
**FINAL REPORT
STRUCTURE FOUNDATION EXPLORATION
TUS-NEIGHBOR STREET BRIDGE
PID: 118634
NEWCOMERSTOWN, OHIO**

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NEAS PROJECT 24-0061

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EXECUTIVE SUMMARY

A bridge replacement project (TUS-Neighbor Street Bridge PID# 118634) is planned to replace the existing bridge carrying Neighbor Street over Buckhorn Creek in Newcomerstown, Tuscarawas County, Ohio. The report presents a summary of the encountered surficial and subsurface conditions and our recommendations for bridge foundation design and construction.

National Engineering and Architectural Services Inc. (NEAS) has been contracted to perform geotechnical engineering services for the project. The purpose of the geotechnical engineering services is to perform geotechnical explorations within the project limits to obtain information concerning the subsurface soil and groundwater conditions relevant to the design and construction of the project. As part of the referenced explorations, NEAS advanced 2 project borings and conducted laboratory testing to characterize the soils/rocks for engineering purposes. The report presents a summary of the encountered surficial and subsurface conditions and our recommendations for bridge foundation design and construction.

The subsurface profile at the proposed bridge replacement site generally consists primarily of very loose to very dense granular materials, with secondarily very stiff to hard cohesive soils. Both project borings encountered a 10.5 ft thick fill layer beneath the pavement section. Bedrock was encountered in both project borings at depths ranging from 38.3 ft to 60.5 ft below ground surface, corresponding to elevations between 738.8 ft and 761.0 ft above mean sea level.

According to our subgrade analysis, unstable subgrade conditions, including areas of weak soils and high moisture content, were encountered in one of the two project borings within the project limit. Therefore, NEAS recommends that spot stabilization be performed from the forward abutment to the end of the project. Spot stabilization should be in the form of Excavate and Replace (Item 204 with Geotextile). Excavations are estimated to extend to a depth of 12 inches, with the excavated material being replaced with material in accordance with Section 608 "Excavate and Replace (Item 204)" of the ODOT GDM (ODOT, 2024). Stabilization limits should extend 18-inches beyond the edge of the proposed paved roadway, shoulder or median and it is recommended removing any topsoil, existing pavement materials or abandoned structure foundation materials.

A foundation review was completed for a deep foundation system for the referenced replacement bridge, based on the following design information: 1) the Site Plan for the Bridge conducted by The Thrasher Group; 2) subsurface exploration data; and 3) load and scour information provided by The Thrasher Group. HP piles, typically designed as end-bearing piles on bedrock, provide high axial working capacity. Per the Thrasher Group's email dated August 22, 2025, the maximum axial service and factored loads at the rear abutment are 102 kips and 140 kips per pile, respectively. At the forward abutment, the maximum axial service and factored loads are 76 kips and 104 kips per pile, respectively. The maximum lateral loads per pile are 19 kips at the rear abutment and 9 kips at the forward abutment. Scour information provided via email on September 11, 2025, indicates that the 50-year local scour at both abutments is 0 ft, meaning no scour is anticipated at the abutments.

For end-bearing HP piles, axial resistance is determined by structural considerations. According to Section 305.3.3 of the ODOT Bridge Design Manual (BDM, 2024), the maximum factored structural resistance for HP 10x42 piles is 310 kips, which exceeds the required axial load. Based on our axial analyses, the estimated pile lengths are approximately 30 ft at the rear abutment and 55 ft at the forward abutment.

The project site is classified as Site Class of E - Soft Soil.

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1. INTRODUCTION

1.1. General

National Engineering and Architectural Services Inc. (NEAS) presents our Structure Foundation Exploration Report for the planned replacement of the existing bridge that carries Neighbor Street over Buckhorn Creek in Newcomerstown, Tuscarawas County, Ohio. The report presents a summary of the encountered surficial and subsurface conditions and our recommendations for bridge foundation design and construction. Our recommendations are in accordance with ODOT's 2025 LRFD Bridge Design Manual (BDM) (ODOT, 2025), ODOT's 2025 Geotechnical Design Manual (GDM) (ODOT, 2025).

The exploration was conducted in general accordance with NEAS, Inc.'s proposal to The Thrasher Group, Inc. dated June 10, 2024, and with the provisions of ODOT's *Specifications for Geotechnical Explorations* (SGE) (ODOT, 2025).

The scope of work performed included: 1) a review of published geotechnical information; 2) performing 2 total test borings; 3) laboratory testing of soil samples in accordance with the SGE; 4) geotechnical engineering analysis to assess foundation design and construction considerations; and 5) the development of this summary report.

1.2. Proposed Construction

The existing Neighbor Street bridge is a single-span steel beam structure with a reinforced concrete deck and abutments. The bridge is approximately 58 ft in length, with an approximate roadway width of 23 ft.

It is our understanding that the proposed structure is a single-span prestressed concrete box beam bridge with composite deck on semi-integral abutments. The proposed bridge spans 72 ft from center to center of bearing, with a roadway width of 20'-10".

2. GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1. Geology and Physiography

The project site is located within the Muskingum-Pittsburgh Plateau, part of the Allegheny Plateaus (ODGS, 1998). This is a moderately high to high relief (300 to 600 ft), dissected plateau, having broad major valleys that contain outwash terraces and tributaries with lacustrine terraces; medium-grained bedrock sequences coarser than those in the Marietta Plateau but finer than those in the Ironston Plateau. Remnants of ancient Teays-age drainage system are uncommon. Soils in this region are characteristically Wisconsin-age sand, gravel, lacustrine silt, and silt-loam colluvium over Mississippian- to Pennsylvanian-age shales, siltstones, sandstones, and economically important coals and claystones (ODGS, 1998).

Based on the Bedrock Geologic Units Map of Ohio (USGS & ODGS, 2006), bedrock within the project area was characterized as undivided Allegheny and Pottsville Groups consists of shale, siltstone and underclay. Shale was described as black, gray and olive, clayey to silty, locally contains marine fossils. Siltstone was described as gray, greenish and olive, clayey to sandy, thinned bedded to medium bedded, locally contain marine fossils. Underclay was described as gray and olive, generally 3 feet or less in thickness, clayey to silty, commonly rooted and underlying coal beds, nonbedded, locally varies from flint to plastic clay. Sandstone was described as light to medium gray weathers to shades of yellow-brown,

mostly very fine to medium grained, locally quartzose and conglomeratic in lower one-third of unit, thin to massive to cross bedded. Limestone was described as black to light gray, micritic to medium grained, locally grades into flint, thin to medium bedded to discoidal concretions containing marine fossils. Coal was described as mostly banded bituminous, locally cannel, thin to locally as much as 12 feet thick, generally in discreet beds (ODGS, 2003). Based on the ODNR bedrock topography map of Ohio, bedrock elevations at the project site can be expected to be around 700 to 750 ft amsl.

The soils at the project site have been mapped (Web Soil Survey) by the Natural Resources Conservation Service (USDA, 2015) as primarily occasionally flooded Tioga fine sandy loam series. The Tioga fine sandy loam series is comprised of both coarse- and fine-grained soils and classifies as A-1, A-2, A-3, A-4 and A-6 type soils according to the AASHTO method of soil classification.

2.2. Hydrology/Hydrogeology

Groundwater at the project site can be expected to be at an elevation consistent with that of the nearby Buckhorn Creek, as it is the most dominant hydraulic influence in the vicinity of the project's boundaries. The water level of the Buckhorn Creek may generally be representative of the local groundwater table. However, it should be noted that perched groundwater systems may exist in certain areas due to the presence of fine-grained soils, which can hinder the permeation of groundwater to the phreatic surface.

The project is located within a special flood hazard area (Zone A), by the Federal Emergency Management Agency's (FEMA) National Flood Hazard mapping program (FEMA, 2016).

2.3. Mining and Oil/Gas Production

No abandoned underground mines are noted on ODNR's Abandoned Underground Mine Locator within the immediate vicinity of the project site (ODNR [1], 2024).

According to the ODNR's Ohio Oil & Gas Locator map, no oil or gas wells are located in the immediate vicinity of the project site (ODNR [2], 2024).

2.4. Historical Records and Previous Phases of Project Exploration

A historic record search was performed through ODOT's Transportation Information Management System (TIMS). No historical plans or borings are available at the project site.

2.5. Field Reconnaissance

A field reconnaissance visit for the bridge carrying Neighbor St. over Buckhorn Creek was conducted on February 24, 2025, during which site conditions were noted and photographed.

The land use of the area surrounding the proposed project site can be described as consisting of residential properties.

The existing bridge carrying Neighbor St. over Buckhorn Creek consists of a single-span, multi-I-beam structure on full-height abutments with a concrete bridge deck (Photograph 1). In the area of the referenced bridge, Neighbor Street is roughly level, with the terrain rising very gently from east to west. The overall bridge structure appeared to be in poor condition, with signs of distress observed primarily on the bridge superstructure. The main support beams were heavily corroded, and section loss was observed on the webs and flanges of some beams (Photograph 2 & 3). The abutments were in fair to good condition, showing

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signs of erosion and some longitudinal cracks (Photograph 4). One concrete pipe was observed piercing the western abutment (Photograph 5). The stream banks to the east of the bridge were unprotected, experiencing minor erosion and possible slope movement, as indicated by the bends in multiple trees along the banks (Photographs 6 & 7). The scour at the foundations of the bridge appeared to be minor, with sediment buildup observed near both corners of the bridge (Photograph 8).

The bridge deck and the asphalt wearing course were observed to be in fair to good. The asphalt wearing course exhibited moderate severity map cracking, along with longitudinal and transverse cracking (Photograph 9). The concrete bridge deck was also in fair to good condition, with deterioration confined to a few isolated areas (Photographs 10). Regarding drainage, a concrete drainage pipe was observed outletting past the western corner of the bridge and appeared to be well-drained, with no signs of ponding observed during our field visit.

Photograph 1 Overall Bridge View



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Photograph 2 Corrosion and Section Loss at Main Support Beams



Photograph 3 Corrosion and Section Loss at Main Support Beams



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Photograph 4 Overall Condition of Abutments



Photograph 5 Concrete Pipe at the Western Side of the Bridge



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Photograph 6 Stream Banks East of Bridge



Photograph 7 Stream Banks East of Bridge



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Photograph 8 Sediment and Scour



Photograph 9 Overall Condition of Asphalt Wearing Course



Photograph 10 Overall Bridge Deck Condition



3. GEOTECHNICAL EXPLORATION

3.1. Field Exploration Program

The project subsurface exploration was conducted by NEAS between November 5, 2024, and November 6, 2024, and included two borings drilled to depths ranging from 50.2 ft to 70.8 ft below ground surface. The boring locations were selected by NEAS in general accordance with the guidelines contained in the SGE with the intent to evaluate subsurface soil and groundwater conditions. Borings were typically located within the planned project construction areas that were not restricted by underground utilities or dictated by terrain (e.g. steep embankment slopes). Project boring locations were in the field after drilling by NEAS. Each individual project boring log (included within Appendix B) includes the recorded boring latitude and longitude location and the corresponding ground surface elevation. The boring locations are depicted on the Site Plan provided in Appendix A. Latitude/Longitude, elevations and stationing and offsets of the borings are shown on Table 1 below.

Table 1: Project Boring Summary

Boring Number	Latitude	Longitude	Elevation (NAVD 88) (ft)	Depth (ft)	Substructure	Elevation of Top of Bedrock (NAVD 88) (ft)
B-001-0-24	40.276087	-81.612190	799.3	50.2	Rear Abutment	761.0
B-002-0-24	40.276157	-81.611857	799.3	70.8	Forward Abutment	738.8

Project borings were drilled using a CME 55TB truck-mounted drilling rig utilizing 3.25-inch (inner diameter) hollow stem auger. In general, soil samples were recovered continuously to 7.5 ft below ground surface, using an 18-inch split spoon sampler (AASHTO T-206 “Standard Method for Penetration Test and Split Barrel Sampling of Soils.”). The soil samples obtained from the exploration program were visually observed in the field by the NEAS field representative and preserved for review by a Geologist for possible laboratory testing. Standard penetration tests (SPT) were conducted using a CME auto hammer calibrated to be 89% efficient on July 30, 2024, as indicated on the boring logs.

Field /boring logs were prepared by drilling personnel, and included lithological description, SPT results recorded as blows per 6-inch increment of penetration and estimated unconfined shear strength values on specimens exhibiting cohesion (using a hand-penetrometer). Groundwater level observations were recorded both during and after the completion of drilling. These groundwater level observations are included on the individual boring logs. After completing the borings, the boreholes were backfilled with bentonite grout and patched with cold patch asphalt and/or quickset concrete where necessary and appropriate.

3.2. Laboratory Testing Program

The laboratory testing program consisted of classification testing, moisture content determinations and unconfined compressive strength testing. Data from the laboratory testing program was incorporated onto the boring logs (Appendix B).

3.2.1. Classification Testing

Representative soil samples were selected for index properties (Atterberg Limits) and gradation testing for classification purposes on approximately 55% of the samples. At each boring location, samples were selected for testing with the intent of identification and classification of all significant soil units. Soils not selected for testing were compared to laboratory tested samples/strata and classified visually. Moisture content testing was conducted on all samples. The laboratory testing was performed in general accordance with applicable AASHTO specifications.

A final classification of the soil strata was made in accordance with AASHTO M-145 “Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes,” as modified by ODOT “Classification of Soils” once laboratory test results became available. The results of the soil classification are presented on the boring logs provided in Appendix B.

3.2.2. Standard Penetration Test Results

Standard Penetration Tests (SPT) and split-barrel (commonly known as split-spoon) sampling of soils were performed at varying intervals (i.e., continuous, 2.5-ft, or 5.0-ft intervals) in the project borings performed. To account for the high efficiency (automatic) hammers used during SPT sampling, field SPT N-values were converted based on the calibrated efficiency (energy ratio) of the specific drill rig's hammer. Field N-values were converted to an equivalent rod energy of 60% (N_{60}) for use in analysis or for correlation purposes. The resulting N_{60} values are shown on the boring logs provided in Appendix B.

3.2.3. D_{50} Values for Scour Evaluation

Grain size distribution testing was performed on the obtained streambed samples to develop D_{50} values (i.e., the diameter in the particle-size distribution curve corresponding to 50% finer). The calculated D_{50} values are shown in Table 2 below and the developed particle-size distribution curves are included with the associated boring log within Appendix B.

Table 2: D₅₀ Values for Scour Evaluation

Boring Number	Specimen ID	Specimen Elevation (ft)	ODOT (Modified AASHTO) ~ USCS Classification	D ₅₀ (mm)
B-001-0-24	SS-1	797.8' - 796.3'	A-1-b ~ SILTY SAND with GRAVEL(SM)	0.593
	SS-3	794.8' - 793.3'	A-2-4 ~ SILTY SAND with GRAVEL(SM)	0.240
	SS-4	793.3' - 791.8'	A-2-4 ~ SILTY SAND(SM)	0.206
	SS-5	790.8' - 789.3'	A-4a ~ SILTY SAND(SM)	0.115
	SS-6	788.3' - 786.8'	A-3a ~ SILTY SAND(SM)	0.157
	SS-8	783.3' - 781.8'	A-2-4 ~ SILTY SAND with GRAVEL(SM)	0.306
	SS-9	780.8' - 779.3'	A-4a ~ SILTY SAND(SM)	0.099
B-002-0-24	SS-1	797.8' - 796.3'	A-4a ~ SILTY SAND(SM)	0.113
	SS-3	794.8' - 793.3'	A-4a ~ SILTY SAND(SM)	0.118
	SS-4	793.3' - 791.8'	A-4a ~ SILTY SAND with GRAVEL(SM)	0.221
	SS-5	790.8' - 789.3'	A-4a ~ SILTY, CLAYEY SAND(SC-SM)	0.082
	SS-6	788.3' - 786.8'	A-3a ~ SILTY SAND(SM)	0.115
	SS-7	785.8' - 784.3'	A-1-a ~ SILTY SAND with GRAVEL(SM)	2.866

3.2.4. Unconfined Compressive Strength of Rock Core

Unconfined Compressive Strength of rock core samples was conducted in accordance with ASTM D7012 "Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying Sates of Stress and Temperatures". The tests were performed on two rock core samples obtained during the exploration program. The results are summarized in Table 3 below and provided in Appendix C.

Table 3: Unconfined Compressive Strength of Rock Core Results

Boring ID	Depth (ft)	Elevation (ft)	Unconfined Compressive Strength (psi)	Stain at Failure (%)
B-001-0-24	45.1 - 45.4	753.9 - 754.2	862	3.3
B-002-0-24	62.4 - 62.8	736.5 - 736.9	4666	2.2

4. GEOTECHNICAL FINDINGS

The subsurface conditions encountered during NEAS's explorations are described in the following subsections and/or on each boring log presented in Appendix B. The boring logs represent NEAS's interpretation of the subsurface conditions encountered at each boring location based on our site observations, field logs, visual review of the soil samples by NEAS's geologist, and laboratory test results. The lines designating the interfaces between various soil strata on the boring logs represent the approximate interface location; the actual transition between strata may be gradual and indistinct. The subsurface soil and groundwater characterizations included herein, including summary test data, are based on the subsurface findings from the geotechnical explorations performed by NEAS as part of the referenced project, and consideration of the geological history of the site.

4.1. Subsurface Conditions

The subsurface profile at the proposed bridge replacement site generally consists primarily of very loose to very dense granular materials, with secondarily very stiff to hard cohesive soils. Both project borings encountered a 10.5 ft thick fill layer beneath the pavement section. Bedrock was encountered in both project borings at depths ranging from 38.3 ft to 60.5 ft below ground surface, corresponding to elevations between 738.8 ft and 761.0 ft above mean sea level.

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4.1.1. Overburden Soil

At the proposed rear abutment, fills were encountered below the pavement section to 10.5 ft below ground surface (from 799.3 ft to 788.8 ft), including medium dense Gravel and Stone Fragments with Sand (A-1-b), Gravel and Stone Fragments with Sand and Silt (A-2-4), and Sandy Silt (A-4a). Below the fills, the subsurface soils encountered generally consisted of non-cohesive coarse-grained soils underlain by cohesive fine-grained soils. The non-cohesive soils at the rear abutment location, from the elevation of 788.8 ft to 775.5 ft, are described as having a relative compactness ranging from loose to very dense, correlating to N_{60} values between 9 and 111 blows per foot (bpf). The natural moisture content of the non-cohesive soils ranged from 11% to 16%. The cohesive fine-grained soils, from the elevation of 775.5 ft to 761.0 ft (top of bedrock), were classified as Sand Silt (A-4a), Silt (A-4b), and Silt and Clay (A-6a). The cohesive soils can be described as having a hard consistency, based on N_{60} values between 21 bpf and refusal, and unconfined compressive strengths (estimated by means of hand penetrometer) of approximately 4.50 tsf. The natural moisture contents of the cohesive soils ranged from 13% to 17%.

