GEOTECHNICAL
CONSULTANTS INC.

GCI PROJECT No.: 21-G-25284

Structure Foundation Exploration Report

MOE-CR29-06.95 Bridge Replacement, PID 111130
Sunfish Creek Road over Tributary to Sunfish Creek

Adams Township, Monroe County, Ohio

Prepared for:
ADR & Associates, Ltd.

September 7, 2021

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September 7, 2021

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Reference: Structure Foundation Exploration Report
MOE-CR29-06.95 Bridge Replacement
Sunfish Creek Road over Tributary to Sunfish Creek
Adams Township, Monroe County, Ohio
GCI Project No. 21-G-25284

Dear Mr. Hartfield:

As authorized, Geotechnical Consultants, Inc. (GCI) performed a subsurface exploration and prepared this structure foundation exploration report for the referenced project. The purpose of this exploration was to assess subsurface conditions and make recommendations for foundations of the proposed bridge. After you have reviewed the report, feel free to contact GCI with any questions you may have. GCI appreciates the opportunity to provide our services for this project, and we hope to continue service through construction.

Sincerely,
Geotechnical Consultants, Inc.

JEFFREY M. HOLKO
E-82689

Jeffrey M. Holko, P.E.
Project Manager

Todd R. Meek, PE
In-House Reviewer

Distribution: Mr. Justin Hartfield @ ADR & Associates, Ltd. – 1 pdf via e-mail
GCI File – 1 copy

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1.0 EXECUTIVE SUMMARY

The project involves the removal of the existing bridge carrying Sunfish Creek Road over a tributary of Sunfish Creek and replacement with a new bridge at the same location. We performed geotechnical borings to aid in assessing subsurface conditions and making recommendations for foundations of the proposed bridge. The borings found natural deposits of fine- and coarse-grained soils overlying shale bedrock. We have recommended the new bridge be founded on a deep foundation system consisting of H-piles driven to refusal, end-bearing in shale bedrock.

2.0 INTRODUCTION

As requested by Mr. Justin Hartfield, P.E., representing ADR & Associates, Ltd. (ADR), Geotechnical Consultants, Inc. (GCI) performed a subsurface exploration for the proposed bridge replacement project (MOE-CR29-06.95) for Sunfish Creek Road over a tributary of Sunfish Creek in Adams Township, Monroe County, Ohio. Our study consisted of two standard penetration test borings (one behind each of the existing abutments), laboratory soil testing, and walk-over site observations. A boring location plan and copies of the boring logs are included in the appendix.

The intent of this exploration was to evaluate subsoil conditions and offer recommendations relative to foundations for the proposed bridge replacement. This report has been prepared for the exclusive use of ADR and their consultants for specific application to the referenced bridge replacement project in accordance with generally accepted soil and foundation engineering practices. No warranty, expressed or implied, is made.

3.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

The existing bridge spans a tributary of Sunfish Creek. A site aerial below shows the existing bridge location and immediate surrounding area.

Aerial courtesy of Google Earth (Image dated October 2015)

The existing bridge consists of a steel superstructure supported on two abutments. The underside of the structure contains beams spanning abutment to abutment. The bridge deck appeared to be a concrete slab covered with asphalt at the road surface. The abutments consisted of cut stone blocks with mortar. The bridge span was measured at 37’ and the deck width at about 18’. Please see the images on the following page.

Facing East

Facing West

Facing East Abutment

Facing West Abutment

The tributary flows from north / northwest to south / southeast and into Sunfish Creek at about 200’ southeast of this bridge. GCI reviewed topographic information from the United States Geological Survey (USGS), which showed the stream and bank at an elevation range of 700’ – 710’. Grades rise sharply to the south (across Sunfish Creek) and to the north to elevations exceeding 1,200’.

The creek bed was measured at about 9.7’ below the top of bridge deck. On the day of our site visit (June 16, 2021), the creek was about 3” – 6” deep.

Upstream of the bridge, the embankments were vegetated and at about a 1H:1V slope with a 9’ – 10” height. Downstream of the bridge, the embankments were vegetated and at about a 1H:1V slope with a 7’ – 8’ height. Beyond the stream banks, grades are flat before reaching the sharp elevation increases. See the photographs below.

We did note apparent scour beneath the southern end of the east abutment. The bank adjacent to the south end of the east abutment appeared to have erosion and roots exposed as shown in the photos below.

Facing Upstream

Facing Downstream

South End of East Abutment

GCI researched and attained available geotechnical information using the following sources:

- *Physiographic Regions of Ohio* produced by the ODNR – Division of Geological Survey:

  The map notes the site to be within the Little Switzerland Plateau. The map characterizes this plateau to be highly dissected and of high relief (generally 350', and up to 750' near the Ohio River). Landslides are common in this area, along with high-gradient shale bottom streams subject to flash flooding.

- *Bedrock Geology Map of Ohio* produced by the ODNR – Division of Geological Survey:

  The map shows bedrock to consist of lower Pennsylvanian age – sedimentary rocks: mainly shale, sandstone, siltstone, mudstone, and some coal.

- Well boring information from the Ohio Department of Natural Resources – Division of Water Resources, Water Wells Map:

  Utilizing the ODNR Water Wells Map, logs from two nearby wells were reviewed (see diagram below). Well 950449 recorded sandstone at a depth of 4 feet and well 2035627 recorded shale at a depth of 29 feet.

4.0 EXPLORATION

4.1 Historic Borings Review

GCI researched available geotechnical information using the ODOT Transportation Information and Mapping System (TIMS) program. The nearest projects were about 5 miles to the west (PID 101701) and 5 miles to the southwest (PID 109262). Both projects were for bridge preservation. Due to the distance, we do not consider them as reliable comparisons for similar geological conditions. We consider the nearby water well logs (mentioned in the prior section) as the most reliable source as a comparison of similar geological conditions.

4.2 Project Exploration

After performing research on the site geological conditions (as discussed in Section 3.0 and Subsection 4.1), GCI visited the project site. At the project site, we were able to determine the depth from road surface to stream bed; knowing this depth aided us in shaping our exploration program. After review of the ODOT Specifications for Geotechnical Exploration (SGE), GCI created the following boring sequence prior to site mobilization:

- 2.5’ interval spacing from the road surface to stream bed;
- Continuous sampling from the stream bed to 6’ below the stream bed (scour sampling zone);
- Since we expected the bottom of footing / pile cap to be near the stream bed elevation (about 9’ below the road surface), we would drill at 2.5’ intervals for about 14’ beyond the scour sampling zone;
- 5’ sampling intervals to a depth of 50’.

GCI mobilized a truck-mounted rotary drill rig with automatic sampling hammer (calibrated energy rating of 84%) to the site on June 24, 2021. We drilled two borings (B-001-0-21 & B-002-0-21) along the roadway, behind the existing bridge abutments. Both borings were drilled to a depth of 41.5 feet, where auger refusal was encountered on bedrock.

Our N_{60}-values were determined by using the following equation:

$$N_{60} = N_m \text{ x } (ER/60)$$

where: N_m = the field blow counts from the 2nd and 3rd 6-inch intervals

ER = the drill rod energy ratio (84 for the CME 45B used on this project)

Our subsurface findings were generally consistent with the previously noted well boring logs and published geologic data. Our boring findings are described in Section 5.0.

4.3 Laboratory Testing Program

GCI performed a laboratory testing program consisting of natural moisture content, Atterberg Limits, grain size analysis, and hydrometer analysis. Results of the laboratory soil testing have been incorporated into the text of this report and attached boring logs; results are attached in the Appendix.

5.0 FINDINGS

Medium stiff to stiff cohesive soils (A-4a and A-7-6) were encountered in borings B-001-0-21 and B-002-0-21 to respective depths of 4.5’ and 1’, upon which granular deposits were encountered. The granular deposits were loose to medium dense in cohesionless density and classified as A-2-6, A-2-4, A-1-b, and A-1-a, extending to a depth of 20’ in both borings. Shale bedrock was encountered below the granular deposits. We summarize our findings below; please refer to the boring logs for specific information at the boring locations.

B-001-0-21 (drilled behind west abutment of existing bridge):

- 0’ – 0.5’: topsoil;
- 0.5’ – 1.5’: medium stiff dark brown SANDY SILT (A-4a), fill, damp;
- 1.5’ – 4.5’: medium stiff brown CLAY (A-7-6), damp;
- 4.5’ – 12.0’: loose to medium dense brown STONE FRAGMENTS WITH SAND, SILT, AND CLAY (A-1-b), damp to wet;
- 12.0’ – 15.0’: loose brown STONE FRAGMENTS WITH SAND (A-1-b), wet;
- 15.0’ – 17.0’: medium dense STONE FRAGMENTS (A-1-a), wet;
- 17.0’ – 20.0’: medium dense GRAVEL WITH SAND (A-1-b), wet;
- 20.0’ – 22.0’: black highly weathered SHALE, friable;
- 22.0’ – 26.0’: gray highly weathered SHALE, friable;
- 26.0’ – 41.5’: gray moderately weathered SHALE;

- groundwater seepage was encountered at a depth of 9’ during drilling;
- auger refusal at 41.5’.

B-002-0-21 (drilled behind east abutment of existing bridge):

- 0’ – 0.5’: topsoil;
- 0.5’ – 1.0’: stiff dark gray SANDY SILT (A-4a), fill, damp;
- 1.0’ – 4.5’: medium dense brown STONE FRAGMENTS WITH SAND AND SILT (A-2-4), damp; brown clay layers near 4’;
- 4.5’ – 10.5’: loose brown STONE FRAGMENTS WITH SAND (A-1-b), damp to wet; gray clay layers near 8’;
- 10.5’ – 20.0’: loose to medium dense brown STONE FRAGMENTS (A-1-a), wet;
- 20.0’ – 22.0’: dark brown highly weathered SHALE, argillaceous;
- 22.0’ – 41.5’: dark gray moderately weathered SHALE, friable;
- groundwater seepage was encountered at a depth of 9’ during drilling;
- auger refusal at 41.5’.

6.0 ANALYSES AND RECOMMENDATIONS

6.1 Foundations

GCI reviewed two foundation types for the new bridge abutments; our analyses assumed a bottom of pile cap elevation 9 feet below existing pavement based on information provided by ADR:

- Drilled Shafts: In our opinion, drilled shafts bearing within shale bedrock would be a feasible option from a geotechnical standpoint. However, we expect that drilled shafts would not be as economically feasible as driven piles.

- Driven Piles: Driven piles would be driven to end bear in shale bedrock. Section 305.3.5.7 of the 2021 ODOT Bridge Design Manual (BDM) requires that piles attain a minimum 15-foot embedment (soil + rock) if soil depth exceeds 10 feet (as is expected for both abutments. Our analyses showed embedment exceeding 15 feet, so driven piles should be a feasible foundation option for the new bridge.

Drivability analyses were performed using the GRLWEAP computer program. Pile driving was assumed to begin 9 feet below the road surface at both abutments. The subsurface conditions revealed in boring B-001-0-21 were divided into seven (7) layers with these layers entered into GRLWEAP for the west abutment. The four (4) upper layers were soil. Layers 5 – 7 were within shale bedrock. The subsurface materials in boring B-002-0-21 were divided into six (6) layers, with the upper four (4) as soil and bottom two (2) as shale bedrock.

