HAS-Bridge Street Bridge Replacement (PID 120494)

Structure Foundation Exploration Report

Bowerston, Harrison County, Ohio

January 28, 2025 | Terracon Project No. N4235509

Prepared for:

EMH&T Engineers, Surveyors, Planners, Scientists, Inc.
5500 New Albany Road
Columbus, Ohio 43054

Terracon
Explore with us

Nationwide
Terracon.com

- Facilities
- Environmental
- Geotechnical
- Materials

Terracon

800 Morrison Road
Columbus, OH 43230
P (614) 863-3113
Terracon.com

January 28, 2025

EMH&T Engineers, Surveyors, Planners, Scientists, Inc.
5500 New Albany Road
Columbus, Ohio 43054

Attn: Mr. Tyler Adams – Senior Bridge Engineer
P: (614) 775-4602
E: Tadams@emht.com

Re: Structure Foundation Exploration Report
HAS-Bridge Street Bridge Replacement (PID 120494)
Bowerston, Harrison County, Ohio
Terracon Project No. N4235509

Dear Mr. Adams:

Terracon Consultants, Inc. (Terracon) has completed the scope of Geotechnical Engineering Services for the above reference in general accordance with Terracon Proposal No. PN4235509 dated December 5, 2023.

This report presents the findings of the subsurface exploration, laboratory testing results, and the results of our foundation analyses performed for the proposed replacement of the existing Bridge Street bridge located at Harrison County, Ohio.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report or if we may be of further service, please contact us.

Sincerely,

Terracon

Ahmad Al-Hosainat

Ahmad Al-Hosainat, Ph.D.
Senior Staff Engineer

Kevin M. Ernst, P.E.
Principal, Regional Manager

STATE OF OHIO
KEVIN M. ERNST
E-56245
REGISTERED
PROFESSIONAL ENGINEER
01/28/2025

Table of Contents

Appendices

Note: This report was originally delivered in a web-based format. **Blue Bold** text in the report indicates a referenced section heading. The PDF version also includes hyperlinks which direct the reader to that section and clicking on the Terracon logo will bring you back to this page. For more interactive features, please view your project online at **client.terracon.com**. Refer to each individual Attachment for a listing of contents.

Geotechnical Engineering Services Report

HAS-Bridge Street Bridge Replacement (PID 120494)
Bowerston, Harrison County, Ohio
Terracon Project No. N4235509
January 28, 2025

Executive Summary

This report presents the findings of our geotechnical exploration performed for the proposed replacement of the existing bridge located along Bridge Street in Harrison County, Ohio over the Conotton Creek. The bridge is located approximately 170 feet north of the intersection of Bridge Street and Main Street. The existing structure is a single-span, non-composite prestressed concrete box beams on concrete and sandstone abutments bridge with a maximum span of approximately 72 feet center to center of bearings along with sidewalk and steel posts with w-beam rail on structure only (no Bridge Terminal Assembly (BTAs) or guardrail beyond the bridge). The proposed replacement structure is anticipated to include new foundation elements, abutments, and concrete composite deck. The horizontal and vertical alignments will closely replicate the existing alignments.

Terracon performed two (2) borings, designated as Borings B-001-0-24 and B-002-0-24 at the north and south abutments of the Bridge Street bridge to approximate depths of 50 to 60 feet below the existing ground surface, respectively. The borings B-001-0-24 and B-002-0-24 encountered a surficial layer consisting of topsoil approximately 8 inches thick. The boring B-002-0-24 encountered fill material to depth of about 3.5 feet below the existing ground surface. The fill materials consisted of cohesive soil described as silt and clay (A-6a).

Below the surficial layer/existing fill, the native cohesive soils encountered in the borings included very soft to hard, silt and clay (A-6a) and silty clay (A-6b). The native granular soils encountered in the borings included very loose to dense, sandy silt (A-4a), and coarse and fine sand (A-3a). Bedrock was encountered in borings B-001-0-24, and B-002-0-24 at a depth varying from about 38.5 to 50 feet, which corresponds to elevations varying from about EL 901.3 to EL 893.7 feet. The bedrock encountered in the borings consisted of moderately weathered shale, and sandstone.

In boring B-001-0-24 groundwater was encountered at a depth of 16 feet below ground surface during drilling and was not encountered upon completion of drilling. In boring B-002-0-24 groundwater was encountered at a depth of 8.5 feet below ground surface during drilling and was not encountered upon completion of drilling.

Based on the subsurface conditions encountered at the site, and the requirements outlined in sections 305.3 and 305.4 of ODOT Bridge Design Manual (BDM), it is recommended that a deep foundation system consisting of steel H-piles driven to refusal into bedrock or drilled shafts be employed for support of the proposed bridge foundation elements. The recommended pile tip elevation and the corresponding ultimate bearing values (P_r), for each abutment are presented in the report. In addition, the estimated top of rock socket elevations and the corresponding unfactored nominal tip and side resistance for rock socketed drilled shafts at each abutment are presented in this report.

The embankments at the bridge abutments slope down towards Conotton Creek at slope inclinations of about 2 Horizontal (H) to 1 Vertical (V) to 4H to 1V. Additional evaluation including slope stability analyses would be required to determine stability of the embankments. Once the final plan and profile drawings for the bridge are available, we would be able to perform this evaluation, however this would involve additional costs.

This summary should be used in conjunction with the entire report for design purposes. It should be recognized that details were not included or fully developed in this section, and the report must be read in its entirety for a comprehensive understanding of the items contained herein. The section titled **General Comments** should be read for an understanding of the report limitations.

Introduction

A structure foundation exploration has been completed for the proposed replacement of the existing bridge located along Bridge Street over Conotton Creek in the village of Bowerston, Harrison County, Ohio. The bridge is located approximately 170 feet north of the intersection of Bridge Street and Main Street. The existing structure is a single-span, non-composite prestressed concrete box beams on concrete and sandstone abutments bridge with a maximum span of approximately 72 feet center to center of bearings along with sidewalk and steel posts with w-beam rail on structure only (no BTAs or guardrail beyond the bridge).

At the time of writing this report, it is our understanding that the proposed replacement structure is anticipated to include new foundation element, and abutments. In addition, the horizontal and vertical alignments will closely replicate the existing alignments.

Site Location and Description

The following description of site conditions is derived from our site visits in association with the field exploration and our review of publicly available geologic and topographic maps.

Item	Description
Location	The project site is located along Bridge Street in the village of Bowerston, Harrison County, Ohio at the existing bridge over the Conotton Creek. The bridge is located approximately 170 feet north of the intersection of Bridge Street and Main Street. The approximate latitude/longitude coordinates of the site are 40.42568, -81.18719. See **Site Location**
Existing Improvements	The existing structure is a single-span, non-composite prestressed concrete box beams on concrete and sandstone abutments bridge and passes over Conotton Creek, along with sidewalk and steel posts with w-beam rail on structure only (no BTAs or guardrail beyond the bridge).
Existing Topography	Based on our site reconnaissance, surface elevations of the bridge at the north and south abutments are approximately 939 and 944 feet above Mean Sea Level (MSL), respectively.

Project Description

Item	Description
Site Layout	We understand that the new structure will maintain the existing horizontal and vertical alignments. A final plan and profile drawing for the proposed bridge is not available at the time of preparation of the report.
Proposed Construction	It is our understanding that the proposed structure is a single-span precast concrete box beam with composite reinforced concrete deck and reinforced concrete substructure and that the new structure will maintain the existing horizontal and vertical alignments. The proposed replacement structure is anticipated to include new foundation elements, abutments, and deck. The new abutments are planned to be supported on driven piles or drilled shafts.
Grading	A grading plan is currently not available at the time of this report.

We would like the opportunity to review our recommendations and make modifications if required, once plan and profile drawings of the proposed bridge are available. We have assumed for the purposes of this report that the scour analyses will be performed by EMH&T and that protective measures will be provided in the design to mitigate erosion and global slope stability issues at the bridge abutments. However, once the final plan and profile drawings are developed, slope stability analyses should be performed to verify the global factor of safety of the abutment slopes.

Reconnaissance

At the time of our site reconnaissance visit on February 9, 2024, the existing Bridge Street was observed to be a two-lane, asphaltic concrete paved roadway aligned in a northeast to southwest orientation. The bridge consists of a non-composite prestressed concrete box beams structure on concrete and sandstone abutments. The Conotton Creek was observed to be a relatively small, low flow waterway with a general flow direction towards the west at the subject structure. Based on provided preliminary plan and profile of the bridge, the side slopes of the abutment embankment appear to range from 2H:1V to 4H:1V.

General Geology

Based on the Ohio Department of Natural Resources (ODNR) Quaternary Geology Map of Ohio, the project site is mapped within the Muskingum-Pittsburgh Plateau region within the Allegheny Plateaus section of the Appalachian Plateaus physiographic province of Ohio. This region is characterized as a dissected moderate to high relief plateau having broad major valleys that contain outwash terraces, and tributaries with lacustrine terraces; medium-grained bedrock sequences coarser than those in Marietta Plateau but finer than those in Ironton Plateau; remnants of ancient Teays-age drainage system uncommon. The bedrock geology consists of Pennsylvanian-aged Allegheny and Pottsville groups undivided, consisting of shale, siltstone, sandstone, conglomerate, and lesser amounts of limestone, clay, flint, and coal.

Exploration

Field Exploration

A total of two (2) borings, designated as B-001-0-24 and B-002-0-24 were performed at the north and south abutments of the bridge on February 19, 2024, to depths of approximately 50 and 60 feet below the existing ground surface, respectively.

The borings were performed in general accordance with Section 303.3 of the Ohio Department of Transportation (ODOT) Specifications for Geotechnical Explorations (SGE) Type E1 bridge borings.

The approximate locations of the borings are illustrated on the attached **Exploration Plan** and summarized in the following table.

Boring ID	Elevation[1] (feet)	Latitude[1]	Longitude[1]	Boring Total Depth (feet) [2]	Top of Rock Elevation (feet)	Top of Rock Depth (feet) [2]
B-001-0-24	939.8	40.42592	-81.18712	50.0	901.3	38.5
B-002-0-24	943.7	40.42563	-81.18726	60.0	893.7	50.0

1. The boring coordinates were obtained using a handheld GPS. Top of the boring elevations were interpolated from the provided preliminary plan and profile of the bridge. The as-drilled boring locations will be surveyed by others.
2. Below ground surface.

The borings were located in the field prior to drilling operations by Terracon personnel using a handheld GPS unit. The survey information was not available as of this report's preparation. Borings coordinates and elevations presented in the preceding table, and on the boring logs presented in Appendix A, were obtained from handheld GPS readings recorded during our site visit. The location and elevation information should be considered accurate only to the degree implied by the means and methods used to define them.

The borings were drilled with a track-mounted drill rig utilizing a 3¼-inch I.D. continuous flight hollow stem auger to advance the boreholes between sampling attempts. Sampling using a split-barrel sampler was performed at 2.5-foot intervals to a depth of approximately 10 feet, followed by continuous sampling to a depth of 25 feet, and at 5-foot intervals thereafter to the bedrock depth. Our drillers observed and recorded groundwater levels during drilling and upon completion of drilling.

In the split-barrel sampling procedure, the number of blows required to advance a standard 2-inch O.D. split-barrel sampler the last 12 inches of the typical total 18-inch penetration by means of a 140-pound automatic hammer with a free fall of 30 inches, is the standard penetration resistance value (SPT-N). This value is corrected to an equivalent (60 percent) energy ratio (N_{60}) utilizing the hammer efficiency energy ratio which is approximately 78.8% for the equipment used during our exploration.

Rock coring was performed using a NQ-size double tube-swivel core barrel. Percentage of recovery and rock quality designation (RQD) were calculated for the core samples and are noted at their depths of occurrence on the boring logs.

In the field, the samples recovered at the boring locations were examined and field logs were prepared indicating the conditions encountered at each location. Representative portions of soil samples obtained during the field exploration were preserved in sealable glass jars and rock core samples were placed in partitioned boxes. The samples were delivered to our laboratory for additional examination and testing.

Following the completion of drilling, the boreholes were sealed with auger cuttings mixed with bentonite chips.

Laboratory Testing Program

As part of the testing program, all samples were examined in our laboratory by a geotechnical engineer. Soil samples were classified in general accordance with ODOT SGE Section 600 Laboratory Testing based on the texture and plasticity of the soils.

Visual soil classification was performed on all recovered soil and rock samples. Atterberg limits, moisture content, grain size analysis tests were performed on selected soil samples. In addition, unconfined compression and slake durability tests were performed on selected rock samples to obtain rock properties information. The results of lab testing are shown on the boring logs and/or presented in the **Exploration and Laboratory Testing Results** of this report.

Findings

Boring logs have been prepared based on the information obtained from the field logs prepared at the time of drilling, the visual examination performed in the laboratory, and the laboratory testing results. Soil classification was performed in general accordance with the current ODOT SGE. The following sections summarize the subsurface conditions encountered at the boring locations.

Subsurface Profile

Borings B-001-0-24 and B-002-0-24 were performed at the north and south abutments of Bridge Street bridge over the Conotton Creek, respectively. The borings B-001-0-24 and B-002-0-24 encountered a surficial layer consisting of topsoil approximately 8 inches thick. The boring B-002-0-24 encountered fill material to depth of about 3.5 feet below the existing ground surface. The fill materials consisted of cohesive soil described as silt and clay (A-6a).

Below the surficial layer/existing fill, the native cohesive soils encountered in the borings included very soft to hard, silt and clay (A-6a) and silty clay (A-6b). The native granular soils encountered in the borings included very loose to dense, sandy silt (A-4a), and coarse and fine sand (A-3a).

Bedrock

Bedrock was encountered in borings B-001-0-24, and B-002-0-24 at a depth varying from about 38.5 to 50 feet, which corresponds to elevations varying from about EL 901.3 and EL 893.7 feet, respectively. The bedrock encountered in the borings consisted of moderately weathered shale, and sandstone.

Groundwater Conditions

In boring B-001-0-24 groundwater was encountered at a depth of 16 feet below ground surface during drilling and was not encountered upon completion of drilling. In boring B-002-0-24 groundwater was encountered at a depth of 8.5 feet below ground surface during drilling and was not encountered upon completion of drilling.

Groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, water flow from the existing creek, runoff, and other factors not evident at the time the borings were performed. Therefore, groundwater levels during construction or at other times in the life of the proposed structure may be higher or lower than the levels indicated on the boring logs. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.

Analysis and Recommendations

Based on our evaluation of the subsurface conditions encountered at the site, and the requirements outlined in Section 305.3.3 of ODOT Bridge Design Manual (BDM), it is recommended that a deep foundation system including steel H-piles driven to refusal into bedrock, or drilled shafts socketed into bedrock be used to support the proposed structure.

Driven Pile Recommendations

Based on the above-mentioned information, and considering the subsurface conditions encountered at the boring locations, the proposed bridge structure can be supported on driven steel H-piles. As per the ODOT BDM, refusal is met during driving when the pile penetration is an inch or less after receiving at least 20 blows from the pile hammer. The following table shows the recommended factored structural resistance values (P_r) at each abutment.

Boring ID	Ground Surface Elevation[1] (feet)	Pile Type	Est. Pile Top Elevation[2] (feet)	Top of Rock Elevation (feet)	P_r [3, 4] (kips/pile)
B-001-0-24	939.8	HP 10x42	930.0	901.3	310
		HP 12x53			380
		HP 14x73			530
B-002-0-24	943.7	HP 10x42	934.0	893.7	310
		HP 12x53			380
		HP 14x73			530

1. Ground surface elevation is the existing ground elevation at the boring location, interpolated from the provided preliminary plan and profile of the bridge. The elevations should be adjusted to surveyed site elevations once survey data is available.
2. The top of pile elevation is at the proposed bottom of footing elevation at the abutment locations. Per the BDM section 305.3.5.2. A 1-ft embedment depth into the concrete footing is required. Also, a 10-foot minimum pile length is required per the BDM section 305.3.5.2. Pile tip elevation is taken at sample with split-spoon refusal at the nearest boring per ODOT BDM section 305.3.5.2. Estimated pile length should be verified by Structural Engineer.
3. Factored structural resistance per BDM C305.3.3-1 for piles driven to refusal.
4. We assumed that the slopes supporting the abutments will be protected against scour to prevent loss from scour.

Per the most recent version of the ODOT BDM, the factored resistance for piles driven to refusal on bedrock is typically governed by the structural resistance of the pile element. The maximum factored structural resistances listed in the table above assume an axially loaded pile with negligible moment, no appreciable loss of section due to deterioration throughout the life of the structure, a steel yield strength of 50 ksi, a structural resistance factor for H-piles subject to damage due to severe driving conditions (LRFD 6.5.4.2: $\varphi c = 0.50$) and a pile fully braced along its length. In addition, pile points/shoes are required for this project.

These bearing values should not be used for piles that are subjected to bending moments or are not supported by soil for their entire length. Therefore, it is recommended that slope erosion protection (riprap) be specified to line the slopes supporting the abutments to prevent a loss of support from scour.

Please note that it is anticipated that the piles will be able to be driven a short distance into the surficial bedrock before satisfying the driving conditions that meet the refusal criteria. The piles should be driven to refusal as defined in Section 305.3.1.2 of the ODOT BDM. Settlement is estimated to be less than 1.0 inch for H-Piles driven to refusal on bedrock.

The recommendations require the piles bear in the existing shale/siltstone bedrock. Note that weak, highly weathered rock such as shale may exhibit a decrease in capacity after pile driving has ended due to a phenomenon known as relaxation. Therefore, piles should be driven to refusal within the shale bedrock, then re-driven to refusal after the relaxation has occurred. Based on ODOT BDM (Section 600), a wait period of 7 days is recommended before re-strike.

Pile Drivability Analysis

A preliminary drivability analysis for each of the proposed pile types and sizes was performed in accordance with ODOT BDM section 305.3.1.2 and ODOT GDM section 1304.2 for borings B-001-0-24 and B-002-0-24 considering the Delmag D19-42 hammer with a rated energy of up to 44,000-ft-lb. This preliminary analysis was performed using GRLWEAP and considered 50 ksi ASTM A572 Grade 50 steel H-piles. Per ODOT BDM section 305.3.1.2, If the analysis indicates blow counts higher than 100 blows per foot (bpf) prior to reaching the required bearing depth or resistance, it should be considered as practical refusal, and a hammer of higher rated energy should be used.

Results of the GRLWEAP analyses are summarized in the table below. Based on the results of our pile drivability analyses, we recommend HP 10x42 steel piles be utilized at both abutments. All piles should meet AASHTO M270, Grade 50 requirements. All piles should be tipped with high strength steel pile points to minimize damage to the tips during final driving conditions. The piles should be aligned so that the flanges are parallel to the direction of thermal movement. Detailed drivability analysis results for borings B-001-0-24 and B-002-0-24 are provided in the Appendix C - **Supporting Information**.

Boring ID	Pile Type	Steel Grade	Allowable Compressive Stress (ksi)	Max. Blow Counts per foot	Max. compressive Stress (ksi)	Blow Count Criteria	Comp. Stress Criteria	Recommended Hammer
B-001-0-24	HP 10x42	ASTM A572 (50 ksi)	45.0	30	31.2	Yes	Yes	D 19-42
	HP 12X53			48	31.3	Yes	Yes	D 19-42
	HP 14X73			93	29.9	Yes	Yes	D 19-42
B-002-0-24	HP 10x42	ASTM A572 (50 ksi)	45.0	13	26.7	Yes	Yes	D 19-42
	HP 12X53			17	26.9	Yes	Yes	D 19-42
	HP 14X73			22	26.1	Yes	Yes	D 19-42

Prior to construction, the contractor shall perform a drivability analysis using the pile hammer-cushion combination that will be used. Pile driving conditions, hammer efficiency, and stress on the pile during driving could be better evaluated during installation using a Pile Driving Analyzer (PDA) performed by a qualified pile testing contractor on selected piles. Driving criteria for the driven piles should be recommended by the pile testing contractor, based on the PDA results. During driving a maximum of 20 blows per inch is recommended to reduce the potential of damage to the piles. Each pile should be observed and checked for buckling, crimping, and alignment in addition to recording penetration resistance, depth of embedment, and general pile driving operations. The pile driving process should be performed under the direction of the Geotechnical Engineer or their representative. They should document the pile installation process including hammer blow counts, consistency with expected conditions, and details of the installed pile.

Drilled Shaft Recommendations

Alternatively, a deep foundation system consisting of drilled shaft foundations can be considered for supporting the proposed bridge. Based on the test borings, we recommend that the drilled shafts be socketed at least 1.5 times the rock socket diameter into the bedrock below the estimated top of rock socket elevations presented in the table below. The actual socket length may be greater based axial loading/ lateral loading conditions and final shaft lengths should be determined by the designer.

Based on the encountered subsurface conditions, drilled shafts that derive resistance from end bearing and side resistance in bedrock can be used for the proposed bridge structures. The designer should refer to AASHTO LRFD Section 10.8.3.5.4d for guidance on proportioning the resistance between tip resistance and side resistance. The following sections provide recommendations regarding the design of drilled shaft foundations to resist axial compressive and uplift loads, as well as soil and bedrock parameters to design the drilled shafts to resist lateral loads. Our recommendations consider the soil and bedrock conditions encountered in the test borings.

Drilled Shaft Design

Boring ID	Embedment Material	Estimated Top of Rock Unit Elevation (ft) [1]	Minimum Rock Socket Length (ft)	Minimum Shaft Diameter (inch) [2]	Unfactored Nominal Unit Tip Resistance, qp (ksf) [3]	Unfactored Nominal Unit Side Resistance, qs (ksf) [3]	Resistance Factor, φstat
B-001-0-24	Shale	898.8	1.5 x Shaft Diameter	36	130	19	0.50 (Tip) 0.55 (Side) 0.4 (uplift resistance)
B-002-0-24	Shale	893.7			150	25	

1. Below existing ground surface. See Findings and the boring logs for soil and bedrock stratigraphy details. Top of rock socket elevations listed in this table are interpreted from test borings. The drilled shaft lengths will vary depending upon the depth to top of rock of the sandstone and shale bedrock. Due to anticipated variation in top of rock elevation, top of rock socket elevations should be field verified with pre-bored holes per ODOT C&MS Items 524.08 & 524.09 during construction.
2. Rock socket diameter should at least 6 inches less than the actual diameter of the shaft.
3. Rock socketed drilled shaft should be designed following BDM Section 305.4.2. Side resistance of drilled shafts can be used to resist either compressive or uplift forces. The weight of the shaft can also be used to resist any uplift forces. The buoyant weight of the shaft should be used below the anticipated groundwater level to resist uplift forces.

The drilled shaft length will need to be designed to satisfy axial compressive, uplift, and lateral load requirements. The penetration of the drilled shaft into shale/siltstone bedrock may need to be increased over the minimum rock socket for axial compressive capacity based on the lateral resistance or uplift resistance requirements of the drilled shaft foundations. In general, based on the geotechnical resistances provided drilled shafts should be designed per BDM section 305.4.