At the proposed forward abutment, fills were encountered below the pavement section to 10.5 ft below ground surface (from 799.3 ft to 788.8 ft), including loose to medium dense Sandy Silt (A-4a), and hard Sandy Silt (A-4a). Below the fills, the subsurface soils encountered generally consisted of non-cohesive coarse-grained soils underlain by cohesive fine-grained soils. The non-cohesive soils at the forward abutment location, from the elevation of 788.8 ft to 761.0 ft, are described as having a relative compactness of very loose to medium dense, correlating to N_{60} values between 3 and 31 blows per foot (bpf). The natural moisture content of the non-cohesive soils ranged from 9% to 25%. The cohesive fine-grained soils, from the elevation of 761.0 ft to 738.8 ft (top of bedrock), were classified as Silt (A-4b) and Silt and Clay (A-6a). The cohesive soils can be described as having a hard consistency based on N_{60} values between 12 and 36 bpf, and unconfined compressive strengths (estimated by means of hand penetrometer) between 4.00 tsf and 4.50 tsf. Natural moisture contents of the cohesive soils ranged from 11% to 21%.

4.1.2. Groundwater

Groundwater measurements were taken during the drilling procedures and/or immediately following the completion of each borehole. Groundwater was not encountered in any of the project borings during drilling.

It should be noted that groundwater is affected by many hydrologic characteristics in the area and may vary from those measured at the time of the exploration.

4.1.3. Bedrock

At the rear abutment, bedrock was encountered at a depth of 38.3 feet below the ground surface (761.0 feet above mean sea level), while at the forward abutment, it was found at 60.5 feet below the ground surface (738.8 feet above mean sea level).

Based on the exploration and testing conducted, bedrock at the project site was classified as slightly to highly weathered, very weak to moderately strong, and slightly fractured to highly fractured narrow to tight Shale. Recovery of the bedrock core was 98 percent while the Rock Quality Designation (RQD) values ranged from 59 to 73 percent.

5. ANALYSES AND RECOMMENDATIONS

We understand that this project entails replacing the existing bridge with a single span prestressed concrete box beam bridge with composite deck on semi-integral abutments. The proposed bridge spans 75 ft from abutment to abutment, with a span length of 72 ft and a roadway width of 20'-10". The summary and results of our evaluation as well as recommendations are presented in subsequent sections.

5.1. Soil Profile for Analysis

For analysis purposes, each boring log was reviewed, and a generalized material profile was developed for analysis. Utilizing the generalized soil profile, engineering properties for each soil strata were estimated based on their field (i.e., SPT N_{60} Values, hand penetrometer values, etc.) and laboratory (i.e., Atterberg Limits, grain size, etc.) test results using correlations provided in published engineering manuals, research reports and guidance documents. The developed soil profile and estimated engineering soil and rock properties (with cited correlation/reference material) used in our evaluation is summarized per boring within Tables 4 and 5 below.

Table 4: B-001-0-24 Soil Profile

Bridge over Buckhorn Creek : Soil Profile B-001-0-24							
Soil Description	Unit Weight ⁽¹⁾ (pcf)	Moist Unit Weight ⁽¹⁾ (pcf)	Saturated Unit Weight ⁽¹⁾ (pcf)	Undrained Shear Strength ⁽²⁾ (psf)	Effective Cohesion ⁽³⁾ (psf)	Effective Friction Angle ⁽³⁾ (degrees)	Setup Factor (f_{su})
Gravel with Sand (A-1-b) Depth (799.3 ft - 794.8 ft)	115	115	125	-	-	40	1.00
Gravel with Sand and Silt (A-2-4) Depth (794.8 ft - 791.3 ft)	115	115	125	-	-	35	1.20
Sandy Silt (A-4a) Depth (791.3 ft - 788.8 ft)	122	112	122	-	-	31	1.20
Coarse and Fine Sand (A-3a) Depth (788.8 ft - 783.8 ft)	125	115	125	-	-	33	1.00
Gravel with Sand (A-1-b) Depth (783.8 ft - 781.3 ft)	140	130	140	-	-	40	1.00
Sandy Silt (A-4a) Depth (781.3 ft - 775.5 ft)	125	115	125	-	-	31	1.20
Sandy Silt (A-4a) Depth (775.5 ft - 771 ft)	125	115	125	2600	250	26	1.50
Silt (A-4b) Depth (771 ft - 766 ft)	140	130	140	7100	450	30	1.50
Silt and Clay (A-6a) Elevation (766 ft - 761 ft)	128	118	128	4250	300	26	1.50

Notes:

1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405.
2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used.
3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.

Table 5: B-002-0-24 Soil Profile

Bridge over Buckhorn Creek : Soil Profile B-002-0-24							
Soil Description	Unit Weight ⁽¹⁾ (pcf)	Moist Unit Weight ⁽¹⁾ (pcf)	Saturated Unit Weight ⁽¹⁾ (pcf)	Undrained Shear Strength ⁽²⁾ (psf)	Effective Cohesion ⁽³⁾ (psf)	Effective Friction Angle ⁽³⁾ (degrees)	Setup Factor (f_{su})
Sandy Silt (A-4a) Depth (799.3 ft - 791.3 ft)	112	112	122	-	-	31	1.20
Sandy Silt (A-4a) Depth (791.3 ft - 788.8 ft)	120	110	120	1250	150	24	1.50
Coarse and Fine Sand (A-3a) Depth (788.8 ft - 786.3 ft)	122	112	122	-	-	32	1.00
Gravel with Sand (A-1-b) Depth (786.3 ft - 783.8 ft)	122	112	122	-	-	34	1.00
Gravel with Sand (A-1-b) Depth (783.8 ft - 778.8 ft)	118	108	118	-	-	31	1.00
Coarse and Fine Sand (A-3a) Depth (778.8 ft - 771 ft)	120	110	120	-	-	31	1.00
Silt (A-4b) Depth (771 ft - 761 ft)	125	115	125	-	-	32	1.50
Silt (A-4b) Depth (761 ft - 746 ft)	125	115	125	3400	250	27	1.50
Silt and Clay (A-6a) Elevation (746 ft - 738.8 ft)	128	118	128	4100	300	26	1.50

Notes:

1. Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405.
2. Values calculated from Terzaghi and Peck (1967) if $N_{160} < 52$, else Stroud and Butler (1975) was used.
3. Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.

5.2. Pavement Design and Recommendations

The subgrade analysis was performed in accordance with ODOT's GDM criteria utilizing the ODOT provided: *Subgrade Analysis Spreadsheet* (SubgradeAnalysis.xls, Version 14.7 dated November 6, 2024). Input information for the spreadsheet was based on the soil characteristics gathered during NEAS's subgrade exploration (i.e., SPT results, laboratory test results, etc.), and our geotechnical experience. For analysis purposes, the proposed roadway elevations were assumed to be the same as the existing roadway elevations.

A subgrade analysis was performed to identify the method, location, and dimensions (including depth) of recommended subgrade stabilization in the referenced project plan. Appropriate stabilization of the subgrade will ensure a constructible pavement buildup, enhance pavement performance over its life, and help reduce costly extra work change orders (ODOT SGE, 2024). In addition to identifying stabilization recommendations, pavement design parameters are also determined to aid in pavement section design. The subsections below present the results of our subgrade analysis including pavement design parameters and unsuitable/unstable subgrade conditions if any identified within the project limits. Subgrade analysis spreadsheet for the referenced roadway segment is provided in Appendix D.

5.2.1. Pavement Design Recommendations

It is our understanding that pavement analysis and design is to be performed to determine the proposed pavement sections for the segments within the project limits to undergo full depth replacement. A subgrade analysis was performed using the subgrade soil data obtained during our field exploration program to evaluate the soil characteristics and develop pavement parameters for use in pavement design. The subgrade analysis parameters recommended for use in pavement design are presented in Table 6 below. Provided in the table are ranges of maximum, minimum and average N_{60L} values for the indicated segments as well as the design CBR value recommended for use in pavement design.

Table 6: Pavement Design Values

Maximum N_{60L}	Minimum N_{60L}	Average N_{60L}	Average PI Value	Design CBR
15	9	9	0	11

5.2.2. Unsuitable/Unstable Subgrade

Per ODOT's GDM, the presence of select subgrade conditions may require some form of subgrade stabilization within the subgrade zone for new pavement construction. These unsuitable and unstable subgrade conditions generally include the presence of rock, specific soil types, weak soil conditions, and overly moist soil conditions. With respect to the planned roadways, these subgrade conditions are further discussed in the following subsections.

5.2.2.1. Rock

Rock was not encountered within the top 6 inches of the proposed grade in either of the borings performed; therefore, no specialized remediation efforts are necessary.

5.2.2.2. Prohibited Soils

Prohibited soil types, per the GDM, include A-4b, A-2-5, A-5, A-7-5, A-8a, A-8b, and soils with liquid limits greater than 65. No prohibited soils were encountered within the subgrade of the referenced project roadway.

5.2.2.3. Weak Soils

The GDM recommends subgrade stabilization for soils considered unstable in which the N_{60} value of a particular soil sample (SS) at a referenced boring location is less than 12 bpf and in some cases less than 15 bpf (i.e., where moisture content is greater than optimum plus 3 percent). Based on the specific N_{60} value at the subject boring, *Figure 600-1 - Subgrade Stabilization* within the GDM recommends a depth of subgrade stabilization for ODOT standard stabilization methods. It should be noted that although a soil sample's N_{60} value may meet the criteria to be considered an unstable soil, the depth in which the unstable soil is encountered in relation to the proposed subgrade is considered when each individual subgrade boring is analyzed. For example, if the GDM recommends an excavate and replace of 12 inches within a weak soil underlying 18 inches of stable material, it would be unreasonable to recommend the removal of both the stable and unstable material for a total of 30 inches of excavate and replace. A summary of boring locations where weak soil condition was encountered within the limits of the proposed alignment are shown in Table 7 below.

Table 7: High Moisture Content Soils Summary

Boring ID	N_{60L}	Subgrade Depth (ft)
B-002-0-24	9	1.5 - 3.0

5.2.2.4. High Moisture Content Soils

High moisture content soils are defined by the GDM as soils that exceed the estimated optimum moisture content (per Table 600-1 - Optimum Moisture Content within the ODOT GDM) for a given classification by 3 percent or more. Per the GDM, soils determined to be above the identified moisture content levels are a likely indication of the presence of an unstable subgrade and may require some form of subgrade stabilization. Similar to our analysis of unstable soils, although a soil sample's moisture content may meet the criteria to be considered high, the depth in which the high moisture soil is encountered in relation to the proposed subgrade is considered when each individual subgrade boring is analyzed for stabilization recommendations. Summaries of the boring locations where high moisture content conditions were encountered within the limits of each proposed alignment are shown in Table 8 below.

Table 8: High Moisture Content Soils Summary

Boring ID	Soil Type	Moisture Content (%)	Optimum Moisture Content (%)	Depth Below Subgrade (ft)
B-002-0-24	A-4a	15	11	0.0 - 1.5
		15	10	1.5 - 3.0

5.2.3. Stabilization Recommendations

According to our subgrade analysis, unstable subgrade conditions, including areas of weak soils and high moisture content, were encountered in one of the two project borings within the project limit. Therefore, NEAS recommends that spot stabilization be performed from the forward abutment to the end of the project. Spot stabilization should be in the form of Excavate and Replace (Item 204 with Geotextile). Excavations are estimated to extend to a depth of 12 inches, with the excavated material being replaced with material in accordance with Section 608 "Excavate and Replace (Item 204)" of the ODOT GDM (ODOT, 2024). Stabilization limits should extend 18-inches beyond the edge of the proposed paved roadway, shoulder or median and it is recommended removing any topsoil, existing pavement materials or abandoned structure foundation materials.

5.3. Bridge Foundation Analysis and Recommendations

A foundation review was completed for a deep foundation system for the referenced replacement bridge based on the following design information: 1) the site plan for the bridge conducted by The Thrasher Group; 2) subsurface exploration; and 3) load and scour information provided by The Thrasher Group. Bedrock elevations descend from west to east.

5.3.1. Axially Loaded Pile Analysis

Deep foundations are proposed to support the substructures of the Neighbor Street bridge over Buckhorn Creek. According to the site plan provided by The Thrasher Group via email on August 22, 2025, the bottom of the footing is approximately at an elevation of 784.69 ft for the rear abutment and 788.01 ft for the forward abutment. Bedrock was encountered at elevations of 761.0 ft and 738.8 ft in the project borings B-001-0-24 and B-002-0-24, respectively.

HP piles, typically used as end bearing piles bearing on bedrock, provide high axial working capacity. Per the Thrasher Group's email dated August 22, 2025, the maximum axial service and factored loads at the rear abutment are 102 kips and 140 kips per pile, respectively. At the forward abutment, the maximum axial service and factored loads are 76 kips and 104 kips per pile, respectively. The maximum lateral loads per pile are 19 kips at the rear abutment and 9 kips at the forward abutment. Scour information provided via email on September 11, 2025, indicates that the 50-year local scour at both abutments is 0 ft, meaning no scour is anticipated at the abutments.

For end-bearing HP-piles, axial resistance is determined by structural considerations. According to Section 305.3.3 of the ODOT Bridge Design Manual (BDM, 2024), the maximum factored structural resistance for HP 10x42 piles is 310 kips, which exceeds the required axial load.

5.3.2. Pile Foundation Recommendations

Pile lengths based on: 1) the "Estimated Length" and "Order Length" definitions and formulas presented in Section 305.3.5.2 "Estimated Pile Length" of the BDM, are shown in Table 9. It is assumed that the piles will be supported from the elevations at the bottom of footing as shown in the site plan provided by The Thrasher Group via email on August 22, 2025. The calculated 'estimated' length assumes penetration through a 1 ft embedment into the concrete footing for the abutments, and rounding up to the nearest 5 ft.

Table 9: Estimated Pile Lengths Summary

Substructure	Bottom of Pile Cap Elevation (ft amsl)	Assumed Pile Cutoff Elevation (ft amsl)	Top of Bedrock	Pile Type	Nearby Borings	Geotechnical Pile Length (ft)	Estimated Pile Length (ft)	Order Length (ft)
TUS - Neighbor Street Bridge								
Rear Abutment	784.7	785.7	761.0	HP 10x42	B-001-0-24	23.7	30	35
Forward Abutment	788.0	789.0	738.8	HP 10x42	B-002-0-24	49.2	55	60

5.3.1. Pile Drivability

NEAS's pile drivability evaluation estimated a Delmag D19-42 diesel hammer to determine if the HP10X42 steel H-pile at the abutments and piers would be overstressed at any time during pile installation (i.e., compressive stresses experienced by pile during driving are greater than 90% of the yield strength of the steel) or encounter driving refusal (i.e., hammer blow counts higher than 100 blows per foot) at any time during pile installation. The results of the evaluation indicated that the HP 10X42 steel H-pile would not be

overstressed during the pile installation process at the each substructure location, based on: 1) the indicated hammer and steel section; 2) the use of ASTM A 572 Grade 50 steel piles; and, 3) our developed model used in the computer program GRLWEAP by GRL Engineers, Inc (GRLWEAP 14 results can be found in Appendix D). It should be noted that drivability is difficult to assess quantitatively as the field test results (i.e., SPT N_{60} values, pocket penetrometer values, etc.) tend to be very high. Furthermore, pile drivability is highly reliant upon the specific equipment used in construction; therefore, it is recommended that the contractor should provide an analysis to demonstrate that the equipment planned for use is capable of pile installation without over-stressing the pile.

5.4. Global Stability Analysis

For the purposes of evaluating the stability of the abutments, NEAS reviewed the cross-section and project boring logs to determine the subsurface soil conditions that posed the greatest potential for slope instability. Based on our review, NEAS developed a representative cross-sectional model at each abutment to serve as the basis for global stability analyses. The models were developed from NEAS's interpretation of the available information which included: 1) the proposed Bridge Site Plan prepared by The Thrasher Group; 2) a live load surcharge of 250 psf, accounting for traffic induced loads; and, 3) test borings and laboratory data developed as part of this report. With respect to the soil's engineering properties, the provided Soil Profile Estimated Engineering Properties presented in Section 5.1 of this report were used in our analyses. It should be noted that deep foundation was considered at the forward abutment in our analyses.

The above referenced slope stability models were analyzed for long-term (Effective Stress) slope stability utilizing the software entitled Slide 7.0 by Rocscience, Inc. Specifically, the Modified Bishop, Spencer and GLE analysis methods were used to calculate a factor of safety (FOS) for circular type slope failures. The FOS is the ratio of the resisting forces and the driving forces, with the desired safety factor being more than about 1.5 which equates to an AASHTO resistance factor less than 0.65 (per AASHTO, 2020 - the specified resistance factors are essentially the inverse of the FOS that should be targeted in slope stability programs). For this analysis, a resistance factor of 0.65 or lower is targeted as the slope contains or supports a structural element.

Based on our slope stability analyses for the referenced abutment section, the minimum slope stability safety factor for long-term (Effective Stress) conditions exceeded the desired value of 1.54. The results of the analyses are summarized in Table 10. Based on the results of the analysis, it is our opinion that the subsurface conditions encountered at this location are generally satisfactory and the site can be considered to be stable at short-term and long-term condition. The graphical output of the slope stability program (cross-sectional model, calculated safety factor, and critical failure plane) is presented in Appendix D.

Table 10: Global Stability Analysis Summary

Global Stability Analysis at Neighbor Street Bridge						
Location	Boring No.	Water Condition	Description	Minimum Factor of Safety	Equivalent Resistance Factor	Status (OK/NG)
Rear Abutment	B-001-0-24	Normal Water	Short Term	1.56	0.64	OK
			Long Term	1.56	0.64	OK
		HW 100	Short Term	1.67	0.60	OK
			Long Term	1.67	0.60	OK
Forward Abutment	B-002-0-24	Normal Water	Short Term	10.51	0.10	OK
			Long Term	5.77	0.17	OK
		HW 100	Short Term	14.10	0.07	OK
			Long Term	7.38	0.14	OK

5.5. Seismic Site Class

Based on the results of the subsurface exploration, laboratory test data, and the AASHTO Site Class Definitions indicated in Table 3.10.3.1-1 of the *LRFD Bridge Design Specifications, 9th Edition* (AASHTO LRFD, 2020), the average Standard Penetration Test blow count $\bar{N}$ for B-001-0-24 and B-002-0-24 is 21 blows/ft and 11 blows/ft, respectively. To be conservative, the project site is classified as Site Class of E – Soft Soil, with $\bar{N} < 15$ blows/ft.

6. QUALIFICATIONS

This investigation was performed in accordance with accepted geotechnical engineering practices for the purpose of characterizing the subsurface conditions at the site of the proposed TUS-Neighbor Street Bridge (PID# 118634) in Tuscarawas County, Ohio. This report has been prepared for The Thrasher Group, Inc., ODOT and their design consultants to be used solely in evaluating the soils/rock underlying the indicated structures and presenting geotechnical engineering recommendations specific to this project. The assessment of general site environmental conditions or the presence of pollutants in the soil, rock and groundwater of the site was beyond the scope of this geotechnical exploration. Our recommendations are based on the results of our field explorations, laboratory test results from representative soil samples, geotechnical engineering analyses and historical information. The results of the field explorations and laboratory tests, which form the basis of our recommendations, are presented in the appendices as noted. This report does not reflect any variations that may occur between the borings or elsewhere on the site, or variations whose nature and extent may not become evident until a later stage of construction. In the event that any changes occur in the nature, design or location of the proposed structural work, the conclusions and recommendations contained in this report should not be considered valid until they are reviewed and have been modified or verified in writing by a geotechnical engineer.

