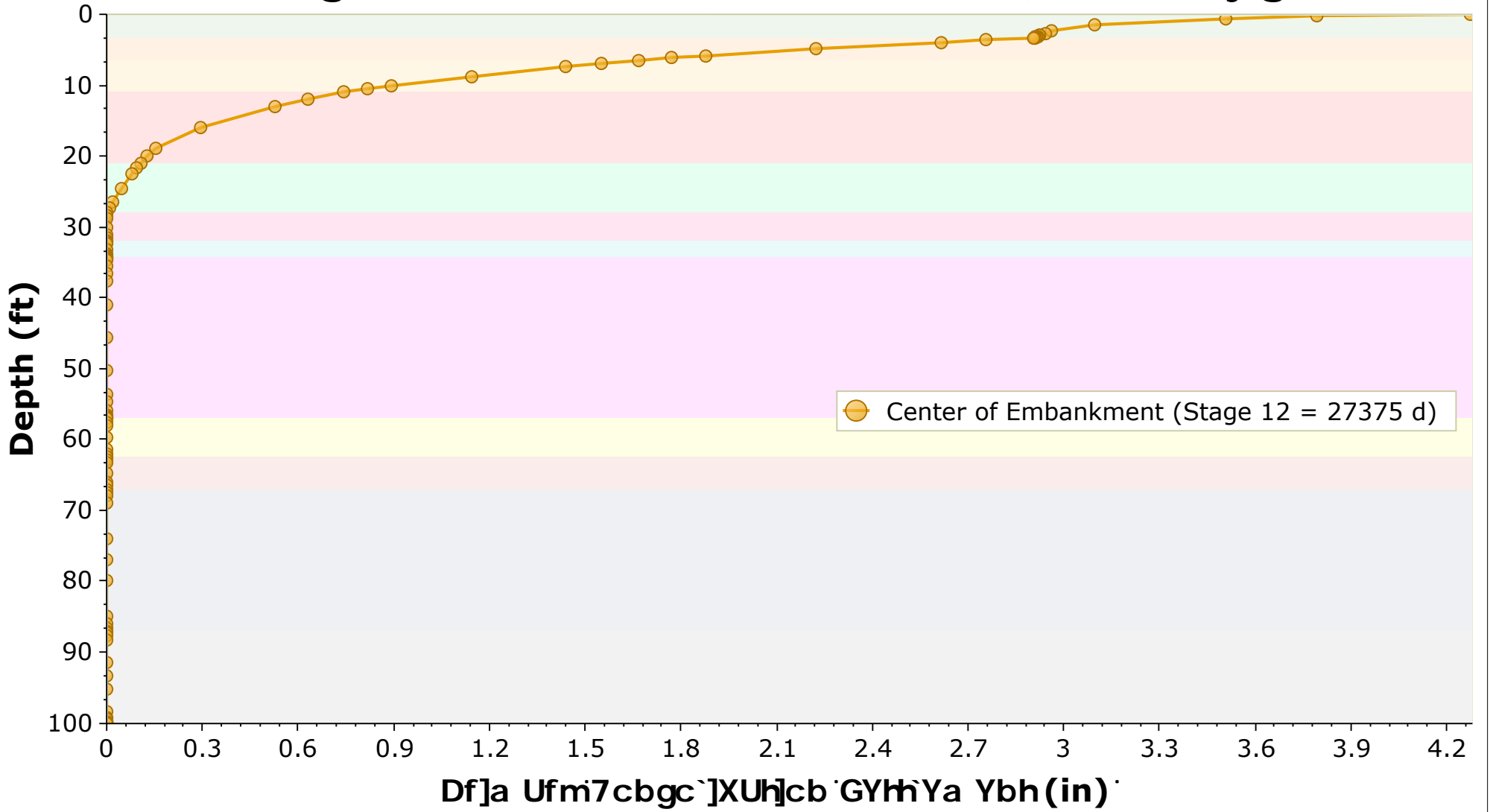
HchU`GYh`Ya Ybh`Uh`GHU"') , , Ž \$\$\$ j g"8Ydh`



SETTLE3 5.025

<i>Project</i>	STA-241-17.37 Roundabout & Bridge		
<i>Analysis Description</i>	Sta. 588+00 - Surcharge Embankment Configuration - Wick Drains Spaced at 3 Feet		
<i>Drawn By</i>	DRS	<i>Company</i>	S&ME
<i>Date</i>	3/12/2025	<i>File Name</i>	Sta 588+00_B-002_ALT-2_Wick Drain_S-3_D-28_Embk+1.s3z

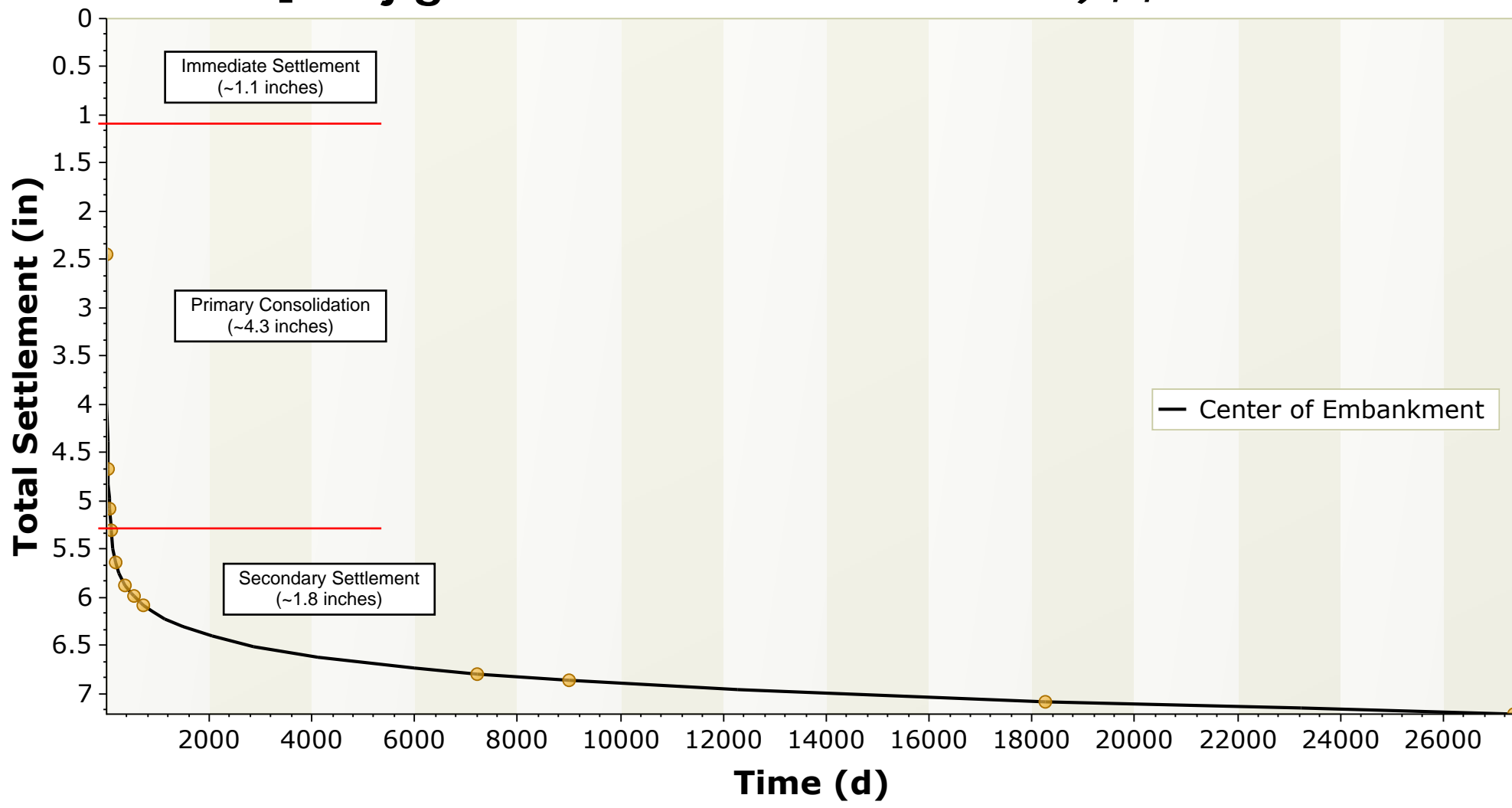
Df]a Ufm7cbgc`]XUh]cb`GYh`Ya Ybh`Uh`GhU" , , Ž \$\$\$ j g" 8YdH`



<i>Project</i>	STA-241-17.37 Roundabout & Bridge		
<i>Analysis Description</i>	Sta. 588+00 - Surcharge Embankment Configuration - Wick Drains Spaced at 3 Feet		
<i>Drawn By</i>	DRS	<i>Company</i>	S&ME
<i>Date</i>	3/12/2025	<i>File Name</i>	Sta 588+00_B-002_ALT-2_Wick Drain_S-3_D-28_Embk+1.s3z

SETTLE3 5.025

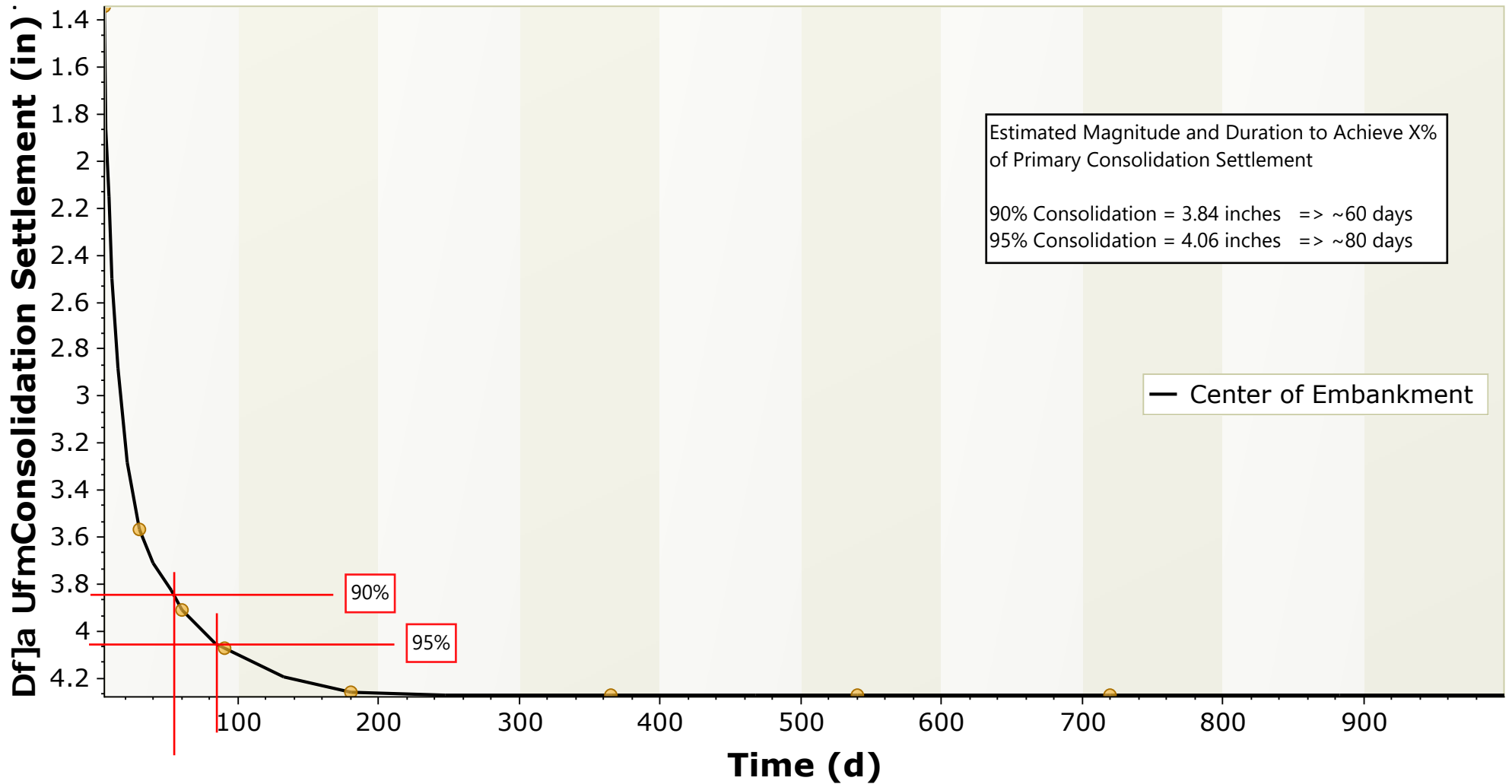
H]a Y'j g''HchU`GYh`Ya Ybh'Uh'GHU'' , , Ž \$\$\$`



SETTLE3 5.025

<i>Project</i>	STA-241-17.37 Roundabout & Bridge		
<i>Analysis Description</i>	Sta. 588+00 - Surcharge Embankment Configuration - Wick Drains Spaced at 3 Feet		
<i>Drawn By</i>	DRS	<i>Company</i>	S&ME
<i>Date</i>	3/12/2025	<i>File Name</i>	Sta 588+00_B-002_ALT-2_Wick Drain_S-3_D-28_Embk+1.s3z

Estimated Magnitude and Duration to Achieve X% of Primary Consolidation Settlement



SETTLE3 5.025

Project	STA-241-17.37 Roundabout & Bridge		
Analysis Description	Sta. 588+00 - Surcharge Embankment Configuration - Wick Drains Spaced at 3 Feet		
Drawn By	DRS	Company	S&ME
Date	3/12/2025	File Name	Sta 588+00_B-002_ALT-2_Wick Drain_S-3_D-28_Embk+1.s3z