Recommended L-Pile Parameters for Lateral Pile Analysis

The following table provides input values for use in LPILE analyses. LPILE estimated values of k_h and E_{50} based on strength; however, non-default values of k_h were used where provided. The soil parameters were estimated based on the test borings, laboratory test results, and our experience with these soil types. The portion of the drilled shaft within 36 inches of finished grade should ignore any lateral soil resistance due to frost considerations.

The tables below present the recommended L-Pile parameters for each boring to be used for lateral pile analysis.

BORING B-001-0-24							
Soil Layer/Type[1]	**Approximate Bottom Depth of Layer (feet)**	**LPILE Model**	**Total Unit Weight (pcf)**	**Undrained Shear Strength (psf)**	**Soil Friction Angle (deg)**	**K (pci)**	**ε_{50}**
Silt and Clay (A-6a)	6.0	Stiff Clay w/o Free Water (Reese)	124	750	--	80	0.015
Sandy Silt (A-4a)	11.5	Sand (Reese)	110	--	28	25	--
Coarse and fine Sand (A-3a)	16.0	Sand (Reese)	110	--	28	25	--
Coarse and fine Sand (A-3a)	20.5	Sand (Reese)	115	--	30	50	--
Coarse and fine Sand (A-3a)	23.5	Sand (Reese)	127	--	35	90	--
Silty Clay (A-6b)	33.5	Stiff Clay with Free Water (Reese)	126	1,500	--	350	0.010
Silty Clay (A-6b)	38.5	Stiff Clay with Free Water (Reese)	131	4,500	--	1,500	0.004
Weathered Bedrock	40.0	Sand (Reese)	132	--	37	135	--
Bedrock [2]	50.0	Weak Rock	See Table Below for Rock Properties				

BORING B-001-0-24

Soil Layer/Type[1]	Approximate Bottom Depth of Layer (feet)	LPILE Model	Total Unit Weight (pcf)	Undrained Shear Strength (psf)	Soil Friction Angle (deg)	K (pci)	ε_{50}

1. See test boring logs and Findings for more details on Stratigraphy.
2. Boring terminated within this layer

Rock Type	Approx. Depth to Bottom of Layer (feet)[1]	Total Unit Weight (pcf)	Rock Compressive Strength (psi)	Elastic Modulus (psi)	RQD (%)	k_{rm}
Sandstone	41.0	145	3,500	28,000	30	0.0005
Shale	50.0	140	4,800	24,000	54	0.0005

1. Below existing ground surface.

BORING B-002-0-24

Soil Layer/Type[1]	Approximate Bottom Depth of Layer (feet)	LPILE Model	Total Unit Weight (pcf)	Undrained Shear Strength (psf)	Soil Friction Angle (deg)	K (pci)	ε_{50}
Silt and Clay (A-6a)	3.5	Stiff Clay w/o Free Water (Reese)	124	750	--	80	0.015
Silty Clay (A-6b)	8.5	Stiff Clay w/o Free Water (Reese)	124	750	--	80	0.015
Silt and Clay (A-6a)	11.5	Stiff Clay w/o Free Water (Reese)	122	250	--	15	0.030
Sandy Silt (A-4a)	14.5	Sand (Reese)	110	--	28	20	--
Coarse and fine Sand (A-3a)	19.0	Sand (Reese)	110	--	28	20	--
Silty Clay (A-6b)	23.5	Stiff Clay with Free Water (Reese)	124	750	--	80	0.015
Silt and Clay (A-6a)	33.5	Stiff Clay with Free	126	1,500	--	350	0.010

BORING B-002-0-24							
Soil Layer/Type[1]	**Approximate Bottom Depth of Layer (feet)**	**LPILE Model**	**Total Unit Weight (pcf)**	**Undrained Shear Strength (psf)**	**Soil Friction Angle (deg)**	**K (pci)**	**ε_{50}**
		Water (Reese)					
Silt and Clay (A-6a)	50.0	Stiff Clay with Free Water (Reese)	131	4,500	--	1,500	0.004
Bedrock [2]	60.0	Weak Rock	See Table Below for Rock Properties				

1. See test boring logs and Findings for more details on Stratigraphy.
2. Boring terminated within this layer.

Rock Type	Approx. Depth to Bottom of Layer (feet)[1]	Total Unit Weight (pcf)	Rock Compressive Strength (psi)	Elastic Modulus (psi)	RQD (%)	k_{rm}
Shale	60.0	140	9,500	25,000	57	0.0005

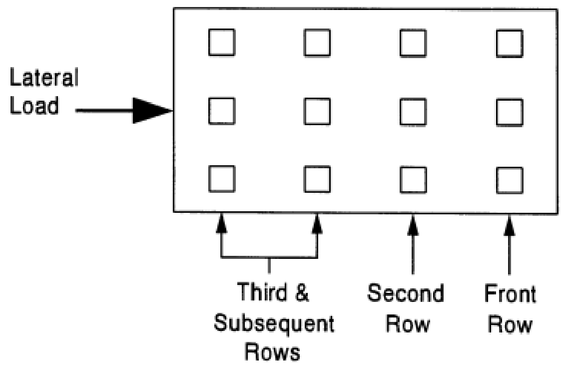
1. Below existing ground surface.

The structural capacity of the drilled shafts should be checked to assure that they can safely accommodate the combined stresses induced by axial and lateral forces. Lateral deflections of drilled shaft foundations should be evaluated using an appropriate analysis method, and will depend upon the element's diameter, length, configuration, stiffness and "fixed head" or "free head" condition. We can provide additional analyses and estimates of lateral deflections for specific loading conditions upon request, at an additional fee. The load-carrying capacity of drilled shaft foundations may be increased by increasing the section. Proper reinforcing steel should be included in the drilled shaft designs for resistance of the combined axial loads and bending moments.

Group action for lateral resistance of drilled shaft foundations should be considered when the center-to-center spacing is less than 6 diameters. For a group of shafts oriented parallel to a lateral load, design parameters for allowable passive resistance within soil should be reduced in accordance with BDM section C305.4.4.1 as shown in the following table. Group reduction factor is not applicable for the portion of the shafts socketed in rock.

Laterally Loaded Shafts – Group Reduction Factors

Shaft Spacing [1] (Diameters)	Leading Row Reduction Factor	Second Row Reduction Factor	Third or Higher Row Reduction Factor
6D	1.0	1.0	1.0
5D	1.0	0.85	0.7
3D	0.8	0.4	0.3



1. Center-to-center spacing in the direction of loading. If the loading direction for a single row of shafts is perpendicular to the row, a group reduction factor should be used if the shaft spacing is less than 5D.

The structural capacity of the drilled shafts should be checked to assure that they can safely accommodate the combined stresses induced by axial and lateral forces. Lateral deflections of drilled shaft foundations should be evaluated using an appropriate analysis method, and will depend upon the element's diameter, length, configuration, stiffness and "fixed head" or "free head" condition. We can provide additional analyses and estimates of lateral deflections for specific loading conditions upon request, at an additional fee. The load-carrying capacity of drilled shaft foundations may be increased by increasing the section. Proper reinforcing steel should be included in the drilled shaft designs for resistance of the combined axial loads and bending moments

Lateral Earth Pressures

Retaining walls, and excavation support systems must be designed to withstand lateral earth pressures, as well as hydrostatic pressure, that may develop behind the structures. The magnitude of lateral earth pressure varies on the basis of soil type, permissible wall movement, and type of the backfill.

In order to minimize lateral earth pressures, the zone behind the structures should be effectively drained. For effective drainage, a zone of porous backfill (ODOT CMS Item 518.03) should be used directly behind the structures for a minimum thickness of 2 feet in accordance with ODOT CMS Item 518.05. The granular zone should be designed to drain to either weepholes or a pipe, to alleviate the build-up of hydrostatic pressures against the walls.

The type of backfill beyond the free-draining granular zone will govern the pressure to be used for structural design. Pressures of a relatively low magnitude will be generated by granular backfill materials, whereas cohesive backfill materials will result in the development of higher lateral pressures. Therefore, it is recommended that granular backfill be utilized whenever possible. Granular backfill behind structures should be placed and compacted in accordance with ODOT CMS Item 203.

Retaining walls that are fixed and unable to rotate or deflect will be subjected to at-rest earth pressure conditions. Earth pressure distributions should be based on the mobilization of active earth pressure conditions for retaining walls that are free to deflect or rotate. Retaining walls exerting a force on the soil (such as soil in front of the footing on the face side of the wall) are subject to a passive resistance. However, due to the potential for erosion, this passive resistance is typically ignored.

The tables presented below include the recommended unfactored and factored equivalent fluid unit weights for walls subject to the mobilization of both at-rest and active earth pressure conditions as described above. A load factor of 1.5 has been used for the determination of the factored equivalent fluid unit weights. The values presented in the following table assume a flat backslope behind the walls, and that the backfill material will not be subject to any additional load (such as uniformly distributed soil surcharge near the top and immediately behind the face of the wall). Two cases have been considered for backfill behind the wall: a two-foot-wide zone of granular porous backfill with filter fabric, and backfilling with a wedge of granular material.

For a two-foot-wide zone of granular porous backfill, the earth pressure was calculated assuming an angle of internal friction of 24 degrees, a moist soil unit weight of 125 pcf, and a soil/concrete interface friction angle of 16 degrees.

Wall Type	Pressure Distribution	Unfactored Equivalent Fluid Weight (pcf)	Factored Equivalent Fluid Weight (pcf)	Earth Pressure Coefficient
Cantilever Retaining Wall – Free Head	Active	47.5	71	K_a = 0.38
Rigid Retaining Wall – Fixed Head	At-rest[1]	74	100	K_o = 0.59

1. Due to the fixity condition at the top of the wall, it is recommended that the triangular pressure distribution should be converted into a uniform or rectangular pressure distribution along the height of the wall.

For a wedge of granular material (assuming 2:1 backslope from bottom of backfill), the earth pressure was computed assuming an angle of internal friction of 30 degrees, a moist soil unit weight of 120 pcf, and a soil/concrete interface friction angle of 20 degrees.

Wall Type	Pressure Distribution	Unfactored Equivalent Fluid Weight (pcf)	Factored Equivalent Fluid Weight (pcf)	Earth Pressure Coefficient
Cantilever Retaining Wall Free Head	Active	36	54	K_a = 0.30
Rigid Retaining Wall Fixed Head	At-rest[1]	60	81	K_o = 0.50

1. Due to the fixity condition at the top of the wall, it is recommended that the triangular pressure distribution should be converted into a uniform or rectangular pressure distribution along the height of the wall.

The earth pressure values presented in the preceding tables assume that provisions for positive gravity drainage will be provided, and that the abutments and walls will be backfilled with free-draining coarse aggregate, such as ODOT No. 57 stone.

We do not recommend using passive earth pressures in design of permanent retaining walls and/or bridge abutments due to the potential for erosion, or possibility of removal of the soils in front of the wall in the future.

Scour Data

SPT sampling was performed at 2.5-foot intervals to a depth of approximately 10 feet, followed by continuous sampling to a depth of 25 feet, and at 5-foot intervals thereafter to the bedrock depth. The sampling was performed to determine the median grain size (D_{50}) of the collected soil samples. Based on the conditions encountered at the boring locations, it is anticipated that the streambed soils will consist of granular soils consisting of sandy silt (A-4a), and coarse and fine sand (A-3a) and cohesive soils consisting of silt and clay (A-6a), and silty clay (A-6b). Note that specific borings were not drilled within the creek as part of this exploration. Recovered soil samples evaluated for potential scour were from borings performed behind the existing abutments. As such, actual soil conditions and potential scour within the creek may vary from the conditions encountered in the borings performed behind the abutments. Based on the grain size analyses performed by Terracon, the following table summarizes the D_{50} values from testing of samples from the borings. Also, the critical shear stress (τ_c), the equivalent D_{50} ($D_{50,\ equiv}$), Erosion Category (EC), and Erodibility Index (K), were calculated based on the equations provided in GDM sections 1302.1, 1302.2 and 1403, and summarized in the following table.

Boring Number	Sample Number	Elevation (feet)	D_{50} (mm)	Erodibility Index, K	τ_c (psf)	$D_{50, equiv}$ (mm)	Erosion Category, EC
B-001-0-24	SS-6	928.3-926.8	0.148	--	0.0015	0.0733	2.211
	SS-7	926.8-925.3	0.141	--	0.0025	0.1182	2.211
	SS-8	925.3-923.8	0.158	--	0.0012	0.0591	2.211
	SS-9	923.8-922.3	0.241	--	0.0050	0.241	1.458
	SS-10 to SS-13[1]	922.3-916.3	0.250	--	0.0052	0.250	1.478
	SS-14 to SS-16[1]	916.3-901.3	0.060	--	0.1162	5.562	3.255
	Sandstone	901.3-898.8	--	48.782	36.930	--	2.084
	Shale	899.0-890.0	--	134.666	61.359	--	2.656
B-002-0-24	SS-5	933.7-932.2	0.062	--	0.0245	1.1705	3.255
	SS-6	932.2-930.7	0.078	--	0.0078	0.3739	2.211
	SS-7	930.7-929.2	0.089	--	0.0081	0.3865	2.211
	SS-8	929.2-927.7	0.156	--	0.0015	0.0717	2.211
	SS-9 and SS-10[1]	927.7-924.7	0.150	--	0.0026	0.1237	2.211
	SS-11 to SS-13[1]	924.7-920.2	0.060	--	0.1287	6.1603	3.255
	SS-14 to SS-17[1]	920.2-900.2	0.062	--	0.1222	5.8476	3.075
	SS-18 and SS-19[1]	900.2-893.7	0.065	--	0.2153	10.304	2.975
	Shale	893.7-883.7	--	279.527	88.402	--	2.656

1. Soil data required to calculate scour parameters were estimated for these samples based on the similar conditions obtained from borings B-001-0-24 and B-002-0-24 and experience with similar conditions.

Seismic Site Classification

Code Used	Site Classification
AASHTO LRFD Bridge Design Specifications, Ninth Edition, 2020 [1]	C [2]

1. In general accordance with Section 3.10.3 of the AASHTO LRFD Bridge Design Specifications, Ninth Edition, 2020.
2. AASHTO LRFD Bridge Design Specifications, requires a site subsurface profile determination extending to a depth of 100 feet for seismic site classification. Borings for this study extended to a maximum depth of approximately 60 feet and this seismic site class definition considers that bedrock continue below the maximum depth of the subsurface exploration. Additional exploration to deeper depths could be performed to confirm the conditions below the current depth of exploration. Alternatively, a geophysical exploration could be utilized in order to attempt to justify a higher seismic site class. The current scope requested does not include the required 100-foot subsurface profile determination.

Construction Considerations

All site work should conform to local codes and to the latest ODOT Construction and Material Specifications (CMS), including that all structure removal, excavation and embankment preparation and construction should follow ODOT CMS Item 200 (Earthwork).

The geotechnical engineer should be retained during the construction phase of the project to observe earthwork and to perform necessary tests and observations during subgrade preparation, proof-rolling, placement and compaction of controlled compacted fills, and backfilling of any excavations into the completed subgrade.

Earthwork Considerations

Subgrade preparation for the new foundations, pavement, shoulder areas, and embankments should be performed in accordance with ODOT CMS Items 203 and 204. Prior to subgrade preparation, perform clearing and grubbing, including removal of stumps and roots, in accordance with ODOT CMS Item 201. Remove existing pavement and base materials as well as other structures or obstructions, as necessary, in accordance with ODOT CMS Item 202. The subgrade should be stripped of any topsoil, organics, or other deleterious or unsuitable materials.

All embankment materials should be spread and compacted in accordance with Items 203.06 and 203.07 and subgrade materials should be spread and compacted in accordance with Items 204.07 and 204.03. Frozen materials should not be incorporated into any new fill nor should new fill, pavement materials, or structures be placed on top of frozen materials. Material to be utilized as borrow should be restricted to conform to Item 203.02R and 203.3 for embankment construction and Item 204.2 for subgrade. Clay with high plasticity should not be used for the embankment.

Earthwork, including subgrade preparation should be performed in accordance with respective items in Section 200 of the current ODOT CMS. Consideration may be given to using the in-situ soils or from the local borrow sources. However, the material may require moisture adjustments to achieve proper compaction. Potentially, chemical treatment may be used for any borrow materials and existing embankment soil with high moisture contents. Chemical treatment should be performed in accordance with ODOT Item 205.

If applicable, it is recommended that any benching required for embankment construction for the project be performed in accordance with “A. General Case: Special Benched Embankment Construction” of ODOT Geotechnical Bulletin 2 (GB-2).

Grading and Drainage

During construction, site grading should be developed to direct surface water flow away from, or around, the site. Exposed subgrades should be sloped to provide positive drainage so that saturation of subgrades is avoided. Surface water should not be permitted to accumulate on the site.

Final surrounding grades should be sloped away from the proposed embankments on all sides to prevent ponding of water. Due to the nature of the soil profile, trapped water infiltration or groundwater seepage may be encountered, particularly after periods of precipitation. In such an event, sump and pumping methods may be used for temporary dewatering.

Excavation Considerations

As a minimum, all excavations should be sloped or braced as required by Occupational Health and Safety Administration (OSHA) regulations to provide stability and safe working conditions. Reference to OSHA 29 CFR, Part 1926, Subpart P should be included in the job specifications. current OSHA excavation and trench safety standards.

The grading contractor, by his contract, is usually 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. Slope heights, slope inclinations and/or excavation depths should in no case exceed those specified in local, state, or federal safety regulations, including the current OSHA Excavation and Trench Safety Standards.

Under no circumstances should the information provided in this report be interpreted to mean that Terracon is responsible for construction site safety or the contractor's activities. Construction site safety is the sole responsibility of the contractor, who shall also be solely responsible for the means, methods, and sequencing of the construction operations.

Groundwater Considerations

In boring B-001-0-24 groundwater was encountered at a depth of 16 feet below ground surface during drilling and was not encountered upon completion of drilling. In boring B-002-0-24 groundwater was encountered at a depth of 8.5 feet below ground surface during drilling and was not encountered upon completion of drilling. Groundwater level upon completion might be affected as water was used as coring fluid for rock coring.

Groundwater is anticipated during construction at the normal water elevation of the creek. Where encountered during construction, proper groundwater control should be employed and maintained to prevent disturbance to excavation bottoms consisting of cohesive soil, and to prevent the possible development of a quick or "boiling" condition where soft silts and/or fine sands are encountered. It is preferable that the groundwater level, if encountered, be maintained at least 5 feet below the deepest excavation. Any seepage or groundwater encountered during foundation excavation should be able to be controlled by pumping from temporary sumps. However, additional measures may be required depending on seasonal fluctuations of the creek/groundwater level. Note that determining and maintaining actual groundwater levels during construction is the responsibility of the contractor.

Slope Stability Analyses

The embankments at the bridge abutments slope down towards the Conotton Creek at slope inclinations of about 3 Horizontal (H) to 1 Vertical (V) to 4H to 1V, additional evaluation including slope stability analyses would be required to determine stability of the embankments. Once the final plan and profile drawings for the bridge are available, we would be able to perform this evaluation, however this would involve an additional fee.

General Comments

Terracon should be retained to review the final design plans and specifications, so comments can be made regarding interpretation and implementation of our geotechnical recommendations in the design and specifications. Terracon should also be retained to provide observation and testing services during grading, excavation, foundation construction and other earth-related construction phases of the project.

This Geotechnical Engineering Report has been prepared to present the findings of our exploration and present our recommendations pertaining to proposed improvements. The analysis and recommendations presented in this report are based upon the data obtained from the borings performed at the indicated locations and from other information discussed in this report. This report does not reflect variations that may occur between borings, across the site, or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. If variations appear, we should be immediately notified so that further evaluation and supplemental recommendations can be provided.

The scope of services for this project does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials, or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

This report has been prepared for the exclusive use of our client for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, either express or implied, are intended or made. Site safety, excavation support, and dewatering requirements are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless Terracon reviews the changes and either verifies or modifies the conclusions of this report in writing.

Site characteristics as provided are for design purposes and not to estimate excavation cost. Any use of our report in that regard is done at the sole risk of the excavating cost estimator as there may be variations on the site that are not apparent in the data that could significantly impact excavation cost. Any parties charged with estimating excavation costs should seek their own site characterization for specific purposes to obtain the specific level of detail necessary for costing. Site safety, and cost estimating including, excavation support, and dewatering requirements/design are the responsibility of others. If changes in the nature, design, or location of the project are planned, our conclusions and recommendations shall not be considered valid unless we review the changes and either verify or modify our conclusions in writing.

Appendices

Appendix A – Field Exploration Information

Contents:

Note: All attachments are one page unless noted above

Site Location

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

Exploration Plan

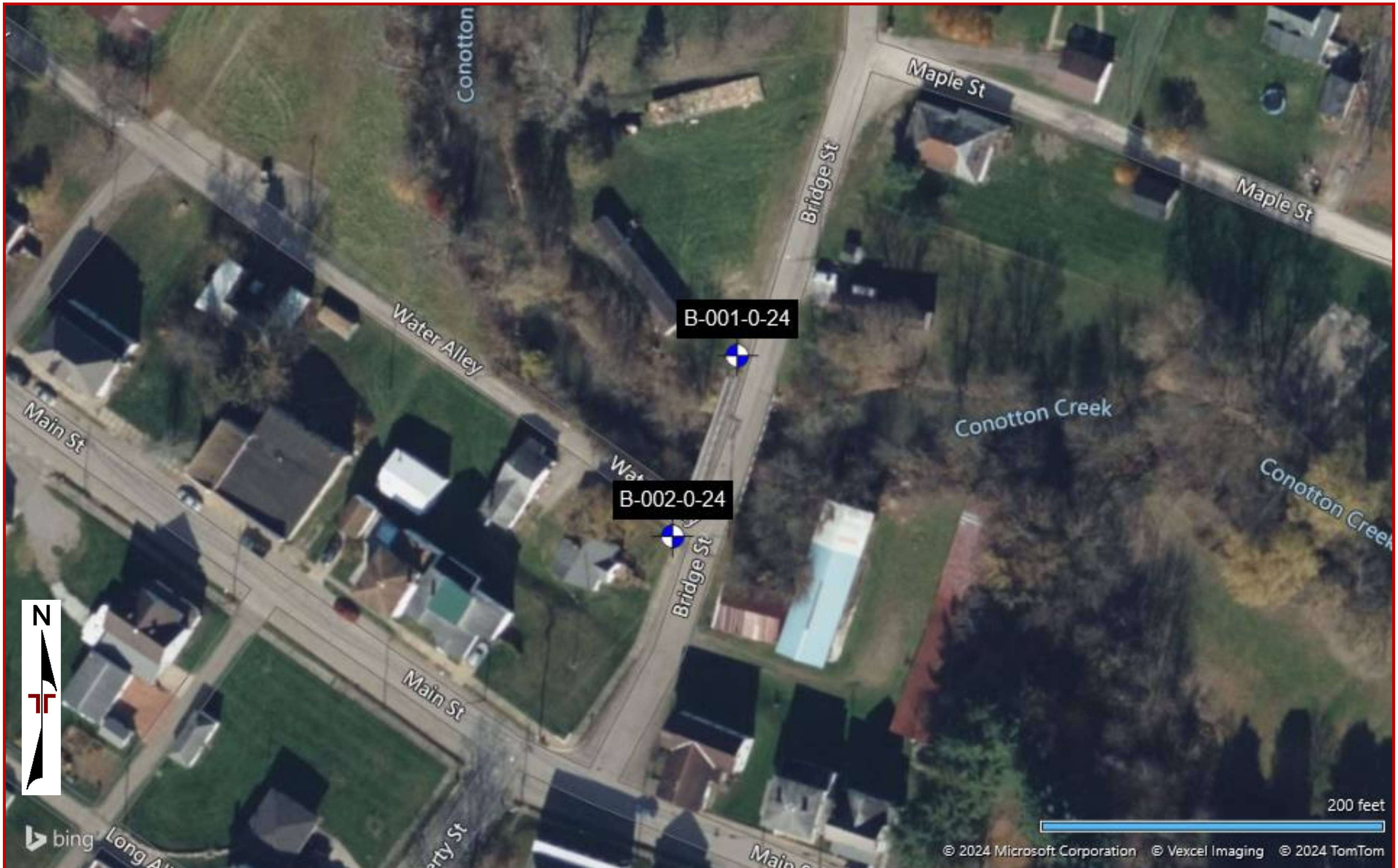


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

Appendix B – Exploration and Laboratory Testing Results

Contents:

Note: All attachments are one page unless noted above.