It has been a pleasure to be of service to The Thrasher Group, Inc. in performing this geotechnical exploration for the TUS-Neighbor Street Bridge (PID# 118634) replacement project. Please call if there are any questions, or if we can be of further service.

Respectfully Submitted,

Chunmei (Melinda) He, Ph.D., P.E.
Project Manager

Zhao Mankoci, Ph.D., P.E.
Geotechnical Engineer

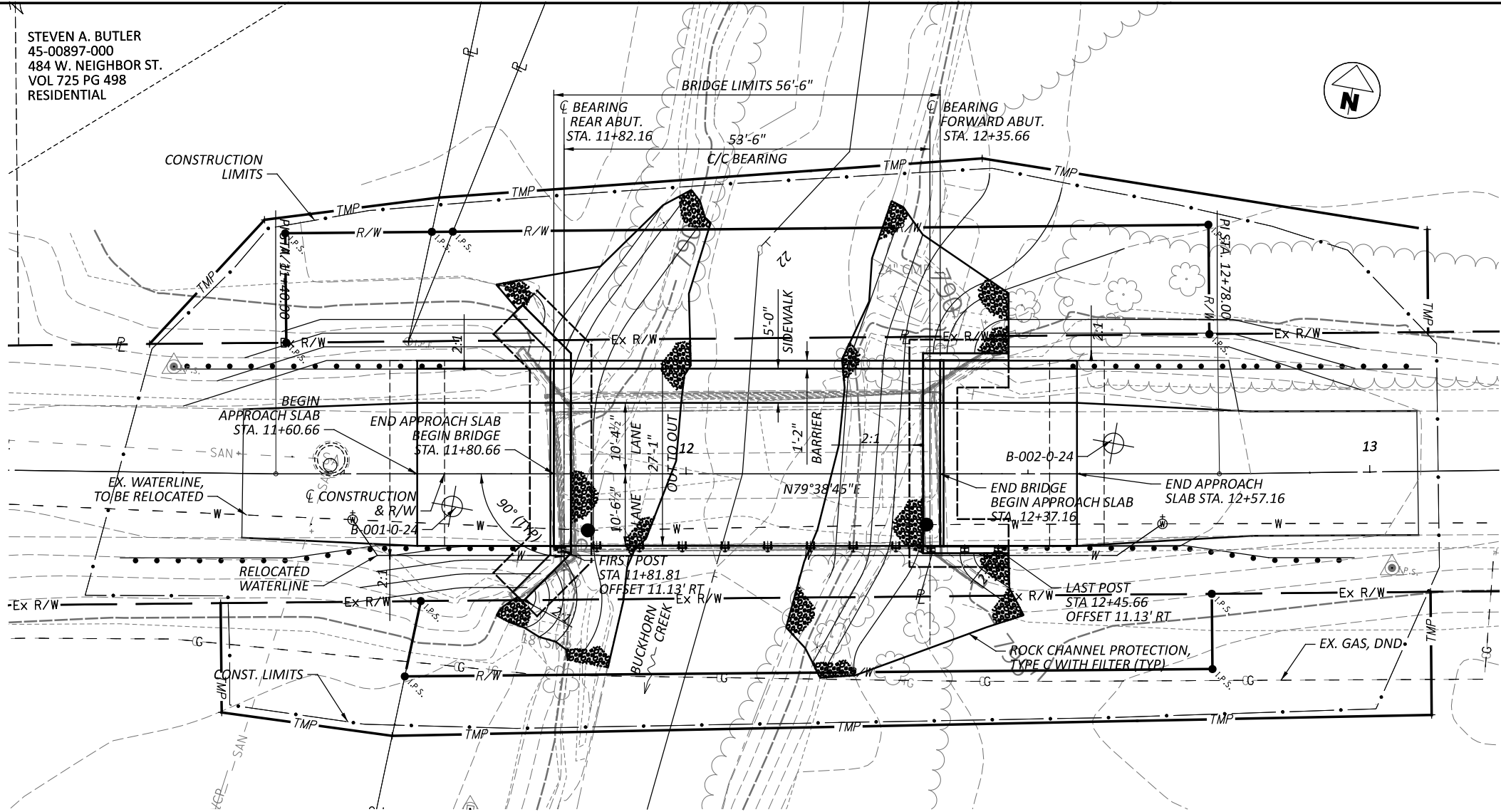
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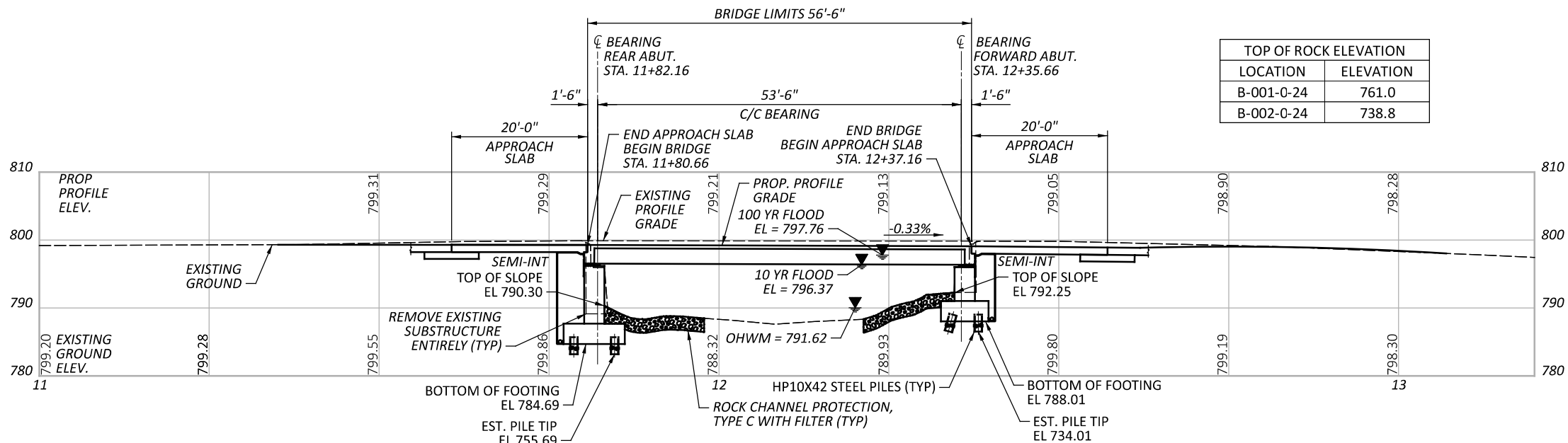
APPENDIX A

SITE PLAN

STEVEN A. BUTLER
45-00897-000
484 W. NEIGHBOR ST.
VOL 725 PG 498
RESIDENTIAL



PLAN



PROFILE ALONG C NEIGHBOR STREET

TOP OF ROCK ELEVATION	
LOCATION	ELEVATION
B-001-0-24	761.0
B-002-0-24	738.8

BENCHMARK DATA

BM #1 STA.	13+03.07,	ELEV.	796.268,	OFFSET	14.09	RIGHT
BM #2 STA.	16+53.18,	ELEV.	795.909,	OFFSET	21.05	RIGHT
BM #3 STA.	11+24.86,	ELEV.	797.473,	OFFSET	15.49	LEFT
BM #4 STA.	11+68.42,	ELEV.	792.779,	OFFSET	49.05	RIGHT

FOR ADDITIONAL BENCHMARK INFORMATION. SEE ROADWAY PLAN SHEET 03

NOTES

EARTHWORK LIMITS SHOWN ARE APPROXIMATE. ACTUAL SLOPES SHALL CONFORM TO PLAN CROSS SECTIONS.

DESIGN TRAFFIC:

2027 ADT =	120	2027 ADTT =	4
2047 ADT =	130	2047 ADTT =	4
DIRECTIONAL DISTRIBUTION =		3%	

LEGEND

⊕ BORING LOCATION

HYDRAULIC DATA

DRAINAGE AREA = 22.9 SQ. MILES
Q (100) = 4,620 CFS V (100) = 3.97 FT/S
Q (10) = 2,420 CFS V (10) = 3.95 FT/S
STRUCTURE CLEARS THE 10 YEAR
DESIGN HW BY 0 FEET.

EXISTING STRUCTURE

TYPE: SINGLE-SPAN STEEL BEAMS ON
REINFORCED CONCRETE DECK AND
ABUTMENTS

SPANS: 53'-6" ±

ROADWAY: 20'-0"± F/F SAFETY CURB

LOADING:

SKEW: 0°

WEARING SURFACE: BITUMINOUS ASPHALT CONCRETE

APPROACH SLABS:

ALIGNMENT: TANGENT

CROWN: 0.016±

STRUCTURE FILE NUMBER: 7960999

DATE BUILT:

DISPOSITION: REMOVE AND REPLACE

PROPOSED STRUCTURE

TYPE: SINGLE SPAN PRESTRESSED CONCRETE
BOX BEAM BRIDGE WITH COMPOSITE
DECK ON SEMI-INTEGRAL ABUTMENTS

SPANS: 53'-6" C/C BEARINGS

ROADWAY: 20'-11" TOE CURB/ RAIL

LOADING: HL93 AND NO FUTURE WEARING SURFACE

SKEW: 0°

WEARING SURFACE: 1" MONOLITHIC CONCRETE

APPROACH SLABS: 20'-0" LONG (AS-1-15, AS-2-15)

ALIGNMENT: TANGENT

CROWN: 0.016 FT/FT

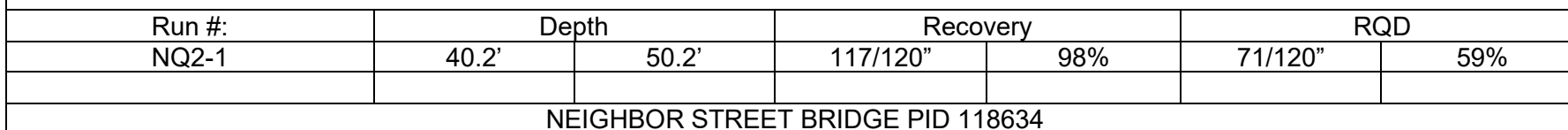
DECK AREA: 1530 SF

COORDINATES: LATITUDE 40° 16' 34.12"
LONGITUDE -81° 36' 43.42"

APPENDIX B

BORING LOGS

B-001-0-24





OHIO DEPARTMENT OF TRANSPORTATION
OFFICE OF GEOTECHNICAL ENGINEERING

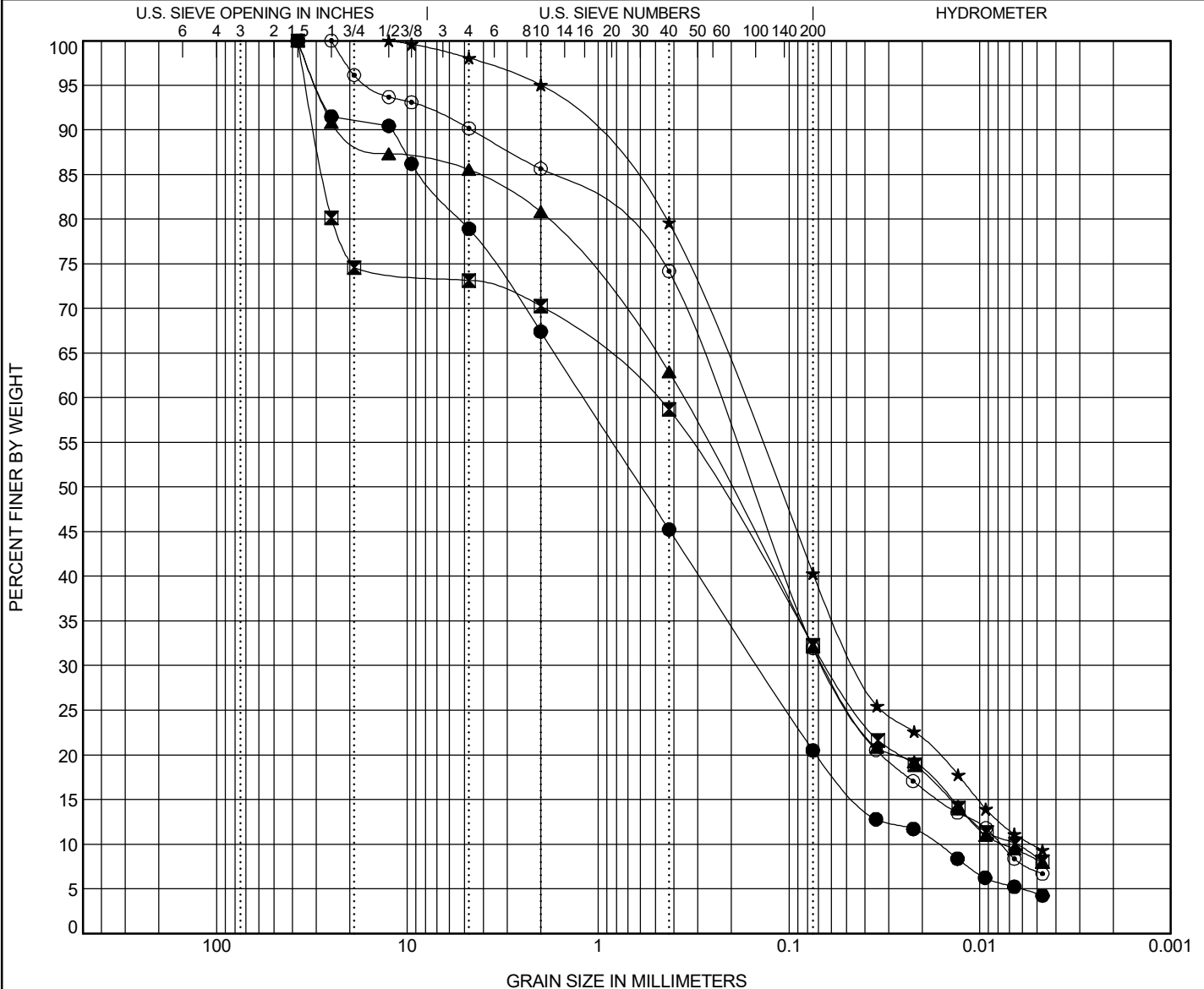
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PROJECT NEIGHBOR ST

PID

OGE NUMBER 0

PROJECT TYPE



COBBLES	GRAVEL	SAND		SILT	CLAY
		coarse	fine		

Specimen Identification			ODOT (Modified AASHTO) ~ USCS Classification								LL	PL	PI
●	B-001-0-24	1.5	A-1-b ~ SILTY SAND with GRAVEL(SM)								NP	NP	NP
☒	B-001-0-24	4.5	A-2-4 ~ SILTY SAND with GRAVEL(SM)								NP	NP	NP
▲	B-001-0-24	6.0	A-2-4 ~ SILTY SAND(SM)								NP	NP	NP
★	B-001-0-24	8.5	A-4a ~ SILTY SAND(SM)								NP	NP	NP
◎	B-001-0-24	11.0	A-3a ~ SILTY SAND(SM)								NP	NP	NP
Specimen Identification			D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
●	B-001-0-24	1.5	12.141	0.593	0.146	0.017	33	22	25	16	4	1.05	69.99
☒	B-001-0-24	4.5	30.565	0.24	0.063	0.006	30	12	26	24	8	1.23	78.43
▲	B-001-0-24	6.0	21.165	0.206	0.065	0.007	19	18	31	24	8	1.57	48.25
★	B-001-0-24	8.5	1.205	0.115	0.044	0.005	6	15	39	30	10	2.01	33.47
◎	B-001-0-24	11.0	4.588	0.157	0.066	0.008	15	11	42	25	7	2.35	30.51

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OHIO DEPARTMENT OF TRANSPORTATION
OFFICE OF GEOTECHNICAL ENGINEERING

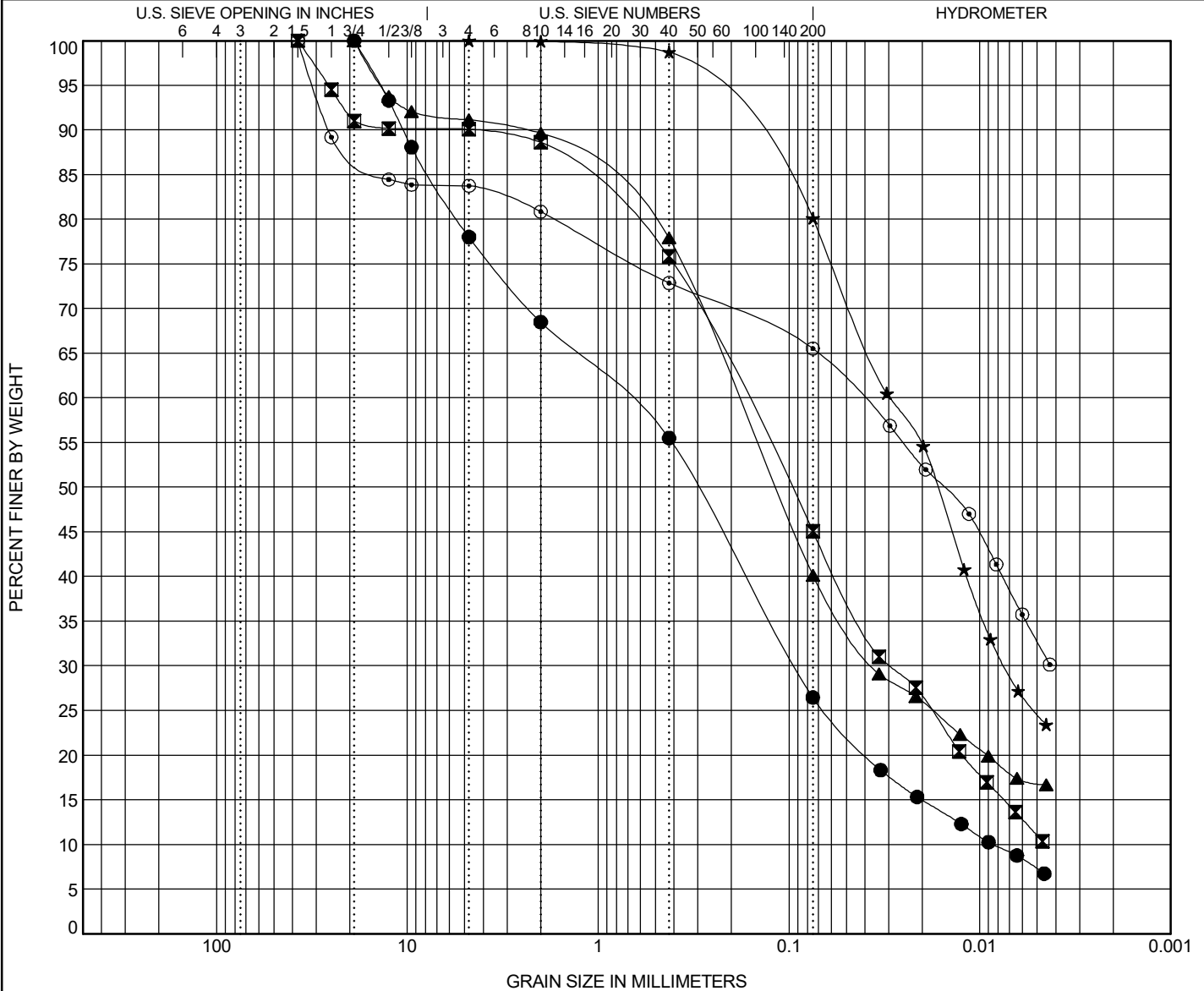
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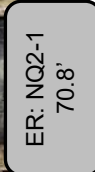
PROJECT TYPE