SETTLEMENT ANALYSES

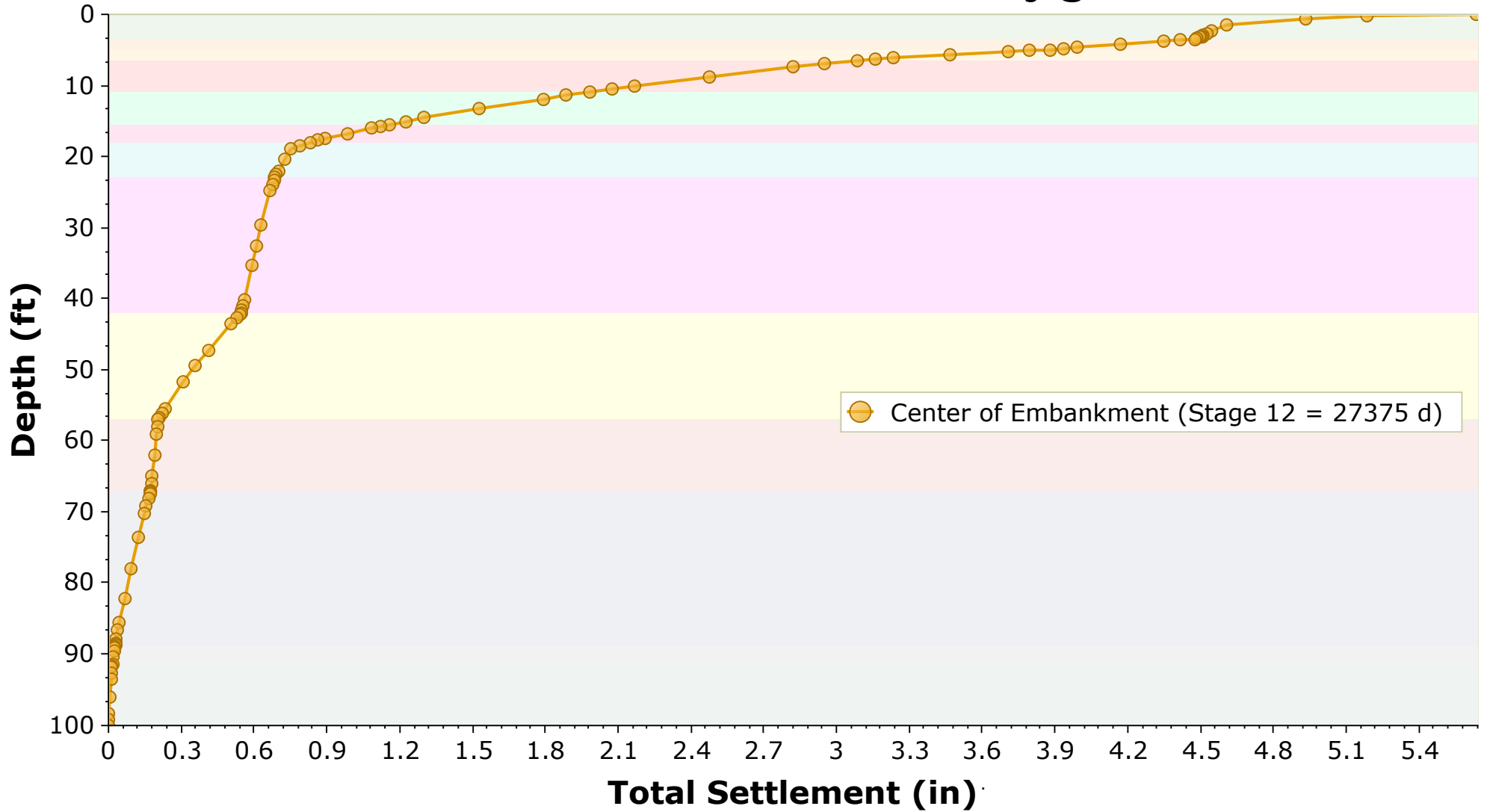
STA. 589+00 (FORWARD ABUTMENT)

PROPOSED EMBANKMENT

CONFIGURATION

(NO SURCHARGE)

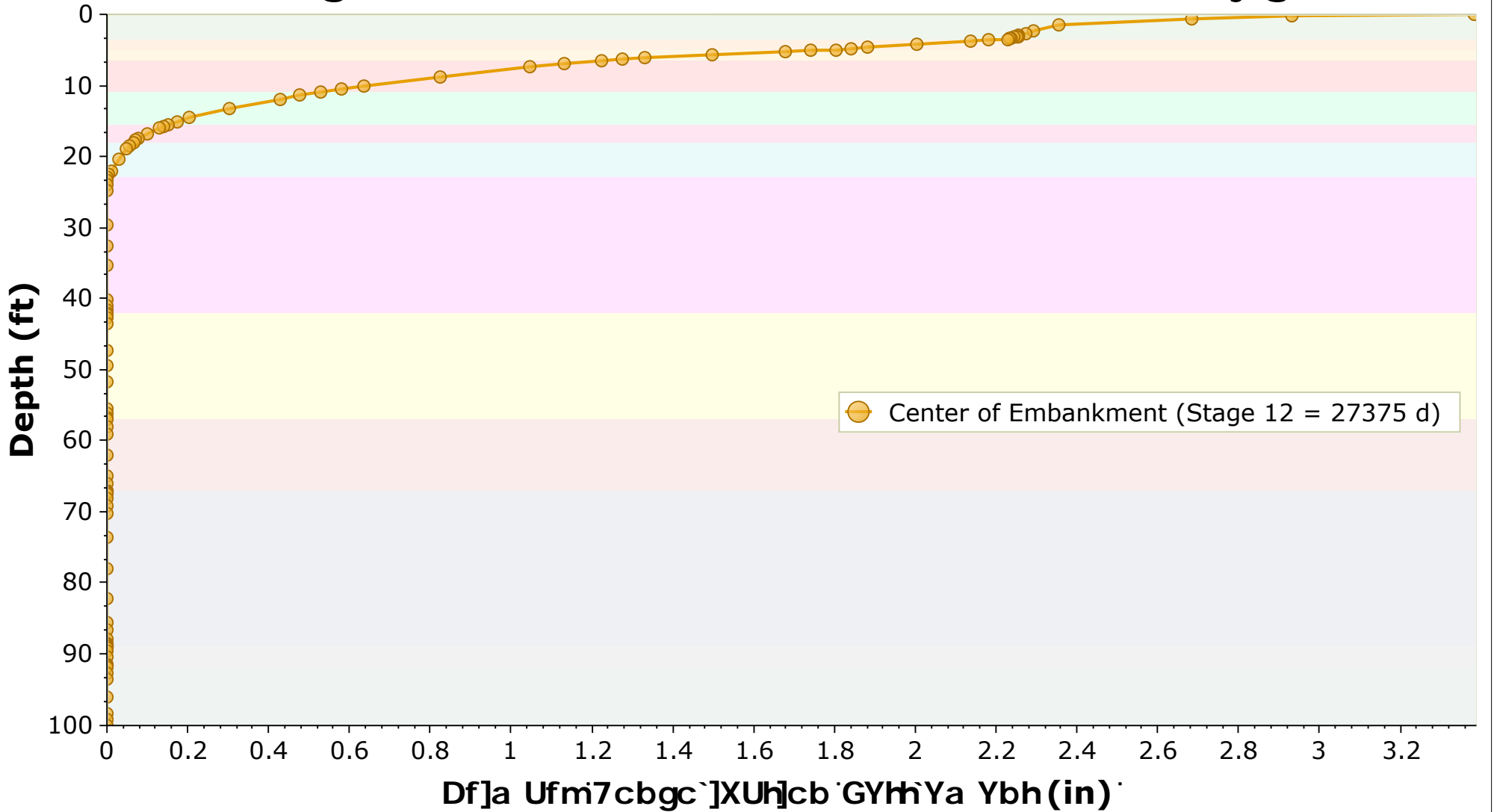
HchU`GYh`Ya Ybh`Uh`GHU"`) , - Ž \$\$\$ j g" 8Ydh`



SETTLE3 5.025

<i>Project</i>	STA-241-17.37 Roundabout & Bridge		
<i>Analysis Description</i>	Sta. 589+00 - Final Embankment Configuration - Wick Drains Spaced at 3 Feet! 'Bc`G`fWUf[Y`		
<i>Drawn By</i>	DRS	<i>Company</i>	S&ME
<i>Date</i>	3/12/2025	<i>File Name</i>	Sta 589+00_B-003_ALT-2_Wick Drain_S-3_D-24.s3z

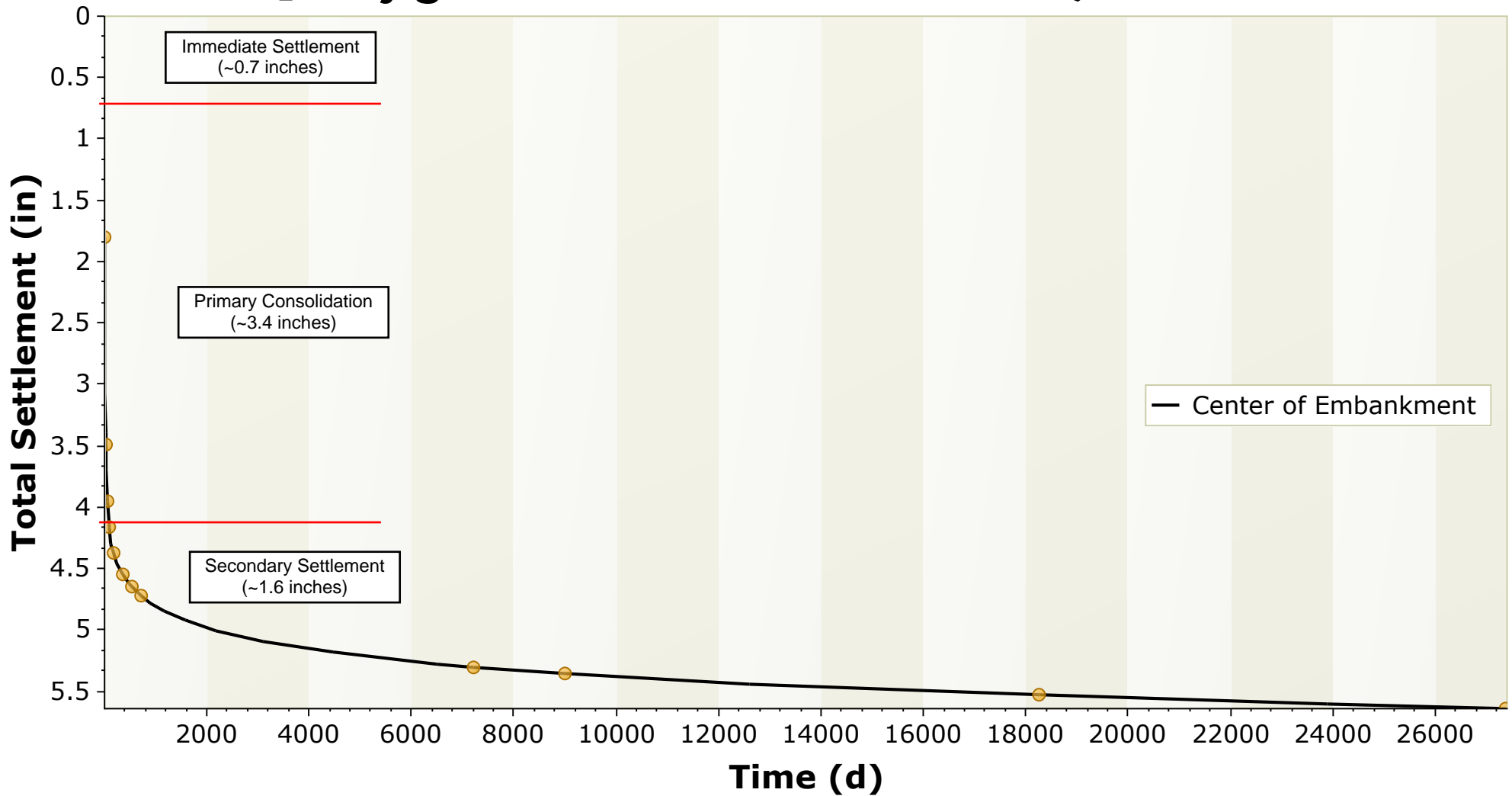
Df]a Ufm7cbgc`]XUh]cb`GYh`Ya Ybh`Uh`GhU"') , - Ž \$\$ j g" 8Ydh`



SETTLE3 5.025

Project	STA-241-17.37 Roundabout & Bridge		
Analysis Description	Sta. 589+00 - Final Embankment Configuration - Wick Drains Spaced at 3 Feet! 'Bc`G`fWUf[Y`		
Drawn By	DRS	Company	S&ME
Date	3/12/2025	File Name	Sta 589+00_B-003_ALT-2_Wick Drain_S-3_D-24.s3z

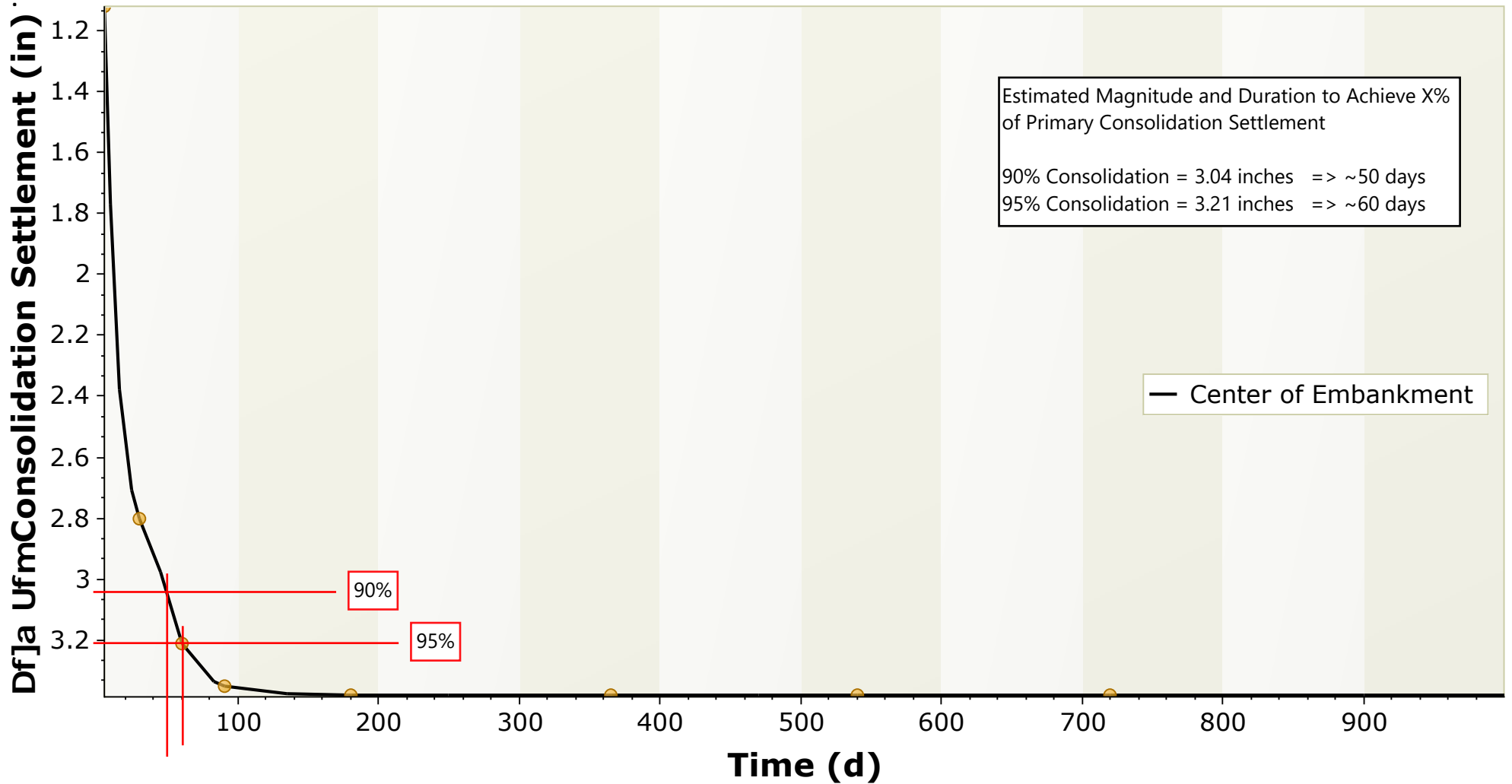
Final Settlement Analysis - Center of Embankment



SETTLE3 5.025

Project	STA-241-17.37 Roundabout & Bridge		
Analysis Description	Sta. 589+00 - Final Embankment Configuration - Wick Drains Spaced at 3 Feet		
Drawn By	DRS	Company	S&ME
Date	3/12/2025	File Name	Sta 589+00_B-003_ALT-2_Wick Drain_S-3_D-24.s3z