PROJECT: HAS-BRIDGE STREET BRIDGE	DRILLING FIRM / OPERATOR: TC / K. REINHART	DRILL RIG: MOBILE B-57 (DR#613)	STATION / OFFSET: 202+31, 6' LT.	EXPLORATION ID B-001-0-24
TYPE: BRIDGE	SAMPLING FIRM / LOGGER: TC / T. CAROTHERS	HAMMER: AUTOMATIC HAMMER	ALIGNMENT: BRIDGE STREET	
PID: 120494 SFN: 3431789	DRILLING METHOD: 3.25" HSA / NQ2	CALIBRATION DATE: 8/11/22	ELEVATION: 939.8 (MSL) EOB: 50.0 ft.	PAGE 1 OF 2
START: 2/20/24 END: 2/20/24	SAMPLING METHOD: SPT / NQ2	ENERGY RATIO (%): 78.8	LAT / LONG: 40.425919, -81.187127	

MATERIAL DESCRIPTION AND NOTES	ELEV. 939.8	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
TOPSOIL (8")	939.1																	
MEDIUM STIFF, BROWN, **SILT AND CLAY**, LITTLE SAND, LITTLE GRAVEL, MOIST		1, 2	3 3 3	8	33	SS-1	-	-	-	-	-	-	-	-	-	16	A-6a (V)	
		3																
	933.8	4, 5	2 2 2	5	33	SS-2	1.00	-	-	-	-	-	-	-	-	20	A-6a (V)	
VERY LOOSE TO LOOSE, BROWN, **SANDY SILT**, TRACE GRAVEL, MOIST TO WET		6, 7	2 2 2	5	67	SS-3	-	-	-	-	-	-	-	-	-	18	A-4a (V)	
		8																
		9, 10	1 0 1	1	100	SS-4	-	-	-	-	-	-	-	-	-	22	A-4a (V)	
	928.3	11	1 1 1	3	78	SS-5	-	-	-	-	-	-	-	-	-	29	A-4a (V)	
VERY LOOSE TO LOOSE, BROWN AND GRAY, **COARSE AND FINE SAND**, TRACE TO LITTLE GRAVEL, WET		12, 13	1 1 0	1	100	SS-6	-	0	5	74	21		NP	NP	NP	30	A-3a (0)	
		14	1 1 2	4	100	SS-7	-	0	4	72	24		NP	NP	NP	27	A-3a (0)	
		W 923.8 15, 16	1 0 1	1	33	SS-8	-	1	12	65	22		NP	NP	NP	35	A-3a (0)	
	922.3	17	1 2 4	8	56	SS-9	-	15	19	49	17		NP	NP	NP	21	A-3a (0)	
MEDIUM DENSE TO DENSE, GRAY, **COARSE AND FINE SAND**, TRACE GRAVEL, WET		18, 19	2 4 5	12	100	SS-10	-	-	-	-	-	-	-	-	-	20	A-3a (V)	
		20	5 5 4	12	50	SS-11	-	-	-	-	-	-	-	-	-	23	A-3a (V)	
		21, 22	10 15 17	42	100	SS-12	-	-	-	-	-	-	-	-	-	21	A-3a (V)	
	916.3	23	11 15 20	46	100	SS-13	-	-	-	-	-	-	-	-	-	24	A-3a (V)	
STIFF, GRAY, **SILTY CLAY**, TRACE TO LITTLE SAND, MOIST		24, 25	2 3 8	14	100	SS-14	1.00	-	-	-	-	-	-	-	-	18	A-6b (V)	
	911.3	26, 27, 28																
STIFF, BROWN, **SILTY CLAY**, LITTLE SAND, MOIST		29	2 3 5	11	100	SS-15	1.00	-	-	-	-	-	-	-	-	22	A-6b (V)	

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 6/12/24 10:31 - N:\PROJECTS\2023\N4235509\WORKING FILES\LABORATORY-FIELD DATA-BORING LOGS\N4235509 HAS-BRIDGE

PID: 120494	SFN: 3431789	PROJECT: HAS-BRIDGE STREET BRIDGE	STATION / OFFSET: 202+31, 6' LT.	START: 2/20/24	END: 2/20/24	PG 2 OF 2	B-001-0-24

MATERIAL DESCRIPTION AND NOTES	ELEV. 909.8	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
STIFF, BROWN, **SILTY CLAY**, LITTLE SAND, MOIST *(continued)*		31–33																
		34–35	10 18 13	41	100	SS-16	1.00	-	-	-	-	-	-	-	-	17	A-6b (V)	
	901.3	36–38 (TR)																
SANDSTONE, GRAY, SEVERELY WEATHERED.	899.8	39–40	50/3"	-	100	SS-17	-	-	-	-	-	-	-	-	-	-	Rock (V)	
SANDSTONE, GRAY, MODERATELY WEATHERED, SLIGHTLY STRONG, FINE GRAINED, THIN BEDDED, FRACTURED; RQD 30%, REC 100%.	898.8	40–41																
SHALE, GRAY, MODERATELY WEATHERED, MODERATELY STRONG, FINE GRAINED, THIN BEDDED, FRACTURED; RQD 54%, REC 60%. @41.5' - 42.0'; Unit weight = 161 pcf; Qu = 4,831 psi		41–45	55		100	NQ2-1											CORE	
	891.8	45–48	47		100	NQ2-2											CORE	
SHALE, GRAY, MODERATELY WEATHERED, STRONG, FINE GRAINED, THIN BEDDED, MODERATELY FRACTURED TO SLIGHTLY FRACTURED; RQD 83%, REC 100%.	889.8	48–50 (EOB)																

NOTES: WATER ENCOUNTERED AT 16 FEET WHILE DRILLING; WATER NOT ENCOUNTERED UPON COMPLETION

ABANDONMENT METHODS, MATERIALS, QUANTITIES: AUGER CUTTINGS MIXED WITH BENTONITE CHIPS

PROJECT: HAS-BRIDGE STREET BRIDGE	DRILLING FIRM / OPERATOR: TC / K. REINHART	DRILL RIG: MOBILE B-57 (DR#613)	STATION / OFFSET: 201+17, 5' LT.	EXPLORATION ID B-002-0-24
TYPE: BRIDGE	SAMPLING FIRM / LOGGER: TC / T. CAROTHERS	HAMMER: AUTOMATIC HAMMER	ALIGNMENT: BRIDGE STREET	
PID: 120494 SFN: 3431789	DRILLING METHOD: 3.25" HSA / NQ2	CALIBRATION DATE: 8/11/22	ELEVATION: 943.7 (MSL) EOB: 60.0 ft.	PAGE 1 OF 2
START: 2/19/24 END: 2/19/24	SAMPLING METHOD: SPT / NQ2	ENERGY RATIO (%): 78.8	LAT / LONG: 40.425629, -81.187259	

MATERIAL DESCRIPTION AND NOTES	ELEV. 943.7	DEPTHS	SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
TOPSOIL (8")	943.0																	
DARK BROWN, **SILT AND CLAY**, TRACE SAND, **FILL**, DAMP		1–2	2 2 2	5	39	SS-1	-	-	-	-	-	-	-	-	-	31	A-6a (V)	
	940.2	3																
MEDIUM STIFF TO STIFF, BROWN, **SILTY CLAY**, LITTLE SAND, MOIST @3.5' to 5.0'; SAMPLE CONTAINS SANDSTONE FRAGMENTS		4–5	6 5 4	12	67	SS-2	1.00	-	-	-	-	-	-	-	-	18	A-6b (V)	
		6–7	2 2 2	5	39	SS-3	1.00	-	-	-	-	-	-	-	-	17	A-6b (V)	
	935.2	W 935.2 / 8																
VERY SOFT, GRAY, **SILT AND CLAY**, SOME SAND, LITTLE GRAVEL, WET		9	2 0 1	1	28	SS-4	-	-	-	-	-	-	-	-	-	33	A-6a (V)	
	932.2	10–11	0 0 0	0	100	SS-5	-	13	1	34	24	28	29	16	13	40	A-6a (5)	
VERY LOOSE, GRAY, **SANDY SILT**, WET		12	0 0 1	1	100	SS-6	-	1	1	49	- 49 -		NP	NP	NP	31	A-4a (3)	
	929.2	13–14	0 1 0	1	56	SS-7	-	0	2	53	- 45 -		NP	NP	NP	28	A-4a (2)	
VERY LOOSE, GRAY, **COARSE AND FINE SAND**, WET		15	0 0 1	1	33	SS-8	-	0	5	77	- 18 -		NP	NP	NP	26	A-3a (0)	
		16–17	0 0 0	0	0	SS-9	-	-	-	-	-	-	-	-	-	-	A-3a (V)	
	924.7	18	0 0 1	1	28	SS-10	-	-	-	-	-	-	-	-	-	33	A-3a (V)	
SOFT TO MEDIUM STIFF, GRAY, **SILTY CLAY**, LITTLE SAND, MOIST		19–20	1 2 1	4	67	SS-11	0.50	-	-	-	-	-	-	-	-	23	A-6b (V)	
		21	1 2 3	7	44	SS-12	-	-	-	-	-	-	-	-	-	20	A-6b (V)	
	920.2	22–23	3 3 2	7	83	SS-13	-	-	-	-	-	-	-	-	-	16	A-6b (V)	
STIFF TO HARD, BROWN, **SILT AND CLAY**, TRACE SAND, TRACE GRAVEL, MOIST		24	2 6 8	18	100	SS-14	4.50	-	-	-	-	-	-	-	-	17	A-6a (V)	
		25–28																
@28.5' to 30.0'; INTERBEDDED SAND LAYER		29	1 2 4	8	100	SS-15	-	-	-	-	-	-	-	-	-	23	A-6a (V)	

PID: 120494	SFN: 3431789	PROJECT: HAS-BRIDGE STREET BRIDGE	STATION / OFFSET: 201+17, 5' LT.	START: 2/19/24	END: 2/19/24	PG 2 OF 2	B-002-0-24

MATERIAL DESCRIPTION AND NOTES	ELEV. 913.7	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
STIFF TO HARD, BROWN, **SILT AND CLAY**, TRACE SAND, TRACE GRAVEL, MOIST *(continued)*		31 – 34																
		34 – 35	10 15 17	42	100	SS-16	-	-	-	-	-	-	-	-	-	15	A-6a (V)	
		39 – 40	15 20 20	53	100	SS-17	-	-	-	-	-	-	-	-	-	16	A-6a (V)	
	900.2	43.5																
HARD, GRAY, **SILT AND CLAY**, LITTLE SAND, TRACE GRAVEL, MOIST @43.5' to 45.0'; SAMPLE CONTAINS SANDSTONE FRAGMENTS		44 – 45	30 39 41	105	100	SS-18	-	-	-	-	-	-	-	-	-	11	A-6a (V)	
		48.5 – 49	50/5"	-	100	SS-19	-	-	-	-	-	-	-	-	-	20	A-6a (V)	
	893.7	TR 50																
SHALE, GRAY, MODERATELY WEATHERED, SLIGHTLY TO MODERATELY STRONG, FINE GRAINED, THIN BEDDED, FRACTURED; RQD 47%, REC 80%. @52.0' - 52.5'; Unit weight = 164 pcf; Qu = 9,509 psi @53.3' - 53.9'; Unit weight = 163 pcf; Qu = 9,670 psi	888.7	50 – 55	47		80	NQ2-1											CORE	
SHALE, GRAY, MODERATELY WEATHERED, WEAK TO SLIGHTLY STRONG, FINE GRAINED, THIN BEDDED, FRACTURED; RQD 80%, REC 100%.	883.7	55 – 60 EOB	67		100	NQ2-2											CORE	

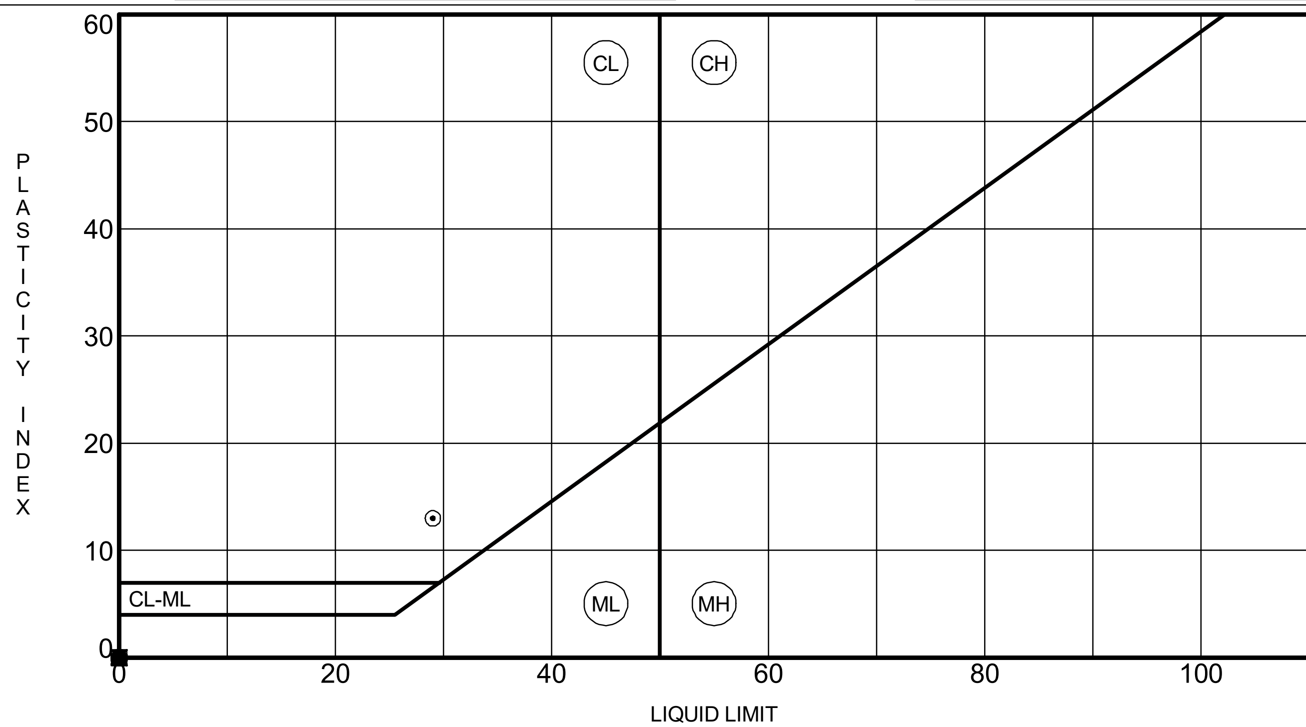
NOTES: WATER ENCOUNTERED AT 8.5 FEET WHILE DRILLING; WATER NOT ENCOUNTERED UPON COMPLETION

ABANDONMENT METHODS, MATERIALS, QUANTITIES: AUGER CUTTINGS MIXED WITH BENTONITE CHIPS

ATTERBERG LIMITS' RESULTS

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT HAS-BRIDGE STREET BRIDGE **PID** 120494
OGE NUMBER N4235509 **PROJECT TYPE** STRUCTURE FOUNDATION



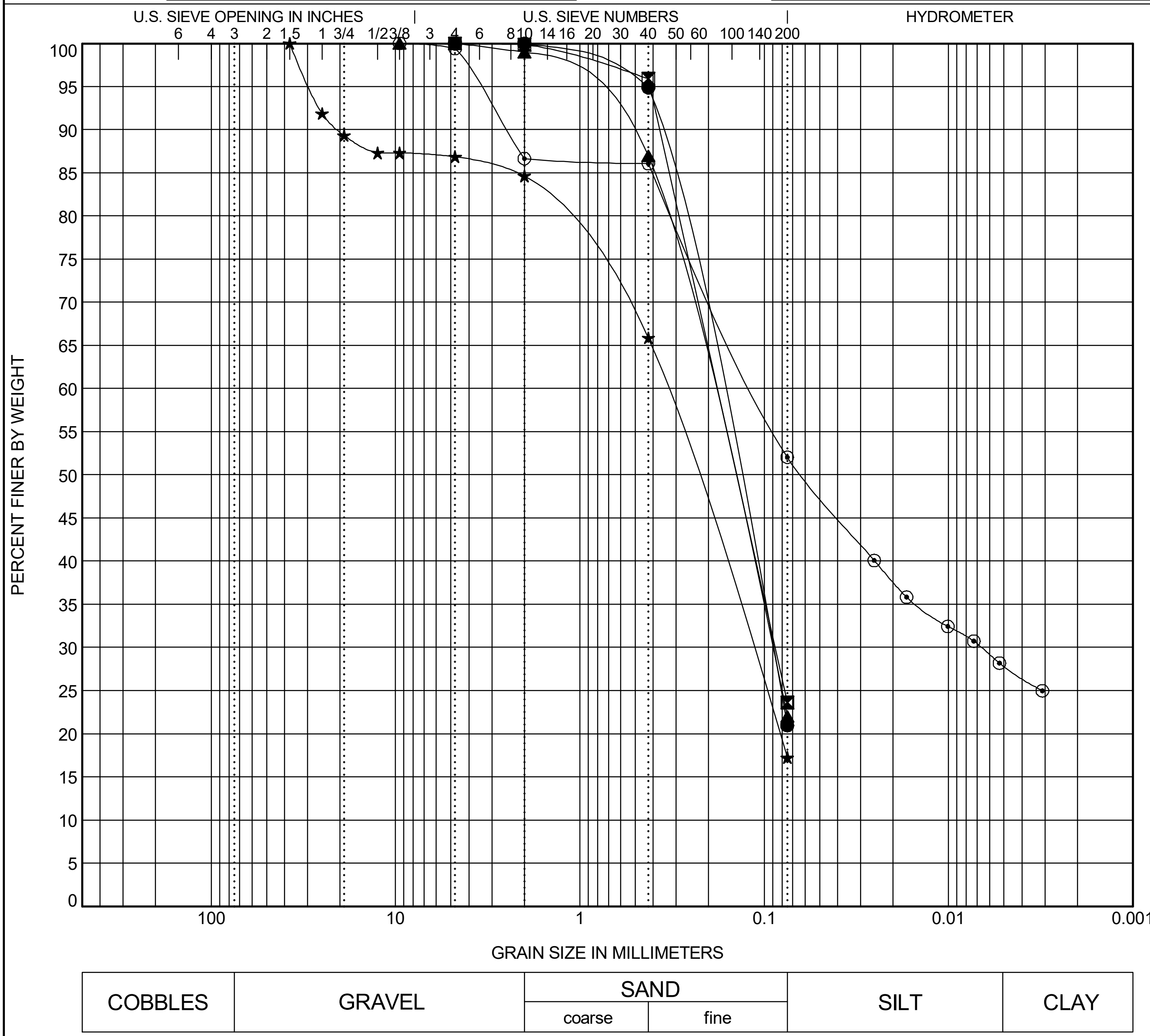
	Specimen Identification		LL	PL	PI	Fines	Classification
●	**B-001-0-24**	**11.5**	**NP**	**NP**	**NP**	**21**	**A-3a ~ SILTY SAND(SM)**
⧗	**B-001-0-24**	**13.0**	**NP**	**NP**	**NP**	**24**	**A-3a ~ SILTY SAND(SM)**
▲	**B-001-0-24**	**14.5**	**NP**	**NP**	**NP**	**22**	**A-3a ~ SILTY SAND(SM)**
★	**B-001-0-24**	**16.0**	**NP**	**NP**	**NP**	**17**	**A-3a ~ SILTY SAND(SM)**
⊙	**B-002-0-24**	**10.0**	**29**	**16**	**13**	**52**	**A-6a ~ SANDY LEAN CLAY(CL)**
✿	**B-002-0-24**	**11.5**	**NP**	**NP**	**NP**	**49**	**A-4a ~ SILTY SAND(SM)**
○	**B-002-0-24**	**13.0**	**NP**	**NP**	**NP**	**45**	**A-4a ~ SILTY SAND(SM)**
△	**B-002-0-24**	**14.5**	**NP**	**NP**	**NP**	**18**	**A-3a ~ SILTY SAND(SM)**

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT HAS-BRIDGE STREET BRIDGE **PID** 120494

OGE NUMBER N4235509 **PROJECT TYPE** STRUCTURE FOUNDATION



Specimen Identification			ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
●	B-001-0-24	11.5	**A-3a ~ SILTY SAND(SM)**	NP	NP	NP
⧗	B-001-0-24	13.0	**A-3a ~ SILTY SAND(SM)**	NP	NP	NP
▲	B-001-0-24	14.5	**A-3a ~ SILTY SAND(SM)**	NP	NP	NP
★	B-001-0-24	16.0	**A-3a ~ SILTY SAND(SM)**	NP	NP	NP
⊙	B-002-0-24	10.0	**A-6a ~ SANDY LEAN CLAY(CL)**	29	16	13

Specimen Identification			D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
●	B-001-0-24	11.5	0.379	0.148	0.093		0	5	74	21			
⧗	B-001-0-24	13.0	0.369	0.141	0.087		0	4	72	24			
▲	B-001-0-24	14.5	0.628	0.158	0.093		1	12	65	22			
★	B-001-0-24	16.0	20.432	0.241	0.118		15	19	49	17			
⊙	B-002-0-24	10.0	2.51	0.062	0.007		13	1	34	24	28		