Per AASHTO LRFD Article 10.7.3.2.2, “soft rock” (which is what we consider this shale to be, due to spoon and auger penetration), bearing resistance design should be in accordance with AASHTO LRFD Article 10.7.3.8. Side resistance (skin friction) was determined using the α-Method presented in AASHTO LRFD Article 10.7.3.8.6b. Tip resistance (end bearing) was evaluated using the method presented in AASHTO LRFD Article 10.7.3.8.6e. Note that both of these methods use the undrained shear strength (s_u). The unconfined compressive strength (q_u) of the shale bedrock ($q_u = 2 \bullet s_u$) was evaluated using the document “Modified Standard Penetration Test-Based Drilled Shalt Design Method for Weak Rocks” (research report no. FHWA-ICT-17-018, dated December 2017). Our analyses and boring log layers are shown on sheets 1 to 10 in the Appendix.

After attaining our soil / shale input values, our drivability analyses were performed. BDM Section C305.3.1.2 states single-acting diesel pile driving hammers having a rated energy of up to 44,000-ft-lbs are commonly available in Ohio; consistent with this section, GCI used a Delmag D 14-42 open-ended diesel pile driving hammer for our drivability analyses. The GRLWEAP program lists the Delmag D 14-42 hammer having a ram weight of 3,086 lbs and energy per blow of 34.501-ft-lbs. Per BDM Section 305.3.1.2, piles end bearing in bedrock attain refusal with 20 hammer blows over one-inch of penetration. A blow count of 240 blows / foot (≈ 20 blows / inch) was used as our assumed tip depth. The results of our drivability analyses are given below:

West Abutment (boring B-001-0-21):

- Pile penetration will begin 9 feet below top of road surface / boring surface
- HP10x42 used in analysis
- Driving refusal (240 blows / foot) at ≈ 29 feet of penetration; this is about 38 feet below the road surface / boring surface
- The maximum compressive strength in the pile shown in the drivability analysis was at ≈ 35 ksi near driving refusal; this is below the maximum internal stress of 45 ksi attained from AASHTO LRFD Equation 10.7.8-1.
- Per BDM Section 305.3.5.2, the estimated pile length is 30’
- Per BDM Section 305.3.5.2, the pile order length is 35’

East Abutment (boring B-002-0-21):

- Pile penetration will begin 9 feet below top of road surface / boring surface
- HP10x42 used in analysis
- Driving refusal (240 blows / foot) at ≈ 20 feet of penetration; this is about 29 feet below the road surface / boring surface

- The maximum compressive strength in the pile shown in the drivability analysis was at ≈ 38 ksi near driving refusal; this is below the maximum internal stress of 45 ksi attained from AASHTO LRFD Equation 10.7.8-1.
- Per BDM Section 305.3.5.2, the estimated pile length is 25’
- Per BDM Section 305.3.5.2, the pile order length is 30’

The factored structural resistance (P_r) for abutment piles driven to refusal on bedrock will be governed by the structural capacity of the pile itself. BDM Section 305.3.3 notes the following maximum factored structural resistance values for select H-pile sections:

H-Pile Size	P_r
HP10X42	310 kips
HP12X53	380 kips
HP14X73	530 kips

These values assume:

- An axially loaded pile with negligible moment
- No appreciable loss of section due to deterioration throughout the life of the structure
- A minimum steel yield strength of 50 ksi
- A structural resistance factor for H-piles of 0.5 due to severe driving conditions
- A fully braced pile along its length
- Per BDM Section 305.3.5.6, Condition D, steel points should be provided for the H-piles.

Based on our analysis, we conclude it is feasible to drive H-piles to refusal on shale bedrock. The contractor will need to properly select the pile hammer size large enough to achieve the ultimate bearing value for piles driven to refusal on bedrock, without overstressing the pile. Pile design should consider unbraced lengths due to potential scour.

6.2 Scour

A scour study was beyond the scope of our services for the project. As a minimum for scour mitigation, we recommend the placement of Rock Channel Protection along the entire length of abutments and wing walls. As stated in the Federal Highways Administration (FHWA) “Hydraulic Engineering Circular No. 18” (HEC-18), rip-rap is not a permanent countermeasure against scour, nor does it eliminate the potential for scour. Therefore, we recommend that the bridge be periodically inspected, particularly after major storm events, to ensure the rip-rap blanket is properly preserved. D_{50} values from our borings are presented below.

Boring	Sample	Depth	D_{50} values	Boring	Sample	Depth	D_{50} values
B-001-0-21	SS-1	0’ – 1.5’	0.1649 mm	B-002-0-21	SS-1	0’ – 1.5’	0.2171 mm
B-001-0-21	SS-2	2.5’ – 4’	0.0064 mm	B-002-0-21	SS-2	2.5’ – 4’	0.8183 mm
B-001-0-21	SS-4	7.5’ – 9’	3.0996 mm	B-002-0-21	SS-3	5’ – 6.5’	4.1787 mm
B-001-0-21	SS-6	10.5’ – 12’	0.4362 mm	B-002-0-21	SS-4	7.5’ – 9’	5.6804 mm
B-001-0-21	SS-7	12’ – 13.5’	9.7436 mm	B-002-0-21	SS-5	9’ – 10.5’	3.4 mm
B-001-0-21	SS-9	15’ – 16.5’	26.7928 mm	B-002-0-21	SS-6	10.5’ – 12’	8.8875 mm
B-001-0-21	SS-10	17.5’ – 19’	1.0777 mm	B-002-0-21	SS-7	12’ – 13.5’	8.5775 mm
				B-002-0-21	SS-8	13.5’ – 15’	6.0301 mm
				B-002-0-21	SS-9	15’ – 16.5’	3.6211 mm

7.0 CONSTRUCTION MATERIALS ENGINEERING AND TESTING

GCI provides construction materials engineering and testing services. For project continuity throughout construction, we recommend GCI be retained to observe, test, and document:

- earthwork procedures,
- driven pile installation observations,
- reinforcing steel and concrete observation and testing, and
- structural steel (welds, bolts, etc.).

The purpose of this work is to assess that our recommendations are being followed and to make timely changes to our recommendations (as needed) in the event site conditions vary from those encountered in our borings. Please contact our field department to initiate these services.

8.0 FINAL

In the event that changes to the nature, design, or location of the proposed bridge are planned, the conclusions and recommendations contained in this report shall not be considered valid, unless the changes are reviewed and conclusions of this report are modified or verified by Geotechnical Consultants, Inc. This report is for design purposes only and is not sufficient to prepare an accurate bid. GCI appreciates the opportunity to work with you on this project. If you have any questions or the need for additional service, please call.

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APPENDIX – MOE-CR29-06.95 Bridge Replacement – Monroe County, Ohio

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 procedes 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% -50%

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		LL_O/LL × 100*	% Pass #40	% Pass #200	Liquid Limit (LL)	Plastic Index (PI)	Group Index Max.	REMARKS
		AASHTO	OHIO							
○	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

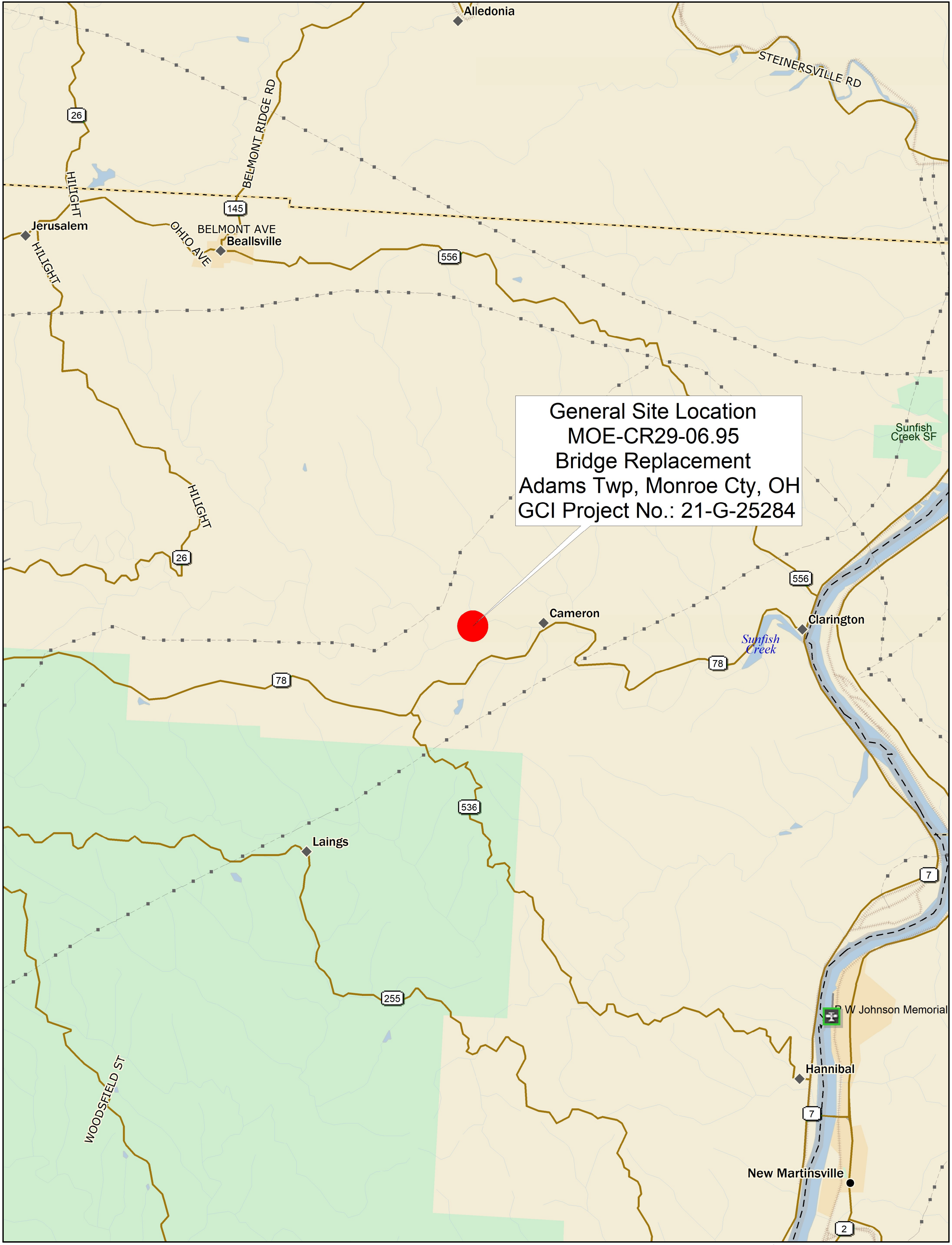
☒ Pavement or Base

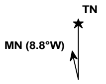
⁂ Uncontrolled Fill (Describe)