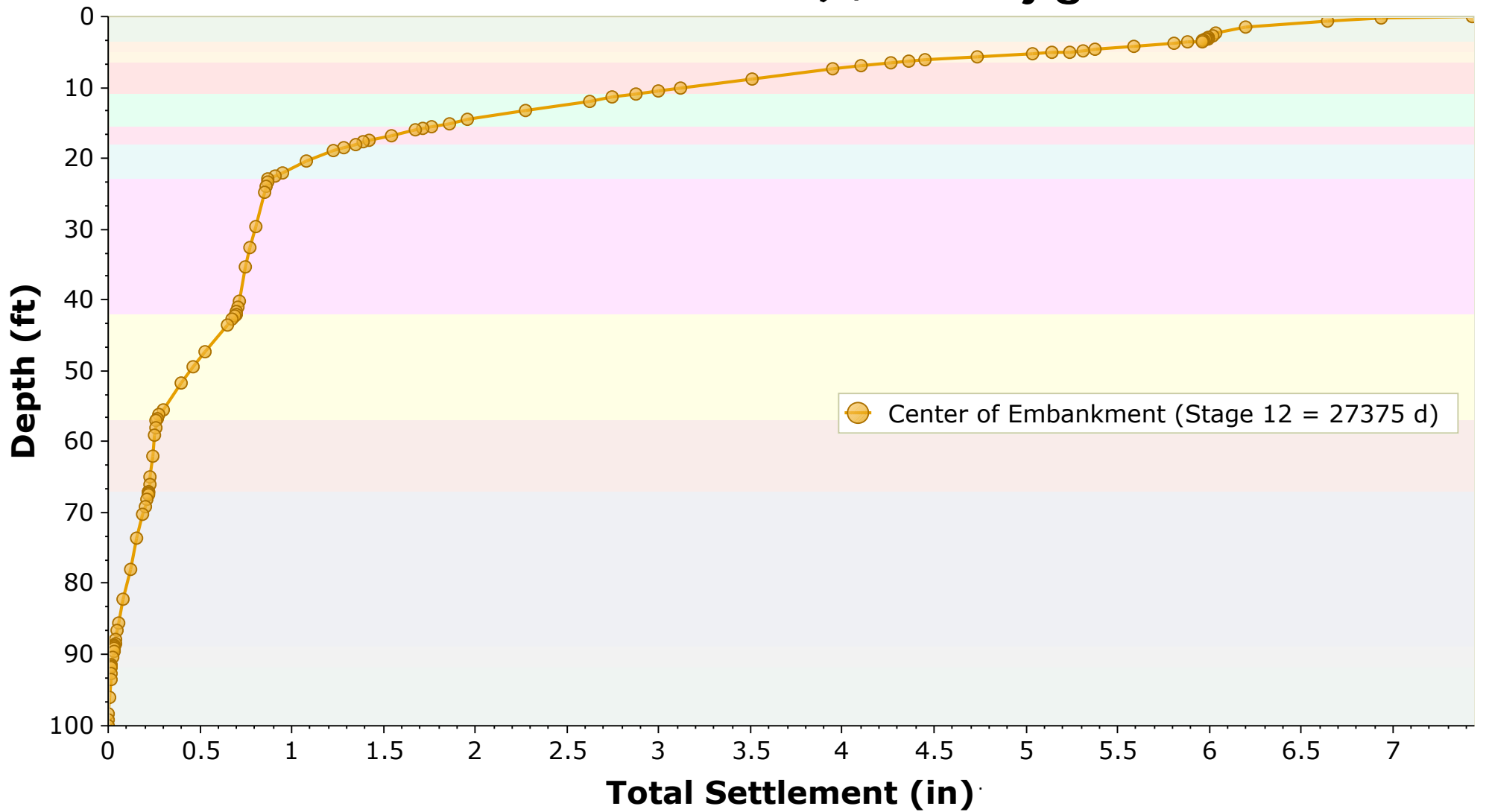
H]a Y'j g''Df]a Ufm7cbgc`]XUh]cb`GYh`Ya Ybh`Uh`GhU'' , - Ž \$\$\$`



<i>Project</i>	STA-241-17.37 Roundabout & Bridge		
<i>Analysis Description</i>	Sta. 589+00 - Final Embankment Configuration - Wick Drains Spaced at 3 Feet		
<i>Drawn By</i>	DRS	<i>Company</i>	S&ME
<i>Date</i>	3/12/2025	<i>File Name</i>	Sta 589+00_B-003_ALT-2_Wick Drain_S-3_D-24.s3z

SETTLEMENT ANALYSES
STA. 589+00 (FORWARD ABUTMENT)
SURCHARGE EMBANKMENT
CONFIGURATION

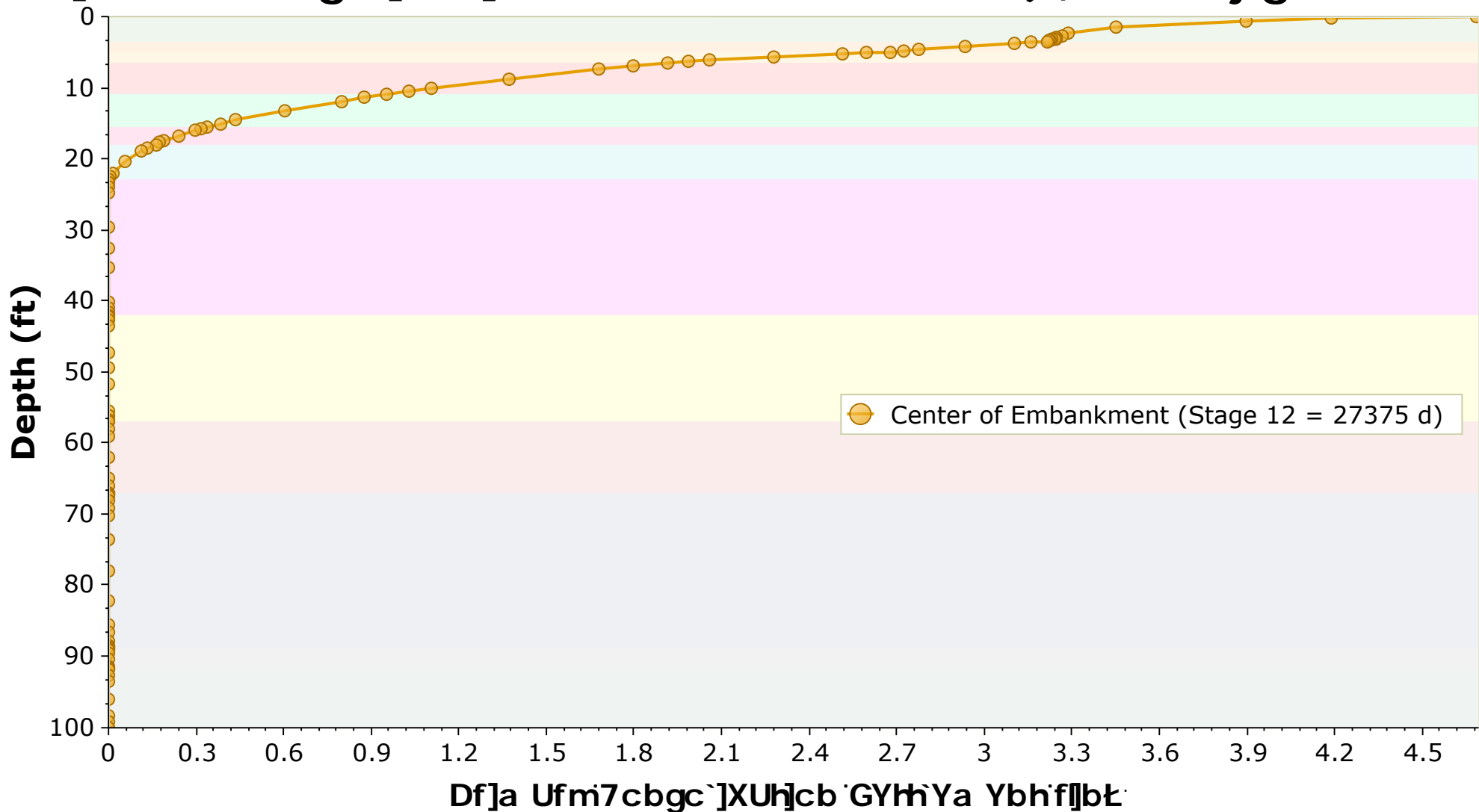
HchU`GYh`Ya Ybh`Uh`GHU"') , - Ž \$\$\$ j g" 8Ydh`



SETTLE3 5.025

<i>Project</i>	STA-241-17.37 Roundabout & Bridge		
<i>Analysis Description</i>	Sta. 589+00 - Surcharge Embankment Configuration - Wick Drains Spaced at 3 Feet		
<i>Drawn By</i>	DRS	<i>Company</i>	S&ME
<i>Date</i>	3/12/2025	<i>File Name</i>	Sta 589+00_B-003_ALT-2_Wick Drain_S-3_D-24_Embk+1.s3z

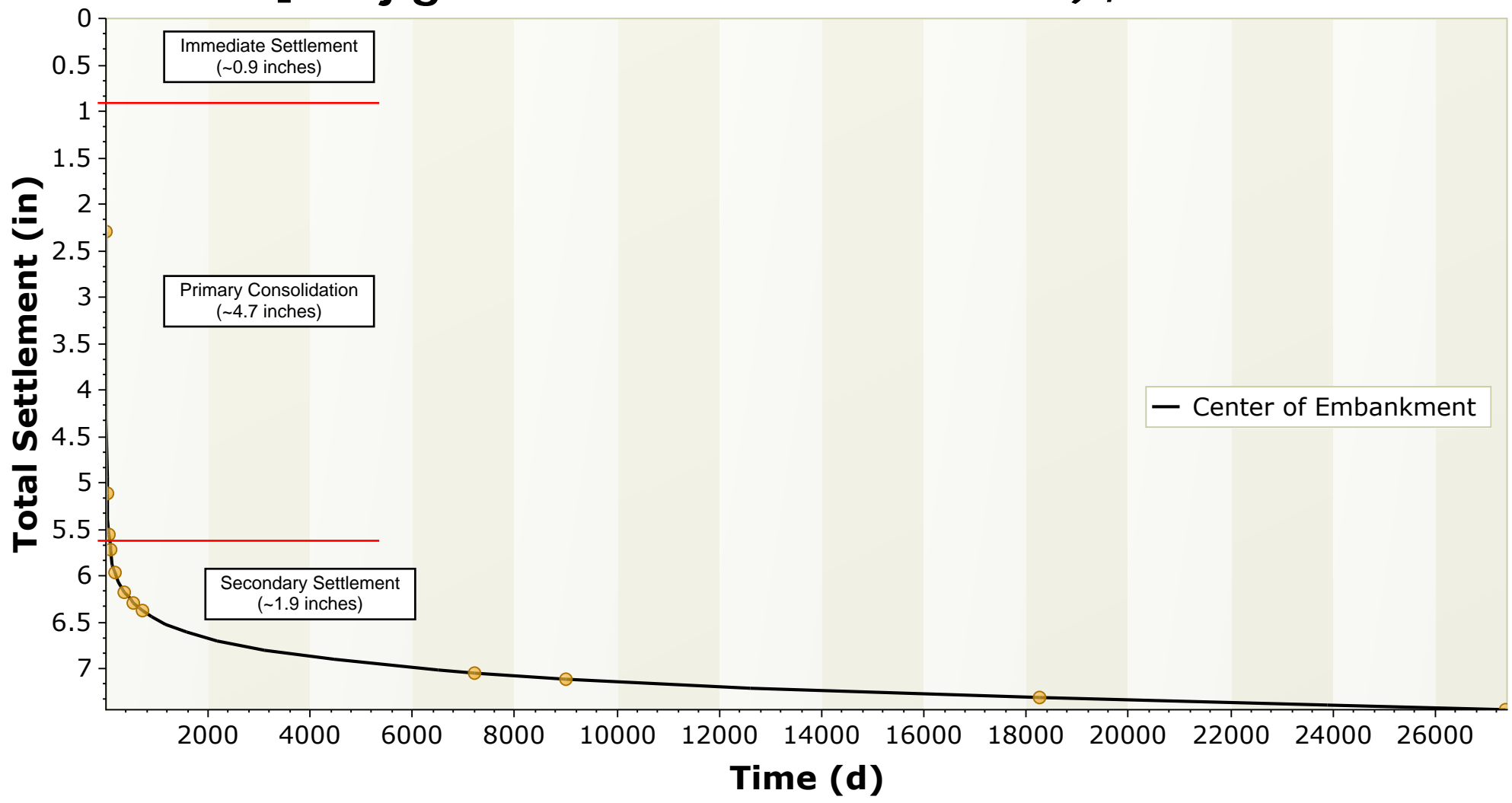
Df]a Ufm7cbgc`]XUh]cb`GYh`Ya Ybh`Uh`GHU"`) , - Ž \$\$\$ j g" 8Ydh`



SETTLE3 5.025

Project	STA-241-17.37 Roundabout & Bridge		
Analysis Description	Sta. 589+00 - Surcharge Embankment Configuration - Wick Drains Spaced at 3 Feet		
Drawn By	DRS	Company	S&ME
Date	3/12/2025	File Name	Sta 589+00_B-003_ALT-2_Wick Drain_S-3_D-24_Embk+1.s3z

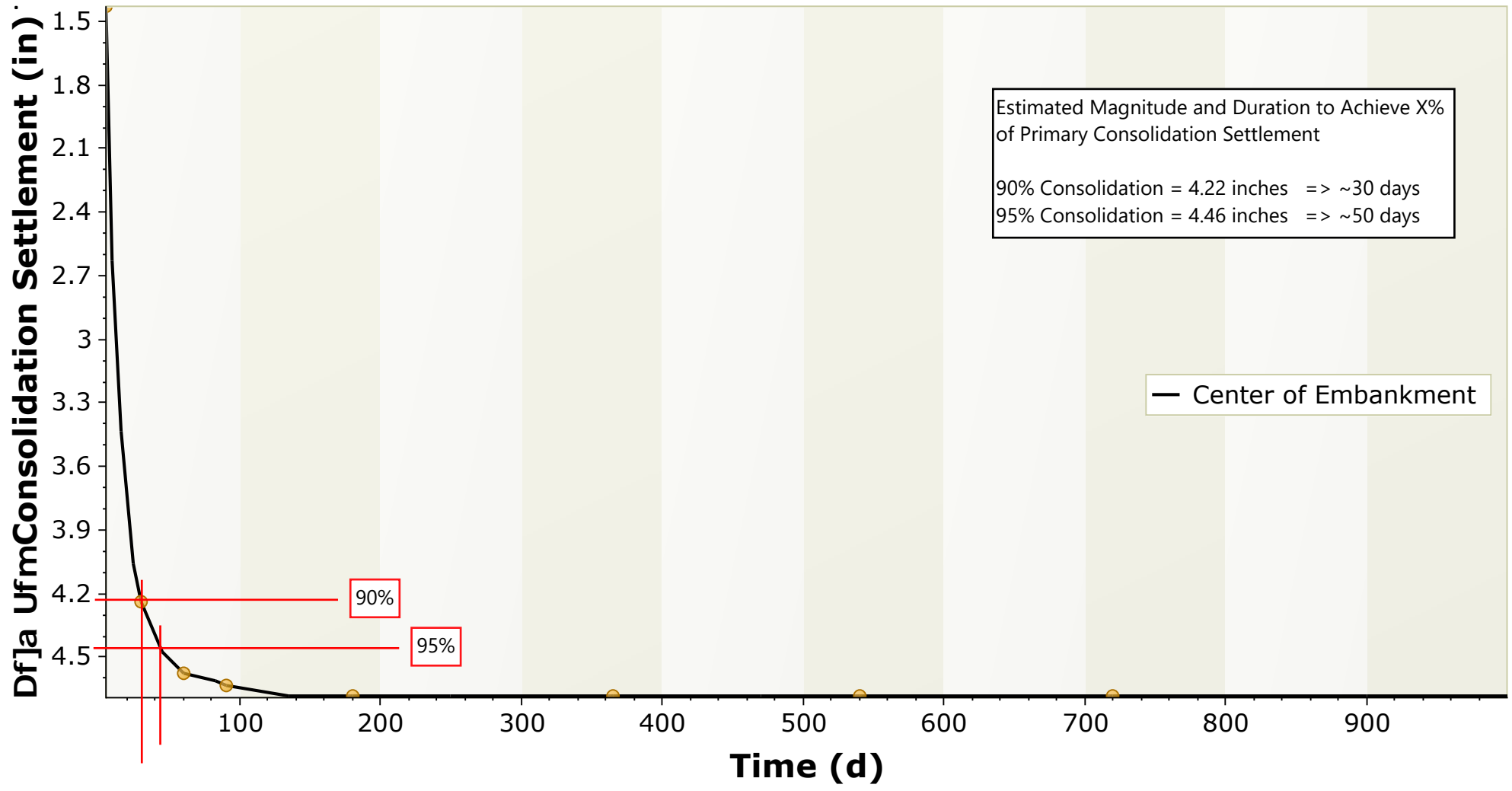
H]a Y'j g''HchU`GYh`Ya Ybh'Uh'GHU''), - Ž \$\$\$`