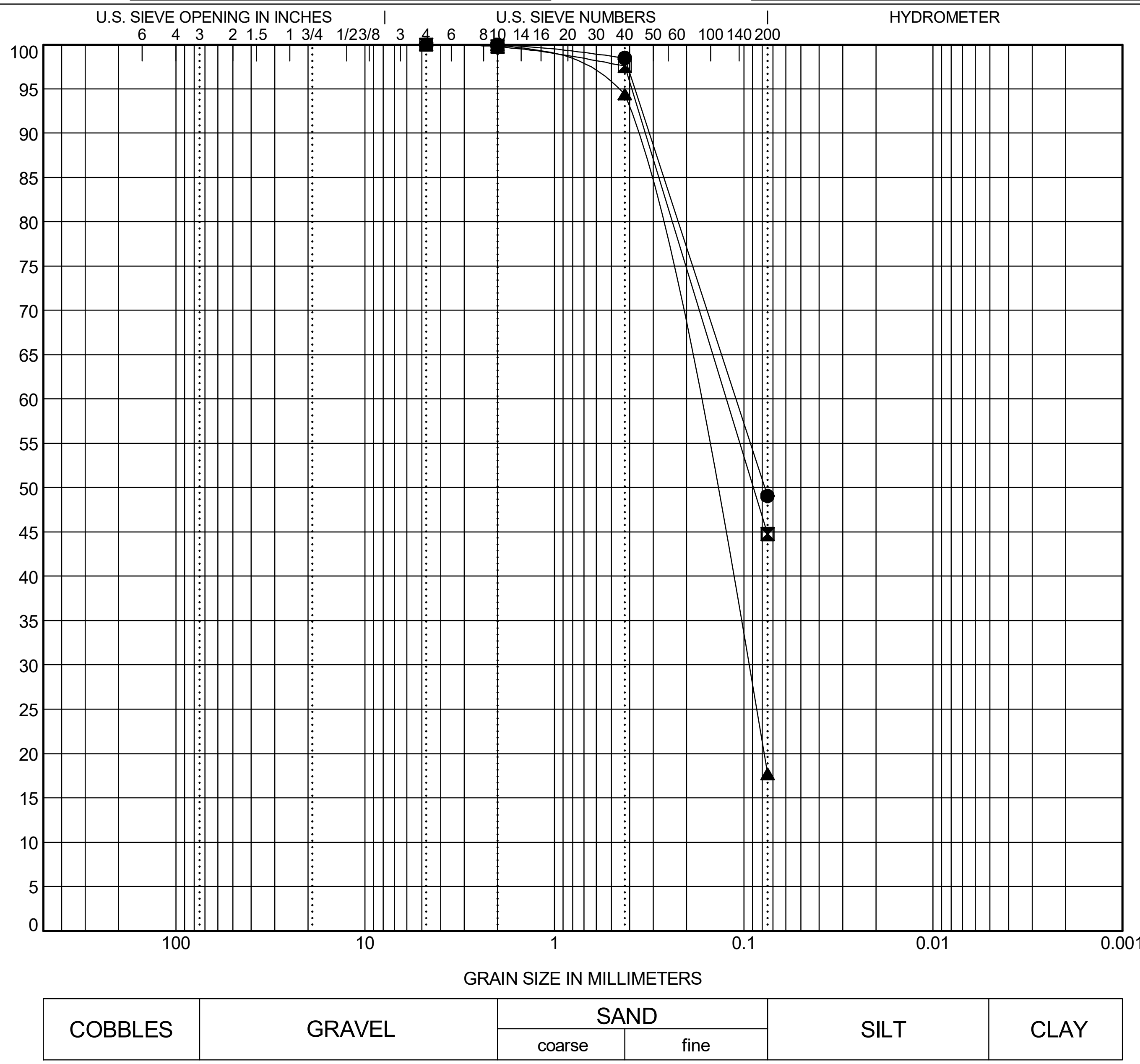
GRAIN SIZE - OH DOT.GDT - 3/18/24 16:09 - E:\PROJECTS\2023\N4235509\WORKING FILES\LABORATORY-FIELD DATA-BORING LOGS\N4235509 HAS-BRIDGE ST BRIDGE ODOT.GPJ

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT HAS-BRIDGE STREET BRIDGE **PID** 120494

OGE NUMBER N4235509 **PROJECT TYPE** STRUCTURE FOUNDATION



	Specimen Identification		ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
●	**B-002-0-24**	**11.5**	**A-4a ~ SILTY SAND(SM)**	**NP**	**NP**	**NP**
⧗	**B-002-0-24**	**13.0**	**A-4a ~ SILTY SAND(SM)**	**NP**	**NP**	**NP**
▲	**B-002-0-24**	**14.5**	**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**	**11.5**	**0.316**	**0.078**			**1**	**1**	**49**	**49**		
⧗	**B-002-0-24**	**13.0**	**0.331**	**0.089**			**0**	**2**	**53**	**45**		
▲	**B-002-0-24**	**14.5**	**0.385**	**0.156**	**0.099**		**0**	**5**	**77**	**18**		

OHIO DEPARTMENT OF TRANSPORTATION
DIVISION OF ENGINEERING

Office of Geotechnical Engineering

B-001-0-24

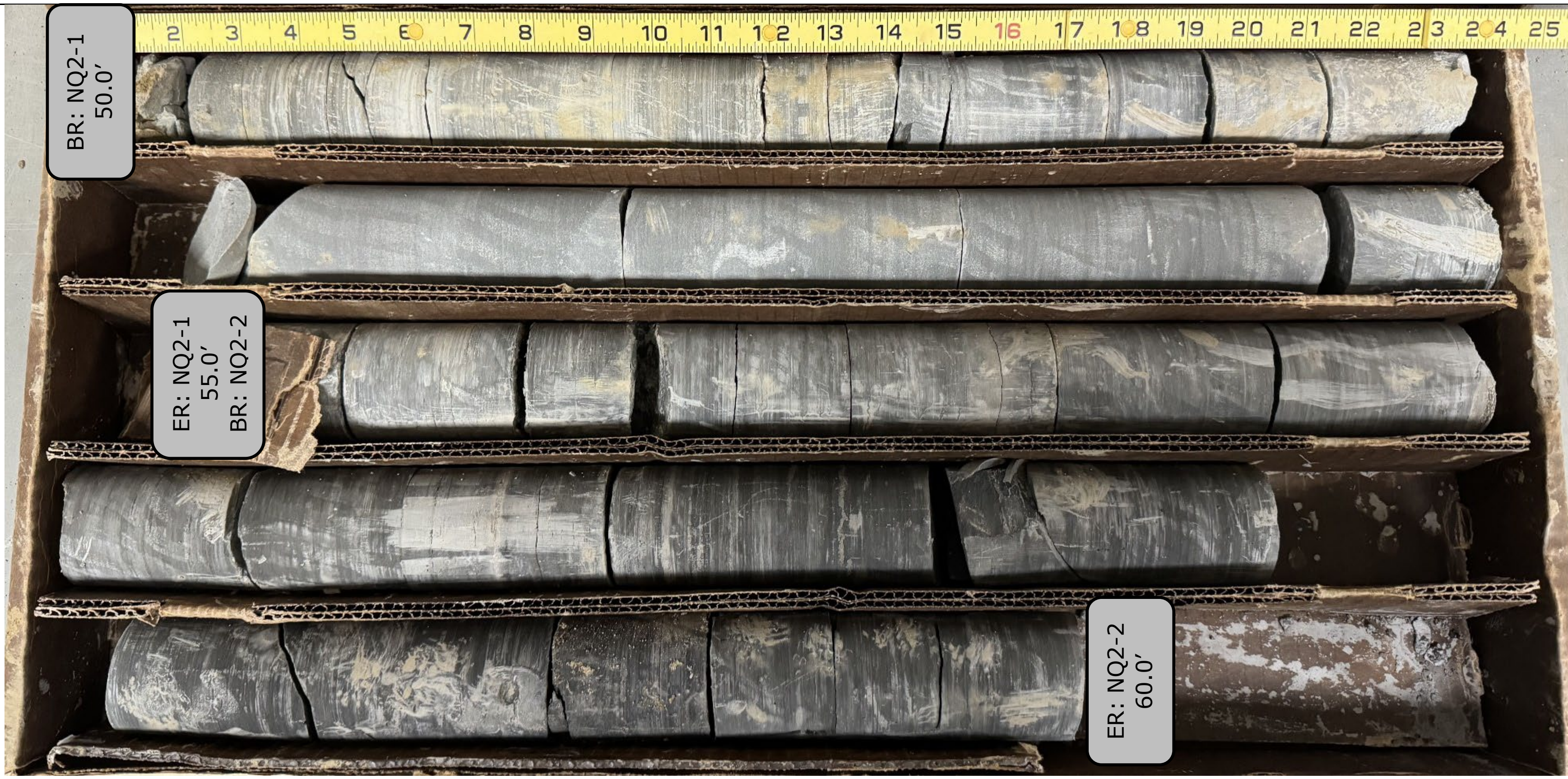
Run #:	Depth		Recovery		RQD	
NQ2-1	40.0′	45.0′	60/60	100%	33/60	55%
NQ2-2	45.0′	50.0′	60/60	100%	28/60	47%

HAS Bridge Street Bridge Replacement

OHIO DEPARTMENT OF TRANSPORTATION
DIVISION OF ENGINEERING

Office of Geotechnical Engineering

B-002-0-23



Run #:	Depth		Recovery		RQD	
NQ2-1	50.0′	55.0′	48/60	80%	28/60	47%
NQ2-2	55.0′	60.0′	60/60	100%	40/60	67%

HAS Bridge Street Bridge Replacement

Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying Stress and Temperatures
ASTM D 7012 Method C

Terracon

Project No.:	N4235509	Tested By:	JMR	Date:	3/4/2024
Project Name:	HAS-Bridge St. Bridge Replacement	Calculated By:	JMR	Date:	3/4/2024
		Checked By:	AA-H	Date:	3/4/2024
Boring No.	B-001-0-24	Run No.:	1		
Depth (ft):	41.5-42.0'				
Description:	SHALE				

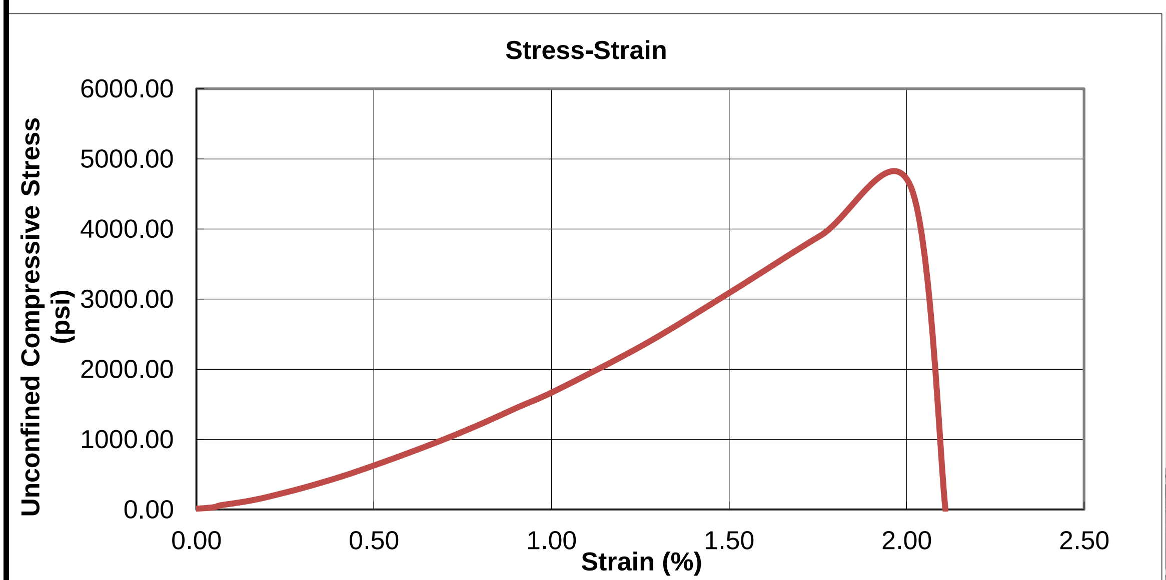
Rock Sample Moisture Condition at Test: [X] As Received [ ] Saturated [ ] See Remarks [ ] Oven Dry

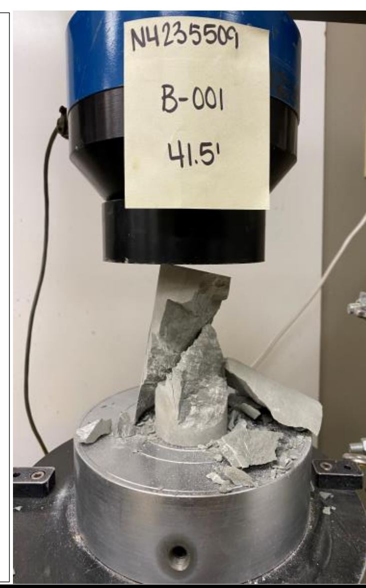
TOLERANCE CHECK

Side Straightness		Maxumum Gap ≤ 0.020 in.						Tolerance Met	No
End Flatness: Max.	Diameter 1a	0.0001	in	Diameter 1b	0.0006	in	≤ 0.0020	Tolerance Met	Yes
End Flatness: Max.	Diameter 2a	0.0011	in	Diameter 2b	0.0010	in	≤ 0.0020	Tolerance Met	Yes
Perpendicularity Slope	Diameter 1a	0.00005		Diameter 1b	0.00056		≤ 0.0043	Tolerance Met	Yes
Perpendicularity Slope	Diameter 2a	0.00030		Diameter 2b	0.00051		≤ 0.0043	Tolerance Met	Yes

	1)	2)	3)	Avg.	
Length (in):	3.987	3.975	3.975	3.979	in
Diameter (in):	1.975	1.980	1.975	1.977	in

Uniaxial Compressive Strength:	**4,831** psi	Mass:	516.55 g
Load:	**14,826** lbs.	**Wet Unit Weight:**	**161.2** pcf
L/D:	**2.0**	**Dry Unit Weight:**	**159.2** pcf
Water Content:	**1.2** %		





Terracon

Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying Stress and Temperatures
ASTM D 7012 Method C

Project No.:	N4235509	Tested By:	JMR	Date:	3/4/2024
Project Name:	HAS-Bridge St. Bridge Replacement	Calculated By:	JMR	Date:	3/4/2024
		Checked By:	AA-H	Date:	3/4/2024
Boring No.	B-002-0-24	Run No.:	1		
Depth (ft):	52-52.5'				
Description:	SHALE				

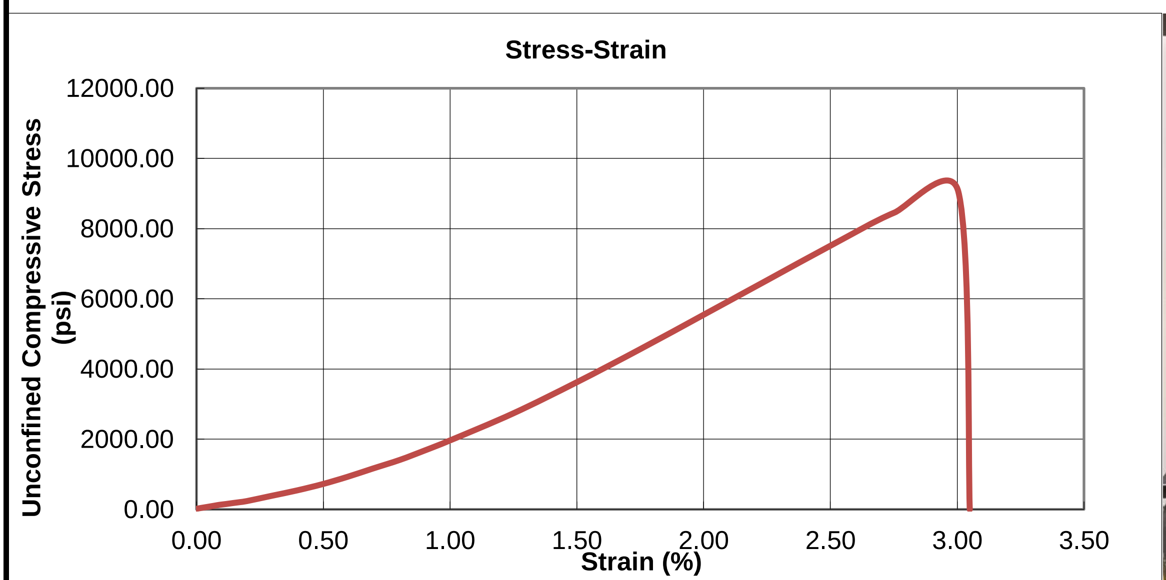
Rock Sample Moisture Condition at Test: [X] As Received [ ] See Remarks [ ] Saturated [ ] Oven Dry

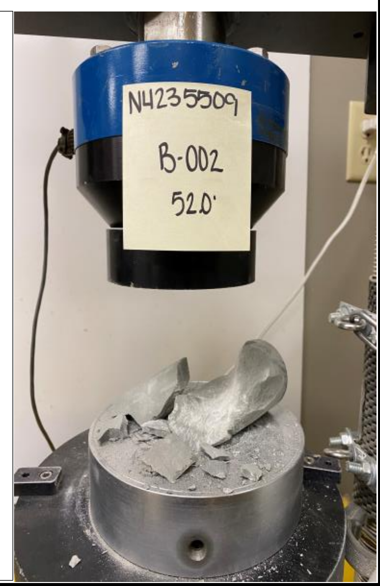
TOLERANCE CHECK

Side Straightness		Maxumum Gap ≤ 0.020 in.							Tolerance Met	No
End Flatness: Max.	Diameter 1a	0.0011	in	Diameter 1b	0.0002	in	≤ 0.0020		Tolerance Met	Yes
End Flatness: Max.	Diameter 2a	0.0010	in	Diameter 2b	0.0004	in	≤ 0.0020		Tolerance Met	Yes
Perpendicularity Slope	Diameter 1a	0.00056		Diameter 1b	0.00050		≤ 0.0043		Tolerance Met	Yes
Perpendicularity Slope	Diameter 2a	0.00010		Diameter 2b	0.00020		≤ 0.0043		Tolerance Met	Yes

Length (in):	1) 4.033	2) 4.035	3) 4.036	Avg.	4.035 in
Diameter (in):	1) 1.979	2) 1.984	3) 1.981	Avg.	1.981 in

Uniaxial Compressive Strength:	**9,509** psi	Mass:	535.31 g
Load:	**29,317** lbs.	**Wet Unit Weight:**	**163.9** pcf
L/D:	**2.0**	**Dry Unit Weight:**	**162.1** pcf
Water Content:	**1.1** %		





Terracon

Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying Stress and Temperatures
ASTM D 7012 Method C

Project No.:	N4235509	Tested By:	JMR	Date:	3/4/2024
Project Name:	HAS-Bridge St. Bridge Replacement	Calculated By:	JMR	Date:	3/4/2024
		Checked By:	AA-H	Date:	3/4/2024
Boring No.	B-002-0-24	Run No.:	1		
Depth (ft):	53.3-53.9'				
Description:	SHALE				

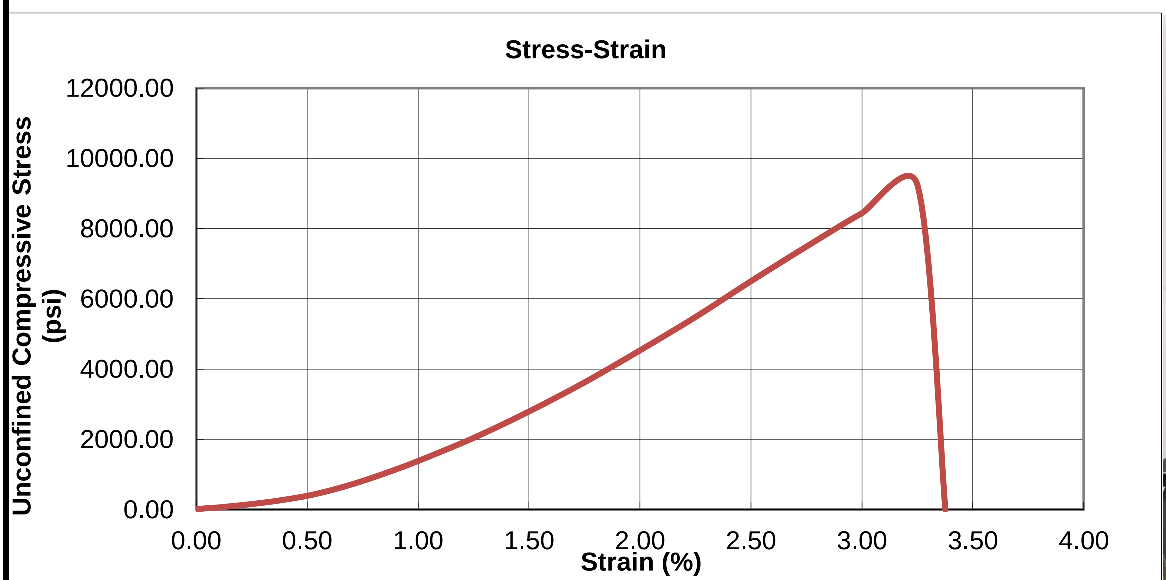
Rock Sample Moisture Condition at Test: [X] As Received [ ] See Remarks [ ] Saturated [ ] Oven Dry

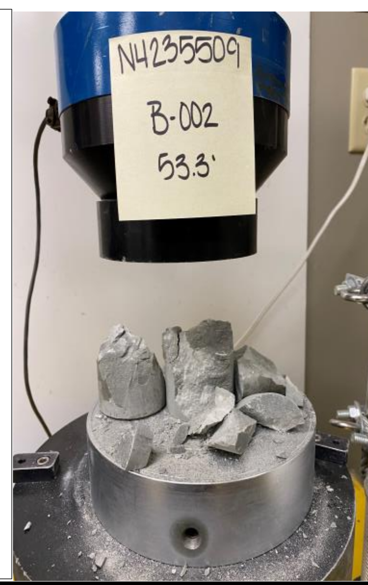
TOLERANCE CHECK

Side Straightness	Maxumum Gap ≤ 0.020 in.							Tolerance Met	No
End Flatness: Max.	Diameter 1a	0.0002	in	Diameter 1b	0.0016	in	≤ 0.0020	Tolerance Met	Yes
End Flatness: Max.	Diameter 2a	0.0004	in	Diameter 2b	0.0002	in	≤ 0.0020	Tolerance Met	Yes
Perpendicularity Slope	Diameter 1a	0.00010		Diameter 1b	0.00020		≤ 0.0043	Tolerance Met	Yes
Perpendicularity Slope	Diameter 2a	0.00081		Diameter 2b	0.00010		≤ 0.0043	Tolerance Met	Yes

Length (in):	1) 4.127	2) 4.129	3) 4.122	Avg.	4.126 in
Diameter (in):	1) 1.989	2) 1.987	3) 1.986	Avg.	1.987 in

Uniaxial Compressive Strength:	**9,670** psi	Mass:	547.10 g
Load:	**29,995** lbs.	**Wet Unit Weight:**	**162.8** pcf
L/D:	**2.1**	**Dry Unit Weight:**	**161.0** pcf
Water Content:	**1.1** %		





SLAKE DURABILITY INDEX (SDI) TEST SUMMARY (ASTM D4644)

Terracon

Client:	EMH&T	Date:	3/12/2024
Project:	HAS-Bridge St. Bridge	Project Number:	N4235509
Location:	Bowerstown, OH		

Boring No.			B-002-0-24
Depth (ft)			52.5
Tare Weight:			843.8
Moist weight (Sample+Tare):			1702.33
Dry weight (Sample+Tare):			1700.10
Natural Moisture Content (%):			0.3
After Cycle No. 1			
Temperature (°F)			Dry Weight (Sample+Tare)
Start	End	Average	
91.8	89.0	90.4	1678.3
After Cycle No. 2			
Temperature (°F)			Dry Weight (Sample+Tare)
Start	End	Average	
77.7	80.2	79.0	1655.3
SLAKE DURABILITY INDEX:			**94.8**
Fragments Retained - Type:			**I**

Before Test

After Test

Material Description:	**SHALE**
Notes/Comments:	

Appendix C – Supporting Information

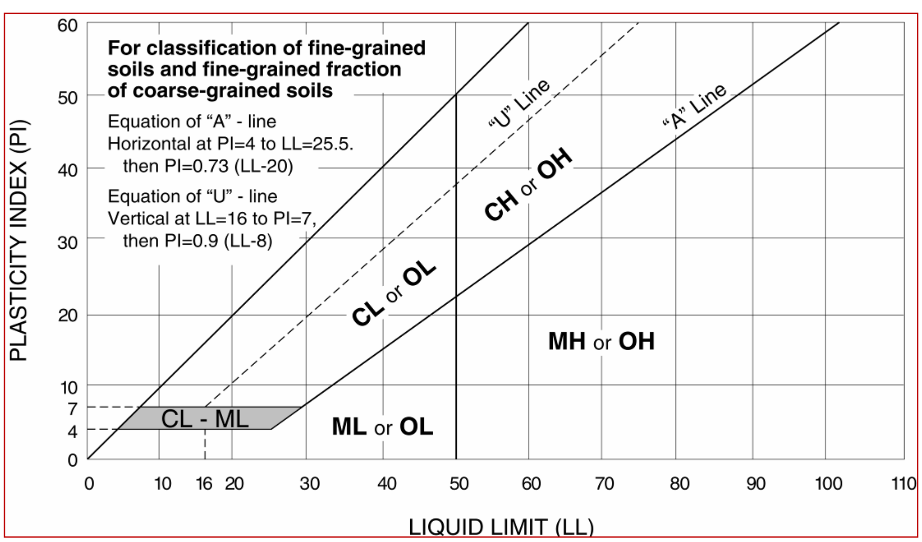
Contents:

Note: All attachments are one page unless noted above.