▪ Bouldery Zone

● Peat, S-Sedimentary W-Woody F-Fibrous L-Loamy & etc

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





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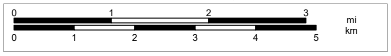
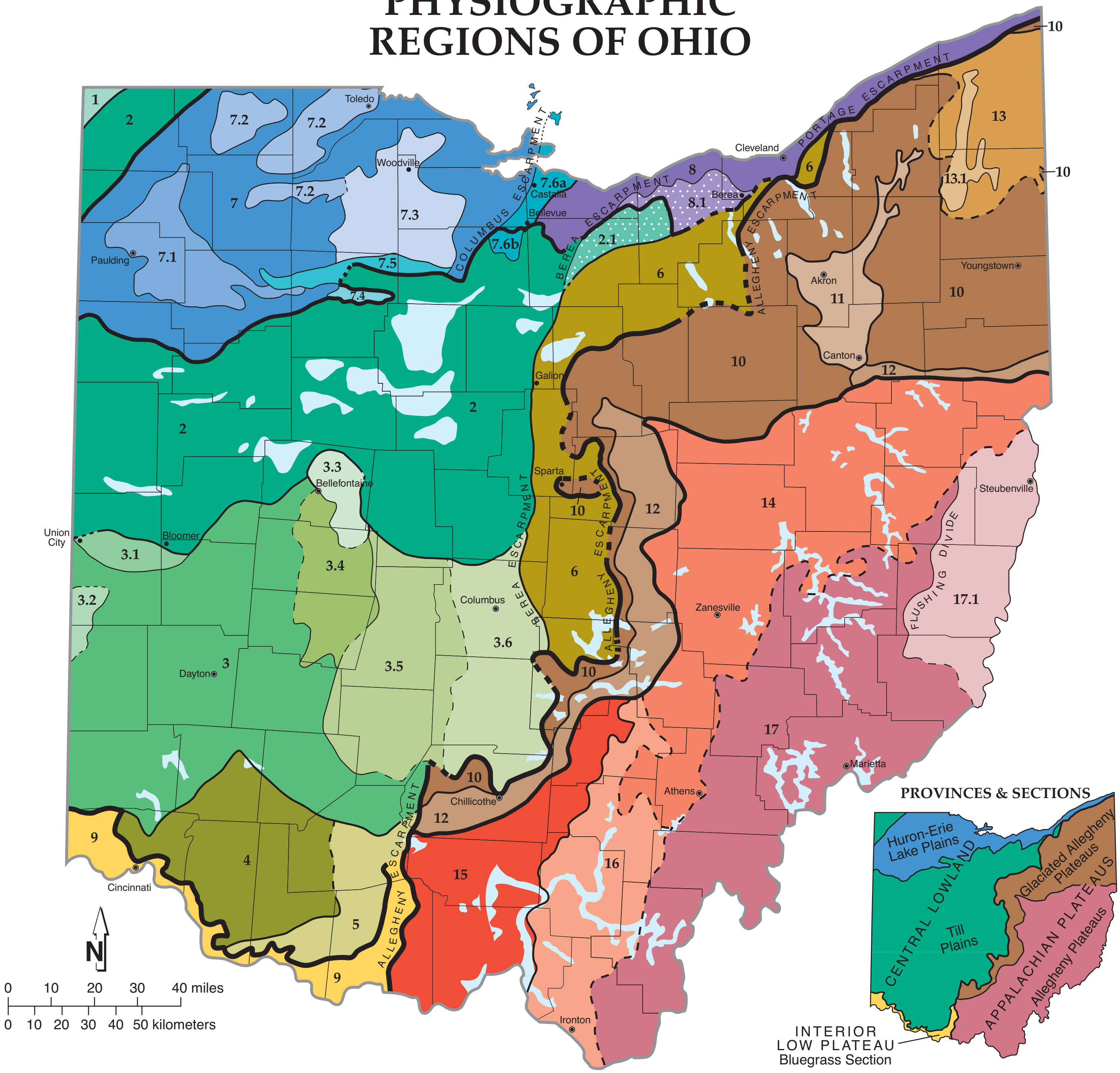


1" = 1.97 mi

Data Zoom 10-6

STATE OF OHIO • DEPARTMENT OF NATURAL RESOURCES • DIVISION OF GEOLOGICAL SURVEY

PHYSIOGRAPHIC REGIONS OF OHIO



Till Plains

- 1. Steuben Till Plain
- 2. Central Ohio Clayey Till Plain
  - 2.1. Berea Headlands of the Till Plain
- 3. Southern Ohio Loamy Till Plain
  - 3.1. Union City-Bloomer Transitional Terrain
  - 3.2. Whitewater Interlobate Plain
  - 3.3. Bellefontaine Upland
  - 3.4. Mad River Interlobate Plain
  - 3.5. Darby Plain
  - 3.6. Columbus Lowland
- 4. Illinoian Till Plain
- 5. Dissected Illinoian Till Plain
- 6. Galion Glaciated Low Plateau

Transitional boundary

Lake basin/deposits outside Huron-Erie Lake Plains

Huron-Erie Lake Plains

- 7. Maumee Lake Plains
  - 7.1. Paulding Clay Basin
  - 7.2. Maumee Sand Plains
  - 7.3. Woodville Lake-Plain Reefs
  - 7.4. Findlay Embayment
  - 7.5. Fostoria Lake-Plain Shoals
  - 7.6a and 7.6b. Bellevue-Castalia Karst Plain
- 8. Erie Lake Plain
  - 8.1. Berea Headlands of the Erie Lake Plain

Bluegrass Section

- 9. Outer Bluegrass Region

Glaciated Allegheny Plateaus

- 10. Killbuck-Glaciated Pittsburgh Plateau
- 11. Akron-Canton Interlobate Plateau
- 12. Illinoian Glaciated Allegheny Plateau
- 13. Grand River Low Plateau
  - 13.1 Grand River Finger-Lake Plain

Allegheny Plateaus

- 14. Muskingum-Pittsburgh Plateau
- 15. Shawnee-Mississippian Plateau
- 16. Ironton Plateau
- 17. Marietta Plateau
  - 17.1. Little Switzerland Plateau

4/98

Recommended citation: Ohio Division of Geological Survey, 1998, Physiographic regions of Ohio: Ohio Department of Natural Resources, Division of Geological Survey, page-size map with text, 2 p., scale 1:2,100,00.