SETTLE3 5.025

Project	STA-241-17.37 Roundabout & Bridge		
Analysis Description	Sta. 589+00 - Surcharge Embankment Configuration - Wick Drains Spaced at 3 Feet		
Drawn By	DRS	Company	S&ME
Date	3/12/2025	File Name	Sta 589+00_B-003_ALT-2_Wick Drain_S-3_D-24_Embk+1.s3z

Estimated Magnitude and Duration to Achieve X% of Primary Consolidation Settlement



SETTLE3 5.025

Project	STA-241-17.37 Roundabout & Bridge		
Analysis Description	Sta. 589+00 - Surcharge Embankment Configuration - Wick Drains Spaced at 3 Feet		
Drawn By	DRS	Company	S&ME
Date	3/12/2025	File Name	Sta 589+00_B-003_ALT-2_Wick Drain_S-3_D-24_Embk+1.s3z

SETTLEMENT ANALYSES

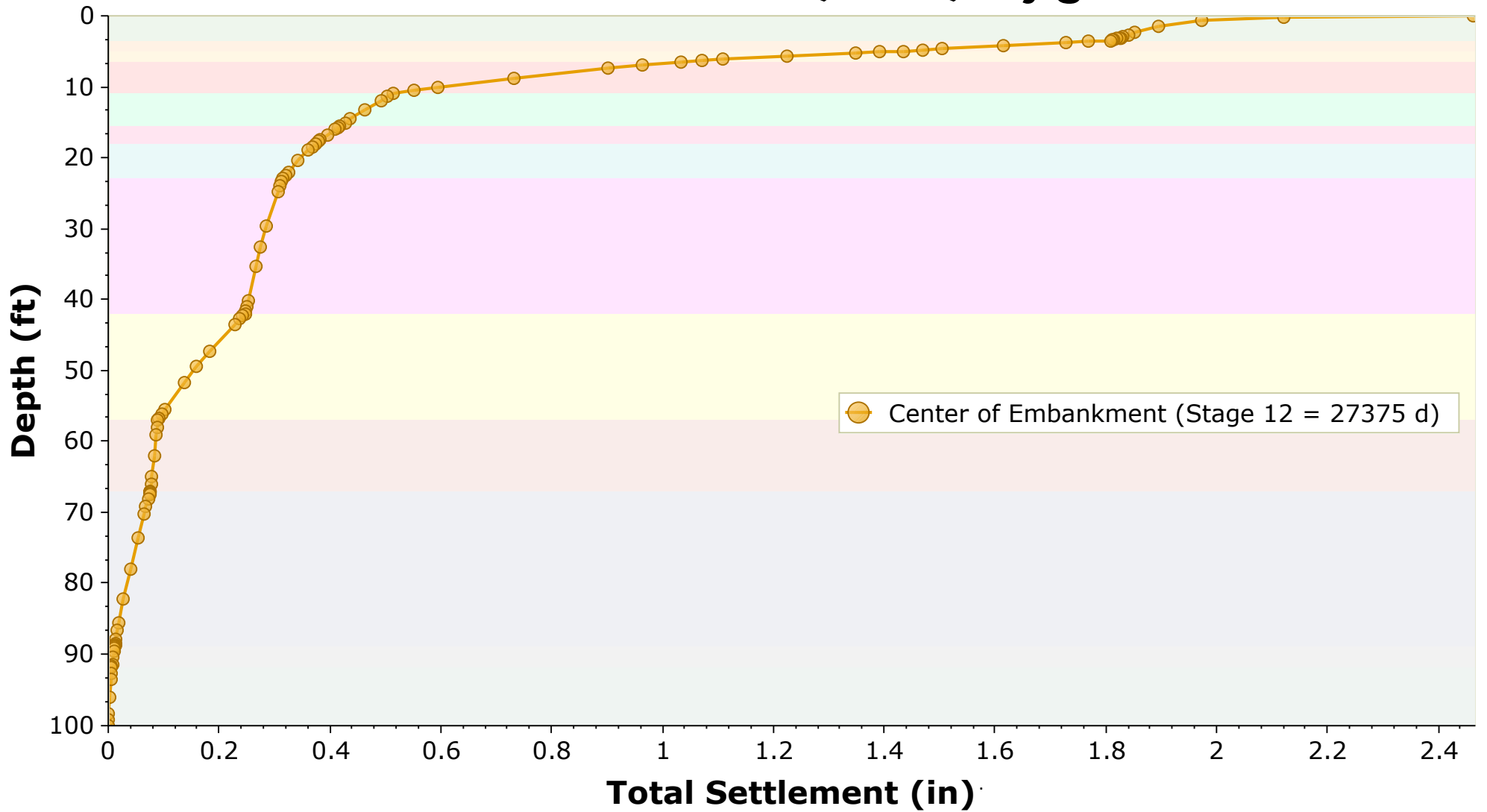
STA. 590+50

PROPOSED EMBANKMENT

CONFIGURATION

(NO SURCHARGE)

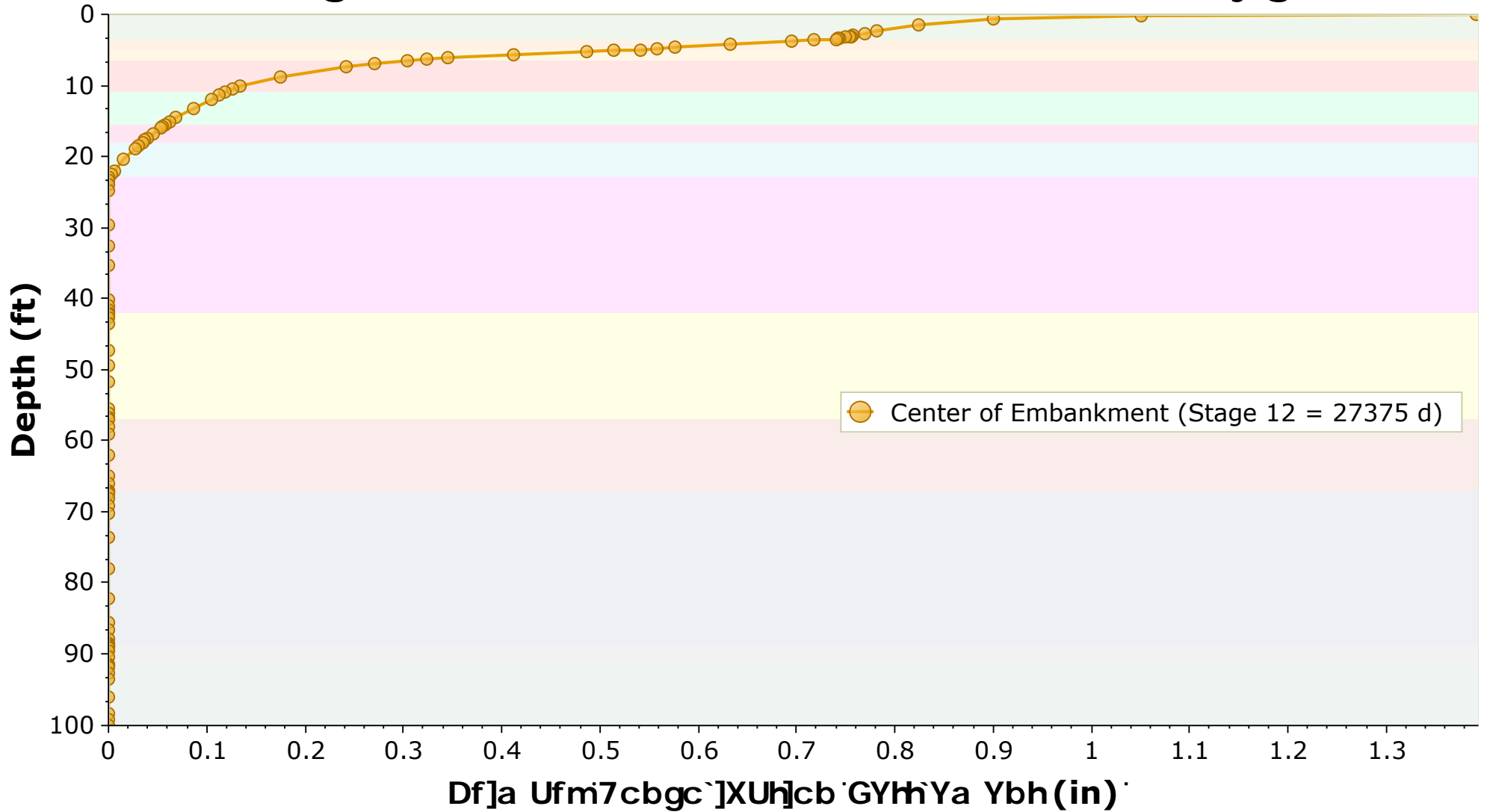
HchU`GYh`Ya Ybh`Uh`GHU``) - \$Ž) \$ j g``8Ydh`



SETTLE3 5.025

<i>Project</i>	STA-241-17.37 Roundabout & Bridge		
<i>Analysis Description</i>	Sta. 590+50 - Final Embankment Configuration - Wick Drains Spaced at 3 Feet! 'Bc`G`fWUf`Y`		
<i>Drawn By</i>	DRS	<i>Company</i>	S&ME
<i>Date</i>	3/12/2025	<i>File Name</i>	Sta 590+50_B-003_ALT-2_Wick Drain_S-3_D-24.s3z

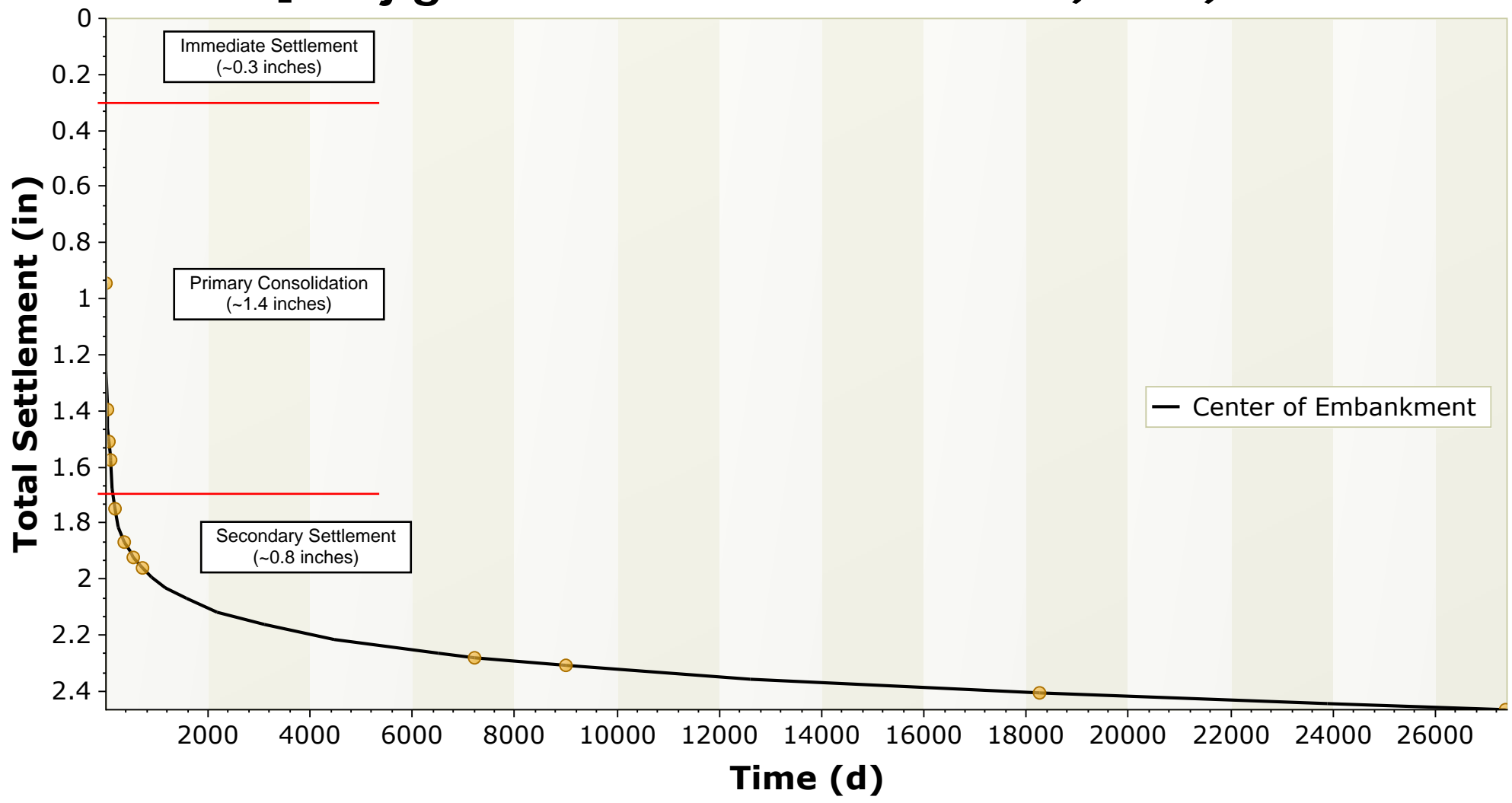
Df]a Ufm7cbgc`]XUh]cb`GYh`Ya Ybh`Uh`GhU"`) - \$Ž) \$ j g"8YdH`