Unified Soil Classification System

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]				Soil Classification: Group Symbol	Soil Classification: Group Name [B]
Coarse-Grained Soils: More than 50% retained on No. 200 sieve	**Gravels:** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels:** Less than 5% fines [C]	Cu≥4 and 1≤Cc≤3 [E]	GW	Well-graded gravel [F]
			Cu<4 and/or [Cc<1 or Cc>3.0] [E]	GP	Poorly graded gravel [F]
		Gravels with Fines: More than 12% fines [C]	Fines classify as ML or MH	GM	Silty gravel [F, G, H]
			Fines classify as CL or CH	GC	Clayey gravel [F, G, H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands:** Less than 5% fines [D]	Cu≥6 and 1≤Cc≤3 [E]	SW	Well-graded sand [I]
			Cu<6 and/or [Cc<1 or Cc>3.0] [E]	SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH	SM	Silty sand [G, H, I]
			Fines classify as CL or CH	SC	Clayey sand [G, H, I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	PI > 7 and plots above "A" line [J]	CL	Lean clay [K, L, M]
			PI < 4 or plots below "A" line [J]	ML	Silt [K, L, M]
		Organic:	$\frac{LL\ oven\ dried}{LL\ not\ dried} < 0.75$	OL	Organic clay [K, L, M, N]
					Organic silt [K, L, M, O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above "A" line	CH	Fat clay [K, L, M]
			PI plots below "A" line	MH	Elastic silt [K, L, M]
		Organic:	$\frac{LL\ oven\ dried}{LL\ not\ dried} < 0.75$	OH	Organic clay [K, L, M, P]
					Organic silt [K, L, M, Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-inch (75-mm) sieve.
[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.
[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay.
[E] $Cu = D_{60}/D_{10}$ $\quad Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$
[F] If soil contains ≥ 15% sand, add "with sand" to group name.
[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
[H] If fines are organic, add "with organic fines" to group name.
[I] If soil contains ≥ 15% gravel, add "with gravel" to group name.
[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.
[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.
[L] If soil contains ≥ 30% plus No. 200 predominantly sand, add "sandy" to group name.
[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.
[N] PI ≥ 4 and plots on or above "A" line.
[O] PI < 4 or plots below "A" line.
[P] PI plots on or above "A" line.
[Q] PI plots below "A" line.



APPENDIX A.1 - ODOT Quick Reference for Visual Description of Soils

1) STRENGTH OF SOIL:

Non-Cohesive (granular) Soils - Compactness

Description	Blows Per Ft.
Very Loose	≤ 4
Loose	5 – 10
Medium Dense	11 – 30
Dense	31 – 50
Very Dense	> 50

Cohesive (fine grained) Soils - Consistency

Description	Qu (TSF)	Blows Per Ft.	Hand Manipulation
Very Soft	<0.25	<2	Easily penetrates 2" by fist
Soft	0.25-0.5	2 - 4	Easily penetrates 2" by thumb
Medium Stiff	0.5-1.0	5 - 8	Penetrates by thumb with moderate effort
Stiff	1.0-2.0	9 - 15	Readily indents by thumb, but not penetrate
Very Stiff	2.0-4.0	16 - 30	Readily indents by thumbnail
Hard	>4.0	>30	Indent with difficulty by thumbnail

2) COLOR :

If a color is a uniform color throughout, the term is single, modified by an adjective such as light or dark. If the predominate color is shaded by a secondary color, the secondary color precedes the primary color. If two major and distinct colors are swirled throughout the soil, the colors are modified by the term "mottled"

3) PRIMARY COMPONENT

Use **DESCRIPTION** from ODOT Soil Classification Chart on Back

4) COMPONENT MODIFIERS:

Description	Percentage By Weight
Trace	0% - 10%
Little	>10% - 20%
Some	>20% - 35%
"And"	>35%

5) Soil Organic Content

Description	% by Weight
Slightly Organic	2% - 4%
Moderately Organic	4% - 10%
Highly Organic	> 10%

6) Relative Visual Moisture

Description	Criteria	
	Cohesive Soil	**Non-cohesive Soils**
Dry	Powdery; Cannot be rolled; Water content well below the plastic limit	No moisture present
Damp	Leaves very little moisture when pressed between fingers; Crumbles at or before rolled to 1/8"; Water content below plastic limit	Internal moisture, but no to little surface moisture
Moist	Leaves small amounts of moisture when pressed between fingers; Rolled to 1/8" or smaller before crumbling; Water content above plastic limit to -3% of the liquid limit	Free water on surface, moist (shiny) appearance
Wet	Very mushy; Rolled multiple times to 1/8" or smaller before crumbles; Near or above the liquid limit	Voids filled with free water, can be poured from split spoon.

CLASSIFICATION OF SOILS

Ohio Department of Transportation

(The classification of a soil is found by proceeding from top to bottom of the chart. The first classification that the test data fits is the correct classification.)

SYMBOL	DESCRIPTION	Classifcation AASHTO	Classifcation OHIO	LL_O/LL × 100*	% Pass #40	% Pass #200	Liquid Limit (LL)	Plastic Index (PI)	Group Index Max.	REMARKS
	Gravel and/or Stone Fragments	A-1-a			30 Max.	15 Max.		6 Max.	0	Min. of 50% combined gravel, cobble and boulder sizes
	Gravel and/or Stone Fragments with Sand	A-1-b			50 Max.	25 Max.		6 Max.	0	
F S	Fine Sand	A-3			51 Min.	10 Max.	NON-PLASTIC		0	
	Coarse and Fine Sand	--	A-3a			35 Max.		6 Max.	0	Min. of 50% combined coarse and fine sand sizes
	Gravel and/or Stone Fragments with Sand and Silt	A-2-4				35 Max.	40 Max.	10 Max.	0	
		A-2-5					41 Min.			
	Gravel and/or Stone Fragments with Sand, Silt and Clay	A-2-6				35 Max.	40 Max.	11 Min.	4	
		A-2-7					41 Min.			
	Sandy Silt	A-4	A-4a	76 Min.		36 Min.	40 Max.	10 Max.	8	Less than 50% silt sizes
	Silt	A-4	A-4b	76 Min.		50 Min.	40 Max.	10 Max.	8	50% or more silt sizes
	Elastic Silt and Clay	A-5		76 Min.		36 Min.	41 Min.	10 Max.	12	
	Silt and Clay	A-6	A-6a	76 Min.		36 Min.	40 Max.	11 - 15	10	
	Silty Clay	A-6	A-6b	76 Min.		36 Min.	40 Max.	16 Min.	16	
	Elastic Clay	A-7-5		76 Min.		36 Min.	41 Min.	≤LL-30	20	
	Clay	A-7-6		76 Min.		36 Min.	41 Min.	>LL-30	20	
	Organic Silt	A-8	A-8a	75 Max.		36 Min.				W/o organics would classify as A-4a or A-4b
	Organic Clay	A-8	A-8b	75 Max.		36 Min.				W/o organics would classify as A-5, A-6a, A-6b, A-7-5 or A-7-6

MATERIAL CLASSIFIED BY VISUAL INSPECTION

- Sod and Topsoil
- Pavement or Base
- Uncontrolled Fill (Describe)
- Bouldery Zone
- P Peat

* Only perform the oven-dried liquid limit test and this calculation if organic material is present in the sample.

APPENDIX A.2 – ODOT Quick Reference Guide for Rock Description

1: ROCK TYPE: Common rock types are: Claystone; Coal; Dolomite; Limestone; Sandstone; Siltstone; & Shale.

2: COLOR: To be determined when rock is wet. When using the GSA Color charts use only Name, not code.

3: WEATHERING

Description	Field Parameter
Unweathered	No evidence of any chemical or mechanical alternation of the rock mass. Mineral crystals have a bright appearance with no discoloration. Fractures show little or no staining on surfaces.
Slightly weathered	Slight discoloration of the rock surface with minor alterations along discontinuities. Less than 10% of the rock volume presents alteration.
Moderately weathered	Portions of the rock mass are discolored as evident by a dull appearance. Surfaces may have a pitted appearance with weathering "halos" evident. Isolated zones of varying rock strengths due to alteration may be present. 10 to 15% of the rock volume presents alterations.
Highly weathered	Entire rock mass appears discolored and dull. Some pockets of slightly too moderately weathered rock may be present and some areas of severely weathered materials may be present.
Severely weathered	Majority of the rock mass reduced to a soil-like state with relic rock structure discernable. Zones of more resistant rock may be present, but the material can generally be molded and crumbled by hand pressures.

4: TEXTURE

Component		Grain Diameter
Boulder		>12"
Cobble		3"-12"
Gravel		0.08"-3"
Sand	Coarse	0.02"-0.08"
	Medium	0.01"-0.02"
	Fine	0.005"-0.01"
	Very Fine	0.003"-0.005"

5: RELATIVE STRENGTH

Description	Field Parameter
Very Weak	Core can be carved with a knife and scratched by fingernail. Can be excavated readily with a point of a pick. Pieces 1 inch or more in thickness can be broken by finger pressure.
Weak	Core can be grooved or gouged readily by a knife or pick. Can be excavated in small fragments by moderate blows of a pick point. Small, thin pieces can be broken by finger pressure.
Slightly Strong	Core can be grooved or gouged 0.05 inch deep by firm pressure of a knife or pick point. Can be excavated in small chips to pieces about 1-inch maximum size by hard blows of the point of a geologist's pick.
Moderately Strong	Core can be scratched with a knife or pick. Grooves or gouges to ¼" deep can be excavated by hand blows of a geologist's pick. Requires moderate hammer blows to detach hand specimen.
Strong	Core can be scratched with a knife or pick only with difficulty. Requires hard hammer blows to detach hand specimen. Sharp and resistant edges are present on hand specimen.
Very Strong	Core cannot be scratched by a knife or sharp pick. Breaking of hand specimens requires hard repeated blows of the geologist hammer.
Extremely strong	Core cannot be scratched by a knife or sharp pick. Chipping of hand specimens requires hard repeated blows of the geologist hammer.

6: BEDDING

Description	Thickness
Very Thick	>36"
Thick	18" – 36"
Medium	10" – 18"
Thin	2" – 10"
Very Thin	0.4" – 2"
Laminated	0.1" – 0.4"
Thinly Laminated	<0.1"

7: DESCRIPTORS

Arenaceous – sandy	Argillaceous - clayey	Brecciated – contains angular to subangular gravel
Calcareous - contains calcium carbonate	Carbonaceous - contains carbon	Cherty- contains chert fragments
Conglomeritic - contains rounded to subrounded gravel	Crystalline – contains crystalline structure	Dolomitic- contains calcium/magnesium carbonate
Ferriferous – contains iron	Fissile – thin planner partings	Fossiliferous – contains fossils
Friable – easily broken down	Micaceous – contains mica	Pyritic – contains pyrite
Siliceous – contains silica	Stylolitic – contain stylotites (suture like structure)	Vuggy – contains openings

APPENDIX A.2 – ODOT Quick Reference Guide for Rock Description

8: Discontinuities

a: Discontinuity Types

Type	Parameters
Fault	Fracture which expresses displacement parallel to the surface that does not result in a polished surface.
Joint	Planar fracture that does not express displacement. Generally occurs at regularly spaced intervals.
Shear	Fracture which expresses displacement parallel to the surface that results in polished surfaces or slickensides.
Bedding	A surface produced along a bedding plane.
Contact	A surface produced along a contact plane. (generally not seen in Ohio)

b: Degree of Fracturing

Description	Spacing
Unfractured	> 10 ft.
Intact	3 ft. – 10 ft.
Slightly fractured	1 ft. – 3 ft.
Moderately fractured	4 in. – 12 in.
Fractured	2 in. – 4 in.
Highly fractured	< 2 in.

c: Aperture Width

Description	Spacing
Open	> 0.2 in.
Narrow	0.05 in. - 0.2 in.
Tight	<0.05 in.

d: Surface Roughness

Description	Criteria
Very Rough	Near vertical steps and ridges occur on the discontinuity surface.
Slightly Rough	Asperities on the discontinuity surface are distinguishable and can be felt.
Slickensided	Surface has a smooth, glassy finish with visual evidence of striation.

11: Recovery

$Run\ Recovery = \left(\frac{R_R}{L_R}\right) * 100$	$Unit\ Recovery = \left(\frac{R_U}{L_U}\right) * 100$
L_R = Run Length R_R – Run Recovery	L_U = Rock Unit Length R_U – Rock Unit Recovery

9: GSI Description

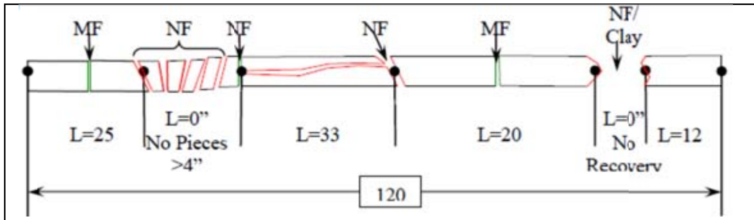
a: Structure

Description	Parameters
Intact or Massive	Intact rock with few widely spaced discontinuities
Blocky	Well interlocked undisturbed rock mass consisting of cubical blocks formed by three interesting discontinuity sets
Very Blocky	Interlocked, partially disturbed mass with multi-faceted angular blocks formed by 4 or more joint sets
Blocky/Disturbed/ Seamy	Angular blocks formed by many intersecting discontinuity sets, Persistence of bedding planes
Disintegrated	Poorly interlocked, heavily broken rock mass with mixture of angular and rounded rock pieces
Laminated/Sheared	Lack of blockiness due to close spacing of weak shear planes

b: Surface Condition

Description	Parameters
Very Good	Very rough, fresh unweathered surfaces
Good	Rough, slightly weathered, iron stained surface
Fair	Smooth, moderately weathered and altered surfaces
Poor	Slickensided, highly weathered surface with compact coatings or fillings or angular fragments
Very Poor	Slickensided, highly weathered surfaces with soft clay coating or fillings

10: RQD



$$RQD = \left(\frac{\sum Length\ \ of\ \ Pieces > 4inches}{Total\ Length\ of\ Core}\right) * 100$$

$$RQD = \left(\frac{25+33+20+12}{120}\right) * 100 = 75\%$$

Client:	EMH&T
Project Name:	HAS-Bridge Street Bridge Replacement
Project No.#	N4235509
Date:	4/15/24
Bedrock Type:	Sandstone (EL 901.5' - EL 899.0')
Boring ID:	B-001-0-24
Calculated By:	SG
Checked By:	NKM / AA-H

Erodibility Index (K) for Bedrock

GDM Section 1302.1.3, BDM Section 305.2.1.2.b(B.6.a) and HEC 18 Equation 4.17

Ms (Qu>10 Mpa)	Kd	Kb
24.13793103	0.25	8.98204

Qu (psi)	Qu (Mpa)
3500	24.13793

BDM Section 305.2.1.2.b(B) and HEC 18 Section 4.7.2

RQD (%)	30	
Jn	3.34	HEC 18 Section 4.7.2 Table 4.23
Jr	1.5	HEC 18 Section 4.7.2 Table 4.24
Ja	6	HEC 18 Section 4.7.2 Table 4.25
Js	0.9	HEC 18 Section 4.7.2 Table 4.26

K
48.7817

Ms = Qu for Qu ≥ 10-MPa, or Ms = (0.78) $Qu^{1.05}$ for Qu < 10-Mpa

$K = (M_s)(K_b)(K_d)(J_s)$, $K_b = RQD/J_n \geq 0.10$, and $K_d = J_r/J_a$

Where RQD = 0, Block Size Parameter $K_b = RQD/J_n = 0$, and subsequently Erodibility Index K = 0. In the scour calculations that depend on K, it is in the denominator, and K = 0 will result in a divide by zero error.

If RQD = 0, do not set Block Size Parameter K_b = 0 and subsequently Erodibility Index K = 0. In this case, set the minimum value of K_b = 0.010. In the scour calculations that depend on K, it is in the denominator, and K = 0 will results in divide by zero error.

Client:	EMH&T
Project Name:	HAS-Bridge Street Bridge Replacement
Project No.#	N4235509
Date:	3/29/2024
Bedrock Type:	Shale (EL 899.0' - EL 890.0')
Boring ID:	B-001-0-24
Calculated By:	SG
Checked By:	NKM / AA-H

Erodibility Index (K) for Bedrock

GDM Section 1302.1.3, BDM Section 305.2.1.2.b(B.6.a) and HEC 18 Equation 4.17

Ms (Qu>10 Mpa)	Kd	Kb
33.31724138	0.25	16.1677

Qu (psi)	Qu (Mpa)
4831	33.31724

BDM Section 305.2.1.2.b(B) and HEC 18 Section 4.7.2

RQD (%)	54	
Jn	3.34	HEC 18 Section 4.7.2 Table 4.23
Jr	1.5	HEC 18 Section 4.7.2 Table 4.24
Ja	6	HEC 18 Section 4.7.2 Table 4.25
Js	1	HEC 18 Section 4.7.2 Table 4.26

K
134.665

Ms = Qu for Qu ≥ 10-MPa, or Ms = (0.78) $Qu^{1.05}$ for Qu < 10-Mpa

$K = (M_s)(K_b)(K_d)(J_s)$, $K_b = RQD/J_n \geq 0.10$, and $K_d = J_r/J_a$

Where RQD = 0, Block Size Parameter $K_b = RQD/J_n$ = 0, and subsequently Erodibility Index K = 0. In the scour calculations that depend on K, it is in the denominator, and K = 0 will result in a divide by zero error.

If RQD = 0, do not set Block Size Parameter K_b = 0 and subsequently Erodibility Index K = 0. In this case, set the minimum value of K_b = 0.010. In the scour calculations that depend on K, it is in the denominator, and K = 0 will results in divide by zero error.