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PID: _____	SFN: _____	PROJECT: NEIGHBOR ST	STATION / OFFSET: _____					START: 11/6/24		END: 11/6/24		PG 3 OF 3		B-002-0-24									
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	
			737.1								GR	CS	FS	SI	CL	LL	PL	PI					
SHALE , GRAY, SLIGHTLY TO MODERATELY WEATHERED, WEAK TO MODERATELY STRONG, LAMINATED, FISSILE, FOSSILIFEROUS, SILTY, BEDDING DISCONTINUITIES: LOW ANGLE, HIGHLY FRACTURED TO SLIGHTLY FRACTURED, NARROW TO TIGHT, SLIGHTLY ROUGH, BLOCKY, GOOD SURFACE CONDITION; RQD 87%, REC 98%. <i>(continued)</i> @62.4'-62.8'; Qu = 4666 PSI @ 2.2%					73		98	NQ2-1											CORE				
				63																			
				64																			
				65																			
				66																			
				67																			
				68																			
				69																			
	70																						
			728.5	EOB																			
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. DRILLED AS STAKED.																							
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 2 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS																							

B-002-0-24



Run #:	Depth		Recovery		RQD	
NQ2-1	60.8'	70.8'	118/120"	98%	104/120"	87%
NEIGHBOR STREET BRIDGE PID 118634						



OHIO DEPARTMENT OF TRANSPORTATION
OFFICE OF GEOTECHNICAL ENGINEERING

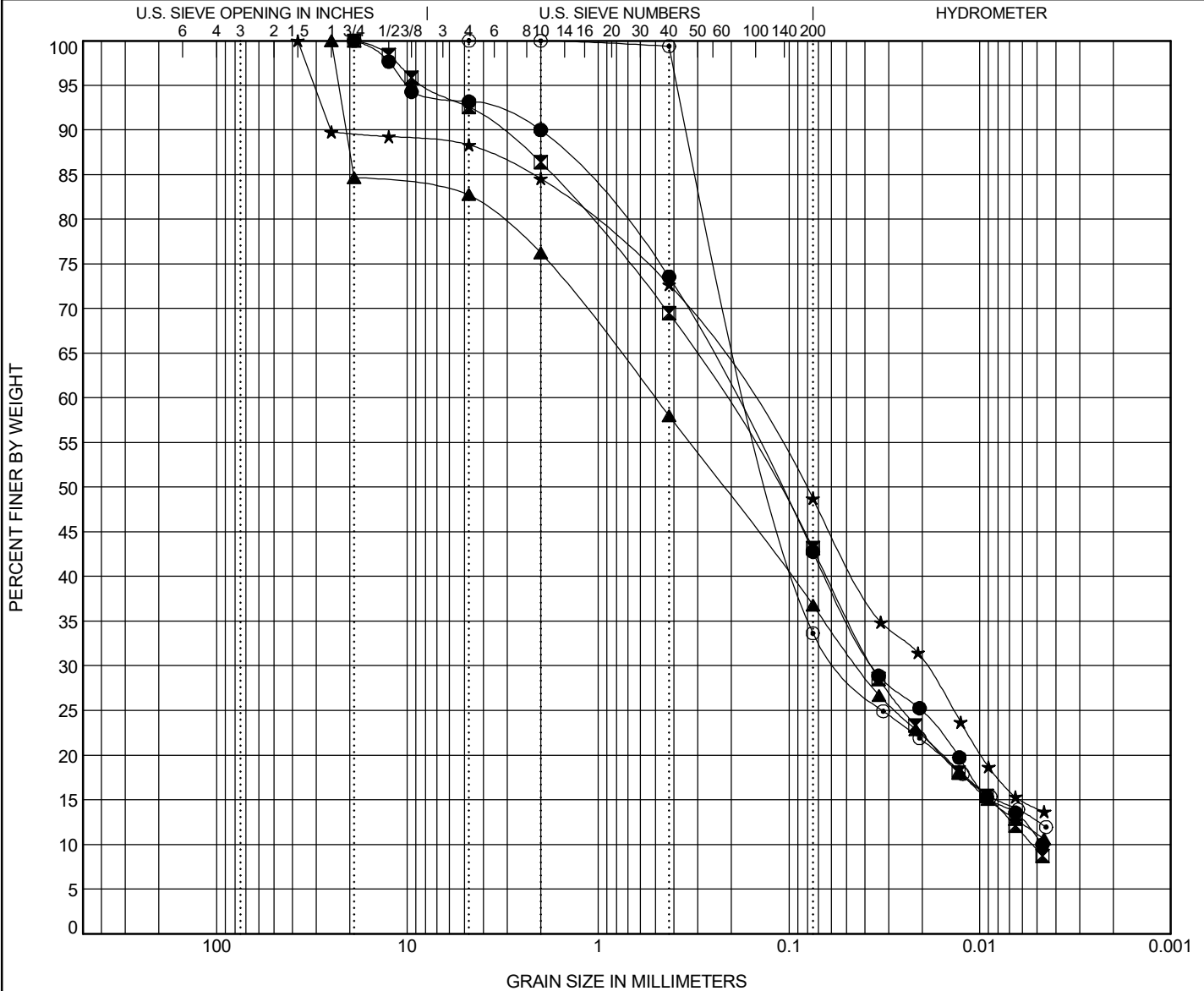
GRAIN SIZE DISTRIBUTION

PROJECT NEIGHBOR ST

PID

OGE NUMBER 0

PROJECT TYPE



COBBLES	GRAVEL	SAND		SILT	CLAY
		coarse	fine		

Specimen Identification			ODOT (Modified AASHTO) ~ USCS Classification								LL	PL	PI
●	B-002-0-24	1.5	A-4a ~ SILTY SAND(SM)								NP	NP	NP
☒	B-002-0-24	4.5	A-4a ~ SILTY SAND(SM)								NP	NP	NP
▲	B-002-0-24	6.0	A-4a ~ SILTY SAND with GRAVEL(SM)								NP	NP	NP
★	B-002-0-24	8.5	A-4a ~ SILTY, CLAYEY SAND(SC-SM)								28	22	6
◎	B-002-0-24	11.0	A-3a ~ SILTY SAND(SM)								NP	NP	NP
Specimen Identification			D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
●	B-002-0-24	1.5	1.999	0.113	0.036	0.005	10	16	31	32	11	1.40	42.02
☒	B-002-0-24	4.5	3.312	0.118	0.037	0.005	14	17	26	34	9	1.11	42.76
▲	B-002-0-24	6.0	20.905	0.221	0.044		24	18	21	26	11		
★	B-002-0-24	8.5	25.216	0.082	0.019		15	12	24	35	14		
◎	B-002-0-24	11.0	0.332	0.115	0.053		0	0	66	21	13		

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OHIO DEPARTMENT OF TRANSPORTATION
OFFICE OF GEOTECHNICAL ENGINEERING

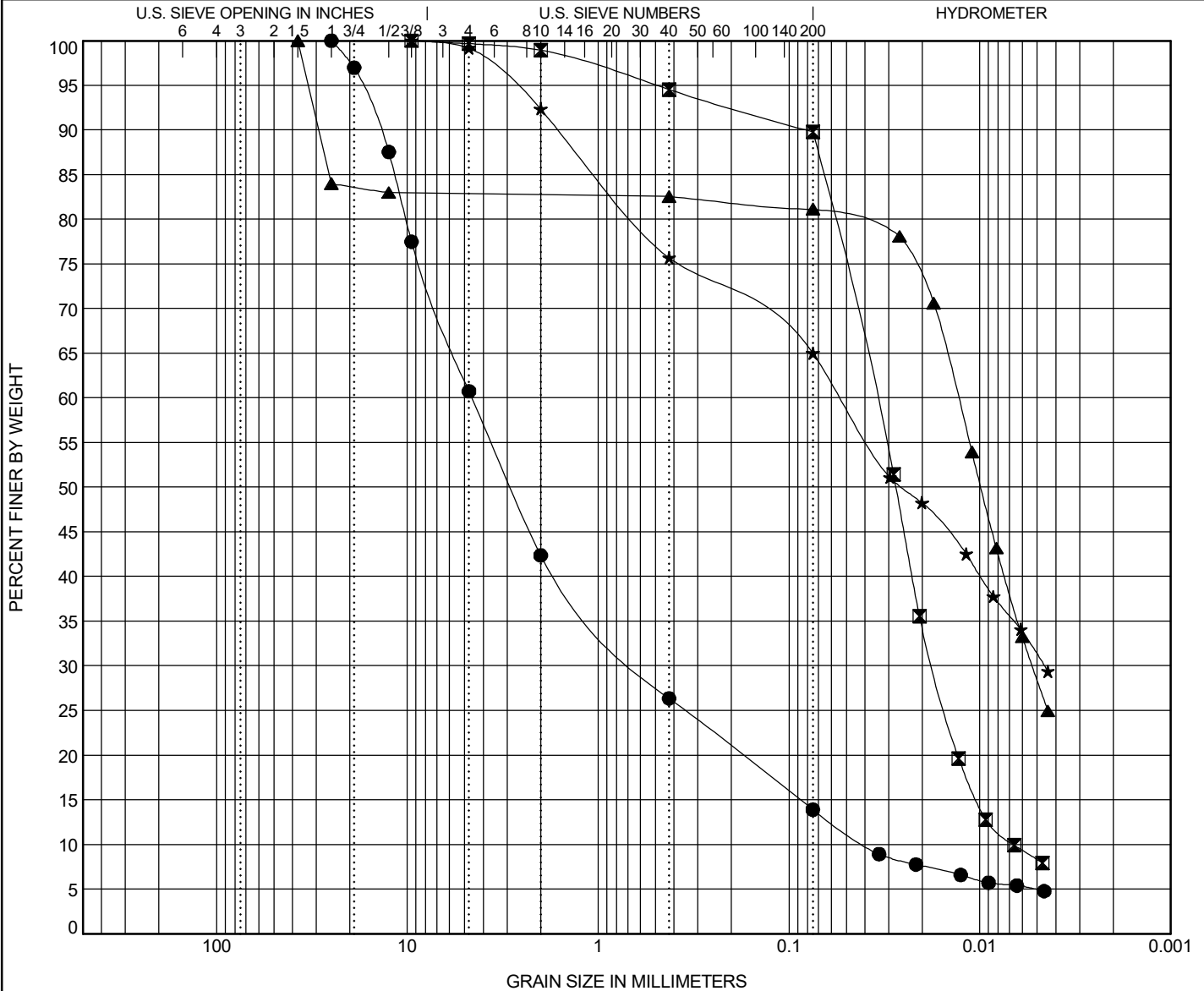
GRAIN SIZE DISTRIBUTION

PROJECT NEIGHBOR ST

PID

OGE NUMBER 0

PROJECT TYPE



COBBLES	GRAVEL	SAND		SILT	CLAY
		coarse	fine		

Specimen Identification			ODOT (Modified AASHTO) ~ USCS Classification								LL	PL	PI
●	B-002-0-24	13.5	A-1-a ~ SILTY SAND with GRAVEL(SM)								NP	NP	NP
☒	B-002-0-24	35.0	A-4b ~ SILT(ML)								NP	NP	NP
▲	B-002-0-24	45.0	A-4b ~ SILT with GRAVEL(ML)								30	24	6
★	B-002-0-24	55.0	A-6a ~ SANDY LEAN CLAY(CL)								37	24	13
Specimen Identification			D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
●	B-002-0-24	13.5	13.943	2.866	0.605	0.04	58	16	12	9	5	1.99	114.17
☒	B-002-0-24	35.0	0.081	0.028	0.018	0.007	1	4	5	82	8	1.31	5.26
▲	B-002-0-24	45.0	29.107	0.01	0.005		18	0	1	53	28		
★	B-002-0-24	55.0	1.607	0.026	0.005		7	17	11	34	31		

APPENDIX C

LAB TEST RESULTS

Unconfined Compressive Strength of Rock Core (ASTM D7012 Method C)

(Project: TUS-Neighbor St, Boring Location: B-001-0-24, NQ2-1, Depth: 45.1-45.4 ft)

Tested Date: 2/19/2025

Specimen Properties

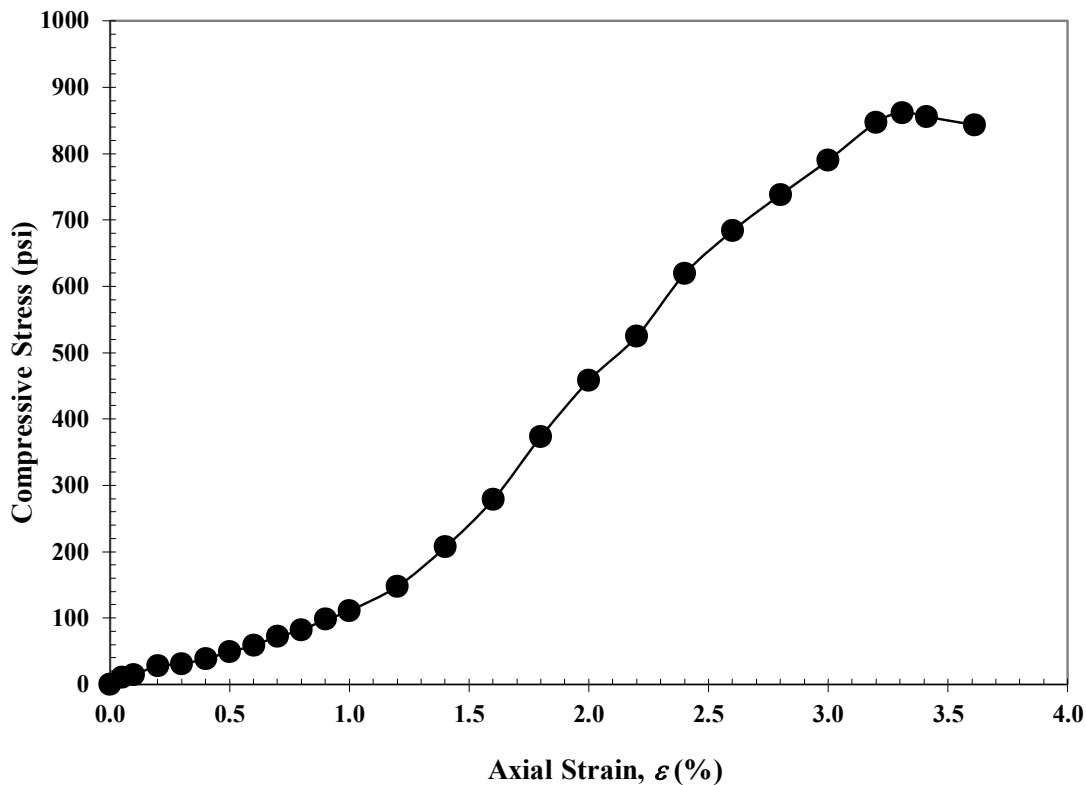
Average Dia., D_{avg} (in):	1.96
Average Height, H_{avg} (in):	3.96
Length to Diameter Ratio:	2.02
Area, A (in ²):	3.03
Volume, V (in ³):	11.99
Wet Mass of Specimen (lb):	1.1
Moisture Content (%):	3.9
Dry Mass of Specimen (lb):	1.0
Wet Unit Weight, γ (lb/ft ³):	156.1
Dry Unit Weight, γ_d (lb/ft ³):	150.3

Final Specimen Figure



Results

Unconfined Compressive Strength (psi):	862	6	(MPa)
Strain (%):	3.3		



Notes: Shale, gray, slightly weathered, weak, fissile, fossiliferous.

Unconfined Compressive Strength of Rock Core (ASTM D7012 Method C)

(Project: TUS-Neighbor St, Boring Location: B-002-0-24, NQ2-1, Depth: 62.4-62.8 ft)

Tested Date: 2/19/2025

Specimen Properties

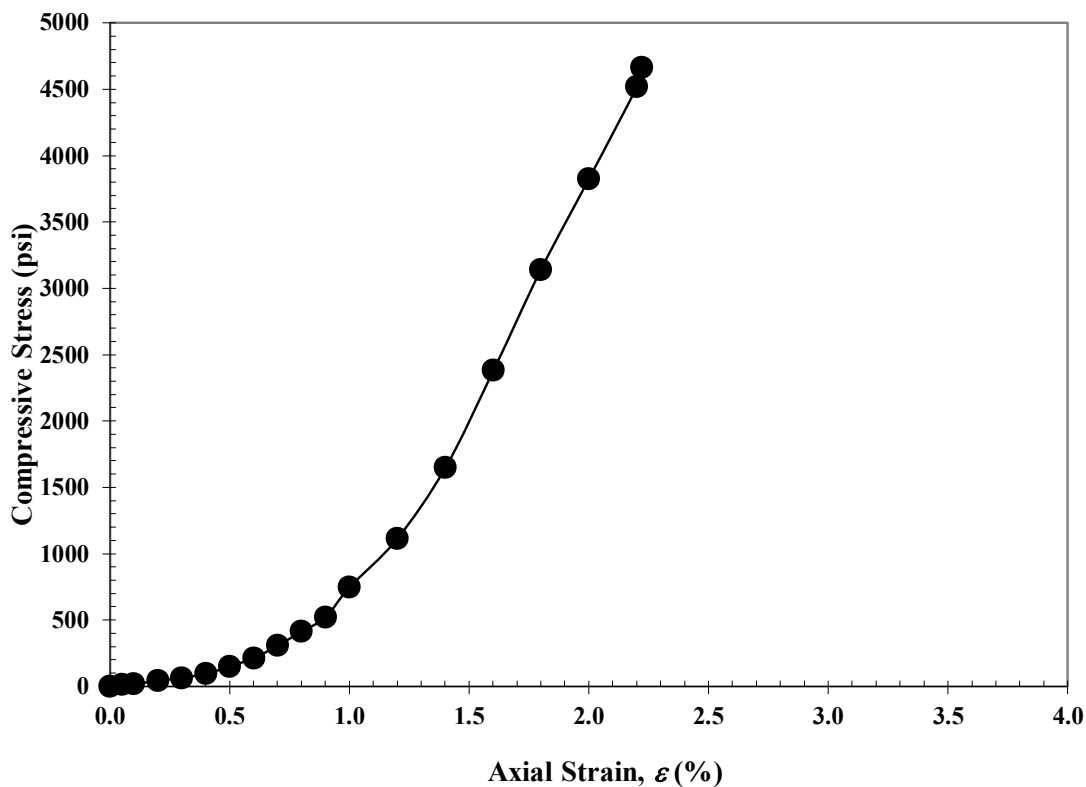
Average Dia., D_{avg} (in):	1.97
Average Height, H_{avg} (in):	4.37
Length to Diameter Ratio:	2.22
Area, A (in ²):	3.05
Volume, V (in ³):	13.33
Wet Mass of Specimen (lb):	1.3
Moisture Content (%):	2.0
Dry Mass of Specimen (lb):	1.2
Wet Unit Weight, γ (lb/ft ³):	163.0
Dry Unit Weight, γ_d (lb/ft ³):	159.8

Final Specimen Figure



Results

Unconfined Compressive Strength (psi):	4666	32	(MPa)
Strain (%):	2.2		



Notes: Shale, gray, slightly weathered, moderately strong, fissile, silty, fossiliferous.