SETTLE3 5.025

<i>Project</i>	STA-241-17.37 Roundabout & Bridge		
<i>Analysis Description</i>	Sta. 590+50 - Final Embankment Configuration - Wick Drains Spaced at 3 Feet!`Bc`G`fWUf[Y`		
<i>Drawn By</i>	DRS	<i>Company</i>	S&ME
<i>Date</i>	3/12/2025	<i>File Name</i>	Sta 590+50_B-003_ALT-2_Wick Drain_S-3_D-24.s3z

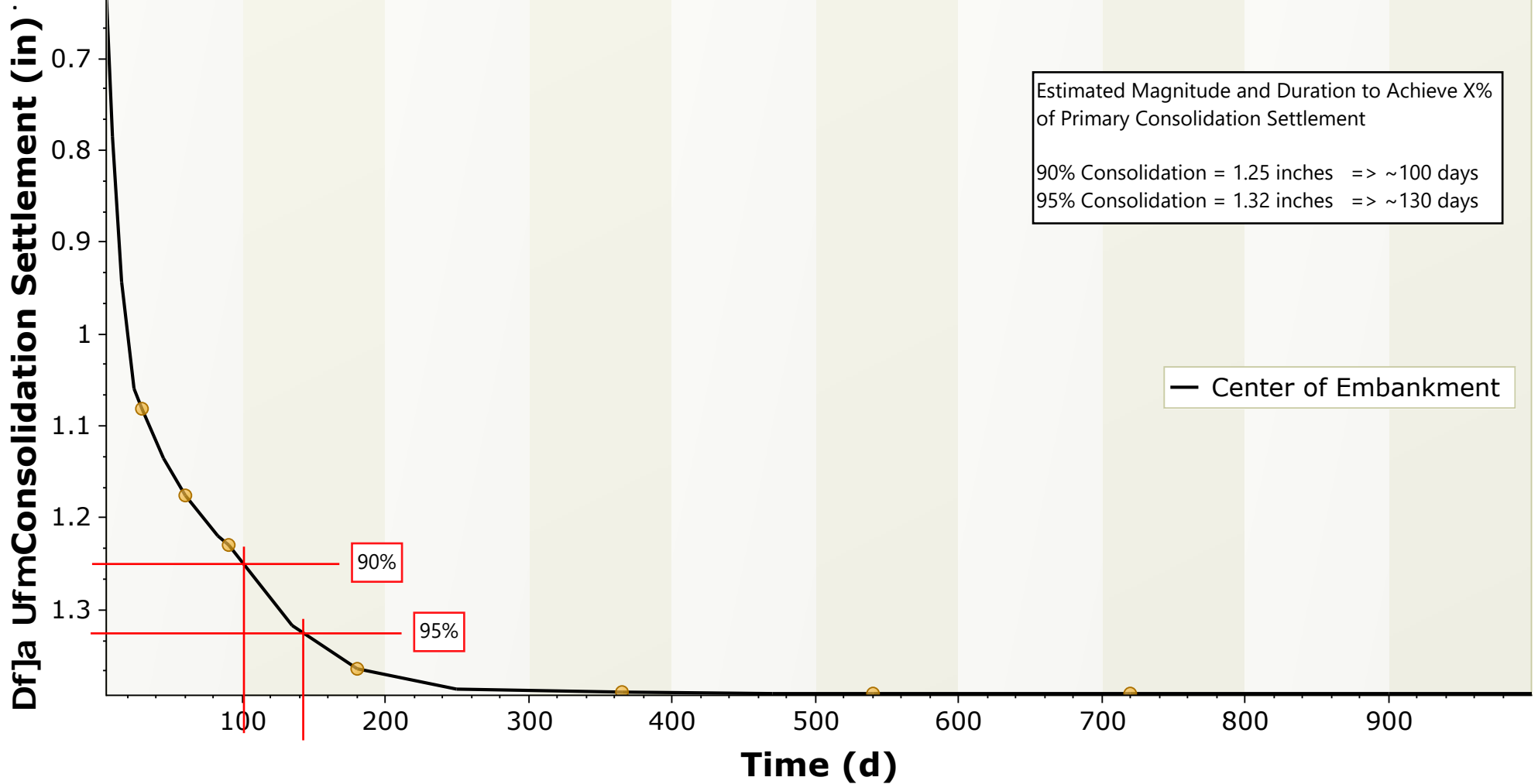
Final Settlement Analysis - Center of Embankment



SETTLE3 5.025

Project	STA-241-17.37 Roundabout & Bridge		
Analysis Description	Sta. 590+50 - Final Embankment Configuration - Wick Drains Spaced at 3 Feet		
Drawn By	DRS	Company	S&ME
Date	3/12/2025	File Name	Sta 590+50_B-003_ALT-2_Wick Drain_S-3_D-24.s3z

Estimated Magnitude and Duration to Achieve X% of Primary Consolidation Settlement



SETTLE3 5.025

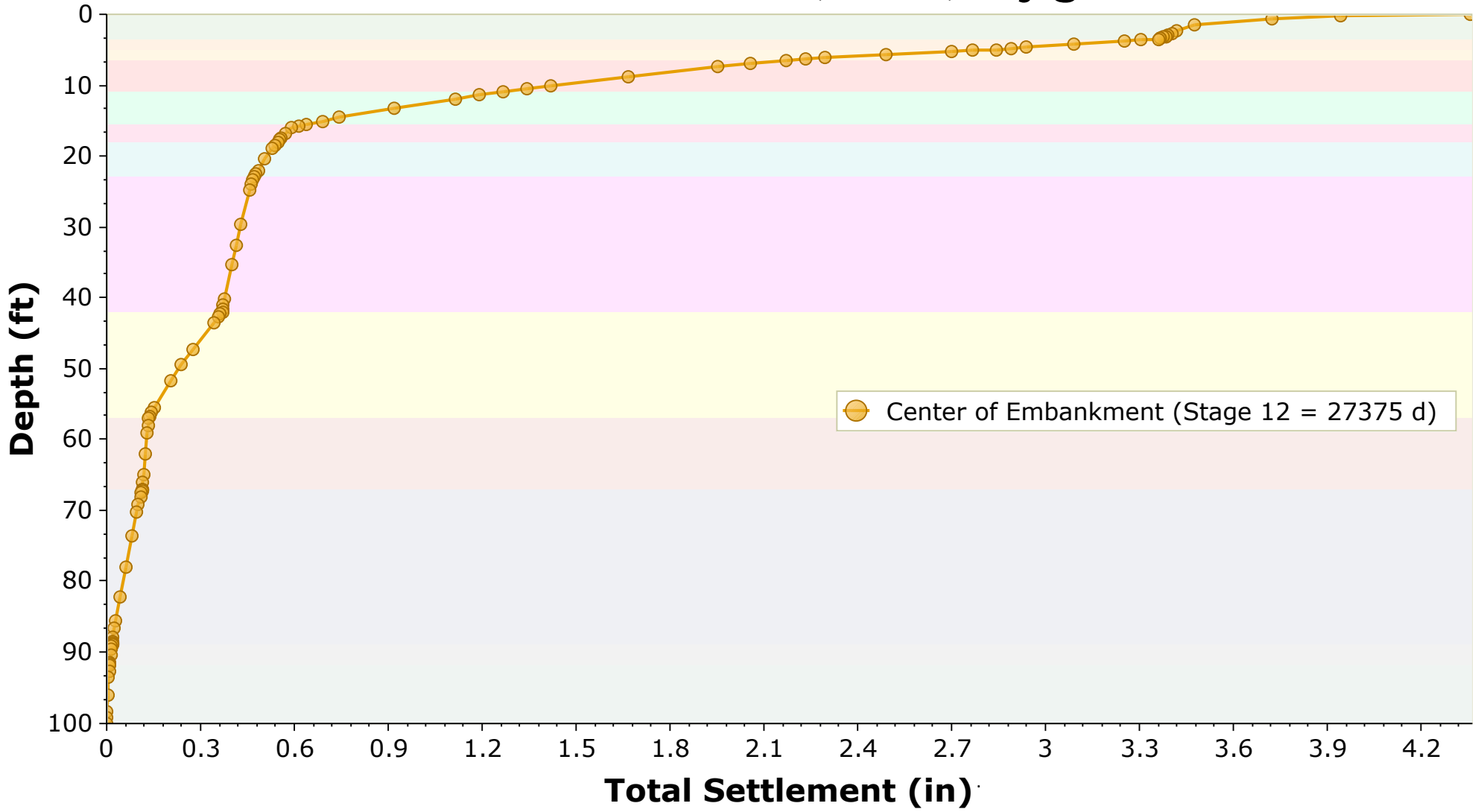
<i>Project</i>	STA-241-17.37 Roundabout & Bridge		
<i>Analysis Description</i>	Sta. 590+50 - Final Embankment Configuration - Wick Drains Spaced at 3 Feet		
<i>Drawn By</i>	DRS	<i>Company</i>	S&ME
<i>Date</i>	3/12/2025	<i>File Name</i>	Sta 590+50_B-003_ALT-2_Wick Drain_S-3_D-24.s3z

SETTLEMENT ANALYSES

STA. 590+50

SURCHARGE EMBANKMENT
CONFIGURATION

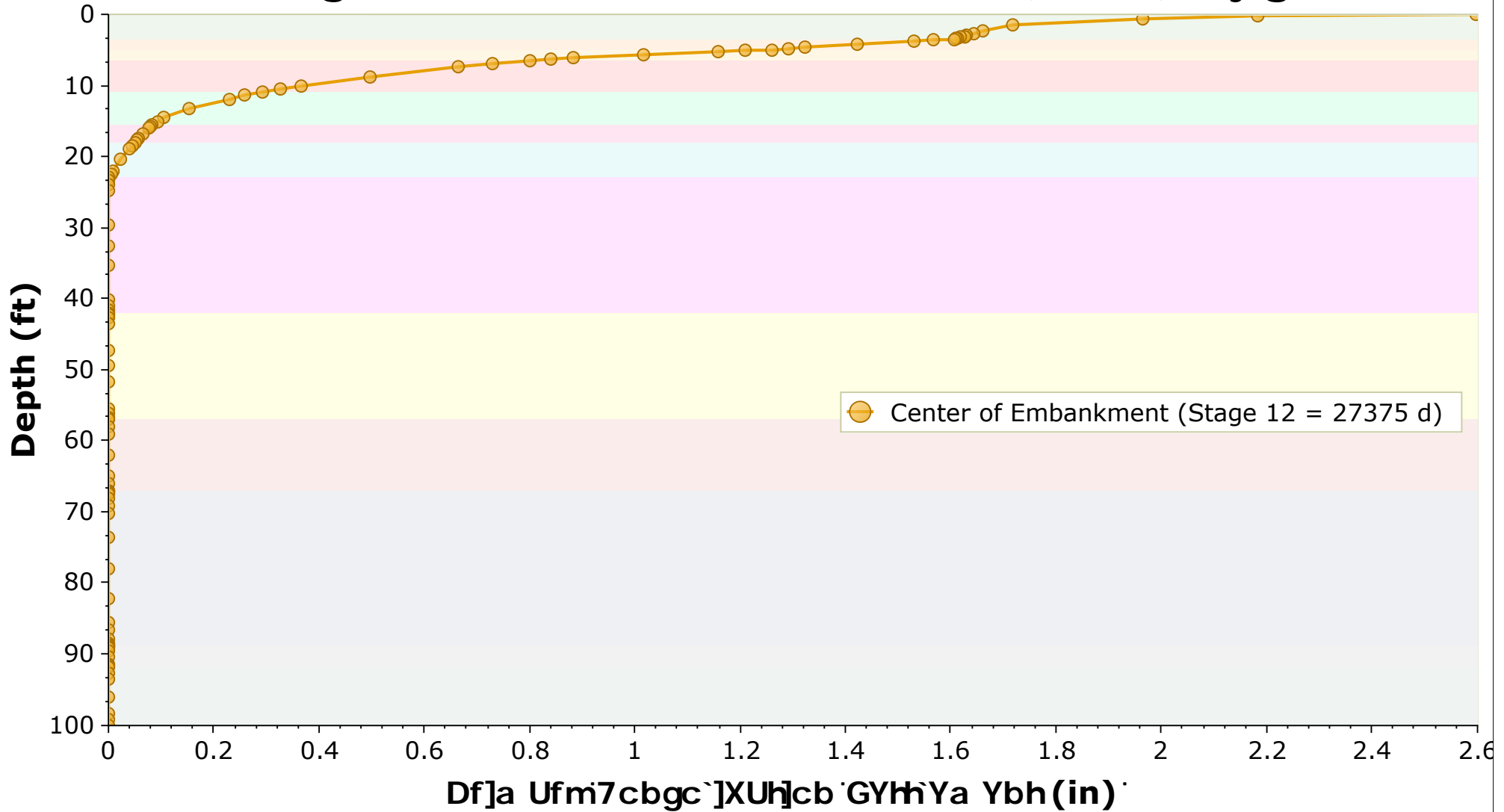
HchU`GYh`Ya Ybh`Uh`GHU``) - \$Ž) \$ j g``8Ydh`



SETTLE3 5.025

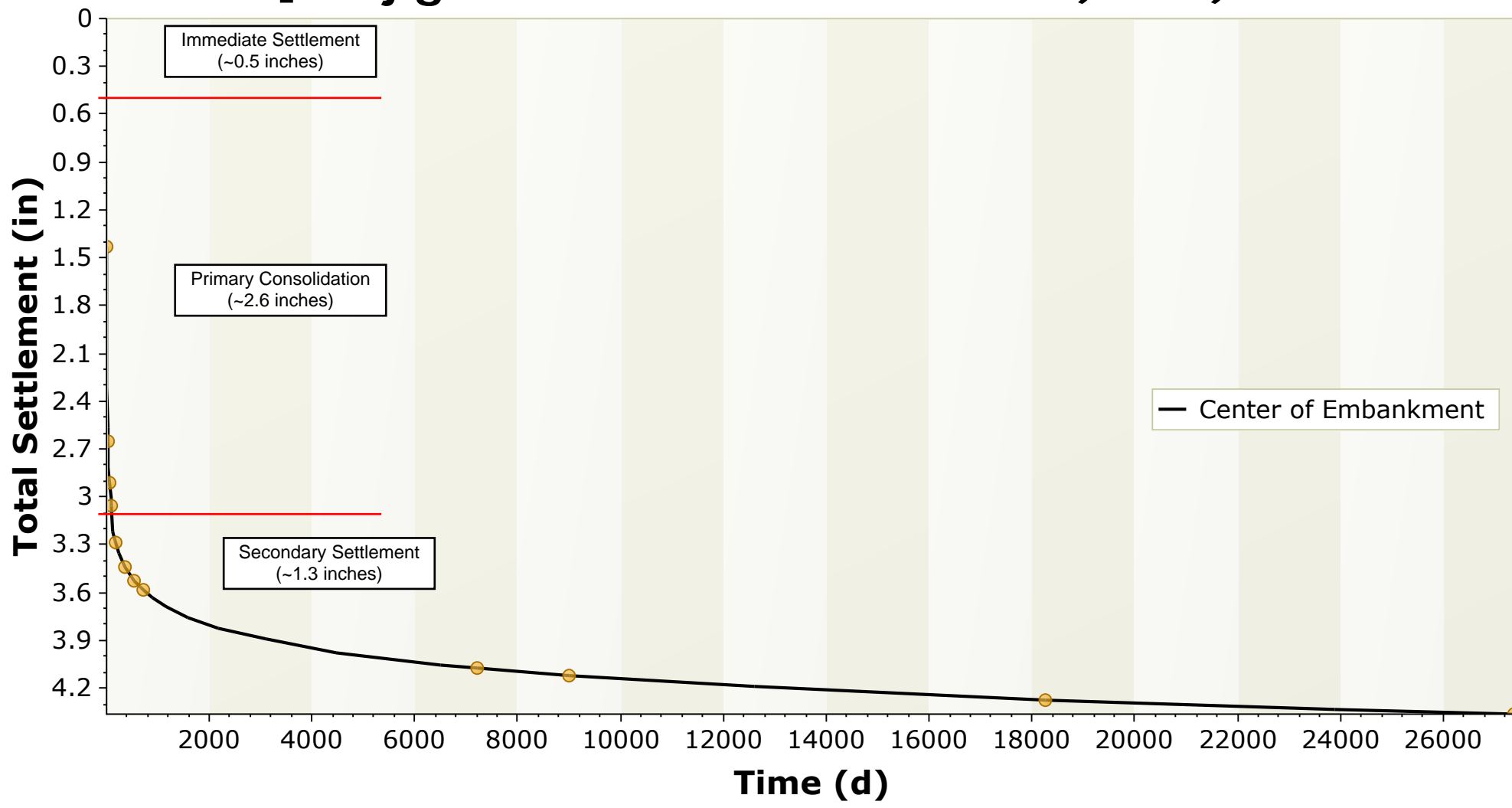
<i>Project</i>	STA-241-17.37 Roundabout & Bridge		
<i>Analysis Description</i>	Sta. 590+50 - Surcharge Embankment Configuration - Wick Drains Spaced at 3 Feet		
<i>Drawn By</i>	DRS	<i>Company</i>	S&ME
<i>Date</i>	3/12/2025	<i>File Name</i>	Sta 590+50_B-003_ALT-2_Wick Drain_S-3_D-24_Embk+1.s3z