Client:	EMH&T
Project Name:	HAS-Bridge Street Bridge Replacement
Project No.#	N4235509
Date:	4/15/24
Bedrock Type:	Shale (EL 894.0' - EL 884.0')
Boring ID:	B-002-0-24
Calculated By:	SG
Checked By:	NKM / AA-H

Erodibility Index (K) for Bedrock

GDM Section 1302.1.3, BDM Section 305.2.1.2.b(B.6.a) and HEC 18 Equation 4.17

Ms (Qu>10 Mpa)	Kd	Kb
65.51724138	0.25	17.0659

BDM Section 305.2.1.2.b(B) and HEC 18 Section 4.7.2

RQD (%)	57	
Jn	3.34	HEC 18 Section 4.7.2 Table 4.23
Jr	1.5	HEC 18 Section 4.7.2 Table 4.24
Ja	6	HEC 18 Section 4.7.2 Table 4.25
Js	1	HEC 18 Section 4.7.2 Table 4.26

Qu (psi)	Qu (Mpa)
9500	65.51724

K
279.5272

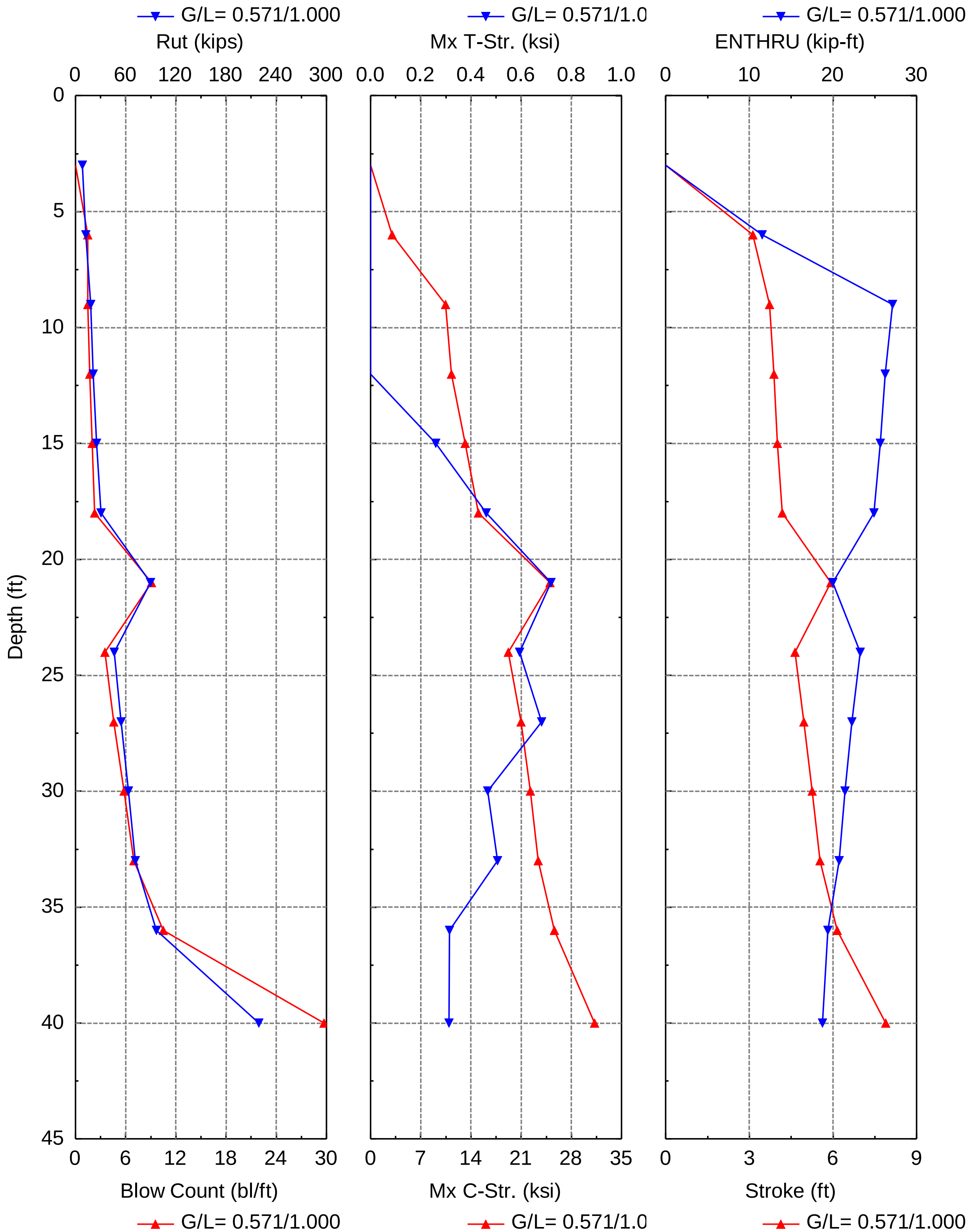
Ms = Qu for Qu ≥ 10-MPa, or Ms = (0.78) $Qu^{1.05}$ for Qu < 10-Mpa

$K = (M_s)(K_b)(K_d)(J_s)$, $K_b = RQD/J_n \geq 0.10$, and $K_d = J_r/J_a$

Where RQD = 0, Block Size Parameter $K_b = RQD/J_n = 0$, and subsequently Erodibility Index K = 0. In the scour calculations that depend on K, it is in the denominator, and K = 0 will result in a divide by zero error.

If RQD = 0, do not set Block Size Parameter K_b = 0 and subsequently Erodibility Index K = 0. In this case, set the minimum value of K_b = 0.010. In the scour calculations that depend on K, it is in the denominator, and K = 0 will results in divide by zero error.

Driveability Analysis Summary



Gain/Loss Factor at Shaft/Toe = 0.571/1.000

Depth	Rut	Rshaft	Rtoe	Blow Ct	Mx C-Str	Mx T-Str.	Stroke	ENTHRU	Hammer
ft	kips	kips	kips	bl/ft	ksi	ksi	ft	kip-ft	-
3.0	8.4	3.8	4.6	0.0	0.000	0.000	0.00	0.0	D 19-42
6.0	12.4	7.8	4.6	1.5	2.977	0.000	3.12	11.5	D 19-42
9.0	18.3	9.7	8.6	1.5	10.425	0.000	3.72	27.1	D 19-42
12.0	21.4	12.4	9.0	1.7	11.253	0.000	3.88	26.2	D 19-42
15.0	25.4	16.4	9.0	2.0	13.158	0.259	4.01	25.6	D 19-42
18.0	30.6	21.5	9.0	2.3	15.039	0.461	4.17	24.9	D 19-42
21.0	89.7	27.8	61.9	9.1	24.977	0.719	5.91	20.0	D 19-42
24.0	46.2	37.1	9.2	3.5	19.186	0.593	4.63	23.2	D 19-42
27.0	54.7	45.6	9.2	4.6	20.953	0.681	4.95	22.2	D 19-42
30.0	63.2	54.0	9.2	5.8	22.266	0.467	5.26	21.4	D 19-42
33.0	71.6	62.5	9.2	7.0	23.365	0.505	5.53	20.7	D 19-42
36.0	96.5	69.0	27.5	10.5	25.649	0.314	6.14	19.3	D 19-42
40.0	219.3	81.3	138.0	29.7	31.213	0.312	7.89	18.7	D 19-42

Total driving time: 5 minutes; Total Number of Blows: 212 (starting at penetration 3.0 ft)

GRLWEAP: Wave Equation Analysis of Pile Foundations

Driven Steel Pile (B-001-0-24) + HP 10x42 1/15/2025
TSVC 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	Soil Type	Spec. Wt	Su	Phi	Unit Rs	Unit Rt
ft	-	lb/ft³	ksf	°	ksf	ksf
0.0	Clay	124.0	0.7	0.0	0.59	6.75
6.0	Clay	124.0	0.7	0.0	0.59	6.75
6.0	Sand	110.0	0.0	28.0	0.19	9.19
11.5	Sand	110.0	0.0	28.0	0.35	13.32
11.5	Sand	110.0	0.0	28.0	0.35	13.32
16.0	Sand	110.0	0.0	28.0	0.47	13.32
16.0	Sand	115.0	0.0	30.0	0.54	13.32
20.5	Sand	115.0	0.0	30.0	0.61	13.32
20.5	Sand	127.0	0.0	35.0	0.90	89.89
23.5	Sand	127.0	0.0	35.0	0.98	97.73
23.5	Clay	126.0	1.5	0.0	1.50	13.50
33.5	Clay	126.0	1.5	0.0	1.50	13.50
33.5	Clay	131.0	4.5	0.0	1.08	40.50
38.5	Clay	131.0	4.5	0.0	1.08	40.50
38.5	Sand	132.0	0.0	37.0	1.74	203.27
40.0	Sand	132.0	0.0	37.0	1.79	203.27

PILE INPUT

Uniform Pile		Pile Type:	H Pile
Pile Length: (ft)	45.000	Pile Penetration: (ft)	40.000
Pile Size: (ft)	0.84	Toe Area: (in²)	97.73

Pile Profile

Lb Top	X-Area	E-Modulus	Spec. Wt	Perim.	Crit. Index
ft	in²	ksi	lb/ft³	ft	-
0.0	12.4	30,000.0	492.0	3.3	0
45.0	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	Ram L.	Ram Ar.	Rtd. Stk	Effic.	Rtd. Energy
-	kips	in	in²	ft	-	kip-ft
41	4.000	129.1	124.7	10.8	0.80	43.2

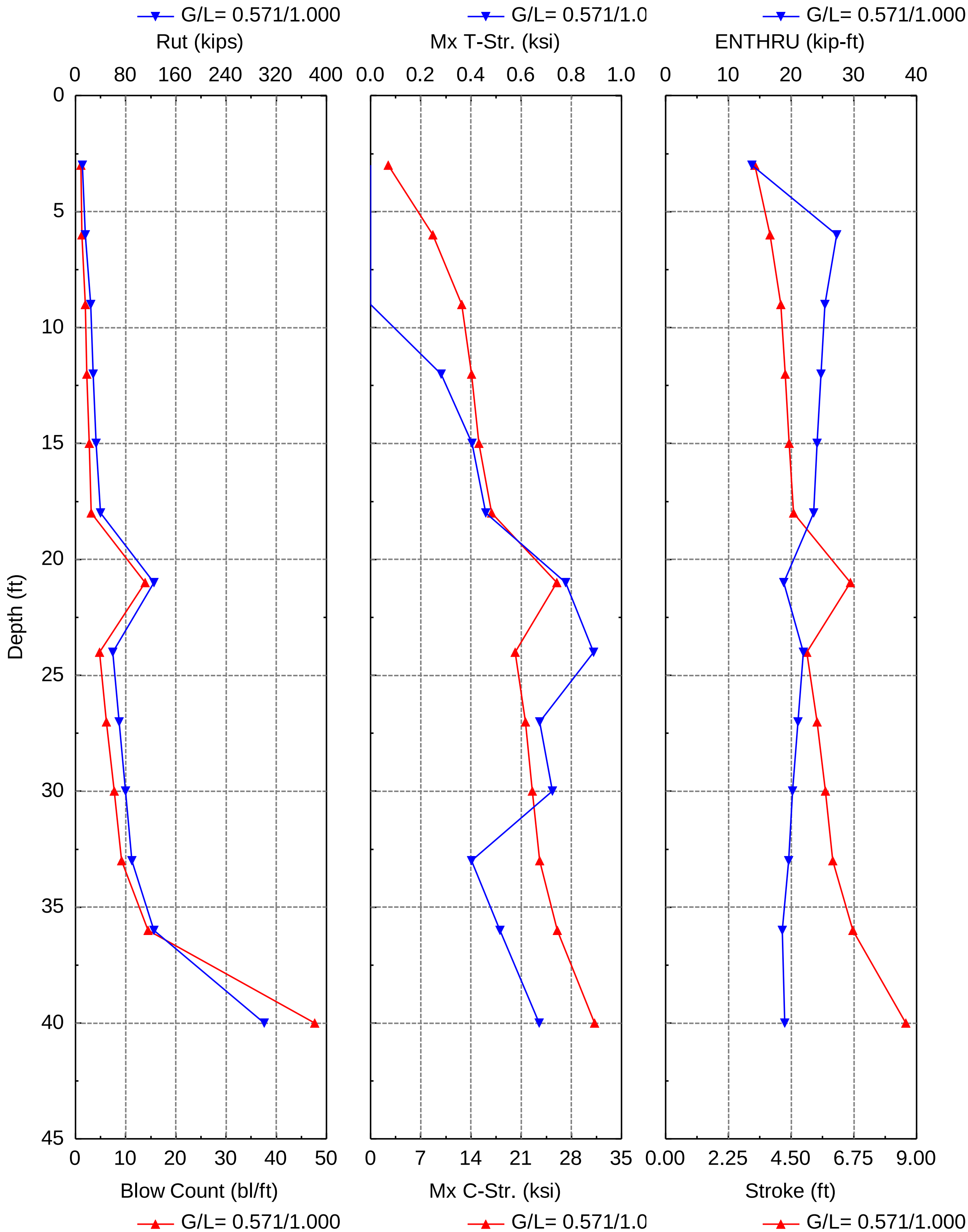
DRIVE SYSTEM FOR DELMAG D 19-42-OED

Type	X-Area	E-Modulus	Thickness	COR	Round-out	Stiffness
-	in²	ksi	in	-	in	kips/in
Hammer C.	227.000	530.000	2.000	0.800	0.120	60155.550
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	in²
0.0	0.6	6.7	0.10	0.13	0.15	0.15	1.5	6.0	168.0	97.7
2.0	0.6	6.7	0.10	0.13	0.15	0.15	1.5	6.0	168.0	97.7
4.0	0.6	6.7	0.10	0.13	0.15	0.15	1.5	6.0	168.0	97.7
6.0	0.6	6.7	0.10	0.13	0.15	0.15	1.5	6.0	168.0	97.7
6.0	0.2	9.2	0.10	0.17	0.10	0.15	1.2	6.0	24.0	97.7
7.8	0.2	11.7	0.10	0.17	0.10	0.15	1.2	6.0	24.0	97.7
9.7	0.3	13.3	0.10	0.17	0.10	0.15	1.2	6.0	24.0	97.7
11.5	0.3	13.3	0.10	0.17	0.10	0.15	1.2	6.0	24.0	97.7
11.5	0.3	13.3	0.10	0.17	0.05	0.15	1.0	6.0	1.0	97.7
13.8	0.4	13.3	0.10	0.17	0.05	0.15	1.0	6.0	1.0	97.7
16.0	0.5	13.3	0.10	0.17	0.05	0.15	1.0	6.0	1.0	97.7
16.0	0.5	13.3	0.10	0.14	0.05	0.15	1.0	6.0	1.0	97.7
18.3	0.6	13.3	0.10	0.14	0.05	0.15	1.0	6.0	1.0	97.7
20.5	0.6	13.3	0.10	0.14	0.05	0.15	1.0	6.0	1.0	97.7
20.5	0.9	89.9	0.10	0.09	0.05	0.15	1.0	6.0	1.0	97.7
22.0	0.9	93.8	0.10	0.09	0.05	0.15	1.0	6.0	1.0	97.7
23.5	1.0	97.7	0.10	0.09	0.05	0.15	1.0	6.0	1.0	97.7
23.5	1.5	13.5	0.10	0.11	0.20	0.15	1.8	6.0	168.0	97.7
25.2	1.5	13.5	0.10	0.11	0.20	0.15	1.8	6.0	168.0	97.7
26.8	1.5	13.5	0.10	0.11	0.20	0.15	1.8	6.0	168.0	97.7
28.5	1.5	13.5	0.10	0.11	0.20	0.15	1.8	6.0	168.0	97.7
30.2	1.5	13.5	0.10	0.11	0.20	0.15	1.8	6.0	168.0	97.7
31.8	1.5	13.5	0.10	0.11	0.20	0.15	1.8	6.0	168.0	97.7
33.5	1.5	13.5	0.10	0.11	0.20	0.15	1.8	6.0	168.0	97.7
33.5	1.1	40.5	0.10	0.08	0.20	0.15	1.8	6.0	168.0	97.7
35.2	1.1	40.5	0.10	0.08	0.20	0.15	1.8	6.0	168.0	97.7
36.8	1.1	40.5	0.10	0.08	0.20	0.15	1.8	6.0	168.0	97.7
38.5	1.1	40.5	0.10	0.08	0.20	0.15	1.8	6.0	168.0	97.7
38.5	1.7	203.3	0.10	0.08	0.10	0.15	1.2	6.0	24.0	97.7
40.0	1.8	203.3	0.10	0.08	0.10	0.15	1.2	6.0	24.0	97.7

Driveability Analysis Summary



Gain/Loss Factor at Shaft/Toe = 0.571/1.000

Depth	Rut	Rshaft	Rtoe	Blow Ct	Mx C-Str	Mx T-Str.	Stroke	ENTHRU	Hammer
ft	kips	kips	kips	bl/ft	ksi	ksi	ft	kip-ft	-
3.0	11.2	4.6	6.7	1.1	2.474	0.000	3.20	13.7	D 19-42
6.0	16.0	9.3	6.7	1.3	8.690	0.000	3.74	27.2	D 19-42
9.0	24.3	11.7	12.5	2.0	12.708	0.000	4.13	25.4	D 19-42
12.0	28.3	15.1	13.1	2.3	14.048	0.280	4.28	24.8	D 19-42
15.0	33.3	20.1	13.1	2.7	15.131	0.405	4.42	24.1	D 19-42
18.0	39.8	26.6	13.1	3.1	16.897	0.459	4.58	23.5	D 19-42
21.0	125.1	34.7	90.4	13.9	25.974	0.776	6.63	18.8	D 19-42
24.0	59.6	46.3	13.3	4.8	20.148	0.888	5.06	21.9	D 19-42
27.0	69.9	56.5	13.3	6.2	21.568	0.674	5.42	21.1	D 19-42
30.0	80.1	66.8	13.3	7.7	22.528	0.725	5.72	20.2	D 19-42
33.0	90.3	77.0	13.3	9.2	23.545	0.401	5.99	19.6	D 19-42
36.0	124.7	84.8	39.9	14.5	26.053	0.516	6.71	18.6	D 19-42
40.0	300.5	100.2	200.3	47.7	31.255	0.672	8.61	18.9	D 19-42

Total driving time: 7 minutes; Total Number of Blows: 306 (starting at penetration 3.0 ft)

GRLWEAP: Wave Equation Analysis of Pile Foundations

Driven Steel Pile (B-001-0-24) + HP 12x53 1/15/2025
TSVC 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	Soil Type	Spec. Wt	Su	Phi	Unit Rs	Unit Rt
ft	-	lb/ft³	ksf	°	ksf	ksf
0.0	Clay	124.0	0.7	0.0	0.59	6.75
6.0	Clay	124.0	0.7	0.0	0.59	6.75
6.0	Sand	110.0	0.0	28.0	0.20	9.19
11.5	Sand	110.0	0.0	28.0	0.36	13.32
11.5	Sand	110.0	0.0	28.0	0.36	13.32
16.0	Sand	110.0	0.0	28.0	0.49	13.32
16.0	Sand	115.0	0.0	30.0	0.56	13.32
20.5	Sand	115.0	0.0	30.0	0.64	13.32
20.5	Sand	127.0	0.0	35.0	0.95	90.46
23.5	Sand	127.0	0.0	35.0	1.04	98.44
23.5	Clay	126.0	1.5	0.0	1.50	13.50
33.5	Clay	126.0	1.5	0.0	1.50	13.50
33.5	Clay	131.0	4.5	0.0	1.08	40.50
38.5	Clay	131.0	4.5	0.0	1.08	40.50
38.5	Sand	132.0	0.0	37.0	1.85	203.27
40.0	Sand	132.0	0.0	37.0	1.91	203.27

PILE INPUT

Uniform Pile		Pile Type:	H Pile
Pile Length: (ft)	45.000	Pile Penetration: (ft)	40.000
Pile Size: (ft)	1.00	Toe Area: (in²)	141.89

Pile Profile

Lb Top	X-Area	E-Modulus	Spec. Wt	Perim.	Crit. Index
ft	in²	ksi	lb/ft³	ft	-
0.0	15.5	30,000.0	492.0	4.0	0
45.0	15.5	30,000.0	492.0	4.0	0

HAMMER INPUT

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

Hammer Data

ID	Ram Wt	Ram L.	Ram Ar.	Rtd. Stk	Effic.	Rtd. Energy
-	kips	in	in²	ft	-	kip-ft
41	4.000	129.1	124.7	10.8	0.80	43.2

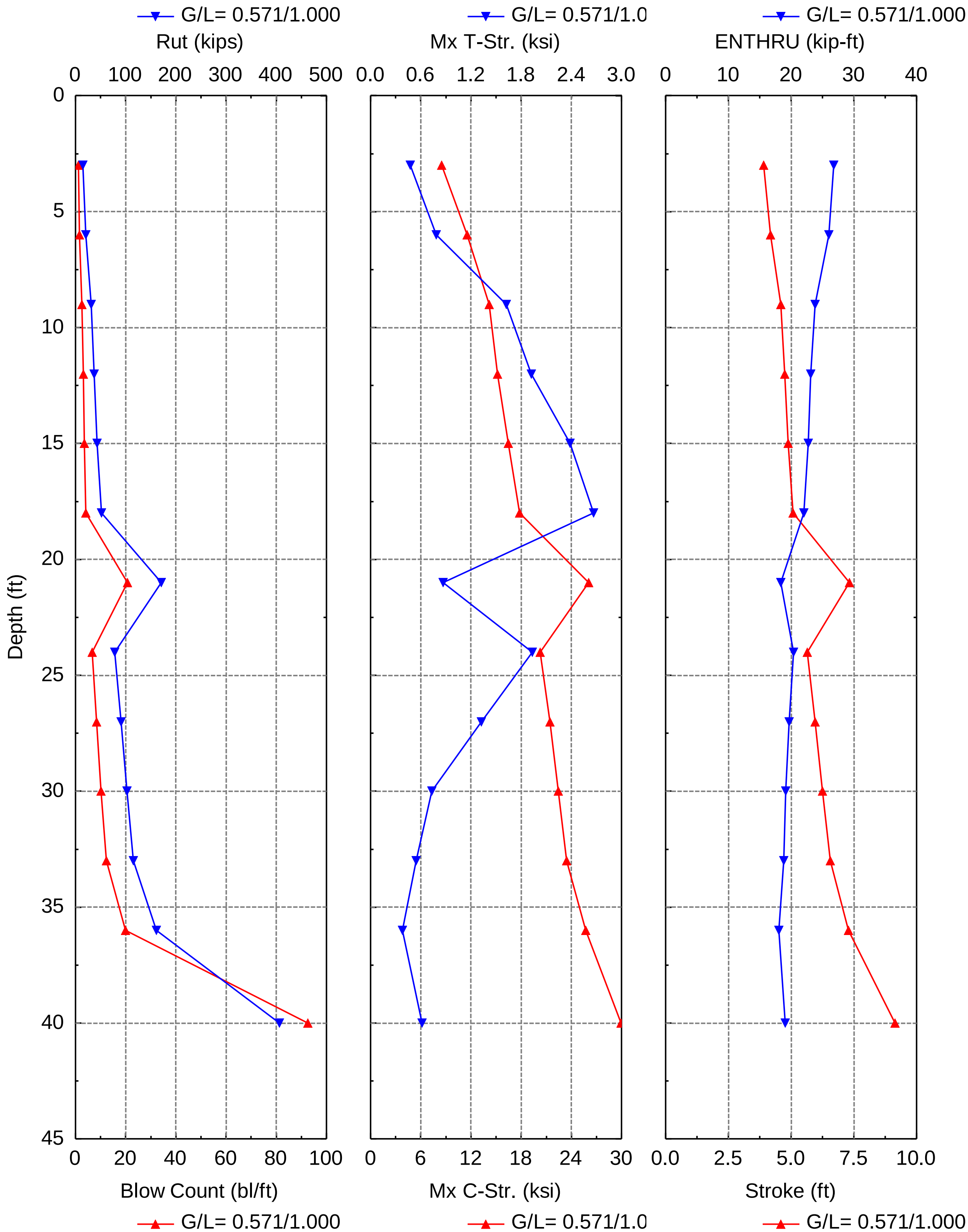
DRIVE SYSTEM FOR DELMAG D 19-42-OED

Type	X-Area	E-Modulus	Thickness	COR	Round-out	Stiffness
-	in²	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	in²
0.0	0.6	6.7	0.10	0.15	0.15	0.15	1.5	6.0	168.0	141.9
2.0	0.6	6.7	0.10	0.15	0.15	0.15	1.5	6.0	168.0	141.9
4.0	0.6	6.7	0.10	0.15	0.15	0.15	1.5	6.0	168.0	141.9
6.0	0.6	6.7	0.10	0.15	0.15	0.15	1.5	6.0	168.0	141.9
6.0	0.2	9.2	0.10	0.20	0.10	0.15	1.2	6.0	24.0	141.9
7.8	0.3	11.7	0.10	0.20	0.10	0.15	1.2	6.0	24.0	141.9
9.7	0.3	13.3	0.10	0.20	0.10	0.15	1.2	6.0	24.0	141.9
11.5	0.4	13.3	0.10	0.20	0.10	0.15	1.2	6.0	24.0	141.9
11.5	0.4	13.3	0.10	0.20	0.05	0.15	1.0	6.0	1.0	141.9
13.8	0.4	13.3	0.10	0.20	0.05	0.15	1.0	6.0	1.0	141.9
16.0	0.5	13.3	0.10	0.20	0.05	0.15	1.0	6.0	1.0	141.9
16.0	0.6	13.3	0.10	0.17	0.05	0.15	1.0	6.0	1.0	141.9
18.3	0.6	13.3	0.10	0.17	0.05	0.15	1.0	6.0	1.0	141.9
20.5	0.6	13.3	0.10	0.17	0.05	0.15	1.0	6.0	1.0	141.9
20.5	1.0	90.5	0.10	0.11	0.05	0.15	1.0	6.0	1.0	141.9
22.0	1.0	94.5	0.10	0.11	0.05	0.15	1.0	6.0	1.0	141.9
23.5	1.0	98.4	0.10	0.11	0.05	0.15	1.0	6.0	1.0	141.9
23.5	1.5	13.5	0.10	0.13	0.20	0.15	1.8	6.0	168.0	141.9
25.2	1.5	13.5	0.10	0.13	0.20	0.15	1.8	6.0	168.0	141.9
26.8	1.5	13.5	0.10	0.13	0.20	0.15	1.8	6.0	168.0	141.9
28.5	1.5	13.5	0.10	0.13	0.20	0.15	1.8	6.0	168.0	141.9
30.2	1.5	13.5	0.10	0.13	0.20	0.15	1.8	6.0	168.0	141.9
31.8	1.5	13.5	0.10	0.13	0.20	0.15	1.8	6.0	168.0	141.9
33.5	1.5	13.5	0.10	0.13	0.20	0.15	1.8	6.0	168.0	141.9
33.5	1.1	40.5	0.10	0.10	0.20	0.15	1.8	6.0	168.0	141.9
35.2	1.1	40.5	0.10	0.10	0.20	0.15	1.8	6.0	168.0	141.9
36.8	1.1	40.5	0.10	0.10	0.20	0.15	1.8	6.0	168.0	141.9
38.5	1.1	40.5	0.10	0.10	0.20	0.15	1.8	6.0	168.0	141.9
38.5	1.8	203.3	0.10	0.10	0.10	0.15	1.2	6.0	24.0	141.9
40.0	1.9	203.3	0.10	0.10	0.10	0.15	1.2	6.0	24.0	141.9