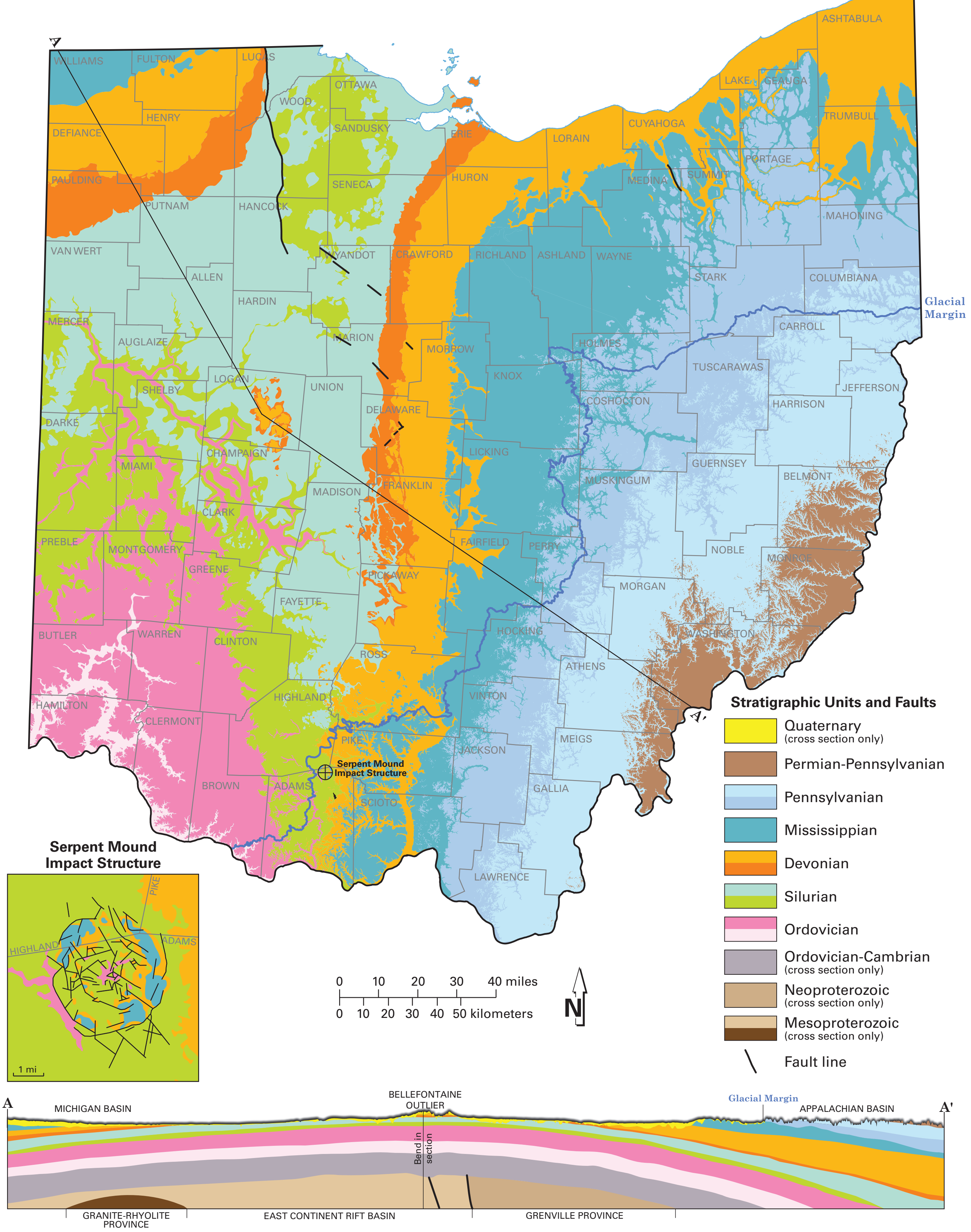
PHYSIOGRAPHIC REGIONS OF OHIO

Major Divisions	Provinces	Sections *	DISTINGUISHING CHARACTERISTICS OF REGIONS & DISTRICTS	GEOLOGY	BOUNDARIES
INTERIOR PLAINS	CENTRAL LOWLAND	Till Plains	**1. Steuben Till Plain.** Hummocky terrain with rolling hills, interspersed flats and closed depressions; wetlands, few streams, deranged drainage; only a small part of the region is in Ohio; elevation 950'-1100', moderately low relief (60')	Wisconsinan-age (latest Ice-Age) loamy till from a northern source (Saginaw glacial lobe) over Mississippian-age Coldwater Shale	Southeast: edge of Wabash Moraine
			2. Central Ohio Clayey Till Plain. Surface of clayey till; well-defined moraines with intervening flat-lying ground moraine and intermorainal lake basins; no boulder belts; about a dozen silt-, clay- and till-filled lake basins range in area from a few to 200 square miles; few large streams; limited sand & gravel outwash; elevation 700'-1150', moderate relief (100')	Clayey, high-lime Wisconsinan-age till from a northeastern source (Erie glacial lobe) and lacustrine materials over Lower Paleozoic-age carbonate rocks and, in the east, shales; loess thin to absent	North: Lake Plain; northeast: limit of Berea Sandstone; east: Berea Escarpment; south: Powell and Union City/Bloomer Moraines; northern segment boundaries: Wabash Moraine and lake plain
			2.1. Berea Headlands of the Till Plain. Gently rolling to flat terrain of thin drift descending to Lake Erie; punctuated by more than 20 streamlined "whalebacks" of Berea Sandstone, 0.5 to 2.5 miles long, 30'-60' high; somewhat poorly drained; elevation 800'-1000', low relief (20')	Thin, clayey, medium-lime Wisconsinan-age till over resistant Mississippian-age Berea Sandstone	South: limit of Berea Sandstone; elsewhere: Berea Escarpment and/or margin of highest Pleistocene lake
			3. Southern Ohio Loamy Till Plain. Surface of loamy till; end and recessional moraines, commonly associated with boulder belts, between relatively flat-lying ground moraine, cut by steep-valleyed large streams; stream valleys filled with outwash and alternate between broad floodplains and narrows; buried valleys common; elevation 530'-1150', moderate relief (200')	Loamy, high-lime Wisconsinan-age till, outwash, and loess over Lower Paleozoic-age carbonate rocks and, in the east, shales	East: Berea and Allegheny Escarpments; north: Powell and Union City/Bloomer Moraines; south: limit of Wisconsinan-age till
			3.1. Union City-Bloomer Transitional Terrain. Well-defined moraines with low-relief, hummocky ground moraine like the Central Ohio Clayey Till Plain to the north; loamy till with loess cap like Southern Ohio Loamy Till Plain to the south; elevation 920'-1075', moderately low relief (30')	Loamy, high-lime Wisconsinan-age till with thin loess cap over Silurian-age dolomites	North: Bloomer Moraine and limit of loamy till; south: Union City Moraine
			3.2. Whitewater Interlobate Plain. An upland between two converging glacial lobes with hummocky moraines, moraine complexes, kames, boulder belts, and broad outwash trains/plains; contains highest elevations in Indiana (1257') and in adjacent Ohio counties (1240'); elevation in Ohio 980'-1240', moderate relief (150')	Loamy, high-lime Wisconsinan-age till and sand and gravel outwash over resistant Silurian-age carbonate rocks (north) and less resistant Ordovician-age shales and limestones (south)	North: limit of Knightstown/Farmersville Moraines and kame fields; east: high, dissected hills draining to Whitewater River
			3.3. Bellefontaine Upland. Moderately high relief (250') dissected topography with moraine complexes, boulder belts, high-gradient major streams, caves and sinkholes; few glacial depressions/kettles compared to surrounding areas; elevation 1100'-1549', includes highest elevation in Ohio (Campbell Hill, 1549')	Loamy, high-lime Wisconsinan-age till over generally deeply buried Silurian- to Devonian-age carbonate rocks and Ohio Shale	North: areas with hilltops above 1200'; elsewhere: hilltops above about 1300'
			3.4. Mad River Interlobate Plain. Area between two major converging glacial lobes with extensive outwash, outwash terraces, and bordering moraines; springs and cool, ground-water-fed surface waters; elevation 800'-1350', moderate relief (200')	Loamy, high-lime Wisconsinan-age till and sand and gravel outwash over Silurian- to Devonian-age carbonate rocks and Ohio Shale	East and north: rear edge of Cable Moraine Complex; south: outwash to Clifton Gorge; west: western edge of Mad River Outwash
			3.5. Darby Plain. Moderately low relief (25'), broadly hummocky ground moraine with several broad, indistinct recessional moraines; between hummocks are broad, poorly drained swales which held wet prairies/meadows in pioneer days; few large streams; elevation 750'-1100'	Loamy, high-lime Wisconsinan-age till and sparse outwash over Silurian- and Devonian-age carbonate rocks and Ohio Shale in the southeast	South and west: front of Reesville and rear of Cable Moraines; north: Powell Moraine; east: increasing eastward slope (see 3.6)
			3.6. Columbus Lowland. Lowland surrounded in all directions by relative uplands, having a broad regional slope toward the Scioto Valley; many larger streams; elevation 600'-850' (950' near Powell Moraine), moderately low relief (25')	Loamy, high-lime (west) to medium-lime (east) Wisconsinan-age till and extensive outwash in Scioto Valley over deep Devonian- to Mississippian-age carbonate rocks, shales, and siltstones	North: Powell Moraine; east and south: Berea and/or Allegheny Escarpments; west: flatter and higher Darby Plain
			4. Illinoian Till Plain. Rolling ground moraine of older till generally lacking ice-constructional features such as moraines, kames, and eskers; many buried valleys; modern valleys alternating between broad floodplains and bedrock gorges; elevation 600'-1100', moderately low relief (50')	Silt-loam, high-lime, Illinoian-age till with loess cap; soils leached several feet; underlain by Ordovician- and Silurian-age carbonate rocks and calcareous shales	North: Wisconsinan glacial margin (Cuba and Hartwell Moraines); elsewhere: limit of common till-covered hillslopes
			5. Dissected Illinoian Till Plain. Hilly former till plain in which glacial deposits have been eroded from many valley sides; relatively high stream density; elevation 600'-1340', moderate relief (200')	Hilltops of high-lime Illinoian-age till with loess cap; slopes of bedrock- and till-derived colluvium and Ordovician- and Silurian-age carbonate rocks and calcareous shales	East: maximum glacial margin; elsewhere: limit of general absence of till on hillslopes
			6. Galion Glaciated Low Plateau. Rolling upland transitional between the gently rolling Till Plain and the hilly Glaciated Allegheny Plateau; mantled with thin to thick drift; elevation 800'-1400', moderate relief (100')	Medium- to low-lime Wisconsinan-age till over Mississippian-age shales and sandstones	North: limit of Berea Sandstone; west: Berea Escarpment; south and east: Allegheny Escarpment
		Huron-Erie Lake Plains	**7. Maumee Lake Plains.** Flat-lying Ice-Age lake basin with beach ridges, bars, dunes, deltas, and clay flats; contained the former Black Swamp; slightly dissected by modern streams; elevation 570'-800', very low relief (5')	Pleistocene-age silt, clay, and wave-planed clayey till over Silurian- and Devonian-age carbonate rocks and shales	Northeast: Lake Erie; elsewhere: margin of highest Pleistocene lake
			7.1. Paulding Clay Basin. Nearly flat lacustrine plain; most clayey of all Lake Plain subregions; low-gradient, highly meandering streams; easily ponded soils; elevation 700'-725', extremely low relief (less than 5')	Pleistocene-age lacustrine clay over clay till and Silurian-age dolomites	Northeast: subdued ("drowned") remnant of Defiance Moraine; elsewhere: limit of lacustrine clay
			7.2. Maumee Sand Plains. Lacustrine plain mantled by sand; includes low dunes, inter-dunal pans, beach ridges, and sand sheets of glacial lakeshores; well to poorly drained; elevation 600'-800', very low relief (10')	Late Wisconsinan-age sand over clay till and lacustrine deposits; Silurian- and Devonian-age carbonate rocks and shales buried deeply.	Limit of sandy deposits and/or low dunes
			7.3. Woodville Lake-Plain Reefs. Very low relief (10') lacustrine plain with low dunes and lake-margin features, punctuated by more than 75 ancient bedrock reefs rising 10' to 40' above the level of the plain and ranging in area from 0.1 to 3.0 square miles; the oblong reefs are thinly draped with drift; elevation 600'-775'	Thin to absent Wisconsinan-age wave-planed clay till, lacustrine deposits, and sand over Silurian-age reefal Lockport Dolomite	Limit of thinly mantled Lockport Dolomite (Bowling Green Fault to the west and the Defiance Moraine to the south)
			7.4. Findlay Embayment. Very low relief (10'), broadly rolling lacustrine plain; embayment of ancestral Lake Erie in which relatively coarse lacustrine sediments collected; elevation 775'-800'	Silty to gravelly Wisconsinan-age lacustrine deposits and wave-planed clayey till over Silurian-age Lockport Dolomite	West: 775' beach ridge; north: Defiance Moraine; south: margin of highest Pleistocene lake level
			7.5. Fostoria Lake-Plain Shoals. Portion of the Defiance Moraine lightly eroded by shallow Lake Maumee with low north-south trending hillocks and shallow, closed depressions; many sandy areas; elevation 750'-825', low relief, decreasing westward (10'-15')	Silty to gravelly Wisconsinan-age lacustrine deposits and wave-planed clay till over deeply covered Silurian-age dolomite	South and east: unmodified Defiance Moraine; elsewhere: very low-relief lake plain
			7.6a and 7.6b. Bellevue-Castalia Karst Plain. Hummocky plain of rock knobs and numerous sinkholes, large solution features, and caves; large springs; thinly mantled by drift; region straddles both Lake Plain (7.6a) and Till Plain (7.6b); 7.6a has greatest relief of any Lake Plain region (25'); elevation 570'-825'	Columbus and Delaware Limestones overlain by thin clay till in 7.6b, and thin silty and sandy Wisconsinan-age lacustrine deposits and wave-planed clay till in 7.6a	Limit of thinly mantled Columbus and Delaware Limestones, which is marked in the west by the Columbus Escarpment
			8. Erie Lake Plain. Edge of very low-relief (10') Ice-Age lake basin separated from modern Lake Erie by shoreline cliffs; major streams in deep gorges; elevation 570'-800'	Pleistocene-age lacustrine sand, silt, clay, and wave-planed till over Devonian- and Mississippian-age shales and sandstones	North: Lake Erie; south: margin of highest Pleistocene lake
			8.1 Berea Headlands of the Erie Lake Plain. Portion of the Erie Lake Plain underlain by resistant Berea Sandstone; several large sandstone headlands jut into the Ice-Age lake basin; contains several streamlined "whalebacks" of Berea Sandstone, 0.5 to 2.0 miles long, 20'-35' high; poorly drained; elevation 670'-800', very low relief (10')	Thin lacustrine deposits over thin, wave-planed, clayey, medium-lime Wisconsinan-age till; underlain by resistant Berea Sandstone	North: portion of Lake Plain underlain by soft shales; south: margin of highest Pleistocene lake
	INT. LOW PLATEAUS	Bluegrass Section	**9. Outer Bluegrass Region.** Moderately high relief (300') dissected plateau of carbonate rocks; in east, caves and other karst features relatively common; in west, thin, early drift caps narrow ridges; elevation 455'-1120'	Ordovician- and Silurian-age dolomites, limestones, and calcareous shales; thin pre-Wisconsinan drift on ridges in west; silt-loam colluvium	Eastern segment: maximum glacial margin and high eastern ridges capped by noncarbonate rocks; connected by Ohio River bluffs to western segment which is bounded by nondissected till plain
APPALACHIAN HIGHLANDS	APPALACHIAN PLATEAUS	Glaciated Allegheny (Southern New York) Plateaus	**10. Killbuck-Glaciated Pittsburgh Plateau.** Ridges and flat uplands generally above 1200', covered with thin drift and dissected by steep valleys; valley segments alternate between broad drift-filled and narrow rock-walled reaches; elevation 600'-1505', moderate relief (200')	Thin to thick Wisconsinan-age clay to loam till over Mississippian- and Pennsylvanian-age shales, sandstones, conglomerates and coals	West and north: resistant sandstones of the Allegheny and Portage Escarpments; south and east: Wisconsinan glacial margin
			11. Akron-Canton Interlobate Plateau. Hummocky area between two converging glacial lobes dominated by kames, kame terraces, eskers, kettles, kettle lakes, and bogs/fens; deranged drainage with many natural lakes; elevation 900'-1200', moderate relief (200')	Sandy Wisconsinan-age and older drift over Devonian- to Pennsylvanian-age sandstones, conglomerates and shales	Limit of common, sandy ice-contact features and deposits
			12. Illinoian Glaciated Allegheny Plateau. Dissected, rugged hills; loess and older drift on ridgetops, but absent on bedrock slopes; dissection similar to unglaciated regions of the Allegheny Plateau; elevation 600'-1400', moderate relief (200')	Colluvium and Illinoian-age till over Devonian- to Pennsylvanian-age shales, siltstones and sandstones	North and west: Wisconsinan glacial margin; south and east: Illinoian (maximum) glacial margin
			13. Grand River Low Plateau. Gently rolling ground and end moraine having thin to thick drift; poorly drained areas and wetlands relatively common; elevation 760'-1200', low relief (20') except near Grand River Valley (200')	Clayey, low-lime Wisconsinan-age till over deeply buried, soft Devonian-age shales and near-surface Mississippian-age sandstones and shales	North: Portage Escarpment; south and west: Defiance Moraine; southeast: increasing relief from proximity of buried Pennsylvanian-age sandstones
			13.1. Grand River Finger-Lake Plain. Very low relief (10') lake deposits in steep-sided troughs (200' relief) within the Grand River Low Plateau; cut by glacial and stream erosion; extensive wetlands; elevation 800'-900'	Surficial lacustrine clay and drift over deeply buried, soft Devonian-age shales	Margins of steeply sloping troughs containing the Grand River and parts of Rock and Mosquito Creeks
		Allegheny (Kanawha) Plateaus	**14. Muskingum-Pittsburgh Plateau.** Moderately high to high relief (300'-600') dissected plateau having broad major valleys that contain outwash terraces, and tributaries with lacustrine terraces; medium-grained bedrock sequences coarser than those in Marietta Plateau (17) but finer than those in Ironton Plateau (16); remnants of ancient Teays-age drainage system uncommon; elevation 650'-1400'	Mississippian and Pennsylvanian-age siltstones, shales, sandstones and economically important coals and claystones; Wisconsinan-age sand, gravel, and lacustrine silt; silt-loam colluvium	North and west: maximum glacial margin; southeast: transition to finer grained bedrock; southwest: transition to coarser grained bedrock
			15. Shawnee-Mississippian Plateau. High relief (400'-800'), highly dissected plateau of coarse and fine grained rock sequences; most rugged area in Ohio; remnants of ancient lacustrine clay-filled Teays drainage system are extensive in lowlands, absent in uplands; elevation 490'-1340'	Devonian- and Mississippian-age shales, siltstones, and locally thick sandstones; Pleistocene-age sandy outwash in Scioto River; Teays-age Minford Clay; silt-loam and channery colluvium	North: Maximum glacial margin; west:: carbonate bedrock; east: limit of Mississippian-age bedrock
			16. Ironton Plateau. Moderately high relief (300') dissected plateau; coarser grained coal-bearing rock sequences more common than in other regions of the Allegheny Plateau; common lacustrine clay-filled Teays Valley remnants; elevation 515'-1060'	Pennsylvanian-age (Pottsville, Allegheny and Conemaugh Groups) cycles of sandstones, siltstones, shales and economically important coals; Pleistocene (Teays)-age Minford Clay; silt-loam and channery colluvium	West: limit of common Pennsylvanian-age bedrock; north and east: gradation to finer rock sequences
			17. Marietta Plateau. Dissected, high-relief (generally 350', to 600' near Ohio River) plateau; mostly fine-grained rocks; red shales and red soils relatively common; landslides common; remnants of ancient lacustrine clay-filled Teays drainage system common; elevation 515'-1400'	Pennsylvanian-age Upper Conemaugh Group through Permian-age Dunkard Group cyclic sequences of red and gray shales, and siltstones, sandstones, limestones and coals; Pleistocene (Teays)-age Minford Clay; red and brown silty-clay loam colluvium; landslide deposits	North and west: transition to medium-grained Lower Conemaugh rocks; east: Flushing Divide
			17.1. Little Switzerland Plateau. Highly dissected, high-relief (generally 450', to 750' along Ohio River) plateau; mostly fine-grained rocks; red shales and red soils relatively common; landslides common; high-gradient shale-bottomed streams subject to flash flooding; no remnants of ancient Teays drainage system; elevation 540'-1400'	Similar to Marietta Plateau but lacking Pleistocene (Teays)-age Minford Clay	North: transition to medium-grained rocks; west and south: Flushing Divide; east: Ohio River