Df]a Ufm7cbgc`]XUh]cb`GYh`Ya Ybh`Uh`GhU"`) - \$Ž) \$ j g" 8Ydh`



<i>Project</i>	STA-241-17.37 Roundabout & Bridge		
<i>Analysis Description</i>	Sta. 590+50 - Surcharge Embankment Configuration - Wick Drains Spaced at 3 Feet		
<i>Drawn By</i>	DRS	<i>Company</i>	S&ME
<i>Date</i>	3/12/2025	<i>File Name</i>	Sta 590+50_B-003_ALT-2_Wick Drain_S-3_D-24_Embk+1.s3z

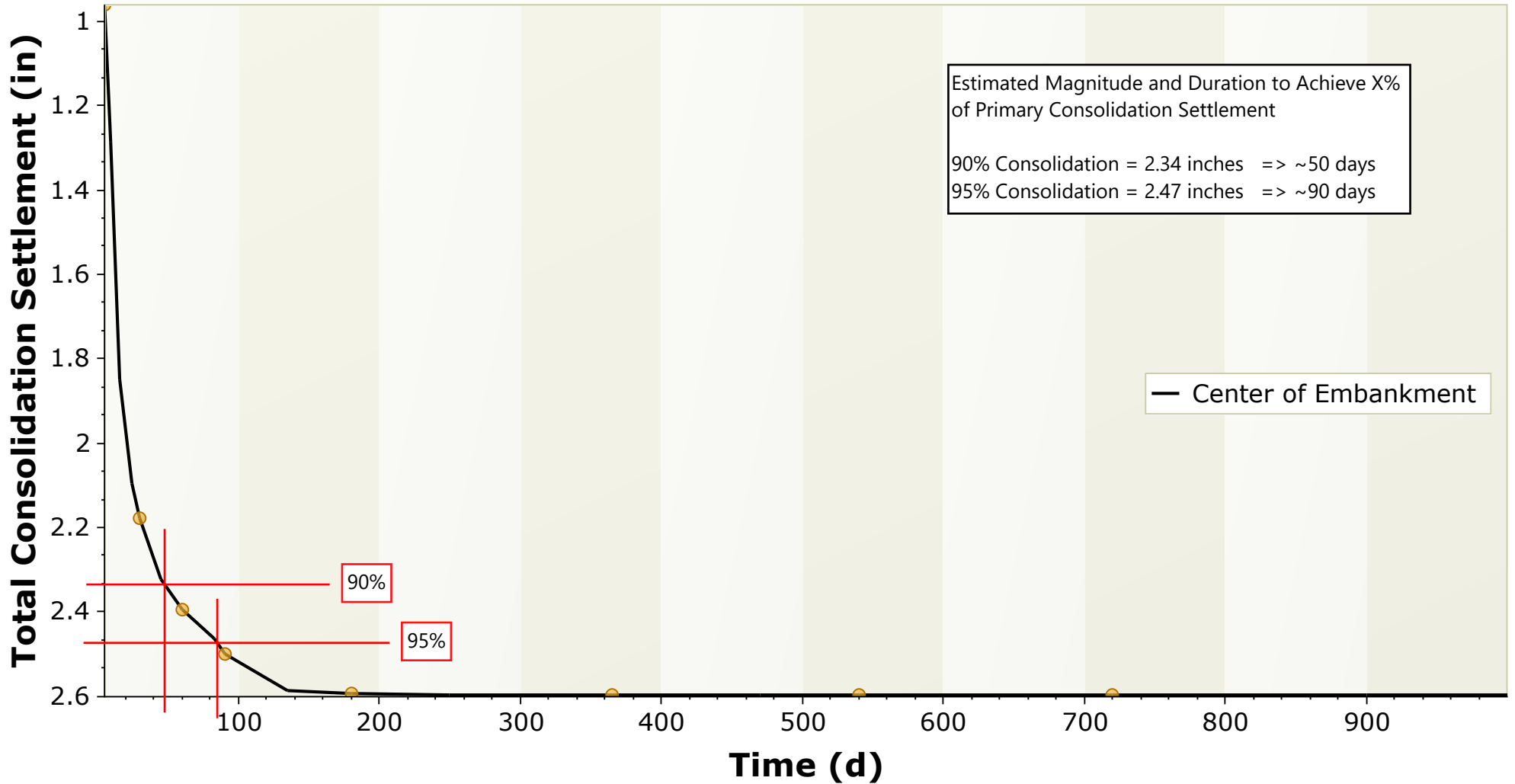
Center of Embankment Settlement Analysis



SETTLE3 5.025

Project	STA-241-17.37 Roundabout & Bridge		
Analysis Description	Sta. 590+50 - Surcharge Embankment Configuration - Wick Drains Spaced at 3 Feet		
Drawn By	DRS	Company	S&ME
Date	3/12/2025	File Name	Sta 590+50_B-003_ALT-2_Wick Drain_S-3_D-24_Embk+1.s3z

H]a Y'j g"Df]a Ufm7cbgc`]XUh]cb`GYh`Ya Ybh`Uh`GhU"" - \$Ž) \$`

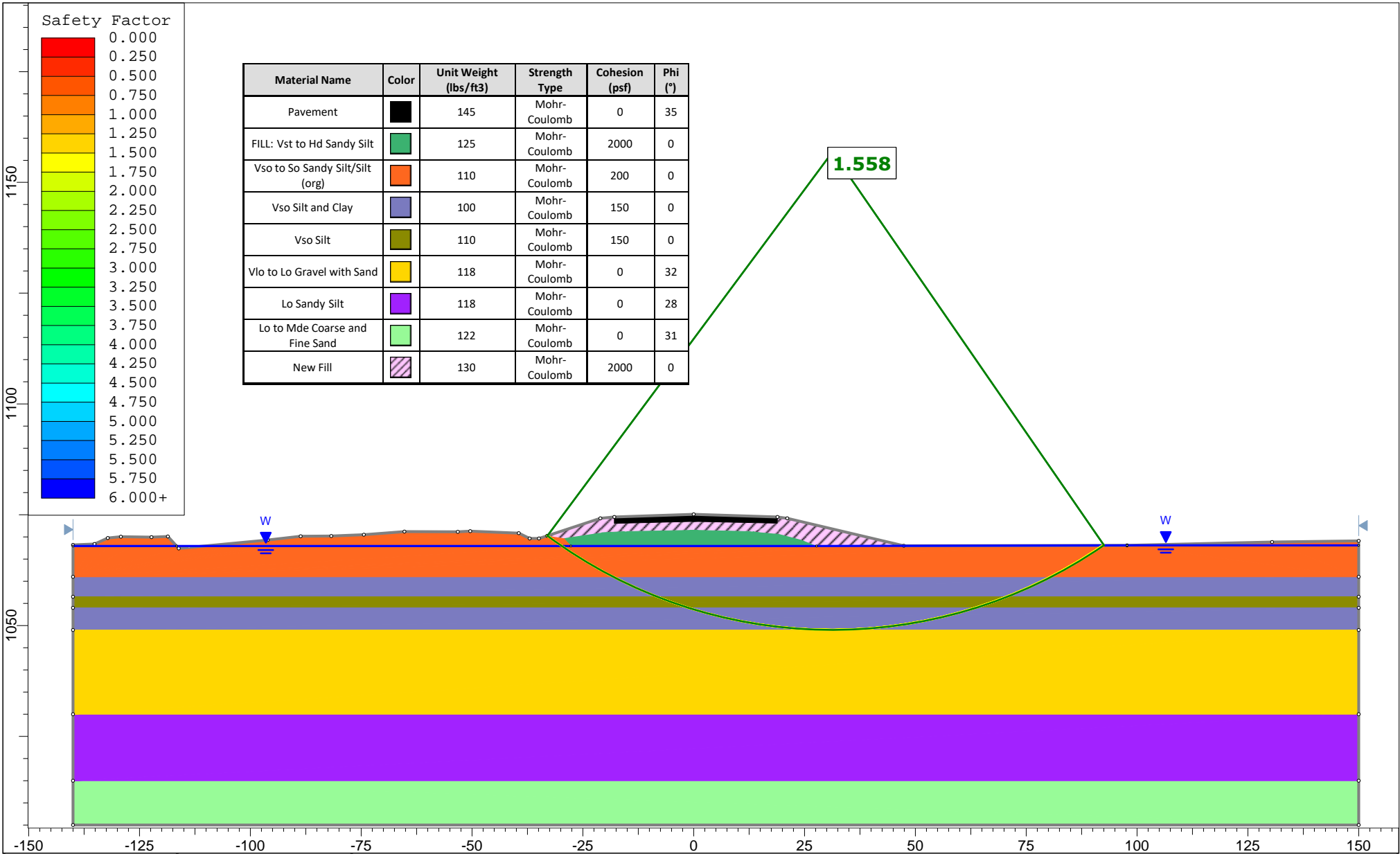


SETTLE3 5.025

<i>Project</i>	STA-241-17.37 Roundabout & Bridge		
<i>Analysis Description</i>	Sta. 590+50 - Surcharge Embankment Configuration - Wick Drains Spaced at 3 Feet		
<i>Drawn By</i>	DRS	<i>Company</i>	S&ME
<i>Date</i>	3/12/2025	<i>File Name</i>	Sta 590+50_B-003_ALT-2_Wick Drain_S-3_D-24_Embk+1.s3z

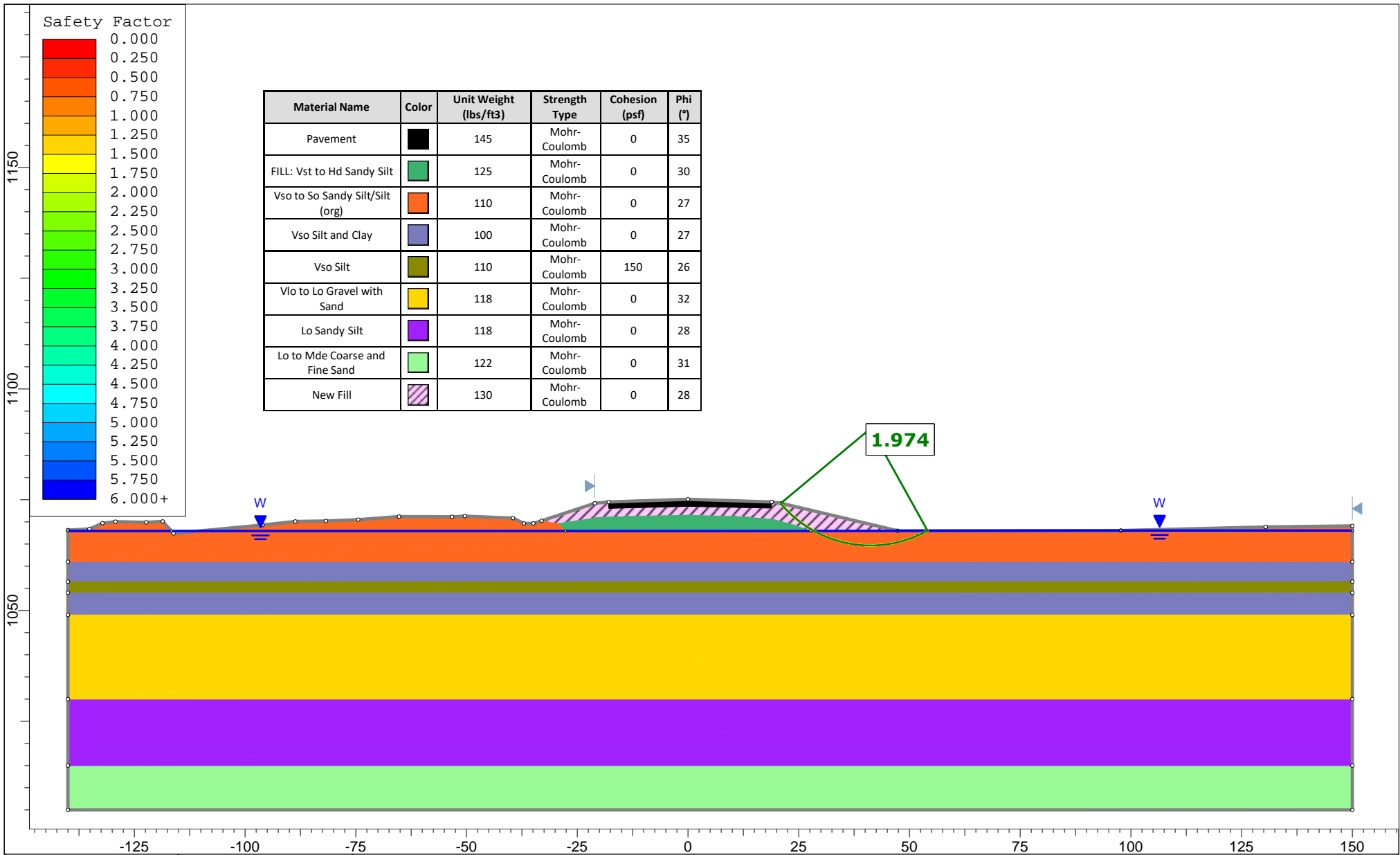
EMBANKMENT STABILITY
ANALYSES

STA. 589+00 (CRITICAL SECTION)



S&ME, Inc.
 6190 Enterprise Ct.
 Dublin, OH 43016
 (614) 793-2226

Project Name		STA-241-17.37 - Embankment Construction	
Scenario		Preliminary Stability - Total Stress Analysis for Final Condition	Analysis Method
			Spencer
Cross Section		Sta. 589+00 (Forward Abutment)	File Name
			Sta 589+00 - Total Stress.slim
Project Number	23170065A	Calc. By	BKS
Date	8/19/2024	Scale	1:360
<small>Note: The results of this analysis are based on the available subsurface information and engineering judgment. Subsurface conditions between explored locations have been approximated based on observations from our site visit(s) and available information.</small>			



<p>S&ME, Inc. 6190 Enterprise Ct. Dublin, OH 43016 (614) 793-2226</p> <p>SLIDEINTERPRET 9.034</p>	Project Name STA-241-17.37 - Embankment Construction			
	Scenario Preliminary Stability - Effective Stress Analysis for Final Condition		Analysis Method Spencer	
	Cross Section Sta. 589+00 (Forward Abutment)		File Name Sta 589+00 - Effective Stress.slim	
	Project Number 23170065A	Calc. By BKS	Date 8/19/2024	Scale 1:360
	<small>Note: The results of this analysis are based on the available subsurface information and engineering judgment. Subsurface conditions between explored locations have been approximated based on observations from our site visit(s) and available information.</small>			

Project Number: 23170065A
 Project Name: STA-241-17.37
 Project Location: Stark County, OH
 Client Name: ODOT District 4

Calculated By: BKS
 Date: 4/2/2025
 Checked By: RSW
 Date: 4/3/2025



Embankment Foundation Bearing Capacity - Sta. 589+00

where:

$$FS = \frac{S_u N_c}{\gamma H}$$

S_u = Undrained shear strength of soil beneath fill

N_c = Bearing capacity factor for $\phi = 0$; 5.14)

γ = Unit weight of embankment fill

H = Height of fill

Based on Eqn. 6-13 from FHWA-NHI-06-068 *Soils and Foundations - Volume 1*.
 Original equation used 6 times undrained shear strength in the denominator. To correlate to bearing capacity failure mode, the 6 has been replaced with N_c .