OHIO DEPARTMENT OF TRANSPORTATION

DETERMINING SULFATE CONTENT IN SOILS

SUPPLEMENT 1122

Project C-R-S: Neighbor St Bridge

PID No:

Report Date: 2/21/2025

Consultant: NEAS Inc.

Technician: L. Rosenbeck

Boring ID & Sample #	Station	Offset	Latitude & Longitude or State Plane Coordinates	Elevation	Soaking Time (hr)	Replicate Sample Readings						Sulfate Content (ppm)
						1		2		3		
						Dilution	Reading	Dilution	Reading	Dilution	Reading	
B-001-0-24 SS-1					18.83	20	18	20	17	20	17	347
B-002-0-24 SS-1					18.83	20	0	20	1	20	1	13

APPENDIX D

CALCULATION

SUBGRADE ANALYSIS

**OHIO DEPARTMENT OF TRANSPORTATION****OFFICE OF GEOTECHNICAL ENGINEERING****PLAN SUBGRADES****Geotechnical Design Manual Section 600**

Instructions: Enter data in the shaded cells only.

(Enter state route number, project description, county, consultant's name, prepared by name, and date prepared. This information will be transferred to all other sheets. The date prepared must be entered in the appropriate cell on this sheet to remove these instructions prior to printing.)

**TUS-Nighbor Street Bridge
118634**

Replace the existing structure with a new single-span prestressed concrete box beam bridge over Buckhorn Creek in Newcomerstown, Tuscarawas County, Ohio

NEAS, Inc.

Prepared By: Zhao Mankoci
Date prepared: Thursday, February 27, 2025

Chunmei (Melinda) He, Ph.D., P.E.
2800 Corporate Exchange Drive
Suite 240
Columbus, OH 43231
614.714.0299 Ext 111
che@neasinc.com

NO. OF BORINGS: **2**



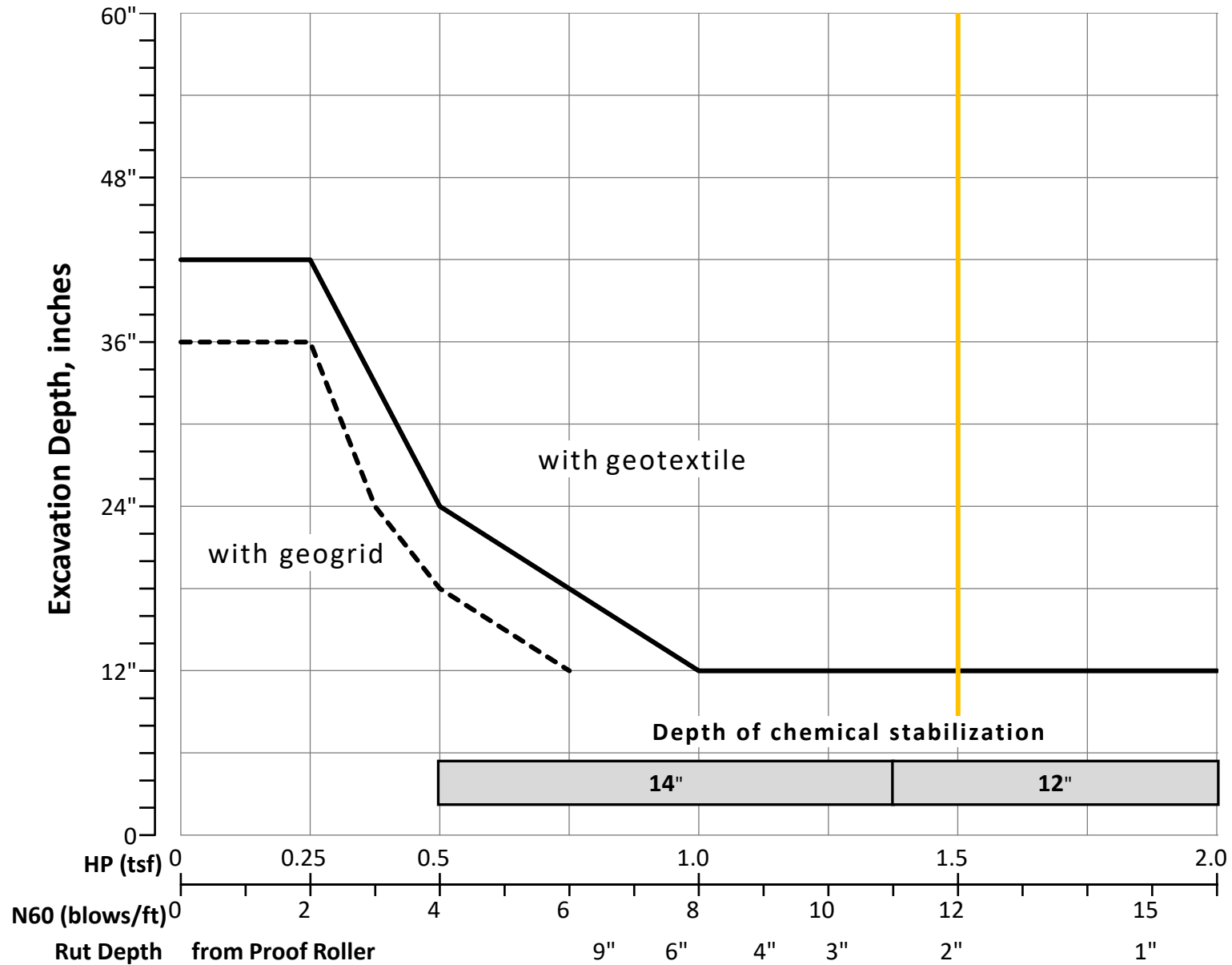
#	Boring ID	Alignment	Station	Offset	Dir	Drill Rig	ER	Boring EL.	Proposed Subgrade EL	Cut Fill
1	B-001-0-24	Neighbor St				CME 55TB	89	799.3	797.8	1.5 C
2	B-002-0-24	Neighbor St				CME 55TB	89	799.3	797.8	1.5 C



#	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 24	SS-1	1.5	3.0	0.0	1.5	28	15		NP	NP	NP	16	4	20	9	6	A-1-b	0	347					
		SS-2	3.0	4.5	1.5	3.0	16								8	6	A-1-b	0							
		SS-3	4.5	6.0	3.0	4.5	22			NP	NP	NP	24	8	32	12	10	A-2-4	0						
		SS-4	6.0	7.5	4.5	6.0	15			NP	NP	NP	24	8	32	13	10	A-2-4	0						
2	B 002-0 24	SS-1	1.5	3.0	0.0	1.5	15	9		NP	NP	NP	32	11	43	15	11	A-4a	2	13		Mc			
		SS-2	3.0	4.5	1.5	3.0	12								15	10	A-4a	8		N ₆₀ & Mc					
		SS-3	4.5	6.0	3.0	4.5	9			NP	NP	NP	34	9	43	12	11	A-4a	2						
		SS-4	6.0	7.5	4.5	6.0	12			NP	NP	NP	26	11	37	13	11	A-4a	0						

[illegible]

Fig. 600-1 – Subgrade Stabilization



OVERRIDE TABLE

Calculated Average	New Values	Check to Override
NP	0.50	☐ HP
12.00	6.00	☐ N60L

Average HP



Average N_{60L}



DRIVABILITY ANALYSIS

GRLWEAP: Wave Equation Analysis of Pile Foundations

Rear Abut + B01_HP10x42

9/12/2025

NATIONAL ENGINEERING AND ARCHITECTURAL

GRLWEAP 14.1.20.1

ABOUT THE WAVE EQUATION ANALYSIS RESULTS

The GRLWEAP program simulates the behavior of a preformed pile driven by either an impact hammer or a vibratory hammer. The program is based on mathematical models, which describe motion and forces of hammer, driving system, pile and soil under the hammer action. Under certain conditions, the models only crudely approximate, often complex, dynamic situations.

A wave equation analysis generally relies on input data, which represents normal situations. In particular, the hammer data file supplied with the program assumes that the hammer is in good working order. All of the input data selected by the user may be the best available information at the time when the analysis is performed. However, input data and therefore results may significantly differ from actual field conditions.

Therefore, the program authors recommend prudent use of the GRLWEAP results. Soil response and hammer performance should be verified by static and/or dynamic testing and measurements. Estimates of bending or other local stresses (e.g., helmet or clamp contact, uneven rock surfaces etc.), prestress effects and others must also be accounted for by the user.

The calculated capacity-blow count relationship, i.e. the bearing graph, should be used in conjunction with observed blow counts for the capacity assessment of a driven pile. Soil setup occurring after pile installation may produce bearing capacity values that differ substantially from those expected from a wave equation analysis due to soil setup or relaxation. This is particularly true for pile driven with vibratory hammers. The GRLWEAP user must estimate such effects and should also use proper care when applying blow counts from restrike because of the variability of hammer energy, soil resistance and blow count during early restriking.

Finally, the GRLWEAP capacities are ultimate values. They **MUST** be reduced by means of an appropriate factor of safety to yield a design or working load. The selection of a factor of safety should consider the quality of the construction control, the variability of the site conditions, uncertainties in the loads, the importance of structure and other factors.

SOIL PROFILE

Depth ft	Soil Type -	Spec. Wt lb/ft ³	Su ksf	Phi °	Unit Rs ksf	Unit Rt ksf
0.0	Sand	125.0	0.0	33.0	0.00	0.00
0.9	Sand	125.0	0.0	33.0	0.02	1.71
0.9	Sand	140.0	0.0	40.0	0.04	6.68
3.4	Sand	140.0	0.0	40.0	0.17	29.95
3.4	Sand	125.0	0.0	31.0	0.08	5.32
9.2	Sand	125.0	0.0	31.0	0.19	13.07
9.2	Clay	125.0	2.6	0.0	2.60	23.40
9.2	Clay	125.0	2.6	0.0	2.60	23.40
9.2	Clay	125.0	2.6	0.0	1.05	23.40
13.7	Clay	125.0	2.6	0.0	1.05	23.40
13.7	Clay	140.0	7.1	0.0	1.70	63.90
18.7	Clay	140.0	7.1	0.0	1.70	63.90
18.7	Clay	128.0	4.2	0.0	1.02	38.25
23.7	Clay	128.0	4.2	0.0	1.02	38.25

PILE INPUT

Uniform Pile		Pile Type:	H Pile
Pile Length: (ft)	23.690	Pile Penetration: (ft)	23.690
Pile Size: (ft)	0.84	Toe Area: (in ²)	97.73

Pile Profile

Lb Top ft	X-Area in ²	E-Modulus ksi	Spec. Wt lb/ft ³	Perim. ft	Crit. Index -
0.0	12.4	30,000.0	492.0	3.3	0
23.7	12.4	30,000.0	492.0	3.3	0

HAMMER INPUT

ID	41	Made By:	DELMAG
Model	D 19-42	Type:	OED

Hammer Data

ID -	Ram Wt kips	Ram L. in	Ram Ar. in ²	Rtd. Stk ft	Effic. -	Rtd. Energy kip-ft
41	4.000	129.1	124.7	10.8	0.80	43.2

DRIVE SYSTEM FOR DELMAG D 19-42-OED

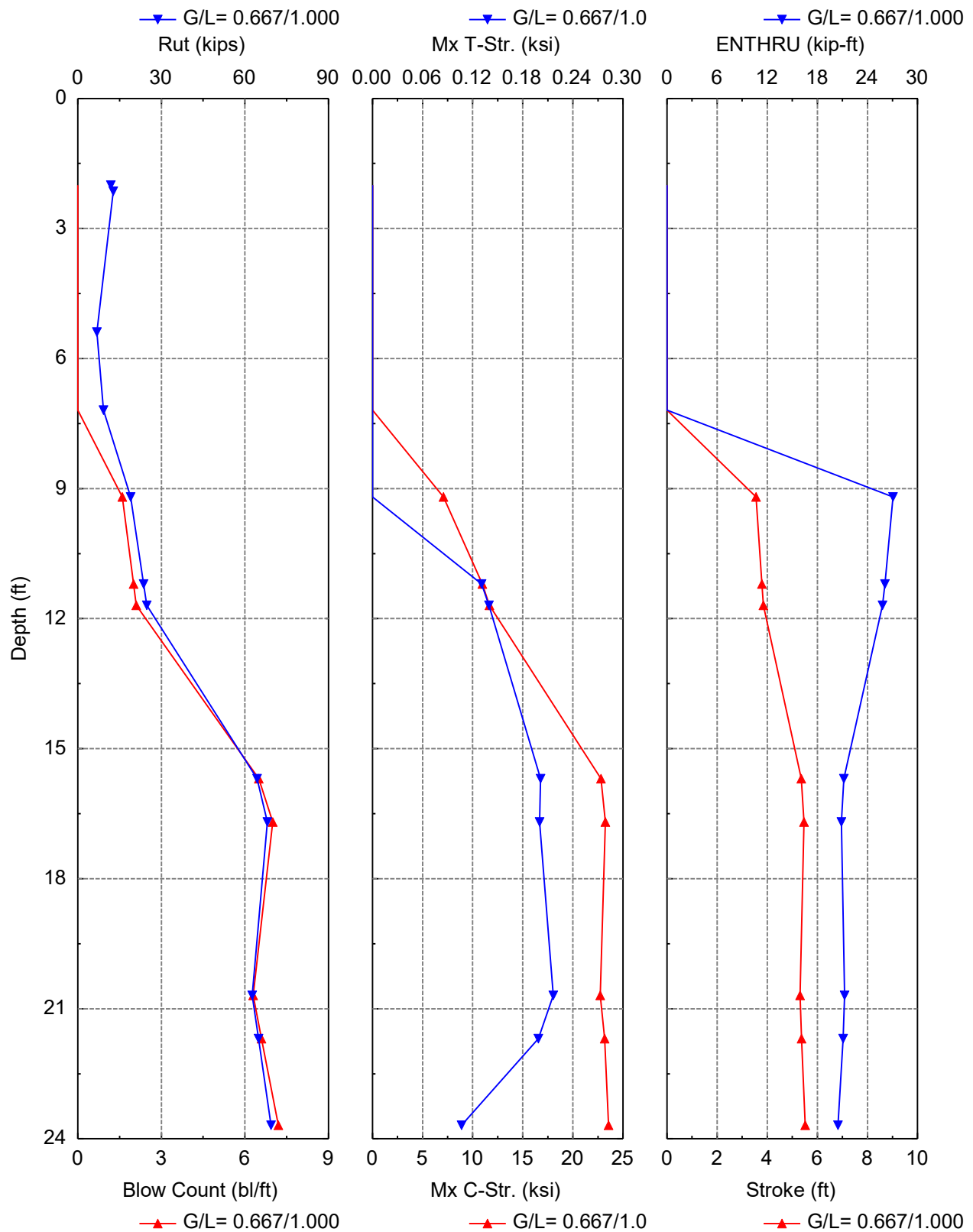
Type -	X-Area in ²	E-Modulus ksi	Thickness in	COR -	Round-out in	Stiffness kips/in
-----------	---------------------------	------------------	-----------------	----------	-----------------	----------------------

Rear Abut + B01_HP10x42		NATIONAL ENGINEERING AND ARCHITECTURAL				
Hammer C.	227.000	530.000	2.000	0.800	0.120	60155.555
Helmet Wt.	1.900	kins				

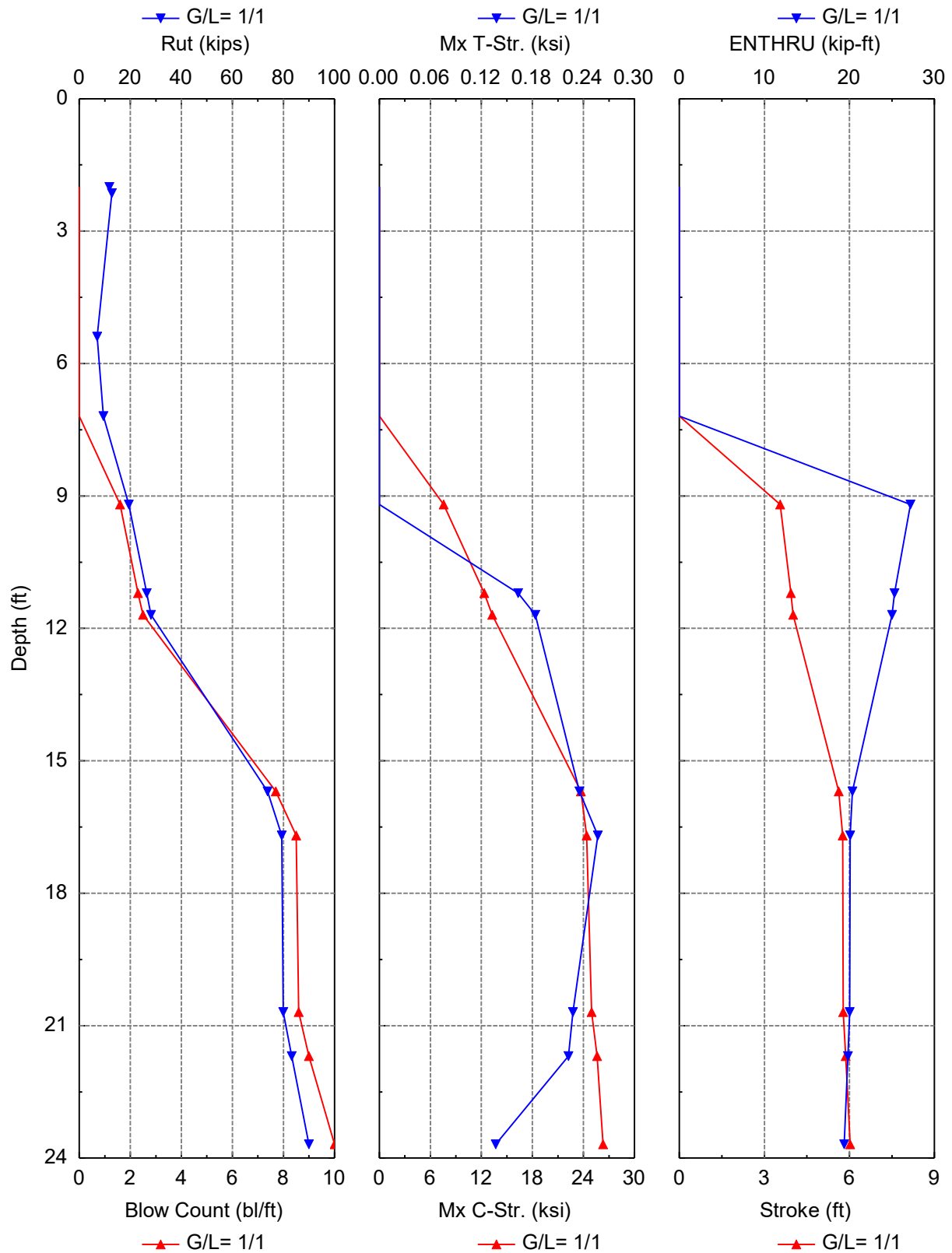
SOIL RESISTANCE DISTRIBUTION

Depth ft	Unit Rs ksf	Unit Rt ksf	Qs in	Qt in	Js s/ft	Jt s/ft	Set. F. -	Limit D. ft	Set. T. Hours	EB Area in ²
0.0	0.0	0.0	0.10	0.13	0.05	0.15	1.0	6.0	1.0	97.7
0.9	0.0	1.7	0.10	0.13	0.05	0.15	1.0	6.0	1.0	97.7
0.9	0.0	6.7	0.10	0.08	0.05	0.15	1.0	6.0	1.0	97.7
2.1	0.1	18.3	0.10	0.08	0.05	0.15	1.0	6.0	1.0	97.7
3.4	0.2	29.9	0.10	0.08	0.05	0.15	1.0	6.0	1.0	97.7
3.4	0.1	5.3	0.10	0.14	0.10	0.15	1.2	6.0	24.0	97.7
5.3	0.1	7.9	0.10	0.14	0.10	0.15	1.2	6.0	24.0	97.7
7.3	0.2	10.5	0.10	0.14	0.10	0.15	1.2	6.0	24.0	97.7
9.2	0.2	13.1	0.10	0.14	0.10	0.15	1.2	6.0	24.0	97.7
9.2	2.6	23.4	0.10	0.11	0.15	0.15	1.5	6.0	168.0	97.7
9.2	2.6	23.4	0.10	0.11	0.15	0.15	1.5	6.0	168.0	97.7
9.2	1.1	23.4	0.10	0.11	0.15	0.15	1.5	6.0	168.0	97.7
11.4	1.1	23.4	0.10	0.11	0.15	0.15	1.5	6.0	168.0	97.7
13.7	1.1	23.4	0.10	0.11	0.15	0.15	1.5	6.0	168.0	97.7
13.7	1.7	63.9	0.10	0.09	0.15	0.15	1.5	6.0	24.0	97.7
15.4	1.7	63.9	0.10	0.09	0.15	0.15	1.5	6.0	24.0	97.7
17.0	1.7	63.9	0.10	0.09	0.15	0.15	1.5	6.0	24.0	97.7
18.7	1.7	63.9	0.10	0.09	0.15	0.15	1.5	6.0	24.0	97.7
18.7	1.0	38.2	0.10	0.11	0.15	0.15	1.5	6.0	168.0	97.7
20.4	1.0	38.2	0.10	0.11	0.15	0.15	1.5	6.0	168.0	97.7
22.0	1.0	38.2	0.10	0.11	0.15	0.15	1.5	6.0	168.0	97.7
23.7	1.0	38.2	0.10	0.11	0.15	0.15	1.5	6.0	168.0	97.7