* Section names modified from Fenneman (1938, 1946).

STATE OF OHIO • DEPARTMENT OF NATURAL RESOURCES • DIVISION OF GEOLOGICAL SURVEY

BEDROCK GEOLOGIC MAP OF OHIO



Recommended citation: Ohio Division of Geological Survey, 2006, Bedrock geologic map of Ohio: Ohio Department of Natural Resources, Division of Geological Survey Map BG-1, generalized page-size version with text, 2 p., scale 1:2,000,000. [Revised 2017.]

This map is a generalization of the *Bedrock Geologic Map of Ohio* (Slucher and others, 2006)—the first statewide 1:500,000-scale bedrock-geology map compiled by the ODNR Division of Geological Survey since 1920 and the first to properly portray the bedrock geology that exists beneath the extensive deposits of Quaternary sediments that cover much of the bedrock in the state[1]. Overall, the bedrock geology of Ohio consists of flat-lying to gently dipping carbonate, siliciclastic, evaporite, and organoclastic strata of sedimentary origin that range in age from Upper Ordovician to Upper Carboniferous-Lower Permian. As illustrated in the cross section, older sedimentary, igneous, and metamorphic rocks occur at depth and range from Lower Ordovician to Mesoproterozoic in age. At the surface, an irregular veneer of mainly unconsolidated Quaternary sediments conceal most bedrock units occurring northward and westward of the glacial margin.

Strata of the Ordovician System are the oldest exposed rocks in Ohio and consist mainly of alternating shale and limestone sequences. Silurian System strata are mostly dolomites with lesser amounts of shale. Rocks of the Devonian System consist of two contrasting types. Lower and Middle Devonian-age strata are mainly carbonate rocks, whereas Upper Devonian-age rocks consist mostly of clastic rocks. In Champaign and Logan Counties, Devonian-age rocks occur on a small erosional remnant referred to by geologists as the Bellefontaine Outlier. Coincidentally, the highest topographic point in Ohio (Campbell Hill at 1,549 feet above sea level) occurs also in this area.

The Carboniferous System is divided into two Subsystems, the Mississippian and Pennsylvanian. Mississippian-age strata are mostly shales and sandstones that occur locally in various proportions. Pennsylvanian-age strata consist mainly of a diverse array of alternating sandstones, siltstones, shales, mudstones, limestones, and underclays; economic coal beds occur also in portions of this sequence. The youngest interval of sedimentary rocks in Ohio, the Dunkard Group, occurs only in southeastern Ohio and consists of strata similar in composition to the underlying Upper Pennsylvanian-age rocks; however, the age of the Dunkard Group has been debated since the late 1800s. Dunkard strata contain a well-studied late Pennsylvanian-age assemblage of plant fossils with infrequent early Permian-age forms. Yet, fossil plant spores found in coal beds in the interval only support a late, but not latest Pennsylvanian age. Thus until more definitive fossils are found, geologists are unable to determine the exact age of the Dunkard Group beyond a combined Permian-Pennsylvanian age assignment.

In west-central Ohio, the ancient Teays River system extended across much of Ohio during the late Neogene to early Quaternary Periods and sculptured an extensive network of deeply dissected valleys into the bedrock surface. The spatial configuration of many geologic units on this map clearly reflects the major channel networks of these former drainage systems. Also, four major regional structural geology elements affect the spatial distribution of rocks in Ohio: the Appalachian and Michigan Basins and the Cincinnati and Findlay Arches, which occur between the two basins. Locally, several high-angle normal faults displace rocks in the state.

The Serpent Mound Impact Structure in southern Ohio is a circular area of deformed and broken rocks that is approximately nine miles in diameter. Recent investigations indicate the feature is the result of a meteorite or comet impact believed to have occurred between 256 and 330 million years ago.

Cross section A–A' traverses Ohio from the northwest to the southeast and intersects the southern portion of the Michigan Basin, the area between the Cincinnati and Findlay Arches, and the western Appalachian Basin, respectively. The stratigraphic units shown in this profile illustrate the broad, arching geometric distortion to the bedrock in Ohio, created mainly by periods of tectonic subsidence within these regional structural basins. For specific details on the various rock units, economic commodities, and geologic hazards within Ohio, see the large-format *Bedrock Geologic Map of Ohio* (Slucher and others, 2006), available for purchase by contacting the ODNR Geologic Records Center at 614-265-6576 or geo.survey@dnr.state.oh.us.

🟨 **Quaternary** (about 1.8 million years ago to present). Unconsolidated sediments: till, gravel, sand, silt, clay, and organic debris. Continental origin. (Shown in cross section only)

Period of widespread erosion

🟫 **Permian and Pennsylvanian** (about 298 to 302 million years ago). Sedimentary rocks: mainly shale, sandstone, siltstone, mudstone, and minor coal. Continental origin.

🟦 **Pennsylvanian** (about 302 to 307 million years ago). Sedimentary rocks: mainly shale, sandstone, siltstone, mudstone, limestone, and some coal. Continental and marine origin.

🟦 **Pennsylvanian** (about 307 to 318 million years ago). Sedimentary rocks: mainly sandstone, siltstone, shale, and conglomerate, with some coal and limestone. Deltaic and marine origin.

Period of widespread erosion

🟦 **Mississippian** (about 322 to 359 million years ago). Sedimentary rocks: sandstone, shale, siltstone, conglomerate, and minor limestone. Marine to marginal marine origin.

🟨 **Devonian** (about 359 to 385 million years ago). Sedimentary rocks: mainly shale and siltstone with some sandstone. Marine to marginal marine origin.

🟧 **Devonian** (about 385 to 407 million years ago). Sedimentary rocks: mainly limestone and dolomite with some shale, and minor sandstone. Marine and eolian origin.

Period of widespread erosion

🟩 **Silurian** (about 416 to 423 million years ago). Sedimentary rocks: dolomite, anhydrite, gypsum, salt, and shale. Marine and restricted marine origin.

🟩 **Silurian** (about 423 to 435 million years ago). Sedimentary rocks: dolomite and shale with some limestone. Marine origin.

Period of widespread erosion

🟪 **Ordovician** (about 446 to 450 million years ago). Sedimentary rocks: shale and limestone. Marine origin.

⬜ **Ordovician** (about 450 to 460 million years ago). Sedimentary rocks: limestone and shale. Marine origin.

Period of widespread erosion

⬜ **Ordovician and Cambrian** (about 486 to 510 million years ago). Sedimentary rocks: mainly dolomite, sandstone, shale, with minor limestone. Marine origin. (Shown in cross section only.)

Period of widespread erosion

🟫 **Neoproterozoic** (between 900 million and 1 billion years ago). Metamorphic rocks: gneiss, schist, amphibolite, and marble; and igneous rocks: granite. Form during collision of tectonic plates. (Shown in cross section only.)

🟫 **Mesoproterozoic** (between 1.0 and 1.2 billion years ago). Sedimentary rocks: sandstone and siltstone; and igneous rocks: basalt and rhyolite. Form during rifting of continental landmass. (Shown in cross section only.)

Period of widespread erosion

🟫 **Mesoproterozoic** (between 1.45 and 1.52 billion years ago). Igneous rocks: granite and rhyolite. Formed during crustal evolution and differentiation. (Shown in cross section only.)

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[1] Slucher, E.R., Swinford, E.M., Larsen, G.E., Schumacher, G.A., Shrake, D.L., Rice, C.L., Caudill, M.R., and Rea, R.G., 2006, Bedrock geologic map of Ohio: Ohio Department of Natural Resources, Division of Geological Survey Map BG-1, Version 6.0, scale 1:500,000.