Driveability Analysis Summary



Gain/Loss Factor at Shaft/Toe = 0.571/1.000

Depth	Rut	Rshaft	Rtoe	Blow Ct	Mx C-Str	Mx T-Str.	Stroke	ENTHRU	Hammer
ft	kips	kips	kips	bl/ft	ksi	ksi	ft	kip-ft	-
3.0	14.7	5.4	9.3	1.2	8.510	0.472	3.90	26.8	D 19-42
6.0	20.2	10.9	9.3	1.6	11.531	0.784	4.18	26.0	D 19-42
9.0	31.6	14.1	17.5	2.6	14.179	1.625	4.58	23.8	D 19-42
12.0	37.0	18.6	18.4	3.1	15.177	1.923	4.74	23.1	D 19-42
15.0	43.5	25.2	18.4	3.6	16.443	2.385	4.88	22.7	D 19-42
18.0	52.1	33.7	18.4	4.2	17.826	2.663	5.07	22.0	D 19-42
21.0	171.2	44.4	126.7	20.7	26.068	0.864	7.32	18.3	D 19-42
24.0	78.6	60.0	18.6	6.6	20.285	1.934	5.64	20.4	D 19-42
27.0	90.7	72.1	18.6	8.4	21.406	1.323	5.95	19.7	D 19-42
30.0	102.8	84.2	18.6	10.2	22.414	0.734	6.25	19.1	D 19-42
33.0	114.9	96.3	18.6	12.3	23.417	0.545	6.55	18.8	D 19-42
36.0	161.4	105.5	55.8	19.9	25.684	0.379	7.27	18.0	D 19-42
40.0	405.9	125.7	280.2	92.6	29.915	0.612	9.12	19.0	D 19-42

Total driving time: 11 minutes; Total Number of Blows: 476 (starting at penetration 3.0 ft)

GRLWEAP: Wave Equation Analysis of Pile Foundations

Driven Steel Pile (B-001-0-24) + HP 14x73 1/15/2025
TSVC 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	Soil Type	Spec. Wt	Su	Phi	Unit Rs	Unit Rt
ft	-	lb/ft³	ksf	°	ksf	ksf
0.0	Clay	124.0	0.7	0.0	0.58	6.75
6.0	Clay	124.0	0.7	0.0	0.58	6.75
6.0	Sand	110.0	0.0	28.0	0.22	9.19
11.5	Sand	110.0	0.0	28.0	0.40	13.32
11.5	Sand	110.0	0.0	28.0	0.40	13.32
16.0	Sand	110.0	0.0	28.0	0.55	13.32
16.0	Sand	115.0	0.0	30.0	0.63	13.32
20.5	Sand	115.0	0.0	30.0	0.71	13.32
20.5	Sand	127.0	0.0	35.0	1.10	90.54
23.5	Sand	127.0	0.0	35.0	1.20	98.97
23.5	Clay	126.0	1.5	0.0	1.50	13.50
33.5	Clay	126.0	1.5	0.0	1.50	13.50
33.5	Clay	131.0	4.5	0.0	1.08	40.50
38.5	Clay	131.0	4.5	0.0	1.08	40.50
38.5	Sand	132.0	0.0	37.0	2.16	203.27
40.0	Sand	132.0	0.0	37.0	2.23	203.27

PILE INPUT

Uniform Pile		Pile Type:	H Pile
Pile Length: (ft)	45.000	Pile Penetration: (ft)	40.000
Pile Size: (ft)	1.22	Toe Area: (in²)	198.50

Pile Profile

Lb Top	X-Area	E-Modulus	Spec. Wt	Perim.	Crit. Index
ft	in²	ksi	lb/ft³	ft	-
0.0	21.4	30,000.0	492.0	4.7	0
45.0	21.4	30,000.0	492.0	4.7	0

HAMMER INPUT

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

Hammer Data

ID	Ram Wt	Ram L.	Ram Ar.	Rtd. Stk	Effic.	Rtd. Energy
-	kips	in	in²	ft	-	kip-ft
41	4.000	129.1	124.7	10.8	0.80	43.2

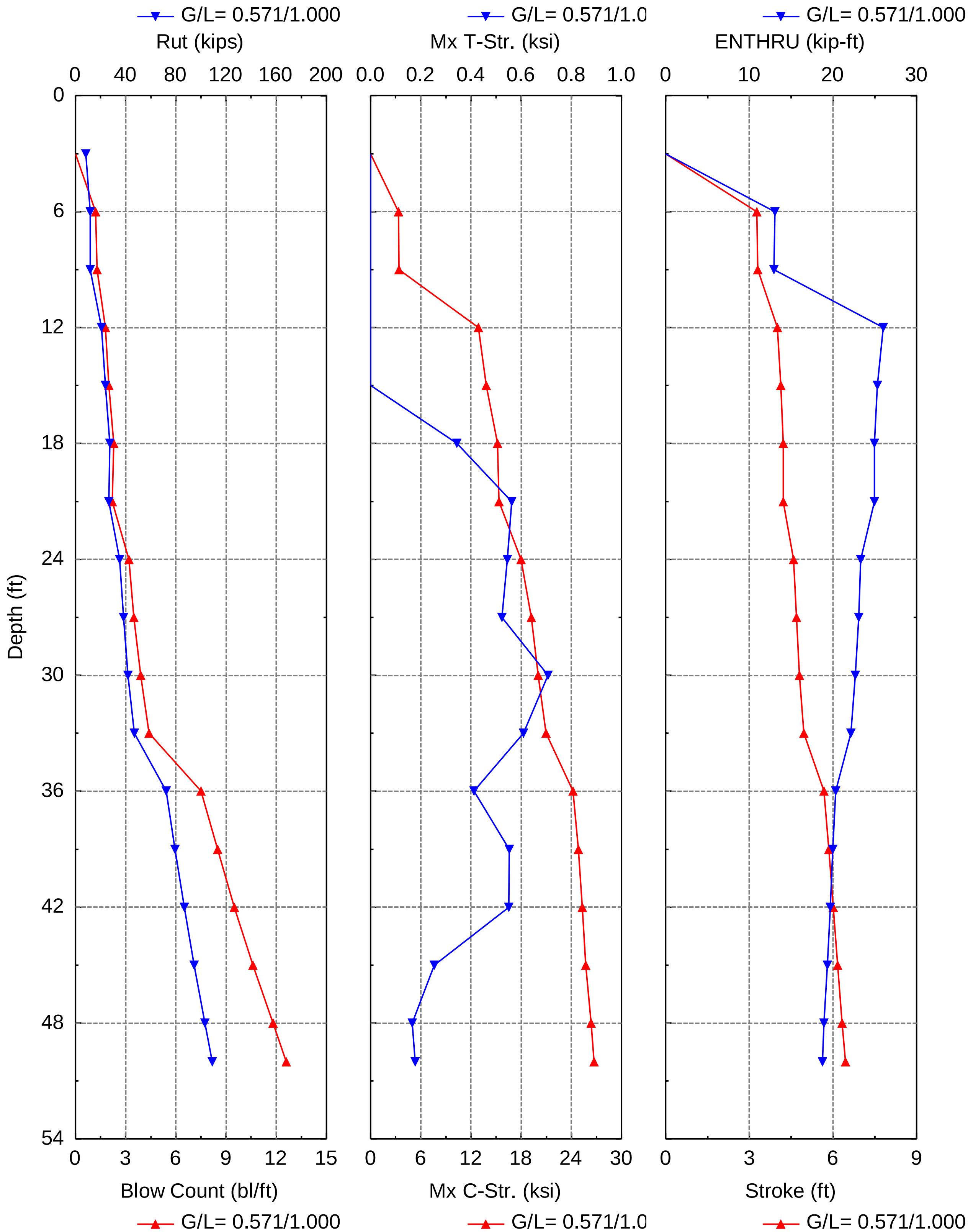
DRIVE SYSTEM FOR DELMAG D 19-42-OED

Type	X-Area	E-Modulus	Thickness	COR	Round-out	Stiffness
-	in²	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	in²
0.0	0.6	6.7	0.10	0.19	0.15	0.15	1.5	6.0	168.0	198.5
2.0	0.6	6.7	0.10	0.19	0.15	0.15	1.5	6.0	168.0	198.5
4.0	0.6	6.7	0.10	0.19	0.15	0.15	1.5	6.0	168.0	198.5
6.0	0.6	6.7	0.10	0.19	0.15	0.15	1.5	6.0	168.0	198.5
6.0	0.2	9.2	0.10	0.24	0.10	0.15	1.2	6.0	24.0	198.5
7.8	0.3	11.7	0.10	0.24	0.10	0.15	1.2	6.0	24.0	198.5
9.7	0.3	13.3	0.10	0.24	0.10	0.15	1.2	6.0	24.0	198.5
11.5	0.4	13.3	0.10	0.24	0.10	0.15	1.2	6.0	24.0	198.5
11.5	0.4	13.3	0.10	0.24	0.05	0.15	1.0	6.0	1.0	198.5
13.8	0.5	13.3	0.10	0.24	0.05	0.15	1.0	6.0	1.0	198.5
16.0	0.5	13.3	0.10	0.24	0.05	0.15	1.0	6.0	1.0	198.5
16.0	0.6	13.3	0.10	0.20	0.05	0.15	1.0	6.0	1.0	198.5
18.3	0.7	13.3	0.10	0.20	0.05	0.15	1.0	6.0	1.0	198.5
20.5	0.7	13.3	0.10	0.20	0.05	0.15	1.0	6.0	1.0	198.5
20.5	1.1	90.5	0.10	0.13	0.05	0.15	1.0	6.0	1.0	198.5
22.0	1.1	94.8	0.10	0.13	0.05	0.15	1.0	6.0	1.0	198.5
23.5	1.2	99.0	0.10	0.13	0.05	0.15	1.0	6.0	1.0	198.5
23.5	1.5	13.5	0.10	0.16	0.20	0.15	1.8	6.0	168.0	198.5
25.2	1.5	13.5	0.10	0.16	0.20	0.15	1.8	6.0	168.0	198.5
26.8	1.5	13.5	0.10	0.16	0.20	0.15	1.8	6.0	168.0	198.5
28.5	1.5	13.5	0.10	0.16	0.20	0.15	1.8	6.0	168.0	198.5
30.2	1.5	13.5	0.10	0.16	0.20	0.15	1.8	6.0	168.0	198.5
31.8	1.5	13.5	0.10	0.16	0.20	0.15	1.8	6.0	168.0	198.5
33.5	1.5	13.5	0.10	0.16	0.20	0.15	1.8	6.0	168.0	198.5
33.5	1.1	40.5	0.10	0.12	0.20	0.15	1.8	6.0	168.0	198.5
35.2	1.1	40.5	0.10	0.12	0.20	0.15	1.8	6.0	168.0	198.5
36.8	1.1	40.5	0.10	0.12	0.20	0.15	1.8	6.0	168.0	198.5
38.5	1.1	40.5	0.10	0.12	0.20	0.15	1.8	6.0	168.0	198.5
38.5	2.2	203.3	0.10	0.12	0.10	0.15	1.2	6.0	24.0	198.5
40.0	2.2	203.3	0.10	0.12	0.10	0.15	1.2	6.0	24.0	198.5

Driveability Analysis Summary



Gain/Loss Factor at Shaft/Toe = 0.571/1.000

Depth	Rut	Rshaft	Rtoe	Blow Ct	Mx C-Str	Mx T-Str.	Stroke	ENTHRU	Hammer
ft	kips	kips	kips	bl/ft	ksi	ksi	ft	kip-ft	-
3.0	8.4	3.8	4.6	0.0	0.000	0.000	0.00	0.0	D 19-42
6.0	11.8	7.2	4.6	1.2	3.347	0.000	3.27	13.0	D 19-42
9.0	11.9	10.3	1.5	1.3	3.416	0.000	3.31	12.9	D 19-42
12.0	20.9	11.9	9.0	1.8	12.856	0.000	4.00	26.0	D 19-42
15.0	23.8	14.7	9.0	2.0	13.848	0.000	4.13	25.3	D 19-42
18.0	27.4	18.3	9.0	2.3	15.184	0.343	4.22	25.0	D 19-42
21.0	26.7	22.1	4.6	2.2	15.325	0.562	4.22	25.0	D 19-42
24.0	35.0	25.9	9.2	3.2	17.977	0.544	4.59	23.3	D 19-42
27.0	38.3	29.2	9.2	3.5	19.196	0.522	4.68	23.1	D 19-42
30.0	41.7	32.5	9.2	3.9	20.037	0.706	4.80	22.6	D 19-42
33.0	46.7	37.6	9.2	4.4	20.940	0.609	4.96	22.1	D 19-42
36.0	72.4	44.9	27.5	7.5	24.214	0.412	5.68	20.3	D 19-42
39.0	79.5	52.0	27.5	8.5	24.858	0.553	5.84	20.0	D 19-42
42.0	86.6	59.2	27.5	9.5	25.308	0.550	6.00	19.7	D 19-42
45.0	94.6	67.2	27.5	10.6	25.717	0.255	6.16	19.3	D 19-42
48.0	103.1	75.6	27.5	11.8	26.372	0.166	6.33	18.9	D 19-42
50.0	108.9	81.4	27.5	12.6	26.724	0.178	6.44	18.7	D 19-42

Total driving time: 5 minutes; Total Number of Blows: 227 (starting at penetration 3.0 ft)

GRLWEAP: Wave Equation Analysis of Pile Foundations

Driven Steel Pile (B-002-0-24) + HP 10x42 1/15/2025
TSVC 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	Soil Type	Spec. Wt	Su	Phi	Unit Rs	Unit Rt
ft	-	lb/ft³	ksf	°	ksf	ksf
0.0	Clay	124.0	0.7	0.0	0.58	6.75
3.5	Clay	124.0	0.7	0.0	0.58	6.75
3.5	Clay	124.0	0.7	0.0	0.60	6.75
8.5	Clay	124.0	0.7	0.0	0.60	6.75
8.5	Clay	110.0	0.2	28.0	0.21	2.25
11.5	Clay	110.0	0.2	28.0	0.21	2.25
11.5	Sand	110.0	0.0	28.0	0.31	13.32
14.5	Sand	110.0	0.0	28.0	0.34	13.32
14.5	Sand	110.0	0.0	28.0	0.34	13.32
19.0	Sand	110.0	0.0	28.0	0.40	13.32
19.0	Clay	124.0	0.7	0.0	0.67	6.75
23.5	Clay	124.0	0.7	0.0	0.67	6.75
23.5	Clay	126.0	1.5	0.0	0.62	13.50
33.5	Clay	126.0	1.5	0.0	0.62	13.50
33.5	Clay	131.0	4.5	34.0	1.17	40.50
50.0	Clay	131.0	4.5	34.0	1.17	40.50

PILE INPUT

Uniform Pile		Pile Type:	H Pile
Pile Length: (ft)	55.000	Pile Penetration: (ft)	50.000
Pile Size: (ft)	0.84	Toe Area: (in²)	97.73

Pile Profile

Lb Top	X-Area	E-Modulus	Spec. Wt	Perim.	Crit. Index
ft	in²	ksi	lb/ft³	ft	-
0.0	12.4	30,000.0	492.0	3.3	0
55.0	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	Ram L.	Ram Ar.	Rtd. Stk	Effic.	Rtd. Energy
-	kips	in	in²	ft	-	kip-ft
41	4.000	129.1	124.7	10.8	0.80	43.2

DRIVE SYSTEM FOR DELMAG D 19-42-OED

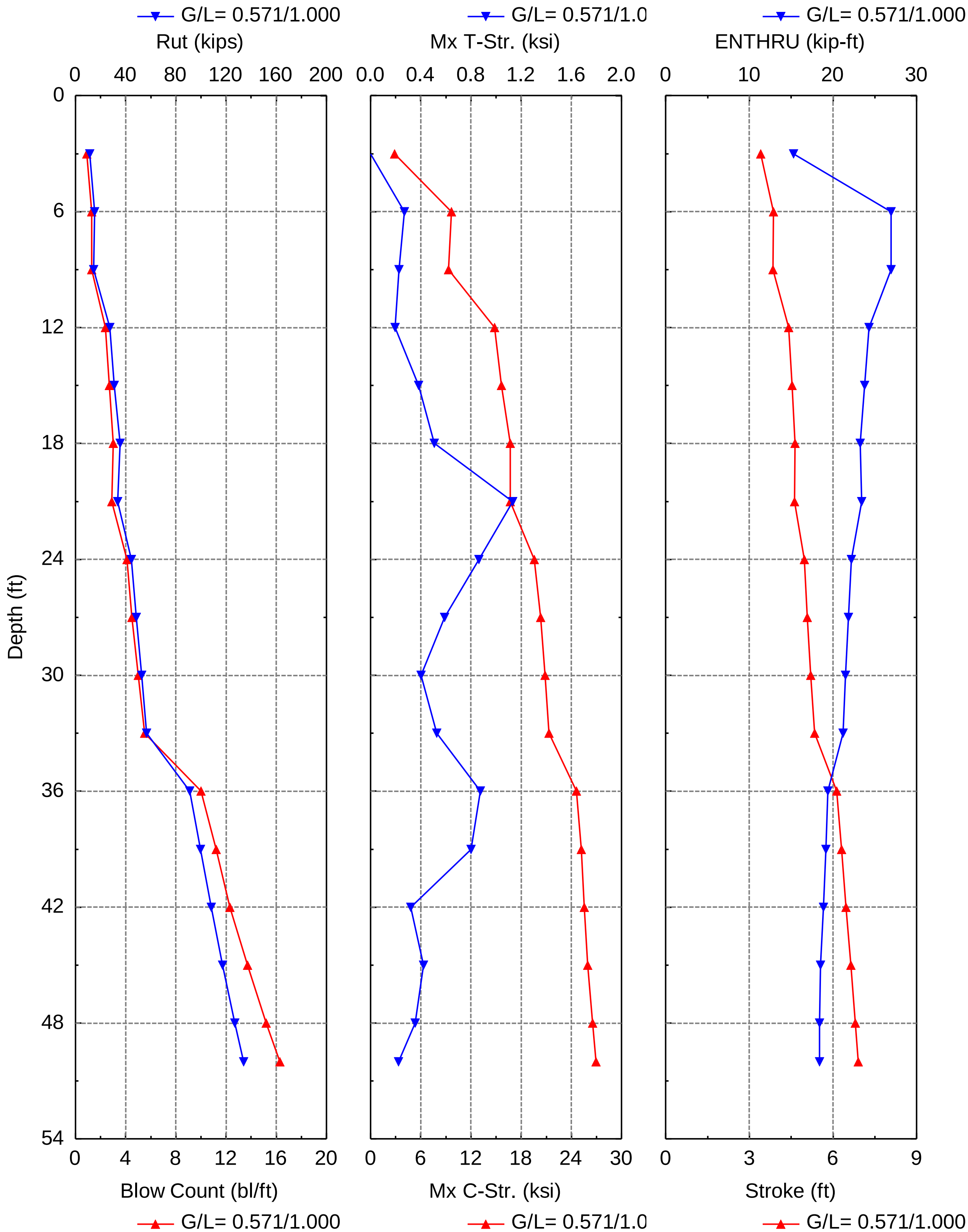
Type	X-Area	E-Modulus	Thickness	COR	Round-out	Stiffness
-	in²	ksi	in	-	in	kips/in
Hammer C.	227.000	530.000	2.000	0.800	0.120	60155.555
Helmet Wt.	1.900	kips				