Driveability Analysis Summary



Driveability Analysis Summary



Gain/Loss Factor at Shaft/Toe = 0.667/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow bl/ft	CtMx ksi	C-StrMx ksi	T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
2.0	11.8	0.3	11.5	0.0	0.000	0.000	0.000	0.00	0.0	D 19-42
2.1	12.8	0.3	12.4	0.0	0.000	0.000	0.000	0.00	0.0	D 19-42
5.4	6.9	1.4	5.4	0.0	0.000	0.000	0.000	0.00	0.0	D 19-42
7.2	9.2	2.1	7.1	0.0	0.000	0.000	0.000	0.00	0.0	D 19-42
9.2	19.0	3.1	15.9	1.6	7.083	0.000	0.000	3.55	27.1	D 19-42
11.2	23.6	7.8	15.9	2.0	10.975	0.131	0.131	3.79	26.1	D 19-42
11.7	24.8	8.9	15.9	2.1	11.669	0.139	0.139	3.84	25.8	D 19-42
15.7	64.4	21.0	43.4	6.5	22.792	0.201	0.201	5.36	21.2	D 19-42
16.7	68.1	24.7	43.4	7.0	23.243	0.200	0.200	5.46	20.9	D 19-42
20.7	62.7	36.7	26.0	6.3	22.728	0.216	0.216	5.31	21.3	D 19-42
21.7	64.9	38.9	26.0	6.6	23.161	0.199	0.199	5.37	21.1	D 19-42
23.7	69.4	43.4	26.0	7.2	23.561	0.107	0.107	5.51	20.5	D 19-42

Total driving time: 1 minutes; Total Number of Blows: 77 (starting at penetration 2.0 ft)

Gain/Loss Factor at Shaft/Toe = 1.000/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow bl/ft	CtMx ksi	C-StrMx ksi	T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
2.0	11.8	0.3	11.5	0.0	0.000	0.000	0.000	0.00	0.0	D 19-42
2.1	12.8	0.3	12.4	0.0	0.000	0.000	0.000	0.00	0.0	D 19-42
5.4	7.0	1.6	5.4	0.0	0.000	0.000	0.000	0.00	0.0	D 19-42
7.2	9.4	2.4	7.1	0.0	0.000	0.000	0.000	0.00	0.0	D 19-42
9.2	19.4	3.6	15.9	1.6	7.571	0.000	0.000	3.56	27.2	D 19-42
11.2	26.4	10.5	15.9	2.3	12.353	0.163	0.163	3.94	25.3	D 19-42
11.7	28.1	12.2	15.9	2.5	13.269	0.183	0.183	4.01	25.0	D 19-42
15.7	73.7	30.4	43.4	7.7	23.708	0.235	0.235	5.63	20.4	D 19-42
16.7	79.4	36.0	43.4	8.5	24.378	0.257	0.257	5.77	20.1	D 19-42
20.7	79.9	54.0	26.0	8.6	24.974	0.228	0.228	5.78	20.0	D 19-42
21.7	83.3	57.3	26.0	9.0	25.599	0.222	0.222	5.86	19.8	D 19-42
23.7	90.0	64.0	26.0	10.0	26.294	0.137	0.137	6.02	19.4	D 19-42

Total driving time: 2 minutes; Total Number of Blows: 96 (starting at penetration 2.0 ft)

GRLWEAP: Wave Equation Analysis of Pile Foundations

Forw Abut + HP10X42_B02

9/15/2025

NATIONAL ENGINEERING AND ARCHITECTURAL

GRLWEAP 14.1.20.1

ABOUT THE WAVE EQUATION ANALYSIS RESULTS

The GRLWEAP program simulates the behavior of a preformed pile driven by either an impact hammer or a vibratory hammer. The program is based on mathematical models, which describe motion and forces of hammer, driving system, pile and soil under the hammer action. Under certain conditions, the models only crudely approximate, often complex, dynamic situations.

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Therefore, the program authors recommend prudent use of the GRLWEAP results. Soil response and hammer performance should be verified by static and/or dynamic testing and measurements. Estimates of bending or other local stresses (e.g., helmet or clamp contact, uneven rock surfaces etc.), prestress effects and others must also be accounted for by the user.

The calculated capacity-blow count relationship, i.e. the bearing graph, should be used in conjunction with observed blow counts for the capacity assessment of a driven pile. Soil setup occurring after pile installation may produce bearing capacity values that differ substantially from those expected from a wave equation analysis due to soil setup or relaxation. This is particularly true for pile driven with vibratory hammers. The GRLWEAP user must estimate such effects and should also use proper care when applying blow counts from restrike because of the variability of hammer energy, soil resistance and blow count during early restriking.

Finally, the GRLWEAP capacities are ultimate values. They **MUST** be reduced by means of an appropriate factor of safety to yield a design or working load. The selection of a factor of safety should consider the quality of the construction control, the variability of the site conditions, uncertainties in the loads, the importance of structure and other factors.

SOIL PROFILE

Depth ft	Soil Type -	Spec. Wt lb/ft ³	Su ksf	Phi °	Unit Rs ksf	Unit Rt ksf
0.0	Sand	122.0	0.0	32.0	0.00	0.00
1.7	Sand	122.0	0.0	32.0	0.04	2.59
1.7	Sand	122.0	0.0	34.0	0.04	3.77
4.2	Sand	122.0	0.0	34.0	0.10	9.29
4.2	Sand	118.0	0.0	31.0	0.08	5.35
9.2	Sand	118.0	0.0	31.0	0.17	11.28
9.2	Sand	120.0	0.0	31.0	0.17	11.28
17.0	Sand	120.0	0.0	31.0	0.31	20.71
17.0	Sand	125.0	0.0	32.0	0.34	24.76
27.0	Sand	125.0	0.0	32.0	0.55	33.09
27.0	Clay	125.0	3.4	0.0	3.40	30.60
27.0	Clay	125.0	3.4	0.0	3.40	30.60
27.0	Clay	125.0	3.4	0.0	0.95	30.60
42.0	Clay	125.0	3.4	0.0	0.95	30.60
42.0	Clay	128.0	4.1	0.0	0.98	36.90
49.2	Clay	128.0	4.1	0.0	0.98	36.90

PILE INPUT

Uniform Pile		Pile Type:	H Pile
Pile Length: (ft)	49.210	Pile Penetration: (ft)	49.210
Pile Size: (ft)	0.84	Toe Area: (in ²)	97.73

Pile Profile

Lb Top ft	X-Area in ²	E-Modulus ksi	Spec. Wt lb/ft ³	Perim. ft	Crit. Index -
0.0	12.4	30,000.0	492.0	3.3	0
49.2	12.4	30,000.0	492.0	3.3	0

HAMMER INPUT

ID	41	Made By:	DELMAG
Model	D 19-42	Type:	OED

Hammer Data

ID	Ram Wt kips	Ram L. in	Ram Ar. in ²	Rtd. Stk ft	Effic. -	Rtd. Energy kip-ft
-						
41	4.000	129.1	124.7	10.8	0.80	43.2

DRIVE SYSTEM FOR DELMAG D 19-42-OED

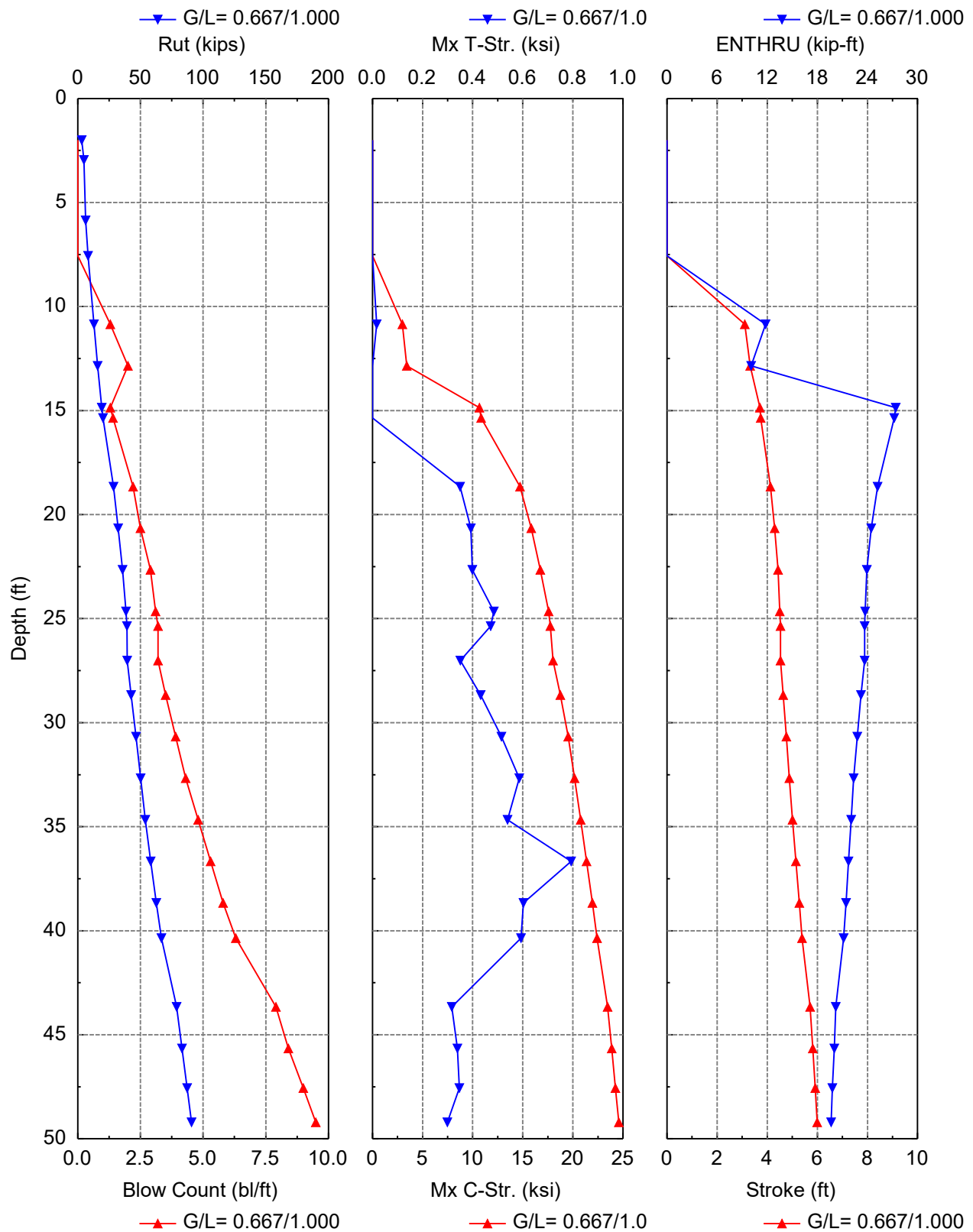
Type	X-Area	E-Modulus	Thickness	COR	Round-out	Stiffness
-	in ²	ksi	in	-	in	kips/in
Hammer C.	227.000	530.000	2.000	0.800	0.120	60155.555
Helmet Wt.	1.900	kips				

SOIL RESISTANCE DISTRIBUTION

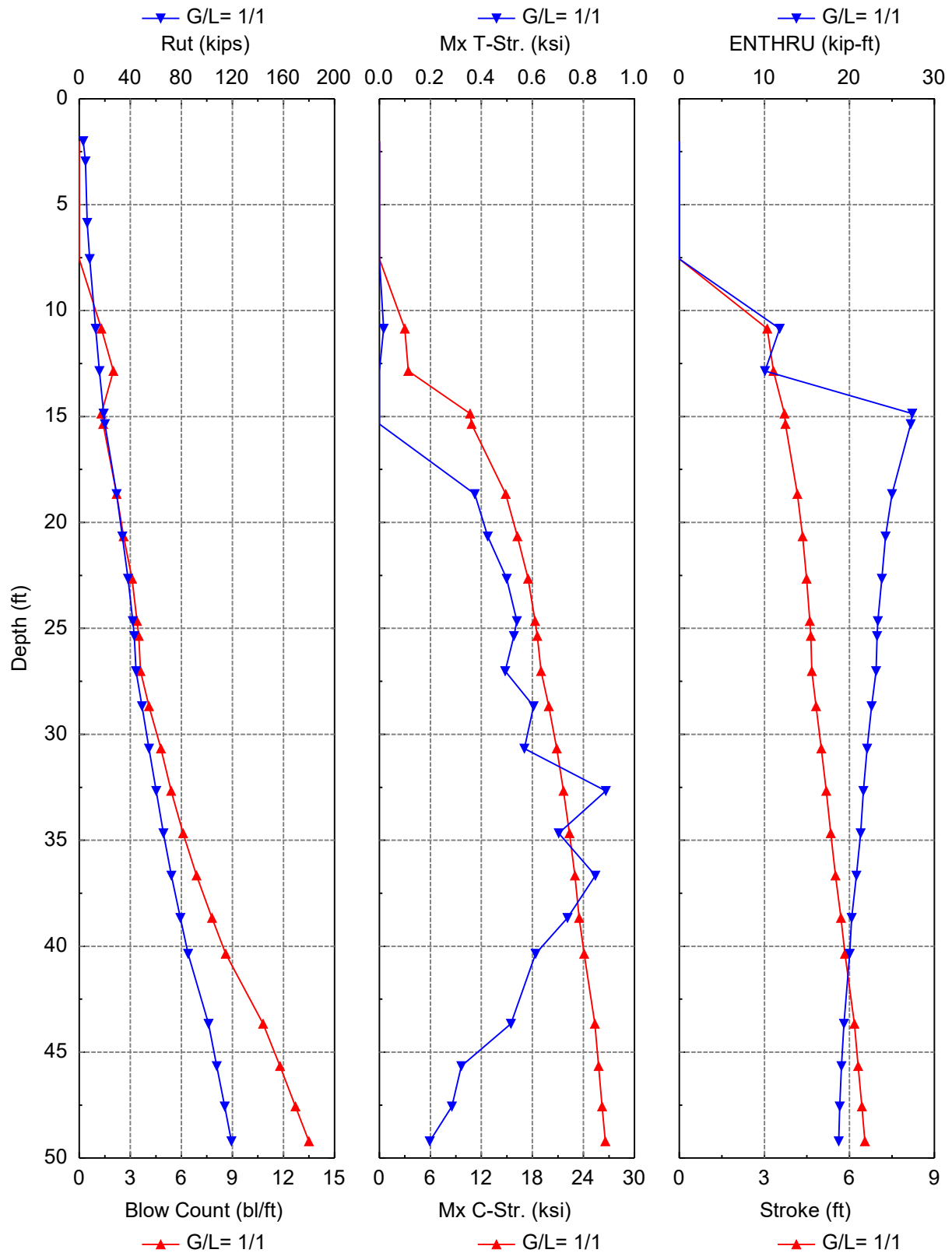
Depth	Unit Rs	Unit Rt	Qs	Qt	Js	Jt	Set. F.	Limit D.	Set. T.	EB Area
ft	ksf	ksf	in	in	s/ft	s/ft	-	ft	Hours	in ²
0.0	0.0	0.0	0.10	0.17	0.05	0.15	1.0	6.0	1.0	97.7
0.9	0.0	1.3	0.10	0.17	0.05	0.15	1.0	6.0	1.0	97.7
1.7	0.0	2.6	0.10	0.17	0.05	0.15	1.0	6.0	1.0	97.7
1.7	0.0	3.8	0.10	0.15	0.05	0.15	1.0	6.0	1.0	97.7
3.0	0.1	6.5	0.10	0.15	0.05	0.15	1.0	6.0	1.0	97.7
4.2	0.1	9.3	0.10	0.15	0.05	0.15	1.0	6.0	1.0	97.7
4.2	0.1	5.3	0.10	0.20	0.05	0.15	1.0	6.0	1.0	97.7
5.9	0.1	7.3	0.10	0.20	0.05	0.15	1.0	6.0	1.0	97.7
7.5	0.1	9.3	0.10	0.20	0.05	0.15	1.0	6.0	1.0	97.7
9.2	0.2	11.3	0.10	0.20	0.05	0.15	1.0	6.0	1.0	97.7
9.2	0.2	11.3	0.10	0.18	0.05	0.15	1.0	6.0	1.0	97.7
11.2	0.2	13.7	0.10	0.18	0.05	0.15	1.0	6.0	1.0	97.7
13.1	0.2	16.1	0.10	0.18	0.05	0.15	1.0	6.0	1.0	97.7
15.1	0.3	18.5	0.10	0.18	0.05	0.15	1.0	6.0	1.0	97.7
17.0	0.3	20.7	0.10	0.18	0.05	0.15	1.0	6.0	1.0	97.7
17.0	0.3	24.8	0.10	0.13	0.10	0.15	1.5	6.0	24.0	97.7
18.7	0.4	27.1	0.10	0.13	0.10	0.15	1.5	6.0	24.0	97.7
20.3	0.4	29.4	0.10	0.13	0.10	0.15	1.5	6.0	24.0	97.7
22.0	0.4	31.6	0.10	0.13	0.10	0.15	1.5	6.0	24.0	97.7
23.7	0.5	33.1	0.10	0.13	0.10	0.15	1.5	6.0	24.0	97.7
25.3	0.5	33.1	0.10	0.13	0.10	0.15	1.5	6.0	24.0	97.7
27.0	0.6	33.1	0.10	0.13	0.10	0.15	1.5	6.0	24.0	97.7
27.0	3.4	30.6	0.10	0.11	0.15	0.15	1.5	6.0	24.0	97.7
27.0	3.4	30.6	0.10	0.11	0.15	0.15	1.5	6.0	24.0	97.7
27.0	1.0	30.6	0.10	0.11	0.15	0.15	1.5	6.0	24.0	97.7
28.7	1.0	30.6	0.10	0.11	0.15	0.15	1.5	6.0	24.0	97.7
30.4	1.0	30.6	0.10	0.11	0.15	0.15	1.5	6.0	24.0	97.7
32.0	1.0	30.6	0.10	0.11	0.15	0.15	1.5	6.0	24.0	97.7
33.7	1.0	30.6	0.10	0.11	0.15	0.15	1.5	6.0	24.0	97.7
35.3	1.0	30.6	0.10	0.11	0.15	0.15	1.5	6.0	24.0	97.7
37.0	1.0	30.6	0.10	0.11	0.15	0.15	1.5	6.0	24.0	97.7
38.7	1.0	30.6	0.10	0.11	0.15	0.15	1.5	6.0	24.0	97.7
40.3	1.0	30.6	0.10	0.11	0.15	0.15	1.5	6.0	24.0	97.7