BORING LOCATION PLAN

MOE-CR29-06.95 (Sunfish Creek Road) Bridge Replacement

Adams Township, Monroe County, Ohio

Aerial obtained from Google Earth, dated October 2015

Project No.: 21-G-25284

Date: 09/07/2021 Drawn By: Jeffrey Holko

Scale: Not to Scale

PROJECT: MOE-CR29-06.95
TYPE: BRIDGE REPLACEMENT
PID: 111130 BR ID: -
START: 6/24/21 END: 6/24/21

DRILLING FIRM / OPERATOR: GCI / R. BRANDUM
SAMPLING FIRM / LOGGER: GCI / R. BRANDUM
DRILLING METHOD: 3.5" SSA
SAMPLING METHOD: SPT

DRILL RIG: CME 45B (RIG 9)
HAMMER: CME AUTOMATIC
CALIBRATION DATE: 12/18/20
ENERGY RATIO (%): 84

STATION / OFFSET: -
ALIGNMENT: -
ELEVATION: - EOB: 41.5 ft.
LAT / LONG: 39.766355, -80.964914

EXPLORATION ID
B-001-0-21

PAGE
1 OF 2

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
Topsoil MEDIUM STIFF, DARK BROWN, **SANDY SILT**, LITTLE SAND, LITTLE GRAVEL, FILL, DAMP		1	2 2 3	7	11	SS-1	1.75	17	13	31	- 39 -		-	-	-	23	A-4a (V)	
MEDIUM STIFF, BROWN, **CLAY**, LITTLE SAND, TRACE GRAVEL, DAMP		2 3 4	3 3 3	8	78	SS-2	1.75	7	9	12	25	47	44	24	20	23	A-7-6 (12)	
LOOSE TO MEDIUM DENSE, BROWN, **STONE FRAGMENTS WITH SAND, SILT, AND CLAY**, SOME SAND, LITTLE SILT, LITTLE CLAY, DAMP		5 6	3 4 4	11	83	SS-3		-	-	-	-	-	-	-	-	13	A-2-6 (V)	
with zones of clay		∇ 7 8	3 3 3	8	56	SS-4		57	11	10	7	15	37	20	17	12	A-2-6 (0)	
wet		W 9 10	3 5 5	14	33	SS-5		-	-	-	-	-	-	-	-	12	A-2-6 (V)	
		11	1 1 1	3	17	SS-6		38	12	16	13	21	35	20	15	23	A-2-6 (1)	
LOOSE, BROWN, **STONE FRAGMENTS WITH SAND**, LITTLE SAND, LITTLE SILT, WET		12 13	1 1 2	4	17	SS-7		70	7	7	- 16 -		NP	NP	NP	29	A-1-b (0)	
		14	1 2 2	6	0	SS-8		-	-	-	-	-	-	-	-	-	A-1-b (V)	
MEDIUM DENSE, BROWN, **STONE FRAGMENTS**, LITTLE SAND, LITTLE SILT, WET		15 16	3 5 6	15	11	SS-9		79	5	5	- 11 -		NP	NP	NP	17	A-1-a (0)	
MEDIUM DENSE, BROWN, **GRAVEL WITH SAND**, SOME SAND, LITTLE SILT, WET		17 18 19	6 6 7	18	56	SS-10		43	26	15	- 16 -		NP	NP	NP	16	A-1-b (0)	
SHALE, BLACK, HIGHLY WEATHERED, FRIABLE		TR 20 21	8 12 40	73	44	SS-11		-	-	-	-	-	-	-	-	-	Rock (V)	
SHALE, GRAY, HIGHLY WEATHERED, FRIABLE		22 23 24	10 19 32	71	33	SS-12		-	-	-	-	-	-	-	-	-	Rock (V)	
		25 26	14 28 50/4"	-	50	SS-13		-	-	-	-	-	-	-	-	-	Rock (V)	
SHALE, GRAY, MODERATELY WEATHERED argillaceous zones		27 28 29	50	-	50	SS-14		-	-	-	-	-	-	-	-	-	Rock (V)	

PID: 111130	BR ID: -	PROJECT: MOE-CR29-06.95	STATION / OFFSET: -	START: 6/24/21	END: 6/24/21	PG 2 OF 2	B-001-0-21

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N60	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI			
SHALE, GRAY, MODERATELY WEATHERED (continued)		31 32 33	50/4"	-	50	SS-15		-	-	-	-	-	-	-	-	-	Rock (V)	
		34 35 36 37 38	50/4"	-	0	SS-16		-	-	-	-	-	-	-	-	-	Rock (V)	
		39 40 41	50/1"	-	0	SS-17		-	-	-	-	-	-	-	-	-	Rock (V)	
		EOB																

Auger refusal at 41.5'

GCI ODOT LOG - GCI OH DOT.GDT - 8/20/21 10:48 - S:\GINT\PROJECTS\21G25284.GPJ

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH AUGER CUTTINGS

GCI Job No: 21-G-25284

PROJECT: MOE-CR29-06.95
TYPE: BRIDGE REPLACEMENT
PID: 111130 BR ID: -
START: 6/24/21 END: 6/24/21

DRILLING FIRM / OPERATOR: GCI / R. BRANDUM
SAMPLING FIRM / LOGGER: GCI / R. BRANDUM
DRILLING METHOD: 3.5" SSA
SAMPLING METHOD: SPT

DRILL RIG: CME 45B (RIG 9)
HAMMER: CME AUTOMATIC
CALIBRATION DATE: 12/18/20
ENERGY RATIO (%): 84

STATION / OFFSET: -
ALIGNMENT: -
ELEVATION: - EOB: 41.5 ft.
LAT / LONG: 39.766457, -80.964803

EXPLORATION ID
B-002-0-21

PAGE
1 OF 2

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS
Topsoil		
STIFF, DARK GRAY, **SANDY SILT**, SOME SAND, SOME GRAVEL, FILL, DAMP		1
MEDIUM DENSE, BROWN, **STONE FRAGMENTS WITH SAND AND SILT**, SOME SAND, LITTLE SILT, LITTLE CLAY, DAMP brown clay layers		2 3 4
LOOSE, BROWN, **STONE FRAGMENTS WITH SAND**, LITTLE TO SOME SAND, LITTLE TO SOME SILT, DAMP with gray clay layers wet		5 6 7 ▽ 8 W 9 10
LOOSE TO MEDIUM DENSE, BROWN, **STONE FRAGMENTS**, LITTLE SAND, LITTLE SILT, WET		11 12 13 14 15 16 17 18 19 TR
SHALE, DARK BROWN, HIGHLY WEATHERED, ARGILLACEOUS		20 21
SHALE, DARK GRAY, MODERATELY WEATHERED, FRIABLE		22 23 24 25 26 27 28 29

SPT/ RQD	N60	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
3 5 5	14	44	SS-1	2.5	26	16	18	24	16	37	27	10	15	A-4a (1)	
					-	-	-	-	-	-	-	-	10	A-2-4 (V)	
5 6 7	18	50	SS-2		42	13	15	18	12	28	19	9	11	A-2-4 (0)	
3 3 3	8	50	SS-3		60	9	10	- 21 -		NP	NP	NP	9	A-1-b (0)	
2 2 3	7	44	SS-4		62	9	13	- 16 -		NP	NP	NP	19	A-1-b (0)	
4 2 3	7	50	SS-5		57	14	11	- 18 -		NP	NP	NP	16	A-1-b (0)	
8 6 6	17	61	SS-6		69	10	9	- 12 -		NP	NP	NP	14	A-1-a (0)	
6 6 8	20	44	SS-7		65	9	13	- 13 -		NP	NP	NP	13	A-1-a (0)	
5 4 3	10	44	SS-8		65	10	11	- 14 -		NP	NP	NP	15	A-1-a (0)	
4 6 10	22	39	SS-9		57	14	14	- 15 -		NP	NP	NP	15	A-1-a (0)	
6 9 11	28	56	SS-10		-	-	-	-	-	-	-	-	18	A-1-a (V)	
10 17 25	59	44	SS-11		-	-	-	-	-	-	-	-	-	Rock (V)	
50	-	67	SS-12		-	-	-	-	-	-	-	-	-	Rock (V)	
50/5"	-	60	SS-13		-	-	-	-	-	-	-	-	-	Rock (V)	
50/1"	-	50	SS-14		-	-	-	-	-	-	-	-	-	Rock (V)	

PID: 111130	BR ID: -	PROJECT: MOE-CR29-06.95	STATION / OFFSET: -	START: 6/24/21	END: 6/24/21	PG 2 OF 2	B-002-0-21

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI			
SHALE, DARK GRAY, MODERATELY WEATHERED, FRIABLE *(continued)*		31 32 33	50/1"	-	50	SS-15		-	-	-	-	-	-	-	-	-	Rock (V)	
		34 35 36 37 38	50/1"	-	0	SS-16		-	-	-	-	-	-	-	-	-	Rock (V)	
		39 40 41	50/0"	-		SS-17		-	-	-	-	-	-	-	-	-	Rock (V)	

EOB

Auger refusal at 41.5'

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH AUGER CUTTINGS AND ASPHALT PATCH