$S_u = \underline{175}$ psf

$N_c = \underline{5.14}$

$\gamma = \underline{130}$ pcf

H = 4.5 ft

FS = 1.54

FS Required = 1.5

FS Met? YES

Embankment Foundation Lateral Squeeze - Sta. 589+00

$$FS = \frac{2S_u}{\gamma D_s \tan \theta} + \frac{4.14S_u}{\gamma H}$$

Eqn. 7-12 from FHWA-NHI-06-068 *Soils and Foundations - Volume 1*.

where:

S_u = Undrained shear strength of soil beneath fill

D_s = Depth of soft soil beneath the toe of the slope

γ = Unit weight of embankment fill

H = Height of fill

θ = Angle of embankment slope

$S_u = \underline{175}$ psf

$D_s = \underline{19}$ ft

$\gamma = \underline{130}$ pcf

H = 4.5 ft

$\theta = \underline{18.4}$ deg

FS = 1.66

FS Required = 1.5

FS Met? YES

Subgrade and Structure Foundation Exploration Report – Final (Rev. #1)
STA-241-17.37 Roundabout and Bridge Replacement (PID 119585)
North Canton, Stark County, Ohio
S&ME Project No. 23170065A/B, Task Order No. 4-01-1



Appendix V – OGE Geotechnical Design Checklists

I. Geotechnical Design Checklists	
Project: STA-241-17.37	PDP Path:
PID: 119585	Review Stage: 2

Checklist	Included in This Submission
II. Reconnaissance and Planning	✓
III. A. Centerline Cuts	
III. B. Embankments	✓
III. C. Subgrade	✓
IV. A. Foundations of Structures (By OGE)	
IV. B. Retaining Wall	
V. A. Landslide Remediation	
V. B. Rockfall Remediation	
V. C. Wetland or Peat Remediation	
V. D. Underground Mine Remediation	
V. E. Surface Mine Remediation	
V. F. Karst Remediation	
VI. A. Geotechnical Profile	✓
VI. D. Geotechnical Reports	✓

II. Reconnaissance and Planning Checklist

C-R-S:	STA-241-17.37	PID:	119585	Reviewer:	BKS	Date:	4/9/2025
Reconnaissance		(Y/N/X)	Notes:				
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:	X	Plans by others				
	Roadway plans						
	Structures plans						
	Geohazards plans						
2	Have the resources listed in Section 302.2.1 of the SGE been reviewed as part of the office reconnaissance?	Y					
3	Have all the features listed in Section 302.3 of the SGE been observed and evaluated during the field reconnaissance?	Y					
4	If notable features were discovered in the field reconnaissance, were the GPS coordinates of these features recorded?	Y					
Planning - General		(Y/N/X)	Notes:				
5	In planning the geotechnical exploration program for the project, have the specific geologic conditions, the proposed work, and historic subsurface exploration work been considered?	Y					
6	Has the ODOT Transportation Information Mapping System (TIMS) been accessed to find all available historic boring information and inventoried geohazards?	Y	No historic information was found				
7	Have the borings been located to develop the maximum subsurface information while using a minimum number of borings, utilizing historic geotechnical explorations to the fullest extent possible?	Y					
8	Have 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					
9	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					

II. Reconnaissance and Planning Checklist

Planning - General		(Y/N/X)	Notes:
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?	Y	
The schedule of borings should present the following information for each boring:			
a.	exploration identification number	Y	
b.	location by station and offset	Y	
c.	estimated amount of rock and soil, including the total for each for the entire program.	Y	
Planning – Exploration Number			
Planning – Exploration Number		(Y/N/X)	Notes:
11	Have the coordinates, stations and offsets of all explorations (borings, soundings, test pits, etc.) been identified?	Y	
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	
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?	X	

II. Reconnaissance and Planning Checklist

Planning – Boring Types		(Y/N/X)	Notes:
14	Based on Sections 303.3 to 303.7.6 of the SGE, have the location, depth, and sampling requirements for the following boring types been determined for the project?	Y	Including CPT soundings performed by ODOT OGE.
	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)		
	Rock Slope (Type C6)		
	Karst (Type C7)		
	Proposed Underground Utilities (Type D)		
	Structure Borings (Type E)		
	Bridges (Type E1)	✓	
	Culverts (Type E2 a,b,c)		
	Retaining Walls (Type E3 a and b)		
	Noise Barrier (Type E4)		
	CCTV & High Mast Lighting Towers (Type E5)		
	Buildings and Salt Domes (Type E6)		

III.B. Embankments Checklist

C-R-S:	STA-241-17.37	PID:	119585	Reviewer:	BKS	Date:	4/9/2025
<i>Use this checklist in conjunction with the Embankment Design Guidance in GDM Section 500</i> <i>If you do not have an embankment on the project, you do not have to fill out this checklist.</i>							
Settlement				(Y/N/X)	Notes:		
1	If soil conditions and project requirements warrant, have settlement issues been addressed? If not applicable (X), go to Question 14			Y			
2	Have consolidation properties of the foundation soils been determined?			Y	CPT dissipation testing		
Check methods used:							
laboratory consolidation tests			✓				
empirical correlations with moisture content and Atterberg values			✓				
	other (describe other methods)			✓			
3	Have calculations been performed to estimate the total expected embankment settlement and the time of consolidation? Indicate method used.			Y			
4	If differing foundation soil and/or loading conditions occur throughout the embankment area, have sufficient analyses been completed to evaluate consolidation at locations representative of the most critical conditions?			Y			
5	Have the total settlement and the time of consolidation analyses indicated acceptable values at all locations for the scope of the embankment work?			Y	Based on discussions with ODOT District 4, wick drains will be required to meet the construction schedule for the project.		
6	If total settlement or time of consolidation is unacceptable, have the stations and lateral extent of the problem areas been defined?			Y	See above answer. See Section 5.4.3.2.		
7	Has a method been chosen as a solution to the settlement issues?			Y			
Check the method(s) used:							
waiting periods with monitoring			✓				
drainage blanket and wick drains			✓				
surcharge (preloading)			✓				
removal and replacement of weak soil							
lowering proposed grade / change alignment							
lightweight fill							
other (describe other methods)							

III.B. Embankments Checklist

Settlement		(Y/N/X)	Notes:
8	Based on accepted design practices, and where applicable, adhering to published guidelines and design recommendations from FHWA, have calculations been performed to evaluate the effectiveness of the chosen solution(s)?	Y	
9	Has an economic analysis been performed to evaluate the cost benefits of the recommended solution compared to others?	X	By others
10	Have all necessary notes, specifications, and details for the chosen solution been determined?	Y	Recommendations for waiting periods, wick drain spacing and surcharge have been provided. Plansnotes prepared by others.
11	Have the need, locations, type, plan notes, and reading schedule for settlement platforms or cells been determined?	Y	See Section 5.4.3.4.
12	Have the effects of the predicted settlement and the chosen solution been determined and accounted for on the construction schedule?	Y	Plan notes prepared by others.
13	Has the effect of any foundation soil consolidation (including differential settlement) been evaluated with regard to adjacent structures (e.g., bridges, buildings, culverts, utilities) which will also undergo settlement and be subject to stresses induced by the consolidation of the surrounding soil?	X	OGE is responsible for bridge foundation design.
Stability		(Y/N/X)	Notes:
14	If soil conditions and project requirements warrant, have stability issues been addressed? If not applicable (X), go to Question 29	Y	
15	Has the total (short term) and effective (long term) shear strength of the foundation soils been determined?	Y	
	Check method used:		
	laboratory shear tests		
	estimation from SPT or field tests	✓	
16	Have the values of shear strength for proposed embankment fill material, as determined from GDM Section 500, been used in the stability analyses?	Y	

III.B. Embankments Checklist

Stability	(Y/N/X)	Notes:
17 Have calculations been performed to determine the F.S. for stability? Indicate which program and which analysis method (Spencer, Bishop, etc) was used.	Y	Spencer
18 Have the following F.S. been met or exceeded, as determined by the calculations, for the given stability conditions:	Y	
a. 1.30 for short term (undrained) condition	Y	
b. 1.30 for long term (drained) condition	Y	
c. 1.10 for rapid drawdown, flood condition	X	
d. 1.50 for embankment containing or supporting a structural element	X	
19 When differing soil or loading conditions occur throughout the embankment area, have sufficient analyses been completed to evaluate the stability at locations representative of the most critical conditions?	Y	
20 If the F.S. was not met or exceeded, have the stations and lateral extent of the problem areas been defined?	X	
21 Has a method been chosen as a solution to the stability issues?	Y	
Check the method(s) used:		
flattening slopes	✓	
counter berm		
lightweight embankment		
reinforced soil slope		
soil nailing		
drainage blanket and wick drains	✓	
removal of soft soil, adding shear key		
reduced grade / change alignment		
staged construction		
controlled rate of fill placement		
drilled shaft slope stabilization		
other (describe other methods)		
22 Based on accepted design practices, and where applicable, adhering to published guidelines and design recommendations from FHWA, have calculations been performed to evaluate the effectiveness of the chosen solution(s)?	Y	
23 Has an economic analysis been performed to evaluate the cost benefits of the recommended solution compared to others?	X	By others.

III.B. Embankments Checklist

Stability		(Y/N/X)	Notes:
24	Have all necessary notes, specifications, and details for the chosen solution been determined?	Y	Plans by others. Recommended notes have been provided to designer.
25	Have the need, location, type, plan notes, and reading schedule for piezometers and inclinometers been determined?	Y	Piezometers and settlement platforms. No inclinometers.
26	If piezometers will be used, has the critical pressure value been determined and the appropriate information included in the plans?	X	
27	Have the effects of the stability solution been determined and accounted for on the construction schedule?	Y	
28	Has the effect of the stability solution been evaluated with regard to structures (e.g., bridges, buildings, culverts, utilities) which may be subject to unusual stresses or require special construction considerations?	Y	
Sidehill Fills			
		(Y/N/X)	Notes:
29	If soil conditions and project requirements warrant, have sidehill fill issues been addressed? If not applicable (X), go to Question 34	N	
30	In accordance with GDM Section 800, have sidehill fills been evaluated to determine if special benching or shear keys are needed?		
31	In accordance with GDM Section 800, if special benching or shear keys are required,		
a.	has Plan Note G109 from L&D3 been included in the General Notes?		
b.	have quantities for both excavation and embankment been calculated for the benched areas and added to the plan General Quantities?		
c.	have the special benching or shear keys been indicated on the appropriate cross sections?		
32	Have water bearing zones been identified and their impact addressed?		
33	Have subsurface drainage controls been adequately addressed?		

III.B. Embankments Checklist

Special	(Y/N/X)	Notes:
34 Have all of the environmental factors, including wetlands, stream mitigation, and landfills, been considered and incorporated prior to design and analysis of embankment settlement and stability, including EPA or other government agencies' involvement, mitigation, or special design or construction considerations?	X	By others
35 If an embankment is to be placed through standing water or over weak, wet soils (with or without a fabric separator), the fill should be placed by the method of end dumping to a given height above the standing water or until compaction is achievable over the soft soil. If end dumping is to be specified,	X	
a. has the material type for the fill to be end dumped been specified?	X	
b. has the need for a fabric separator or filter layer been determined?	X	
c. has the height of fill to be end dumped been determined?	X	
d. have all notes and specifications for end dumping been developed?	X	

III.C. Subgrade Checklist

C-R-S:	STA-241-17.37	PID:	119585	Reviewer:	BKS	Date:	4/9/2025
<p>Use this Checklist in conjunction with the Subgrade design guidance in GDM Section 600 If you do not have any subgrade work on the project, you do not have to fill out this checklist.</p>							
Subgrade		(Y/N/X)	Notes:				
1	Has the subsurface exploration adequately characterized the soil or rock according to GDM Section 600?	Y					
a.	Has each sample been visually classified and inspected for the presence of gypsum? Has a moisture content been performed on each sample?	Y					
b.	Has mechanical classification (Plastic Limit (PL), Liquid Limit (LL), and gradation testing) been done on at least two samples from each boring within six feet of the proposed subgrade?	Y					
c.	Has the sulfate content of at least one sample from each boring within 3 feet of the proposed subgrade been determined, per Supplement 1122, Determining Sulfate Content in Soils?	Y					
d.	Has the sulfate content of all samples that exhibit gypsum crystals been determined?	X	No gypsum crystals observed				
e.	Have A-2-5, A-4b, A-5, A-7-5, A-8a, or A-8b soils within the top 3 feet of the proposed subgrade been mechanically classified?	Y	A-4b soils were encountered in B-002 and B-003 and A-8b soil in B-003; however, these soils are not within the top 3 feet.				
2	If soils classified as A-2-5, A-4b, A-5, A-7-5, A-8a, or A-8b, or having a LL>65, are present at the proposed subgrade (geotechnical profile), do the plans specify that these materials need to be removed and replaced or chemically stabilized?	X	Though A-4b and A-8b soils were encountered, they are not within 3 feet of proposed subgrade.				
a.	If these materials are to be removed and replaced, have the station limits, depth, and lateral limits for the planned removal been provided?	X					
3	If there is any rock, shale, or coal present at the proposed subgrade (C&MS 204.05), do the plans specify the removal of the material?	X					
a.	If removal of any rock, shale, or coal is required, have the station limits, depth, and lateral limits for the planned removal of the material at proposed subgrade been provided?	X					

III.C. Subgrade Checklist

Subgrade	(Y/N/X)	Notes:						
4 In accordance with GDM Section 600, do the SPT (N_{60})/HP values and existing moisture contents for the proposed subgrade soils indicate the need for subgrade stabilization?	Y	Only at B-005-0-23.						
a. If removal and replacement is applicable, has the detail of subgrade removal been shown on the plans, including depth of removal, station limits, lateral extent, replacement material, and plan notes (Item 204 - Subgrade Compaction and Proof Rolling)?	Y	Plans being prepared by others.						
b. If chemical stabilization is applicable, has the detail of this treatment been shown on the plans, including depth, percentage of chemical, station limits, lateral extent, and plan notes? <table border="1" data-bbox="188 795 781 911"> <tr> <td colspan="2">Indicate type of chemical stabilization specified:</td> </tr> <tr> <td>cement stabilization</td> <td></td> </tr> <tr> <td>lime stabilization</td> <td></td> </tr> </table>	Indicate type of chemical stabilization specified:		cement stabilization		lime stabilization		X	Per discussions with ODOT, chemical stabilization will not be used for this project.
Indicate type of chemical stabilization specified:								
cement stabilization								
lime stabilization								
5 If removal and replacement has been specified, do the plans include Plan Note G121 from L&D3?	X	Plans by others.						
6 If drainage or groundwater is an issue with the proposed subgrade, has an appropriate drainage system (e.g., pipe, underdrains) been provided?	X							
7 Has an appropriate quantity of Proof Rolling (C&MS 204.06) and has Plan Note G111 from L&D3 been included in the plans?	X	Plans by others.						
8 Has a design CBR value been provided?	Y	See Section 5.2.2.						