Forw Abut + HP10X42_B02				NATIONAL ENGINEERING AND ARCHITECTURAL						
42.0	1.0	30.6	0.10	0.11	0.15	0.15	1.5	6.0	24.0	97.7
42.0	1.0	36.9	0.10	0.10	0.15	0.15	1.5	6.0	168.0	97.7
43.8	1.0	36.9	0.10	0.10	0.15	0.15	1.5	6.0	168.0	97.7
45.6	1.0	36.9	0.10	0.10	0.15	0.15	1.5	6.0	168.0	97.7
47.4	1.0	36.9	0.10	0.10	0.15	0.15	1.5	6.0	168.0	97.7
49.2	1.0	36.9	0.10	0.10	0.15	0.15	1.5	6.0	168.0	97.7

Driveability Analysis Summary



Driveability Analysis Summary



Gain/Loss Factor at Shaft/Toe = 0.667/1.000

Depth	Rut	Rshaft	Rtoe	Blow	CtMx	C-StrMx	T-Str.	Stroke	ENTHRU	Hammer
ft	kips	kips	kips	bl/ft	ksi	ksi	ksi	ft	kip-ft	-
2.0	3.1	0.1	3.0	0.3	0.000	0.000		10.81	0.0	D 19-42
3.0	4.8	0.3	4.4	0.0	0.000	0.000		0.00	0.0	D 19-42
5.9	6.2	1.2	5.0	0.0	0.000	0.000		0.00	0.0	D 19-42
7.6	8.2	1.9	6.3	0.0	0.000	0.000		0.00	0.0	D 19-42
10.9	12.7	3.7	9.0	1.3	2.990	0.017		3.10	11.8	D 19-42
12.9	15.8	5.1	10.7	2.0	3.419	0.000		3.32	10.1	D 19-42
14.9	19.2	6.8	12.4	1.3	10.673	0.000		3.71	27.4	D 19-42
15.4	20.0	7.3	12.8	1.4	10.844	0.000		3.74	27.2	D 19-42
18.7	28.5	10.2	18.4	2.2	14.737	0.350		4.13	25.2	D 19-42
20.7	32.1	11.9	20.2	2.5	15.839	0.392		4.29	24.5	D 19-42
22.7	35.7	13.8	21.8	2.9	16.739	0.398		4.43	23.9	D 19-42
24.7	38.4	15.9	22.5	3.1	17.575	0.484		4.50	23.7	D 19-42
25.4	39.2	16.7	22.5	3.2	17.762	0.472		4.52	23.7	D 19-42
27.0	39.5	18.7	20.8	3.2	18.007	0.351		4.53	23.7	D 19-42
28.7	42.6	21.8	20.8	3.5	18.755	0.432		4.64	23.2	D 19-42
30.7	46.4	25.6	20.8	3.9	19.532	0.515		4.76	22.8	D 19-42
32.7	50.1	29.3	20.8	4.3	20.171	0.586		4.88	22.4	D 19-42
34.7	53.9	33.1	20.8	4.8	20.768	0.539		5.01	22.0	D 19-42
36.7	58.1	37.3	20.8	5.3	21.365	0.793		5.14	21.7	D 19-42
38.7	62.7	41.9	20.8	5.8	21.958	0.603		5.28	21.4	D 19-42
40.4	66.8	46.0	20.8	6.3	22.412	0.593		5.39	21.2	D 19-42
43.7	78.8	53.8	25.0	7.9	23.455	0.317		5.71	20.2	D 19-42
45.7	83.1	58.1	25.0	8.4	23.866	0.339		5.82	20.0	D 19-42
47.6	87.2	62.2	25.0	9.0	24.242	0.347		5.91	19.8	D 19-42
49.2	90.8	65.8	25.0	9.5	24.574	0.299		5.99	19.7	D 19-42

Total driving time: 3 minutes; Total Number of Blows: 172 (starting at penetration 2.0 ft)

Gain/Loss Factor at Shaft/Toe = 1.000/1.000

Depth	Rut	Rshaft	Rtoe	Blow	CtMx	C-StrMx	T-Str.	Stroke	ENTHRU	Hammer
ft	kips	kips	kips	bl/ft	ksi	ksi	ksi	ft	kip-ft	-
2.0	3.1	0.1	3.0	0.3	0.000	0.000	0.000	10.81	0.0	D 19-42
3.0	4.8	0.3	4.4	0.0	0.000	0.000	0.000	0.00	0.0	D 19-42
5.9	6.2	1.2	5.0	0.0	0.000	0.000	0.000	0.00	0.0	D 19-42
7.6	8.2	1.9	6.3	0.0	0.000	0.000	0.000	0.00	0.0	D 19-42
10.9	12.7	3.7	9.0	1.3	2.990	0.017	0.017	3.10	11.8	D 19-42
12.9	15.8	5.1	10.7	2.0	3.419	0.000	0.000	3.32	10.1	D 19-42
14.9	19.2	6.8	12.4	1.3	10.673	0.000	0.000	3.71	27.4	D 19-42

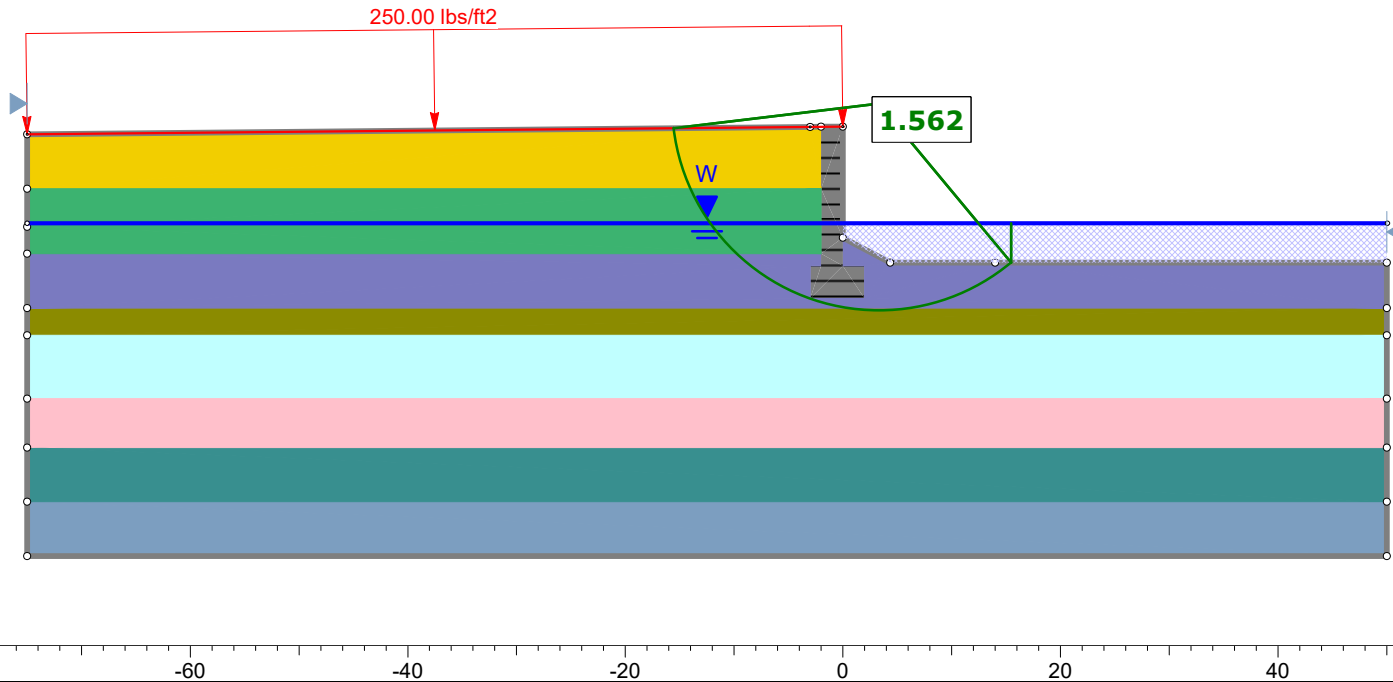
Forw Abut + HP10X42_B02				NATIONAL ENGINEERING AND ARCHITECTURAL					
15.4	20.0	7.3	12.8	1.4	10.844	0.000	3.74	27.2	D 19-42
18.7	29.2	10.8	18.4	2.2	14.846	0.374	4.16	25.0	D 19-42
20.7	33.6	13.4	20.2	2.6	16.229	0.425	4.35	24.2	D 19-42
22.7	38.1	16.3	21.8	3.1	17.491	0.499	4.49	23.8	D 19-42
24.7	41.9	19.5	22.5	3.4	18.328	0.539	4.60	23.3	D 19-42
25.4	43.1	20.7	22.5	3.5	18.574	0.528	4.63	23.3	D 19-42
27.0	44.4	23.6	20.8	3.6	19.002	0.493	4.67	23.1	D 19-42
28.7	49.1	28.3	20.8	4.1	19.935	0.605	4.82	22.6	D 19-42
30.7	54.7	33.9	20.8	4.8	20.869	0.568	5.00	22.1	D 19-42
32.7	60.3	39.6	20.8	5.4	21.659	0.888	5.18	21.6	D 19-42
34.7	66.0	45.2	20.8	6.1	22.345	0.703	5.34	21.3	D 19-42
36.7	72.3	51.5	20.8	6.9	22.996	0.847	5.51	20.8	D 19-42
38.7	79.2	58.4	20.8	7.8	23.501	0.737	5.70	20.3	D 19-42
40.4	85.3	64.6	20.8	8.6	24.090	0.613	5.84	20.0	D 19-42
43.7	101.2	76.2	25.0	10.8	25.348	0.515	6.18	19.4	D 19-42
45.7	107.7	82.7	25.0	11.8	25.770	0.321	6.31	19.1	D 19-42
47.6	113.9	88.9	25.0	12.7	26.183	0.284	6.44	18.9	D 19-42
49.2	119.2	94.2	25.0	13.5	26.559	0.198	6.54	18.7	D 19-42

Total driving time: 5 minutes; Total Number of Blows: 219 (starting at penetration 2.0 ft)

GLOBAL STABILITY ANALYSIS

Material Name	Color	Unit Weight (lbs/ft3)	Saturated U.W. (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Material 1		115	125	Mohr-Coulomb	0	40
Material 2		115	125	Mohr-Coulomb	0	35
Material 4		115	125	Mohr-Coulomb	0	33
Material 5		130	140	Mohr-Coulomb	0	40
Material 6		115	125	Mohr-Coulomb	0	31
Material 7		115	125	Mohr-Coulomb	250	26
Material 8		130	140	Mohr-Coulomb	450	30
Material 9		118	128	Mohr-Coulomb	300	26
Abutment		150	150	Infinite Strength		

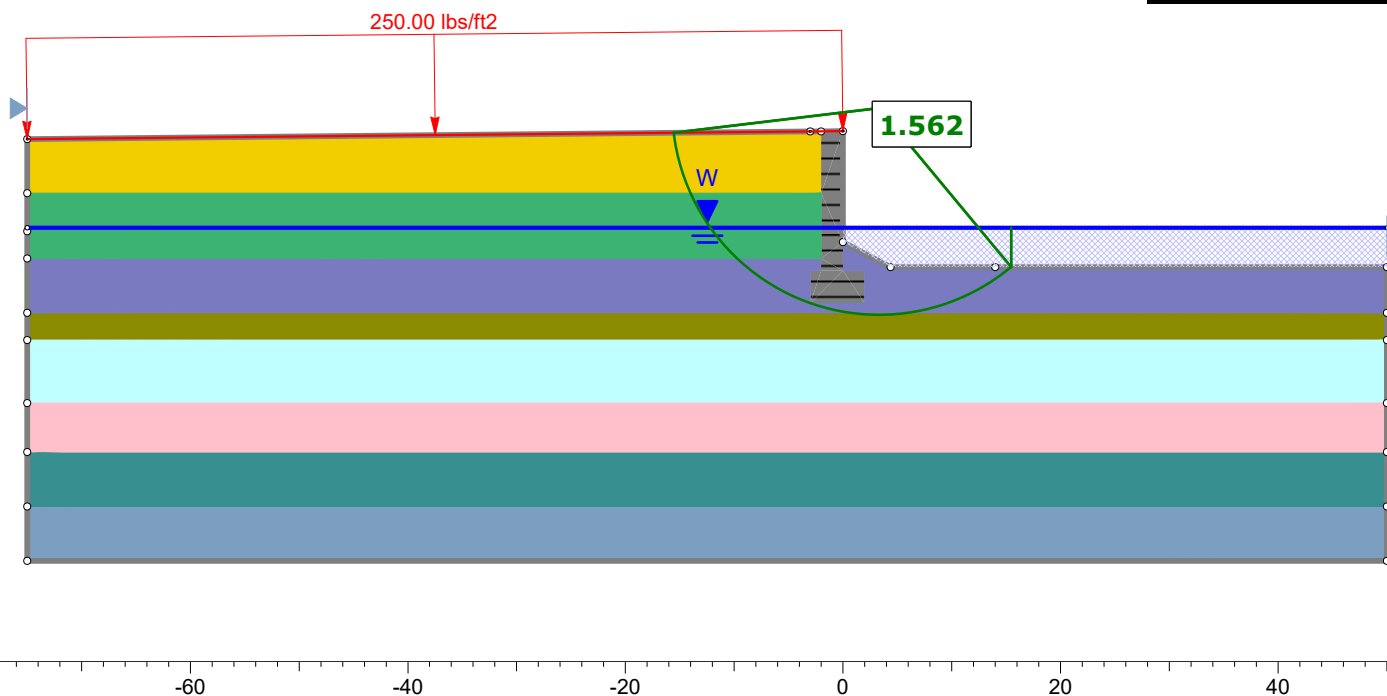
Method Name	Min FS
Bishop simplified	1.562
Spencer	1.583
GLE / Morgenstern-Price	1.591



Project	TUS-Nighbor Street Bridge	
Group	Group 1	Scenario Master Scenario
Drawn By	ZM	Company NEAS, Inc.
Date	3/3/2025	File Name TUS-Nighbor St Bridge_RB_Effective_Normal Water_B01.slmd

Material Name	Color	Unit Weight (lbs/ft3)	Saturated U.W. (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Material 1		115	125	Mohr-Coulomb	0	40
Material 2		115	125	Mohr-Coulomb	0	35
Material 4		115	125	Mohr-Coulomb	0	33
Material 5		130	140	Mohr-Coulomb	0	40
Material 6		115	125	Mohr-Coulomb	0	31
Material 7		115	125	Mohr-Coulomb	2600	0
Material 8		130	140	Mohr-Coulomb	7100	0
Material 9		118	128	Mohr-Coulomb	4250	0
Abutment		150	150	Infinite Strength		

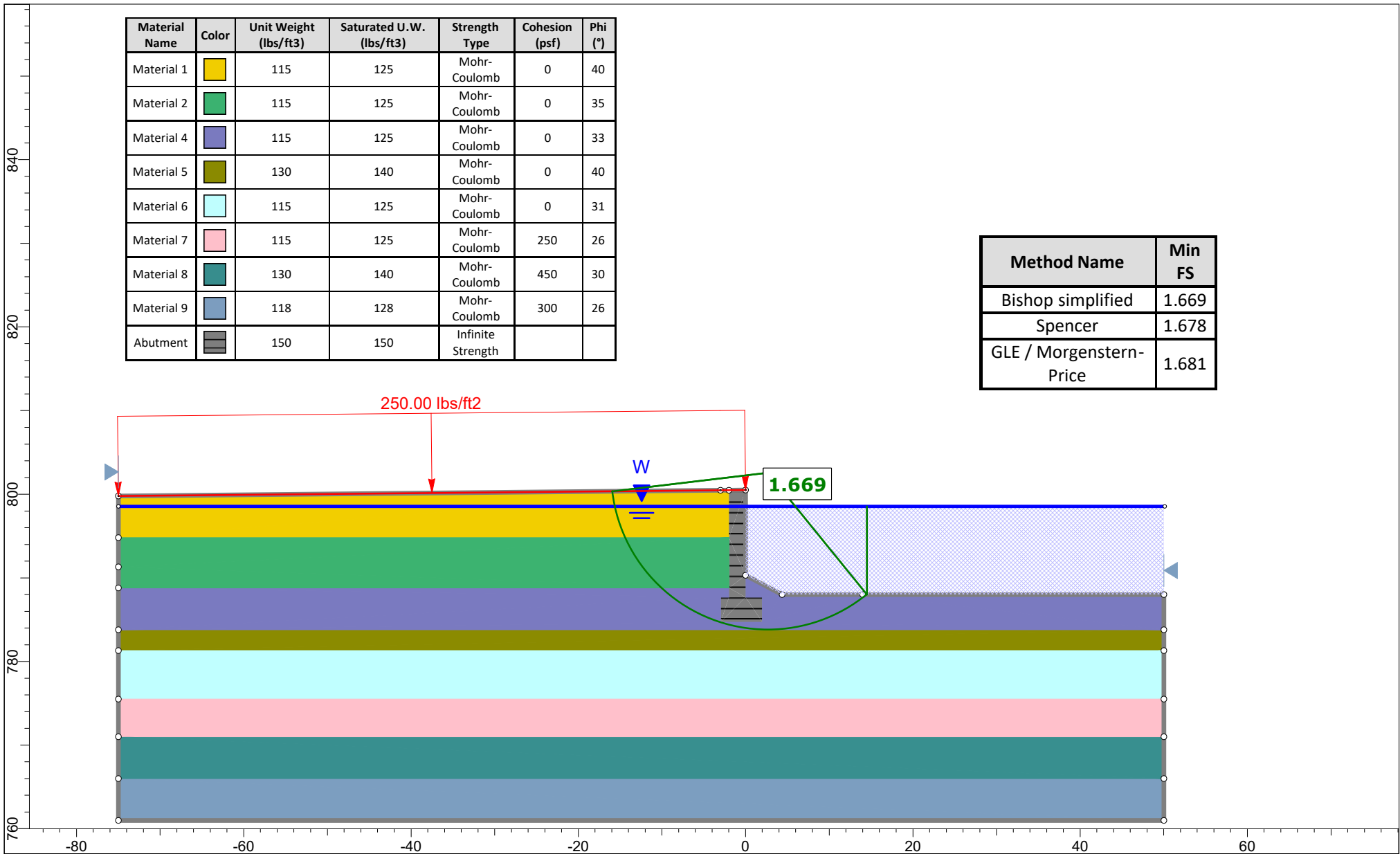
Method Name	Min FS
Bishop simplified	1.562
Spencer	1.583
GLE / Morgenstern-Price	1.591