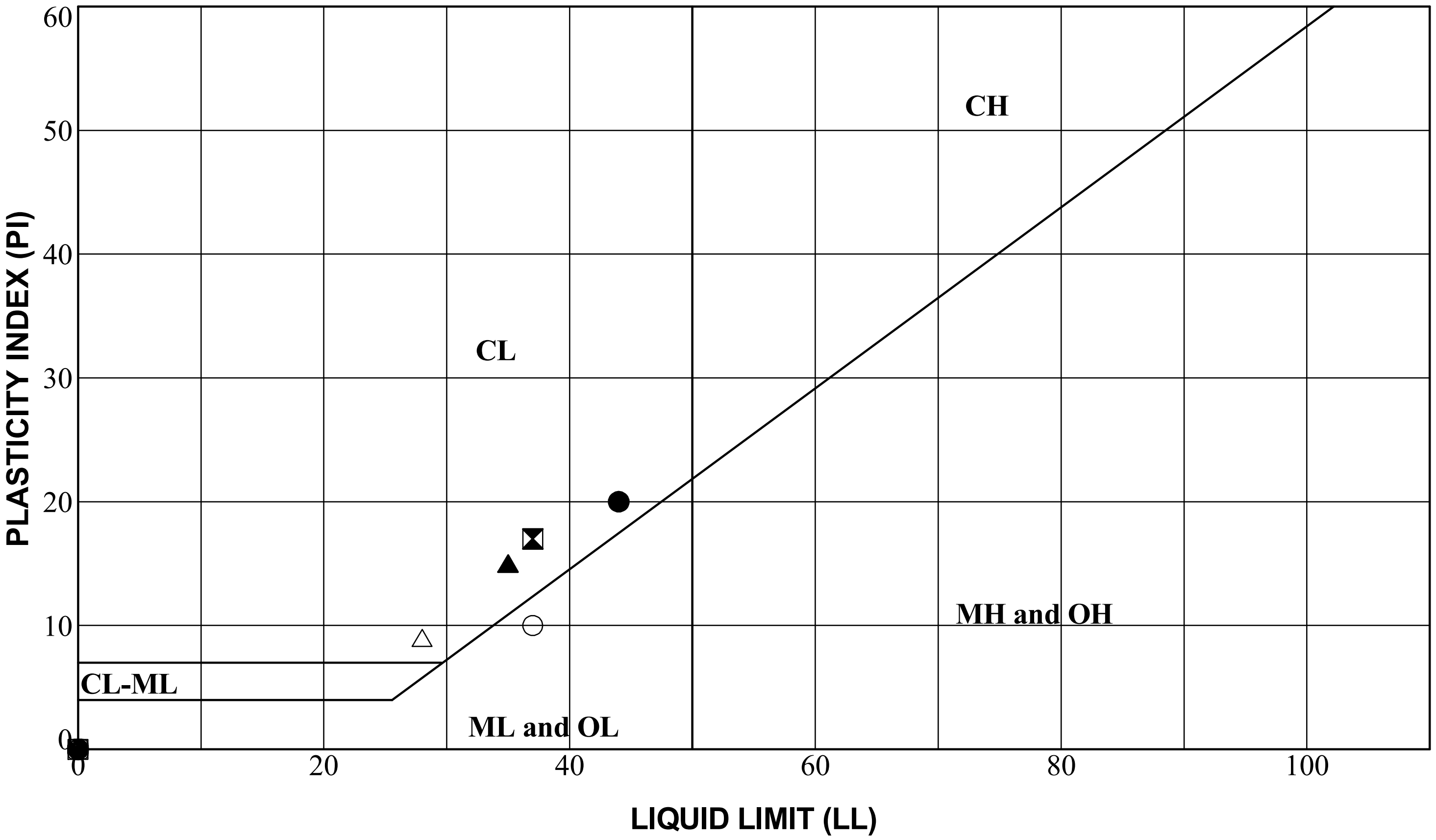
GCI Job No: 21-G-25284

Summary of Laboratory Results

MOE-CR29-06.95
Adams Township, Monroe County, Ohio
GCI Job Number: 21-G-25284

Test Hole	Depth	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	% Fines (< #200 Sieve)	% Clay (< 0.005 mm)	ODOT Class-ification	ODOT Group Index
B-001-0-21	0.0	23.3				38.5			
B-001-0-21	2.5	22.5	44	24	20	71.8	47	A-7-6	12
B-001-0-21	5.0	13.4							
B-001-0-21	7.5	11.8	37	20	17	21.6	15	A-2-6	0
B-001-0-21	9.0	11.8							
B-001-0-21	10.5	23.1	35	20	15	33.6	21	A-2-6	1
B-001-0-21	12.0	28.5	NP	NP	NP	15.8		A-1-b	0
B-001-0-21	15.0	16.7	NP	NP	NP	11.4		A-1-a	0
B-001-0-21	17.5	15.7	NP	NP	NP	16.1		A-1-b	0
B-002-0-21	0.0	14.6	37	27	10	39.9	16	A-4a	1
B-002-0-21	1.0	9.7							
B-002-0-21	2.5	10.5	28	19	9	30.1	12	A-2-4	0
B-002-0-21	5.0	8.8	NP	NP	NP	21.3		A-1-b	0
B-002-0-21	7.5	19.4	NP	NP	NP	15.7		A-1-b	0
B-002-0-21	9.0	16.3	NP	NP	NP	17.6		A-1-b	0
B-002-0-21	10.5	14.4	NP	NP	NP	12.1		A-1-a	0
B-002-0-21	12.0	12.6	NP	NP	NP	13.1		A-1-a	0
B-002-0-21	13.5	15.0	NP	NP	NP	14.0		A-1-a	0
B-002-0-21	15.0	15.1	NP	NP	NP	14.6		A-1-a	0
B-002-0-21	17.5	18.2							

August 2021



LEGEND:

							---------- ODOT ----------	
	TEST HOLE	DEPTH	w_n	LL	PL	PI	CLASS-IFICATION	GROUP INDEX
●	B-001-0-21	2.5	22.5	44	24	20	A-7-6	12
⧗	B-001-0-21	7.5	11.8	37	20	17	A-2-6	0
▲	B-001-0-21	10.5	23.1	35	20	15	A-2-6	1
★	B-001-0-21	12.0	28.5	NP	NP	NP	A-1-b	0
⦿	B-001-0-21	15.0	16.7	NP	NP	NP	A-1-a	0
✿	B-001-0-21	17.5	15.7	NP	NP	NP	A-1-b	0
○	B-002-0-21	0.0	14.6	37	27	10	A-4a	1
△	B-002-0-21	2.5	10.5	28	19	9	A-2-4	0
⊗	B-002-0-21	5.0	8.8	NP	NP	NP	A-1-b	0
⊕	B-002-0-21	7.5	19.4	NP	NP	NP	A-1-b	0
□	B-002-0-21	9.0	16.3	NP	NP	NP	A-1-b	0
❂	B-002-0-21	10.5	14.4	NP	NP	NP	A-1-a	0
◕	B-002-0-21	12.0	12.6	NP	NP	NP	A-1-a	0
☆	B-002-0-21	13.5	15.0	NP	NP	NP	A-1-a	0
❀	B-002-0-21	15.0	15.1	NP	NP	NP	A-1-a	0

Job No: *21-G-25284*

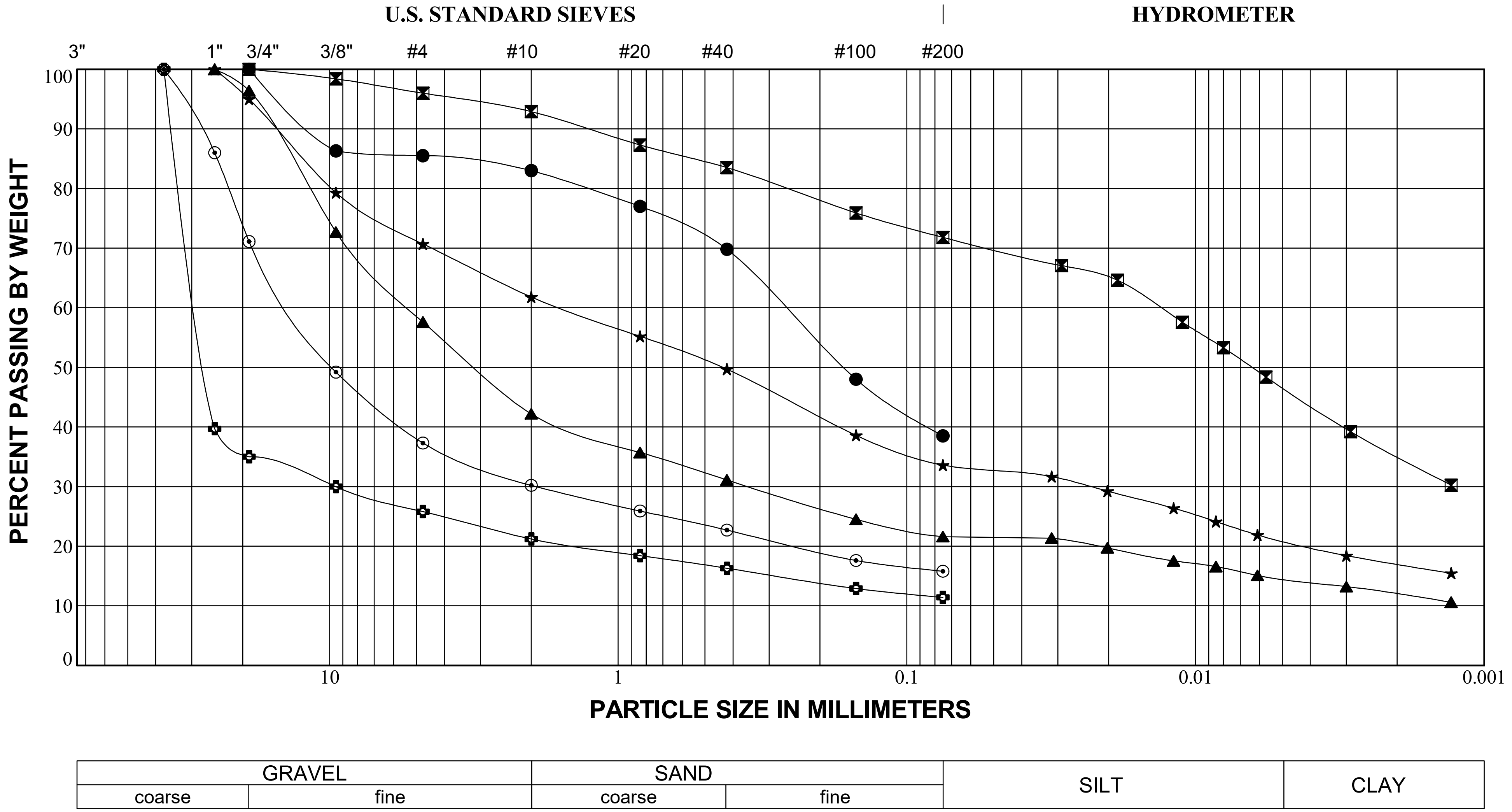
Method: *ASTM D4318*

Date: *August 2021*

ATTERBERG LIMITS TEST RESULTS

MOE-CR29-06.95
Adams Township, Monroe County, Ohio

Geotechnical Consultants, Inc. - Westerville, Ohio 43081



LEGEND:

	TEST HOLE	DEPTH	LL	w_n	PL	ODOT CLASSIFI-CATION	ODOT GROUP INDEX	C.B.R.
●	B-001-0-21	0.0		23.3				---
⧗	B-001-0-21	2.5	44	22.5	24	A-7-6	12	---
▲	B-001-0-21	7.5	37	11.8	20	A-2-6	0	---
★	B-001-0-21	10.5	35	23.1	20	A-2-6	1	---
⊙	B-001-0-21	12.0	NP	28.5	NP	A-1-b	0	---
✚	B-001-0-21	15.0	NP	16.7	NP	A-1-a	0	---