VI.A. Geotechnical Profile Checklist

C-R-S:	STA-241-17.37	PID:	119585	Reviewer:	BKS	Date:	4/9/2025
General Presentation				(Y/N/X)	Notes:		
1	Has an electronic copy of all geotechnical submissions been provided to the District Geotechnical Engineer (DGE)?			Y			
2	Have the cadd files been prepared using the appropriate version of the ODOT CADD standards?			Y			
3	Has the geotechnical specification (title and date) under which the work was performed been clearly identified on every submission (reports, plans, etc.)?			Y			
4	Has the first complete version of all documents being submitted been labeled as 'Draft'?			Y			
5	Subsequent to ODOT's review and approval, has the complete version of the revised documents being submitted been labeled as 'Final'?			X	This is the draft submission for the profile sheets.		
a.	Have the C-R-S, PID number, and product title been included in the folder name?			Y			
6	If the project includes structures, have all structure explorations been presented together under the same cover sheet? (Do not create separate Geotechnical Profile - Bridge Sheets)			Y			
7	Has a scale of 1"=1' been used for cover sheets, laboratory test data sheets, and boring log sheets, if applicable?			Y			
8	Based on the project length, has the correct horizontal scale been used to plot the project data?			Y			
	Check scale used:						
	1" = 5', 10', 20', 25', 40', or 50' for projects 1500' or less (use largest scale appropriate to present entire plan on one sheet)						
	1" = 50' projects greater than 1500'			✓			
9	Has a scale of 1" = 10' been utilized for the vertical scale of the project data?			Y			
10	If the project includes structures, has the plan and profile view been shown at the same scale as the Site Plan for the proposed structure(s), when possible?			Y			

VI.A. Geotechnical Profile Checklist

General Presentation		(Y/N/X)	Notes:
11	If the project includes culverts, have the plan and profile been presented along the flowline of the culvert?	X	
12	Have the cross-sections been plotted at a scale of 1" = 10' (preferred) or 1" = 20' (for higher or wider slopes)?	Y	
Cover Sheet		(Y/N/X)	Notes:
13	Has the following general information been provided on the cover sheet:	Y	
a.	Brief description of the project, including the bridge number of each bridge involved in the plan set, if any?	Y	
b.	Brief description of historic geotechnical explorations referenced in this exploration? State if no historic records are available.	Y	
c.	Generalized information about the geology of the project area, including terrain, soil origin, bedrock types, and age?	Y	
d.	Brief presentation of geological and topographical information derived from the field reconnaissance? Include comments on structure and pavement conditions.	Y	
e.	Brief presentation of test boring and sampling methods? Include date of last calibration and drill rod energy ratio as a percent for the hammer systems used.	Y	
f.	Summary of general soil, bedrock, and groundwater conditions, including a generalized interpretation of findings?	Y	
g.	A statement of which version (date) of the SGE specification the exploration was performed in accordance with?	Y	
h.	Statement of where geotechnical reports are available for review?	Y	
i.	Initials of personnel and dates they performed field reconnaissance, subsurface exploration and preparation of the geotechnical profile?	Y	

VI.A. Geotechnical Profile Checklist

Cover Sheet	(Y/N/X)	Notes:
14 Has a Legend been provided?	Y	
15 Have the following items been included in the Legend:	Y	
a. Symbols and usual descriptions for only the soil and bedrock types presented in the Geotechnical Profile, as per the Soil and Rock Symbology Chart in Appendix D of the SGE?	Y	
b. All miscellaneous symbols and acronyms, used on any of the sheets, defined?	Y	
c. The number of soil samples for each classification that were mechanically classified and visually described in the current exploration?	Y	
16 Has a Location Map, showing the beginning and end stations for the project, been shown on the cover sheet, sized per the L&D3 Manual?	Y	
17 Have the station limits for each plan and profile sheet for projects with multiple alignments, or greater than 1500', been identified in a table?	Y	
18 Have the station limits for any cross section sheets been identified in the same table?	X	
19 Has a list of any structures for which structure foundation explorations been performed been identified in the same table?	Y	
20 If sampling and testing for a scour analysis was performed, has this data been shown in tabular form?	Y	
21 Has a summary table of test data for all roadway and subgrade boring samples been shown?	Y	
22 If borings from previous subsurface explorations are being used, has that data been shown in a separate table?	X	
23 In the summary table, has the data been displayed by roadway and subgrade boring in ascending stationing order for each roadway?	Y	
24 Have the centerline or baseline station, offset, and exploration identification number been provided for each boring presented in the table?	Y	

VI.A. Geotechnical Profile Checklist

Cover Sheet	(Y/N/X)	Notes:
25 For each sample, has the following information been provided in the summary table:	Y	
a. Sample depth interval?	Y	
b. Sample number and type?	Y	
c. N_{60} ?	Y	
d. Percent recovery?	Y	
e. Hand Penetrometer?	Y	
f. Percentage of aggregate, coarse sand, fine sand, silt, and clay size particles?	Y	
g. Liquid limit, plastic limit, plasticity index, and water content, all rounded to the nearest percent or whole number?	Y	
h. ODOT classification and Group Index?	Y	
i. Visual description of samples not mechanically classified, including water content, and estimated ODOT classification with 'Visual' in parentheses?	Y	
j. Sulfate Content test results?	Y	
26 Have all undisturbed test results been displayed in graphical format on the sheet prior to the plan and profile sheets?	Y	
Surface Data	(Y/N/X)	Notes:
27 Has the following information been shown on each roadway plan drawing:	Y	
a. Existing surface features described in Section 702.5.1?	Y	
b. Proposed construction items, as described in Section 702.5.2?	Y	
c. Project and historic boring locations, with appropriate exploration targets and exploration identification numbers?	Y	
d. Notes regarding observations not readily shown by drawings?	X	
28 Have the existing ground surface contours been presented?	Y	
29 If cross sections are to be developed for stationing covered on a plan sheet, has an index for the appropriate cross section sheets been included on the plan sheet?	X	

VI.A. Geotechnical Profile Checklist

Subsurface Data	(Y/N/X)	Notes:
30 Has all the subsurface data been presented in the form of a profile along the centerline or baseline, and on cross sections where applicable?	Y	
31 Have the graphical boring logs been correctly shown, as follows:	Y	
a. Location and depth of boring indicated by a heavy dashed vertical line?	Y	
b. Exploration identification number above the boring?	Y	
c. Logs indicate soil and bedrock layers with symbols 0.4" wide and centered on the heavy dashed vertical line where possible?	Y	
d. Bedrock exposures with 0.4" wide symbols, but without a heavy dashed vertical line?	X	
e. Soil and bedrock symbols as per ODOT Soil and Rock Symbolology chart (SGE - Appendix D)?	Y	
f. Historical borings shown in same manner with the exploration identification number above the boring?	X	
32 Have the proposed groundline and existing groundline been shown on the profile view, according to ODOT CADD standards?	Y	
33 Have the locations of the proposed structure foundation elements been shown on the profile view?	Y	
34 Have the offsets from centerline or baseline been indicated above the borings in the profile view?	Y	
35 Have borings located immediately adjacent to the centerline or baseline and considered representative of centerline or baseline subsurface conditions been referenced directly to the centerline or baseline?	Y	
36 Have offset borings in or near the same elevation interval of a centerline or baseline boring been plotted either on a cross section or immediately above or below the centerline boring in a box containing an elevation scale?	Y	
37 Have cross-sections been developed to show subsurface conditions disclosed by a series of borings drilled transverse to centerline or baseline?	X	

VI.A. Geotechnical Profile Checklist

Subsurface Data	(Y/N/X)	Notes:
38 Have the existing and proposed groundlines been displayed on cross section sheets according to ODOT CADD standards?	Y	
39 Have bedrock exposures shown on the cross sections been plotted along the contour of the cross section?	X	
40 Has the following information been provided adjacent to the graphical logs or bedrock exposure:	Y	
a. Thickness, to the nearest inch, of sod/topsoil or other shallow surface material written above the boring (with corresponding symbology at top of log)?	Y	
b. Moisture content, to nearest whole percent, with the bottom of the text aligned with the bottom of the sample? Label this column as 'WC' at bottom of the boring.	Y	
c. N_{60} , aligned with the bottom of sample? Label column as ' N_{60} ' at bottom of boring.	Y	
d. Free water indicated by a horizontal line with a 'w' attached, and water level at the end of drilling indicated by an open equilateral triangle, point down?	Y	
e. Complete geologic description of each bedrock unit, including unit core loss, unit RQD, SDI, and compressive strength test results? (Do not present geologic descriptions for structure borings for which this information is presented on the boring logs as described in 703.3)	X	
f. Visual description of any uncontrolled fill or interval not adequately defined by a graphical symbol?	X	
g. Organic content with modifiers, per 603.5?	Y	
h. Designate a plastic soil with moisture content equal to or greater than the liquid limit minus three with a 1/8" solid black circle adjacent to the moisture content?	Y	
i. Designate a non-plastic soil with moisture content exceeding 25% or exceeding 19% but appearing wet initially, with a 1/8" open circle with a horizontal line through it adjacent to the moisture content?	Y	
j. The reason for discontinuing a boring prior to reaching the planned depth indicated immediately below the boring?	X	

VI.A. Geotechnical Profile Checklist

Boring Logs	(Y/N/X)	Notes:
41 Have the boring logs of all structure borings, all geohazard borings, and any roadway borings drilled in the vicinity of the structures or geohazard been shown on the boring log sheets following the plan and profile sheets? (Create the logs in accordance with 703.3)	Y	
42 Have the boring logs been developed by integrating the driller's field logs, laboratory test data, and visual descriptions?	Y	
43 Has the following boring information been included in the heading of each boring log:	Y	
a. Exploration identification number?	Y	
b. Project designation (C-R-S) and PID?	Y	
c. Structure File Number (if applicable) and project type?	Y	
d. Centerline or baseline name, station, offset, and surface elevation?	Y	
e. Coordinates?	Y	
f. Method of drilling?	Y	
g. Date started and date completed?	Y	
h. Method and material (including quantity) used for backfilling or sealing, including type of instrumentation, if any (reported in the footer)?	Y	
i. Date of last calibration and drill rod energy ratio (ER) in percent for the hammer system(s) used, not to exceed 90%?	Y	
44 Has the following boring information been included in each boring log:	Y	
a. A depth and elevation scale?	Y	
b. Indication of stratum change?	Y	
c. Description of material in each stratum?	Y	
d. Depth of bottom of boring?	Y	
e. Depth of boulders or cobbles, if encountered?	X	
f. Caving depth?	Y	
g. Water level observations?	Y	
h. Artesian water level and height of rise?	X	
i. Heaving sand?	Y	
j. Cavities or other unusual conditions?	X	
k. Depth interval represented by sample?	Y	
l. Sample number and type?	Y	
m. Percent recovery for each sample?	Y	
n. Measured blow counts for each 6 inches of drive for split spoon samples, not to exceed 18 inches total?	Y	
o. N_{60} to the nearest whole number?	Y	
p. Hand penetrometer?	Y	

VI.A. Geotechnical Profile Checklist

Boring Logs	(Y/N/X)	Notes:
q. Particle-size analysis?	Y	
r. Liquid limit, plastic limit, plasticity index?	Y	
s. Water content?	Y	
t. ODOT soil classifications, with "V" in parentheses for those samples that are not mechanically classified?	Y	
u. Top of bedrock and bedrock descriptions?	X	
v. Rock core run percent recovery?	X	
w. Run RQD?	X	
x. Unit rock core percent recovery?	X	
y. Unit RQD?	X	
z. SDI, if applicable?	X	
aa. Rock compressive strength test results, if applicable?	X	

VI.B. Geotechnical Reports

C-R-S:	STA-241-17.37	PID:	119585	Reviewer:	BKS	Date:	4/9/2025
General		(Y/N/X)	Notes:				
1	Has an electronic copy of all geotechnical submissions been provided to the District Geotechnical Engineer (DGE)?	Y					
2	Has the first complete version of a geotechnical report being submitted been labeled as 'Draft'?	Y					
3	Subsequent to ODOT's review and approval, has the complete version of the revised geotechnical report being submitted been labeled 'Final'?	Y					
4	Has the boring data been submitted in a native format that is DIGGS (Data Interchange for Geotechnical and Geoenvironmental) compatible? gINT files meet this demand?	Y					
5	Does the report cover format follow ODOT's Brand and Identity Guidelines Report Standards found at http://www.dot.state.oh.us/brand/Pages/default.aspx ?	Y					
6	Have all geotechnical reports being submitted been titled correctly as prescribed in Section 706.1 of the SGE?	Y					
Report Body		(Y/N/X)	Notes:				
7	Do all geotechnical reports being submitted contain the following:						
a.	an Executive Summary as described in Section 706.2 of the SGE?	Y					
b.	an Introduction as described in Section 706.3 of the SGE?	Y					
c.	a section titled "Geology and Observations of the Project," as described in Section 706.4 of the SGE?	Y					
d.	a section titled "Exploration," as described in Section 706.5 of the SGE?	Y					
e.	a section titled "Findings," as described in Section 706.6 of the SGE?	Y					
f.	a section titled "Analyses and Recommendations," as described in Section 706.7 of the SGE?	Y					
Appendices		(Y/N/X)	Notes:				
8	Do all geotechnical reports being submitted contain all applicable Appendices as described in Section 706.8 of the SGE?	Y					
9	Do the Appendices present a site Boring Plan showing all boring locations as described in Section 706.8.1 of the SGE?	Y					

VI.B. Geotechnical Reports

Appendices	(Y/N/X)	Notes:
10 Do the Appendices include boring logs and color pictures of rock, if applicable, as described in Section 706.8.2 of the SGE?	Y	
11 Do the Appendices include reports of undisturbed test data as described in Section 706.8.3 of the SGE?	Y	
12 Do the Appendices include calculations in a logical format to support recommendations as described in Section 706.8.4 of the SGE?	Y	