Project	TUS-Nighbor Street Bridge		
Group	Group 1	Scenario	Master Scenario
Drawn By	ZM	Company	NEAS, Inc.
Date	3/3/2025	File Name	TUS-Nighbor St Bridge_RB_Total_Normal Water_B01.slmd

Material Name	Color	Unit Weight (lbs/ft3)	Saturated U.W. (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Material 1		115	125	Mohr-Coulomb	0	40
Material 2		115	125	Mohr-Coulomb	0	35
Material 4		115	125	Mohr-Coulomb	0	33
Material 5		130	140	Mohr-Coulomb	0	40
Material 6		115	125	Mohr-Coulomb	0	31
Material 7		115	125	Mohr-Coulomb	250	26
Material 8		130	140	Mohr-Coulomb	450	30
Material 9		118	128	Mohr-Coulomb	300	26
Abutment		150	150	Infinite Strength		

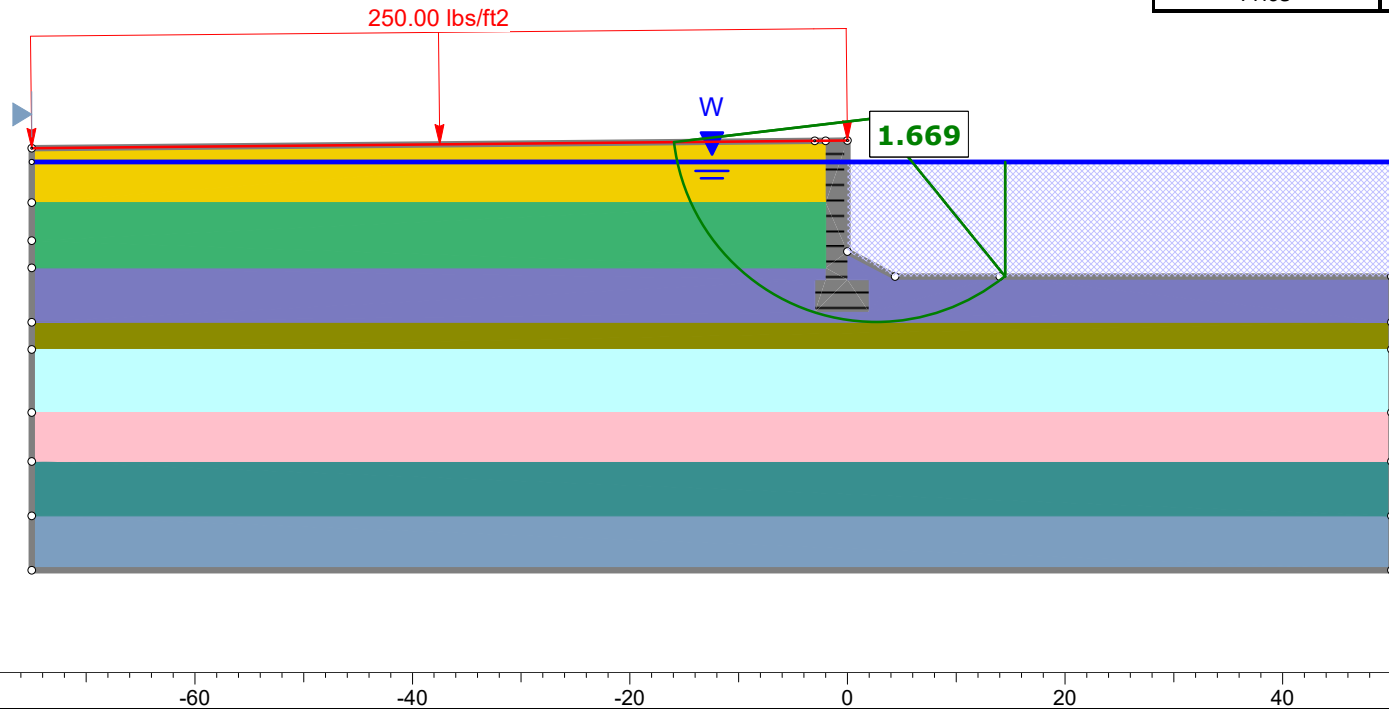
Method Name	Min FS
Bishop simplified	1.669
Spencer	1.678
GLE / Morgenstern-Price	1.681



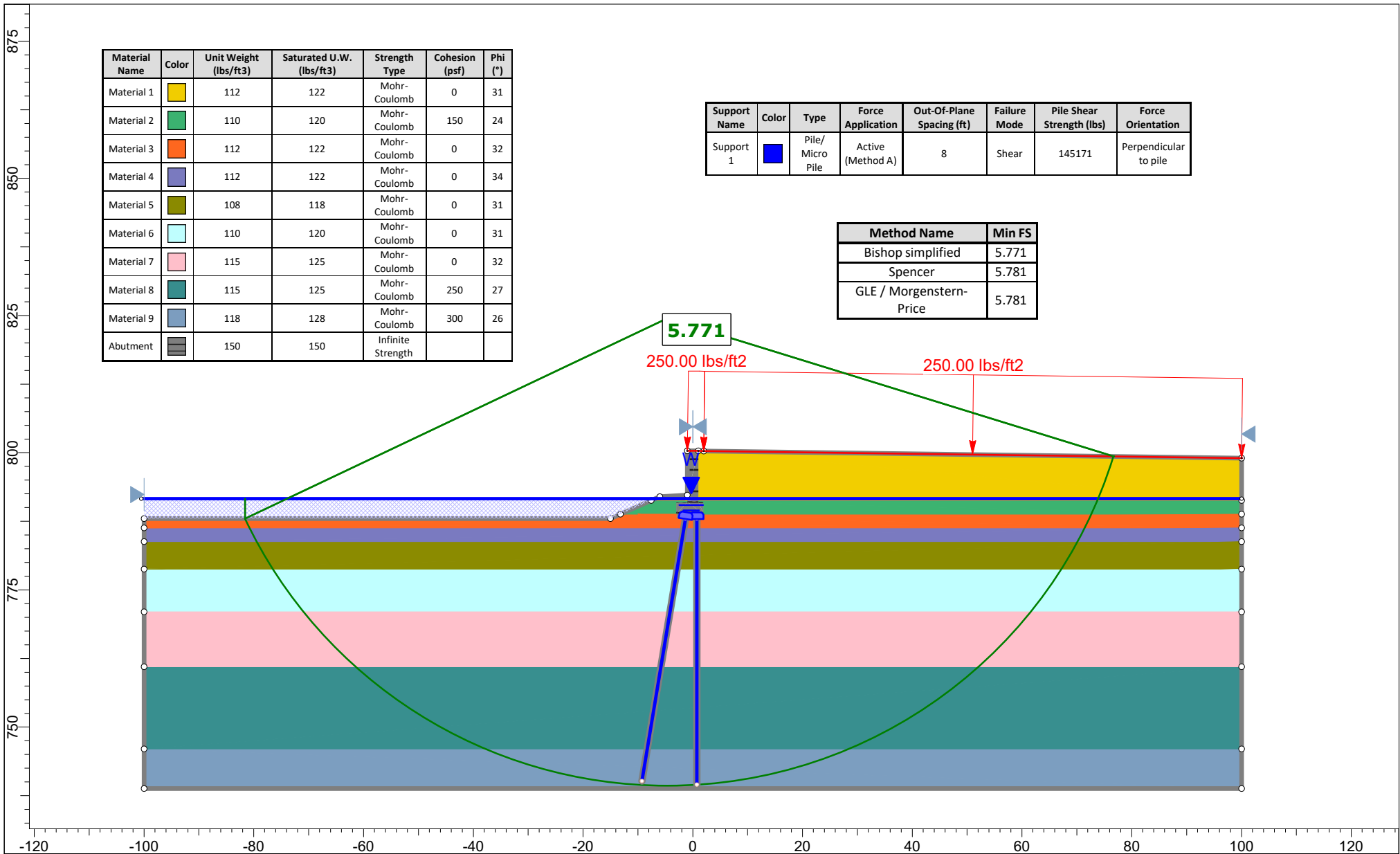
	Project		TUS-Nighbor Street Bridge	
	Group	Group 1	Scenario	Master Scenario
	Drawn By	ZM	Company	NEAS, Inc.
	Date	3/3/2025	File Name	TUS-Nighbor St Bridge_RB_Effective_HW100_B01.slmd

Material Name	Color	Unit Weight (lbs/ft ³)	Saturated U.W. (lbs/ft ³)	Strength Type	Cohesion (psf)	Phi (°)
Material 1		115	125	Mohr-Coulomb	0	40
Material 2		115	125	Mohr-Coulomb	0	35
Material 4		115	125	Mohr-Coulomb	0	33
Material 5		130	140	Mohr-Coulomb	0	40
Material 6		115	125	Mohr-Coulomb	0	31
Material 7		115	125	Mohr-Coulomb	2600	0
Material 8		130	140	Mohr-Coulomb	7100	0
Material 9		118	128	Mohr-Coulomb	4250	0
Abutment		150	150	Infinite Strength		

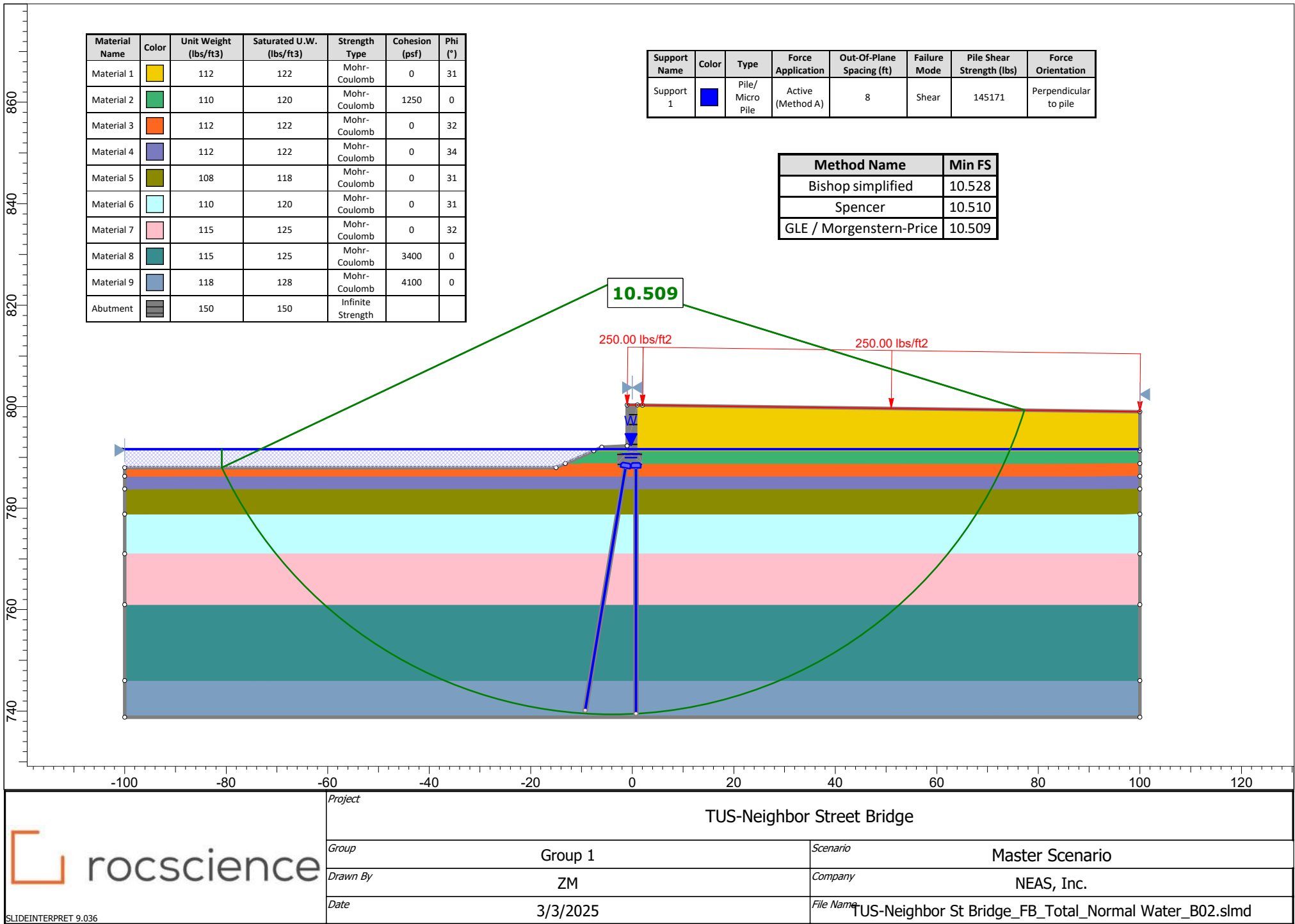
Method Name	Min FS
Bishop simplified	1.669
Spencer	1.678
GLE / Morgenstern-Price	1.681



Project	TUS-Nighbor Street Bridge		
Group	Group 1	Scenario	Master Scenario
Drawn By	ZM	Company	NEAS, Inc.
Date	3/3/2025	File Name	TUS-Nighbor St Bridge_RB_Total_HW100_B01.slmd



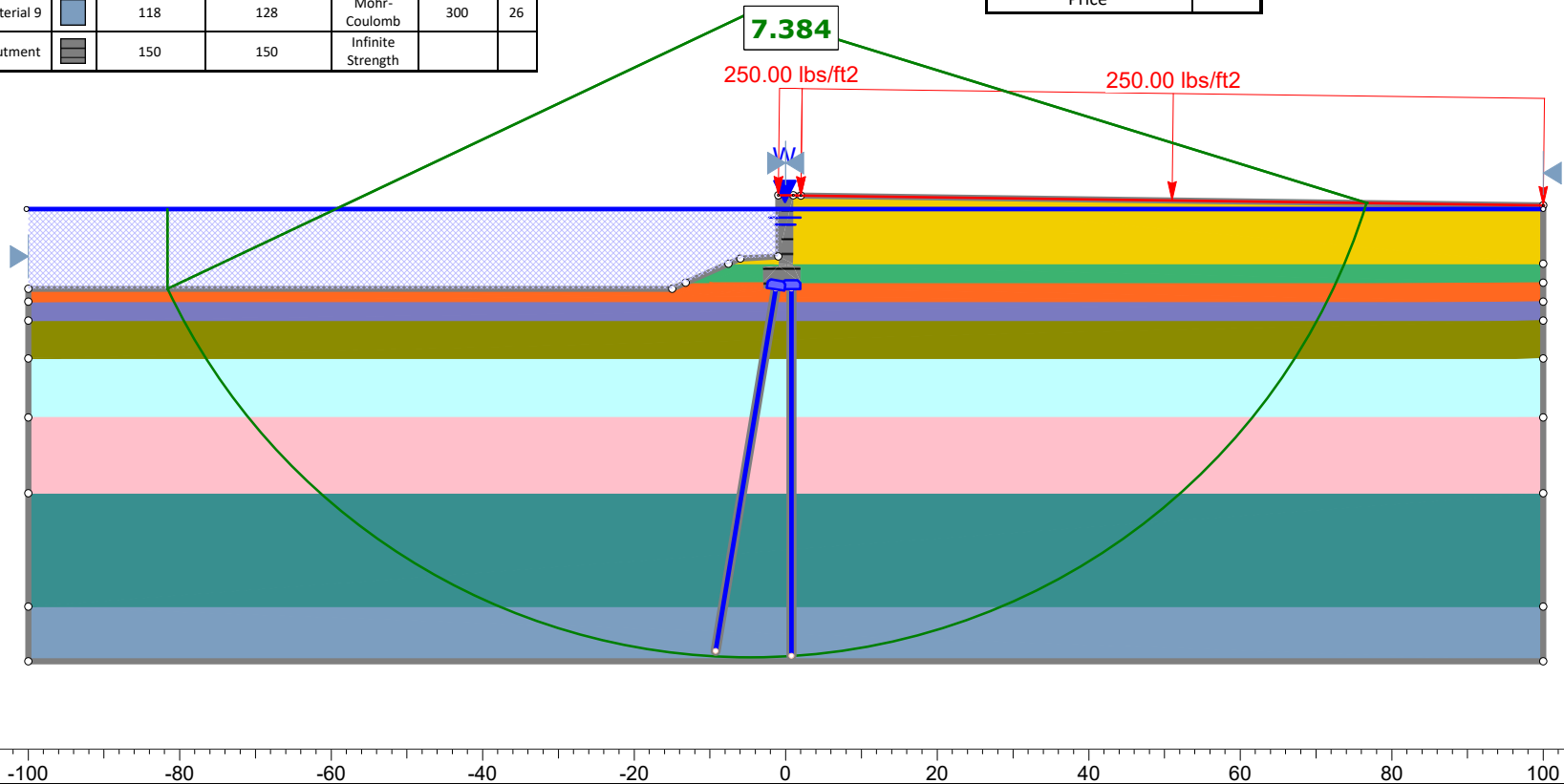
	Project		TUS-Neighbor Street Bridge	
	Group	Group 1	Scenario	Master Scenario
	Drawn By	ZM	Company	NEAS, Inc.
	Date	3/3/2025	File Name	TUS-Neighbor St Bridge_FB_Effective_Normal Water_B02.slmd
	SLIDEINTERPRET 9.036			



Material Name	Color	Unit Weight (lbs/ft3)	Saturated U.W. (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Material 1		112	122	Mohr-Coulomb	0	31
Material 2		110	120	Mohr-Coulomb	150	24
Material 3		112	122	Mohr-Coulomb	0	32
Material 4		112	122	Mohr-Coulomb	0	34
Material 5		108	118	Mohr-Coulomb	0	31
Material 6		110	120	Mohr-Coulomb	0	31
Material 7		115	125	Mohr-Coulomb	0	32
Material 8		115	125	Mohr-Coulomb	250	27
Material 9		118	128	Mohr-Coulomb	300	26
Abutment		150	150	Infinite Strength		

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation
Support 1		Pile/Micro Pile	Active (Method A)	8	Shear	145171	Perpendicular to pile

Method Name	Min FS
Bishop simplified	7.384
Spencer	7.390
GLE / Morgenstern-Price	7.388



Project		TUS-Nighbor Street Bridge	
Group	Group 1	Scenario	Master Scenario
Drawn By	ZM	Company	NEAS, Inc.
Date	3/3/2025	File Name	TUS-Nighbor St Bridge_FB_Effective_HW100_B02.slmd