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	in²
0.0	0.6	6.7	0.10	0.13	0.15	0.15	1.5	6.0	168.0	97.7
1.8	0.6	6.7	0.10	0.13	0.15	0.15	1.5	6.0	168.0	97.7
3.5	0.6	6.7	0.10	0.13	0.15	0.15	1.5	6.0	168.0	97.7
3.5	0.6	6.7	0.10	0.12	0.20	0.15	1.8	6.0	168.0	97.7
5.2	0.6	6.7	0.10	0.12	0.20	0.15	1.8	6.0	168.0	97.7
6.8	0.6	6.7	0.10	0.12	0.20	0.15	1.8	6.0	168.0	97.7
8.5	0.6	6.7	0.10	0.12	0.20	0.15	1.8	6.0	168.0	97.7
8.5	0.2	2.2	0.10	0.17	0.15	0.15	1.5	6.0	168.0	97.7
10.0	0.2	2.2	0.10	0.17	0.15	0.15	1.5	6.0	168.0	97.7
11.5	0.2	2.2	0.10	0.17	0.15	0.15	1.5	6.0	168.0	97.7
11.5	0.3	13.3	0.10	0.17	0.10	0.15	1.2	6.0	24.0	97.7
13.0	0.3	13.3	0.10	0.17	0.10	0.15	1.2	6.0	24.0	97.7
14.5	0.3	13.3	0.10	0.17	0.10	0.15	1.2	6.0	24.0	97.7
14.5	0.3	13.3	0.10	0.17	0.05	0.15	1.0	6.0	1.0	97.7
16.8	0.4	13.3	0.10	0.17	0.05	0.15	1.0	6.0	1.0	97.7
19.0	0.4	13.3	0.10	0.17	0.05	0.15	1.0	6.0	1.0	97.7
19.0	0.7	6.7	0.10	0.13	0.20	0.15	1.8	6.0	168.0	97.7
21.3	0.7	6.7	0.10	0.13	0.20	0.15	1.8	6.0	168.0	97.7
23.5	0.7	6.7	0.10	0.13	0.20	0.15	1.8	6.0	168.0	97.7
23.5	0.6	13.5	0.10	0.11	0.15	0.15	1.5	6.0	168.0	97.7
25.2	0.6	13.5	0.10	0.11	0.15	0.15	1.5	6.0	168.0	97.7
26.8	0.6	13.5	0.10	0.11	0.15	0.15	1.5	6.0	168.0	97.7
28.5	0.6	13.5	0.10	0.11	0.15	0.15	1.5	6.0	168.0	97.7
30.2	0.6	13.5	0.10	0.11	0.15	0.15	1.5	6.0	168.0	97.7
31.8	0.6	13.5	0.10	0.11	0.15	0.15	1.5	6.0	168.0	97.7
33.5	0.6	13.5	0.10	0.11	0.15	0.15	1.5	6.0	168.0	97.7
33.5	1.2	40.5	0.10	0.08	0.15	0.15	1.5	6.0	168.0	97.7
35.2	1.2	40.5	0.10	0.08	0.15	0.15	1.5	6.0	168.0	97.7
36.8	1.2	40.5	0.10	0.08	0.15	0.15	1.5	6.0	168.0	97.7
38.5	1.2	40.5	0.10	0.08	0.15	0.15	1.5	6.0	168.0	97.7
40.1	1.2	40.5	0.10	0.08	0.15	0.15	1.5	6.0	168.0	97.7
41.8	1.2	40.5	0.10	0.08	0.15	0.15	1.5	6.0	168.0	97.7
43.4	1.2	40.5	0.10	0.08	0.15	0.15	1.5	6.0	168.0	97.7

45.1	1.2	40.5	0.10	0.08	0.15	0.15	1.5	6.0	168.0	97.7
46.7	1.2	40.5	0.10	0.08	0.15	0.15	1.5	6.0	168.0	97.7
48.4	1.2	40.5	0.10	0.08	0.15	0.15	1.5	6.0	168.0	97.7
50.0	1.2	40.5	0.10	0.08	0.15	0.15	1.5	6.0	168.0	97.7

Driveability Analysis Summary



Gain/Loss Factor at Shaft/Toe = 0.571/1.000

Depth	Rut	Rshaft	Rtoe	Blow Ct	Mx C-Str	Mx T-Str.	Stroke	ENTHRU	Hammer
ft	kips	kips	kips	bl/ft	ksi	ksi	ft	kip-ft	-
3.0	11.2	4.6	6.7	0.9	2.843	0.000	3.40	15.3	D 19-42
6.0	15.3	8.7	6.7	1.3	9.660	0.268	3.87	26.9	D 19-42
9.0	14.6	12.3	2.2	1.3	9.320	0.225	3.84	27.0	D 19-42
12.0	27.4	14.2	13.1	2.4	14.788	0.197	4.41	24.3	D 19-42
15.0	30.9	17.8	13.1	2.7	15.634	0.382	4.52	23.8	D 19-42
18.0	35.5	22.4	13.1	3.0	16.678	0.506	4.64	23.2	D 19-42
21.0	33.5	26.9	6.7	2.9	16.710	1.132	4.61	23.4	D 19-42
24.0	44.6	31.3	13.3	4.1	19.562	0.862	4.97	22.2	D 19-42
27.0	48.6	35.3	13.3	4.5	20.305	0.590	5.08	21.8	D 19-42
30.0	52.6	39.3	13.3	5.0	20.840	0.402	5.20	21.5	D 19-42
33.0	56.6	43.3	13.3	5.5	21.320	0.525	5.33	21.2	D 19-42
36.0	91.1	51.2	39.9	10.0	24.584	0.875	6.13	19.4	D 19-42
39.0	99.7	59.8	39.9	11.2	25.162	0.800	6.30	19.1	D 19-42
42.0	108.3	68.4	39.9	12.3	25.509	0.319	6.46	18.8	D 19-42
45.0	117.3	77.4	39.9	13.7	25.934	0.420	6.63	18.5	D 19-42
48.0	127.1	87.2	39.9	15.2	26.529	0.357	6.79	18.4	D 19-42
50.0	133.8	93.9	39.9	16.3	26.913	0.223	6.90	18.4	D 19-42

Total driving time: 6 minutes; Total Number of Blows: 295 (starting at penetration 3.0 ft)

GRLWEAP: Wave Equation Analysis of Pile Foundations

Driven Steel Pile (B-002-0-24) + HP 12x53 1/15/2025
TSVC 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	Soil Type	Spec. Wt	Su	Phi	Unit Rs	Unit Rt
ft	-	lb/ft³	ksf	°	ksf	ksf
0.0	Clay	124.0	0.7	0.0	0.58	6.75
3.5	Clay	124.0	0.7	0.0	0.58	6.75
3.5	Clay	124.0	0.7	0.0	0.59	6.75
8.5	Clay	124.0	0.7	0.0	0.59	6.75
8.5	Clay	110.0	0.2	28.0	0.20	2.25
11.5	Clay	110.0	0.2	28.0	0.20	2.25
11.5	Sand	110.0	0.0	28.0	0.32	13.32
14.5	Sand	110.0	0.0	28.0	0.36	13.32
14.5	Sand	110.0	0.0	28.0	0.36	13.32
19.0	Sand	110.0	0.0	28.0	0.42	13.32
19.0	Clay	124.0	0.7	0.0	0.65	6.75
23.5	Clay	124.0	0.7	0.0	0.65	6.75
23.5	Clay	126.0	1.5	0.0	0.51	13.50
33.5	Clay	126.0	1.5	0.0	0.51	13.50
33.5	Clay	131.0	4.5	34.0	1.14	40.50
50.0	Clay	131.0	4.5	34.0	1.14	40.50

PILE INPUT

Uniform Pile		Pile Type:	H Pile
Pile Length: (ft)	55.000	Pile Penetration: (ft)	50.000
Pile Size: (ft)	1.00	Toe Area: (in²)	141.89

Pile Profile

Lb Top	X-Area	E-Modulus	Spec. Wt	Perim.	Crit. Index
ft	in²	ksi	lb/ft³	ft	-
0.0	15.5	30,000.0	492.0	4.0	0
55.0	15.5	30,000.0	492.0	4.0	0

HAMMER INPUT

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

Hammer Data

ID	Ram Wt	Ram L.	Ram Ar.	Rtd. Stk	Effic.	Rtd. Energy
-	kips	in	in²	ft	-	kip-ft
41	4.000	129.1	124.7	10.8	0.80	43.2

DRIVE SYSTEM FOR DELMAG D 19-42-OED

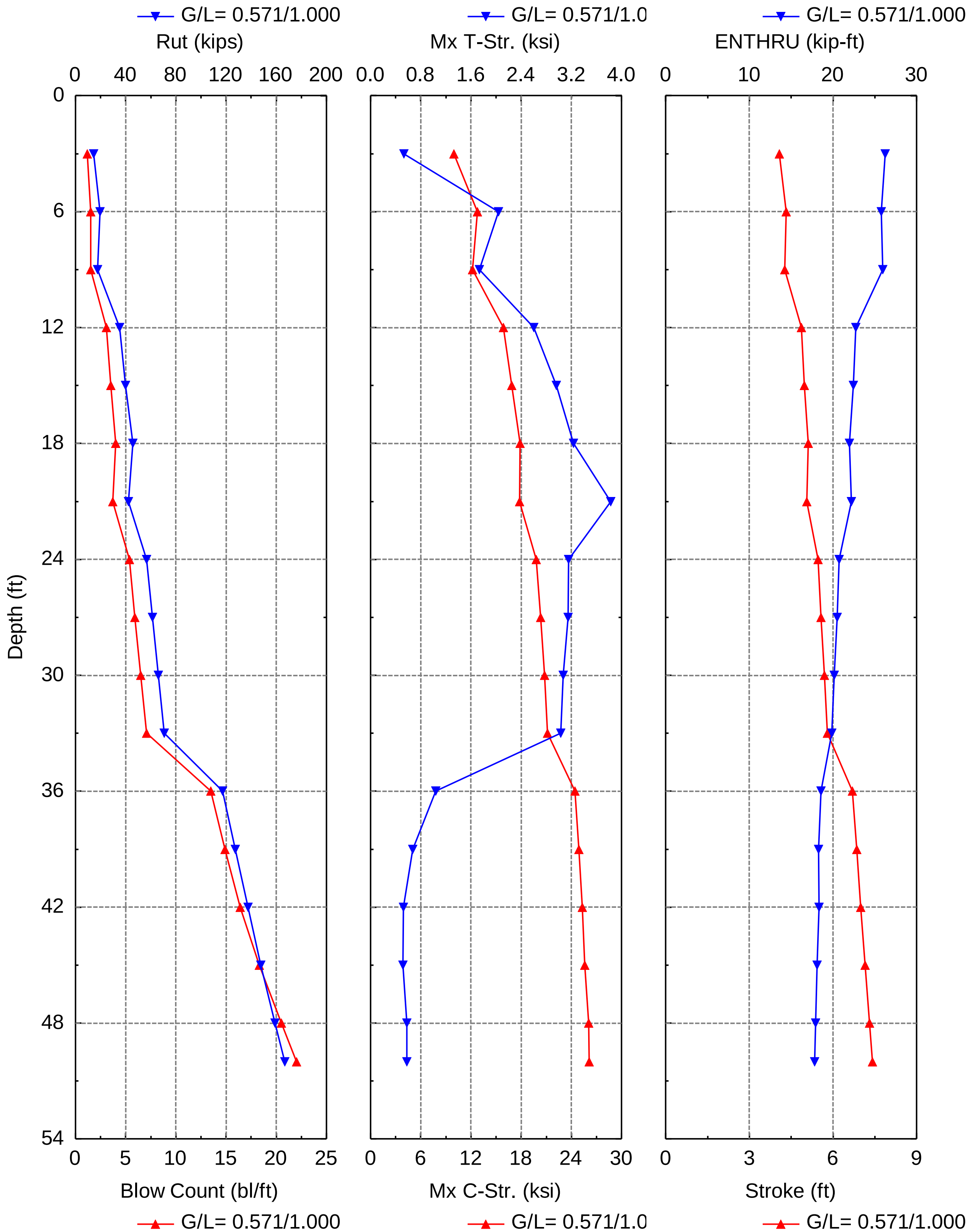
Type	X-Area	E-Modulus	Thickness	COR	Round-out	Stiffness
-	in²	ksi	in	-	in	kips/in
Hammer C.	227.000	530.000	2.000	0.800	0.120	60155.555
Helmet Wt.	1.900	kips				

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	in²
0.0	0.6	6.7	0.10	0.16	0.15	0.15	1.5	6.0	168.0	141.9
1.8	0.6	6.7	0.10	0.16	0.15	0.15	1.5	6.0	168.0	141.9
3.5	0.6	6.7	0.10	0.16	0.15	0.15	1.5	6.0	168.0	141.9
3.5	0.6	6.7	0.10	0.14	0.20	0.15	1.8	6.0	168.0	141.9
5.2	0.6	6.7	0.10	0.14	0.20	0.15	1.8	6.0	168.0	141.9
6.8	0.6	6.7	0.10	0.14	0.20	0.15	1.8	6.0	168.0	141.9
8.5	0.6	6.7	0.10	0.14	0.20	0.15	1.8	6.0	168.0	141.9
8.5	0.2	2.2	0.10	0.20	0.15	0.15	1.5	6.0	168.0	141.9
10.0	0.2	2.2	0.10	0.20	0.15	0.15	1.5	6.0	168.0	141.9
11.5	0.2	2.2	0.10	0.20	0.15	0.15	1.5	6.0	168.0	141.9
11.5	0.3	13.3	0.10	0.20	0.10	0.15	1.2	6.0	24.0	141.9
13.0	0.3	13.3	0.10	0.20	0.10	0.15	1.2	6.0	24.0	141.9
14.5	0.4	13.3	0.10	0.20	0.10	0.15	1.2	6.0	24.0	141.9
14.5	0.4	13.3	0.10	0.20	0.05	0.15	1.0	6.0	1.0	141.9
16.8	0.4	13.3	0.10	0.20	0.05	0.15	1.0	6.0	1.0	141.9
19.0	0.4	13.3	0.10	0.20	0.05	0.15	1.0	6.0	1.0	141.9
19.0	0.7	6.7	0.10	0.15	0.20	0.15	1.8	6.0	168.0	141.9
21.3	0.7	6.7	0.10	0.15	0.20	0.15	1.8	6.0	168.0	141.9
23.5	0.7	6.7	0.10	0.15	0.20	0.15	1.8	6.0	168.0	141.9
23.5	0.5	13.5	0.10	0.13	0.15	0.15	1.5	6.0	168.0	141.9
25.2	0.5	13.5	0.10	0.13	0.15	0.15	1.5	6.0	168.0	141.9
26.8	0.5	13.5	0.10	0.13	0.15	0.15	1.5	6.0	168.0	141.9
28.5	0.5	13.5	0.10	0.13	0.15	0.15	1.5	6.0	168.0	141.9
30.2	0.5	13.5	0.10	0.13	0.15	0.15	1.5	6.0	168.0	141.9
31.8	0.5	13.5	0.10	0.13	0.15	0.15	1.5	6.0	168.0	141.9
33.5	0.5	13.5	0.10	0.13	0.15	0.15	1.5	6.0	168.0	141.9
33.5	1.1	40.5	0.10	0.10	0.15	0.15	1.5	6.0	168.0	141.9
35.2	1.1	40.5	0.10	0.10	0.15	0.15	1.5	6.0	168.0	141.9
36.8	1.1	40.5	0.10	0.10	0.15	0.15	1.5	6.0	168.0	141.9
38.5	1.1	40.5	0.10	0.10	0.15	0.15	1.5	6.0	168.0	141.9
40.1	1.1	40.5	0.10	0.10	0.15	0.15	1.5	6.0	168.0	141.9
41.8	1.1	40.5	0.10	0.10	0.15	0.15	1.5	6.0	168.0	141.9
43.4	1.1	40.5	0.10	0.10	0.15	0.15	1.5	6.0	168.0	141.9

45.1	1.1	40.5	0.10	0.10	0.15	0.15	1.5	6.0	168.0	141.9
46.7	1.1	40.5	0.10	0.10	0.15	0.15	1.5	6.0	168.0	141.9
48.4	1.1	40.5	0.10	0.10	0.15	0.15	1.5	6.0	168.0	141.9
50.0	1.1	40.5	0.10	0.10	0.15	0.15	1.5	6.0	168.0	141.9

Driveability Analysis Summary



Gain/Loss Factor at Shaft/Toe = 0.571/1.000

Depth	Rut	Rshaft	Rtoe	Blow Ct	Mx C-Str	Mx T-Str.	Stroke	ENTHRU	Hammer
ft	kips	kips	kips	bl/ft	ksi	ksi	ft	kip-ft	-
3.0	14.7	5.4	9.3	1.2	9.926	0.531	4.07	26.2	D 19-42
6.0	19.5	10.2	9.3	1.5	12.754	2.037	4.31	25.8	D 19-42
9.0	17.6	14.5	3.1	1.5	12.187	1.737	4.26	25.9	D 19-42
12.0	35.2	16.8	18.4	3.1	15.841	2.603	4.86	22.7	D 19-42
15.0	39.8	21.5	18.4	3.5	16.882	2.963	4.97	22.4	D 19-42
18.0	45.8	27.5	18.4	4.0	17.842	3.230	5.10	22.0	D 19-42
21.0	42.3	33.0	9.3	3.7	17.813	3.829	5.05	22.2	D 19-42
24.0	56.7	38.1	18.6	5.4	19.774	3.155	5.45	20.7	D 19-42
27.0	61.4	42.8	18.6	5.9	20.331	3.146	5.57	20.5	D 19-42
30.0	66.2	47.6	18.6	6.5	20.785	3.068	5.69	20.1	D 19-42
33.0	70.9	52.3	18.6	7.1	21.127	3.026	5.80	19.9	D 19-42
36.0	117.4	61.6	55.8	13.5	24.444	1.035	6.70	18.5	D 19-42
39.0	127.5	71.7	55.8	14.9	24.885	0.669	6.84	18.3	D 19-42
42.0	137.7	81.9	55.8	16.4	25.298	0.521	6.99	18.3	D 19-42
45.0	147.8	92.0	55.8	18.3	25.598	0.517	7.14	18.1	D 19-42
48.0	159.0	103.1	55.8	20.5	26.056	0.581	7.30	17.9	D 19-42
50.0	166.7	110.9	55.8	22.0	26.123	0.576	7.42	17.8	D 19-42

Total driving time: 8 minutes; Total Number of Blows: 391 (starting at penetration 3.0 ft)

GRLWEAP: Wave Equation Analysis of Pile Foundations

Driven Steel Pile (B-002-0-24) + HP 14x73 1/15/2025
TSVC 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	Soil Type	Spec. Wt	Su	Phi	Unit Rs	Unit Rt
ft	-	lb/ft³	ksf	°	ksf	ksf
0.0	Clay	124.0	0.7	0.0	0.57	6.75
3.5	Clay	124.0	0.7	0.0	0.57	6.75
3.5	Clay	124.0	0.7	0.0	0.59	6.75
8.5	Clay	124.0	0.7	0.0	0.59	6.75
8.5	Clay	110.0	0.2	28.0	0.20	2.25
11.5	Clay	110.0	0.2	28.0	0.20	2.25
11.5	Sand	110.0	0.0	28.0	0.36	13.32
14.5	Sand	110.0	0.0	28.0	0.40	13.32
14.5	Sand	110.0	0.0	28.0	0.40	13.32
19.0	Sand	110.0	0.0	28.0	0.46	13.32
19.0	Clay	124.0	0.7	0.0	0.64	6.75
23.5	Clay	124.0	0.7	0.0	0.64	6.75
23.5	Clay	126.0	1.5	0.0	0.50	13.50
33.5	Clay	126.0	1.5	0.0	0.50	13.50
33.5	Clay	131.0	4.5	34.0	1.12	40.50
50.0	Clay	131.0	4.5	34.0	1.12	40.50

PILE INPUT

Uniform Pile		Pile Type:	H Pile
Pile Length: (ft)	55.000	Pile Penetration: (ft)	50.000
Pile Size: (ft)	1.22	Toe Area: (in²)	198.50

Pile Profile

Lb Top	X-Area	E-Modulus	Spec. Wt	Perim.	Crit. Index
ft	in²	ksi	lb/ft³	ft	-
0.0	21.4	30,000.0	492.0	4.7	0
55.0	21.4	30,000.0	492.0	4.7	0

HAMMER INPUT

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

Hammer Data

ID	Ram Wt	Ram L.	Ram Ar.	Rtd. Stk	Effic.	Rtd. Energy
-	kips	in	in²	ft	-	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
-	in²	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	in²
0.0	0.6	6.7	0.10	0.19	0.15	0.15	1.5	6.0	168.0	198.5
1.8	0.6	6.7	0.10	0.19	0.15	0.15	1.5	6.0	168.0	198.5
3.5	0.6	6.7	0.10	0.19	0.15	0.15	1.5	6.0	168.0	198.5
3.5	0.6	6.7	0.10	0.17	0.20	0.15	1.8	6.0	168.0	198.5
5.2	0.6	6.7	0.10	0.17	0.20	0.15	1.8	6.0	168.0	198.5
6.8	0.6	6.7	0.10	0.17	0.20	0.15	1.8	6.0	168.0	198.5
8.5	0.6	6.7	0.10	0.17	0.20	0.15	1.8	6.0	168.0	198.5
8.5	0.2	2.2	0.10	0.24	0.15	0.15	1.5	6.0	168.0	198.5
10.0	0.2	2.2	0.10	0.24	0.15	0.15	1.5	6.0	168.0	198.5
11.5	0.2	2.2	0.10	0.24	0.15	0.15	1.5	6.0	168.0	198.5
11.5	0.4	13.3	0.10	0.24	0.10	0.15	1.2	6.0	24.0	198.5
13.0	0.4	13.3	0.10	0.24	0.10	0.15	1.2	6.0	24.0	198.5
14.5	0.4	13.3	0.10	0.24	0.10	0.15	1.2	6.0	24.0	198.5
14.5	0.4	13.3	0.10	0.24	0.05	0.15	1.0	6.0	1.0	198.5
16.8	0.4	13.3	0.10	0.24	0.05	0.15	1.0	6.0	1.0	198.5
19.0	0.5	13.3	0.10	0.24	0.05	0.15	1.0	6.0	1.0	198.5
19.0	0.6	6.7	0.10	0.19	0.20	0.15	1.8	6.0	168.0	198.5
21.3	0.6	6.7	0.10	0.19	0.20	0.15	1.8	6.0	168.0	198.5
23.5	0.6	6.7	0.10	0.19	0.20	0.15	1.8	6.0	168.0	198.5
23.5	0.5	13.5	0.10	0.16	0.15	0.15	1.5	6.0	168.0	198.5
25.2	0.5	13.5	0.10	0.16	0.15	0.15	1.5	6.0	168.0	198.5
26.8	0.5	13.5	0.10	0.16	0.15	0.15	1.5	6.0	168.0	198.5
28.5	0.5	13.5	0.10	0.16	0.15	0.15	1.5	6.0	168.0	198.5
30.2	0.5	13.5	0.10	0.16	0.15	0.15	1.5	6.0	168.0	198.5
31.8	0.5	13.5	0.10	0.16	0.15	0.15	1.5	6.0	168.0	198.5
33.5	0.5	13.5	0.10	0.16	0.15	0.15	1.5	6.0	168.0	198.5
33.5	1.1	40.5	0.10	0.12	0.15	0.15	1.5	6.0	168.0	198.5
35.2	1.1	40.5	0.10	0.12	0.15	0.15	1.5	6.0	168.0	198.5
36.8	1.1	40.5	0.10	0.12	0.15	0.15	1.5	6.0	168.0	198.5
38.5	1.1	40.5	0.10	0.12	0.15	0.15	1.5	6.0	168.0	198.5
40.1	1.1	40.5	0.10	0.12	0.15	0.15	1.5	6.0	168.0	198.5
41.8	1.1	40.5	0.10	0.12	0.15	0.15	1.5	6.0	168.0	198.5
43.4	1.1	40.5	0.10	0.12	0.15	0.15	1.5	6.0	168.0	198.5

45.1	1.1	40.5	0.10	0.12	0.15	0.15	1.5	6.0	168.0	198.5
46.7	1.1	40.5	0.10	0.12	0.15	0.15	1.5	6.0	168.0	198.5
48.4	1.1	40.5	0.10	0.12	0.15	0.15	1.5	6.0	168.0	198.5
50.0	1.1	40.5	0.10	0.12	0.15	0.15	1.5	6.0	168.0	198.5