Job No: *21-G-25284*

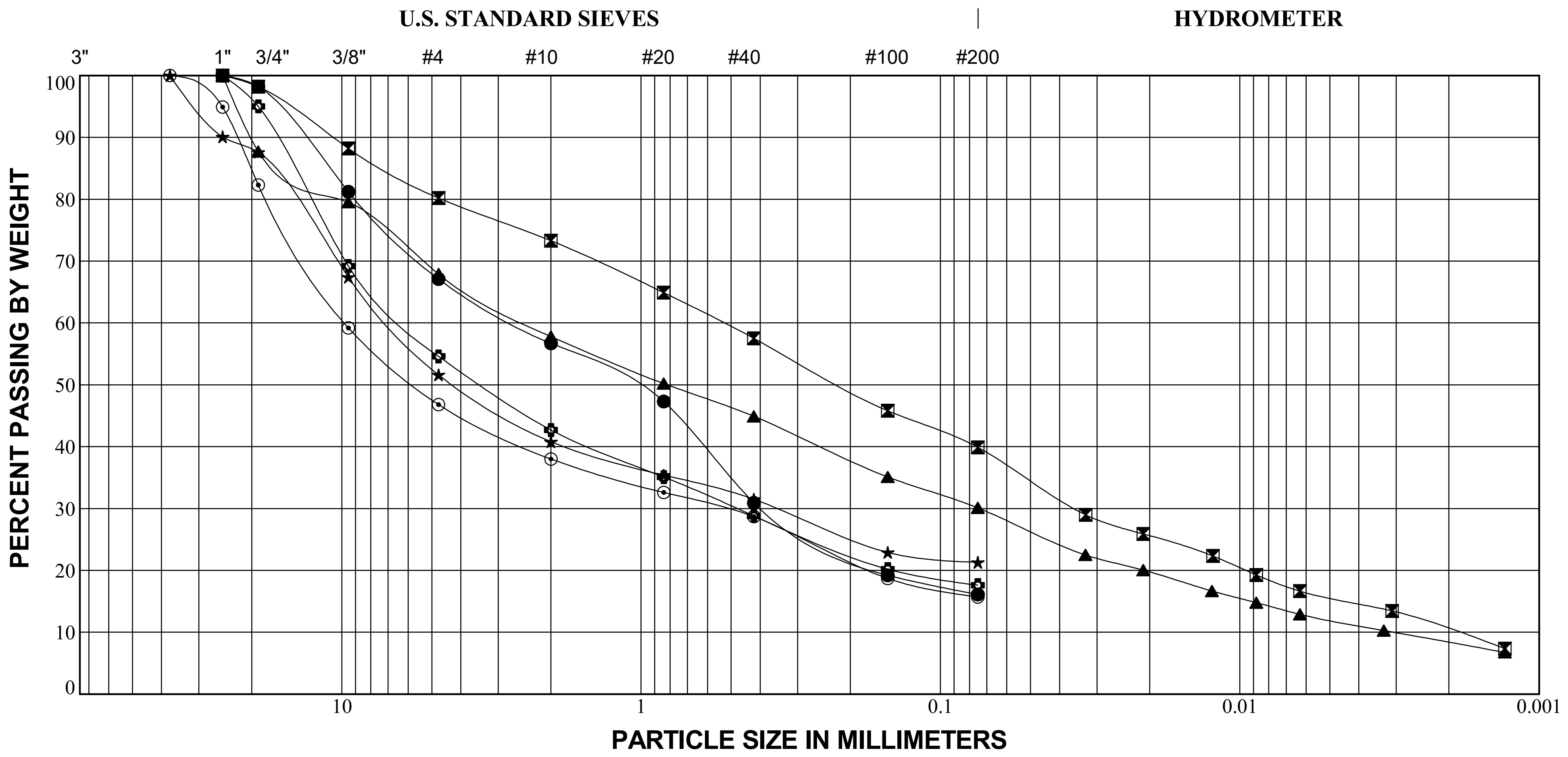
Method: *ASTM D421 D422*

Date: *August 2021*

COMBINED PARTICLE SIZE DISTRIBUTION

MOE-CR29-06.95 - Adams Township, Monroe County, Ohio

Geotechnical Consultants, Inc. - Westerville, Ohio 43081



GRAVEL		SAND		SILT	CLAY
coarse	fine	coarse	fine		

LEGEND:

	TEST HOLE	DEPTH	LL	w_n	PL	-------- ODOT -------- CLASSIFI-CATION	GROUP INDEX	C.B.R.
●	B-001-0-21	17.5	NP	15.7	NP	A-1-b	0	---
☒	B-002-0-21	0.0	37	14.6	27	A-4a	1	---
▲	B-002-0-21	2.5	28	10.5	19	A-2-4	0	---
★	B-002-0-21	5.0	NP	8.8	NP	A-1-b	0	---
⊙	B-002-0-21	7.5	NP	19.4	NP	A-1-b	0	---
✚	B-002-0-21	9.0	NP	16.3	NP	A-1-b	0	---

Job No: 21-G-25284

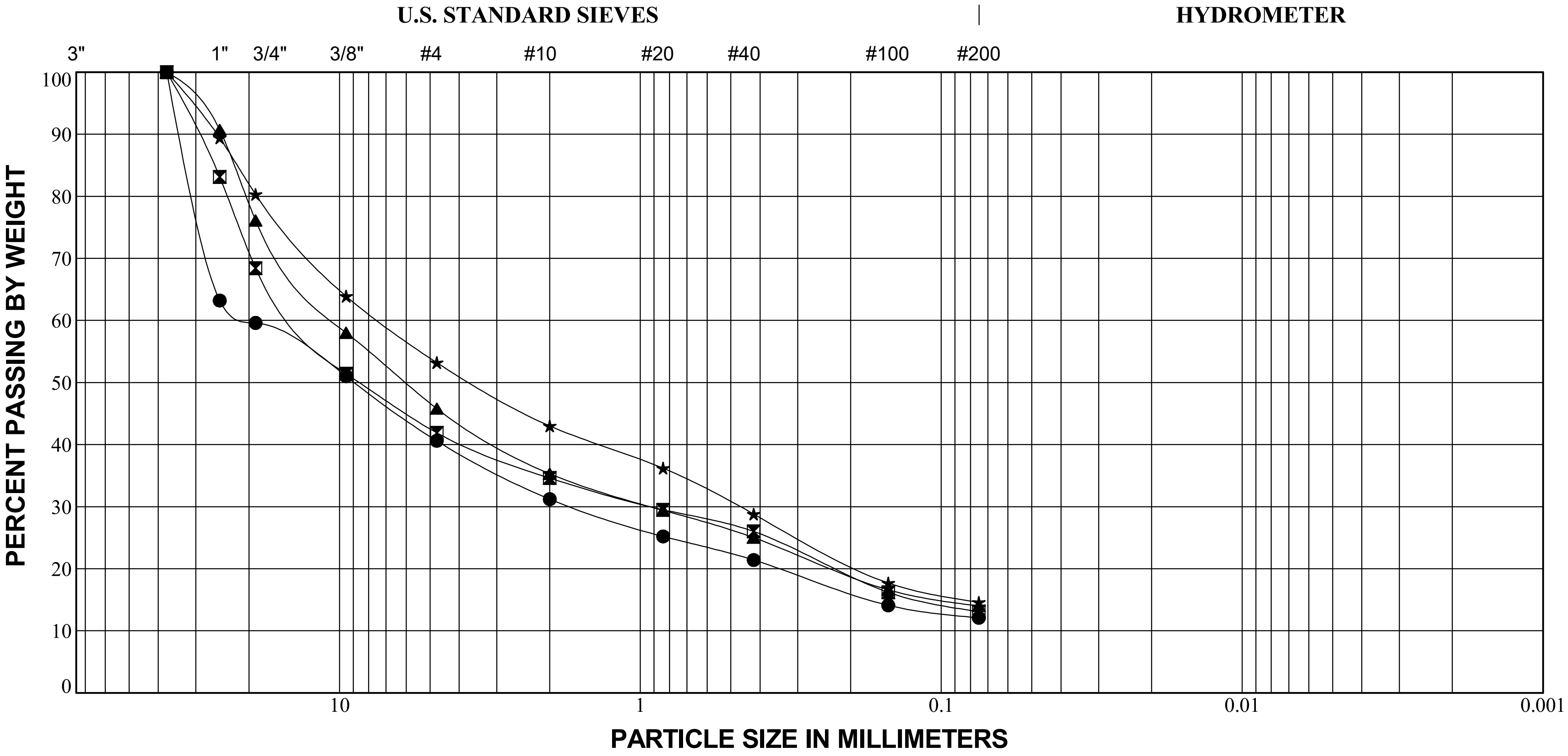
Method: ASTM D421 D422

Date: August 2021

COMBINED PARTICLE SIZE DISTRIBUTION

MOE-CR29-06.95 - Adams Township, Monroe County, Ohio

Geotechnical Consultants, Inc. - Westerville, Ohio 43081



GRAVEL		SAND		SILT	CLAY
coarse	fine	coarse	fine		

LEGEND:

						-------- ODOT --------		
	TEST HOLE	DEPTH	LL	w_n	PL	CLASSIFICATION	GROUP INDEX	C.B.R.
●	B-002-0-21	10.5	NP	14.4	NP	A-1-a	0	---
☒	B-002-0-21	12.0	NP	12.6	NP	A-1-a	0	---
▲	B-002-0-21	13.5	NP	15.0	NP	A-1-a	0	---
★	B-002-0-21	15.0	NP	15.1	NP	A-1-a	0	---

Job No: *21-G-25284*

Method: *ASTM D421 D422*

Date: *August 2021*

COMBINED PARTICLE SIZE DISTRIBUTION

MOE-CR29-06.95 - Adams Township, Monroe County, Ohio

Geotechnical Consultants, Inc. - Westerville, Ohio 43081

GEOTECHNICAL
CONSULTANTS INC.

JOB MOE-CR29-06.95
SHEET NO. 1 OF 12
CALCULATED BY JHolko DATE 8/11/21
CHECKED BY DATE
SCALE

FOUNDATION SELECTION AND PARAMETERS

1) Consider the following foundation types:

a) Spread Footings → Bedrock is too deep for this to be feasible.

b) Drilled Shafts → This would be feasible from a geotechnical standpoint, but likely not as economical as driven piles.

c) Driven Piles → Driven piles with refusal in shale bedrock is the most feasible option.

GCI will recommend a deep foundation system of driven piles.

2) Driven Pile Design Parameters

a) Need to determine the following (per BDM Section 201.2.1.3.b) →
- pile size
- pile type
- estimated pile length
- order pile length
- nominal and factored geotechnical resistance
- driveability analysis

b) BDM Section 305.3.1.1 → Steel H-piles meet ASTM A572 Grade 50 (HP10x42, HP12x53, or HP14x73)

c) BDM Section 305.3.1.2 → For piles bearing on bedrock, select a hammer that is capable of reaching and penetrating bedrock; refusal is met when pile penetrates into bedrock 1" or less after 20 blows

* Single-acting diesel pile driving hammers having a rated energy of up to 44,000 ft-lb are commonly available in Ohio.

d) Set up profile for boring B-001-0-21 in GRLWEAP (West Abutment)

- assume that bottom of pile cap is 9' below road surface (also, top of boring)
- See attached boring log for separation of layers

Layer 1 → pile depth of 0'-1.5' → N=14

Layer 2 → pile depth of 1.5'-6' → Avg N=4.3 ≈ 4

Layer 3 → pile depth of 6'-8' → N=15

Layer 4 → pile depth of 8'-11' → N=18

Layer 5 → • We will treat the shale as a very hard cohesive material. GRLWEAP requires the the side friction and end bearing.

pile depth of 11'-15.5'

- Get q_u from Modified Standard Penetration Test (MSPT) presented in "Modified Standard Penetration Test - Based Drilled Shaft Design Method for Weak Rocks" from the Illinois Center for Transportation, December 2017. Article 2.4 presents the following equation:

$$(N_{rate})_{90} = \frac{N_{rate} \times E_m \times C_B \times C_S \times C_R}{90}$$

$EM = 84$
$C_B = 1.0$ (Table Q.1 from MSPT Article)
$C_S = 1.0$ ("   ")
$C_R = 0.95$ with 20'-30' rod length and 1.0 for deeper

$$\text{Avg } N_{rate} = \frac{(12+40)+(19+32)}{2} = 51.5 \text{ blows/ft}$$

$$(N_{rate})_{90} = \frac{51.5 \times 84 \times 1 \times 1 \times 0.95}{90} = 45.7 \text{ blows/ft}$$

Eq. 2.2 from MSPT publication gives → $q_u = 0.092 \times (N_{rate})_{90} = 0.092 \times 45.7 = 4.2$ ksf

Next, select side friction and end bearing values using AASHTO LRFD Section 10.7.3.2.

We will consider the shale as "soft rock"; in accordance with Article 10.7.3.2.2, the bearing resistance should be analyzed the same as soil using Article 10.7.3.8.

Per AASHTO LRFD Article 10.7.8.3.4, a wave equation analysis will be used to determine driving criteria. Per Article C10.7.8.3.4, soil input values can be obtained from a static bearing analysis.

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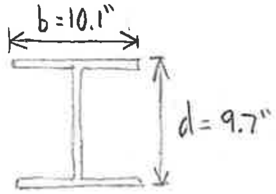
JOB MOE-CR29-06.95
SHEET NO. 3 OF 12
CALCULATED BY JHolko DATE 8/19/21
CHECKED BY DATE
SCALE

The side friction will be determined by AASHTO LRFD Article 10.7.3.8b (α-method)

$q_s = \alpha S_u \longrightarrow S_u = 0.5 q_u = (0.5)(4.2\ ksf) = 2.1\ ksf$

$\alpha \longrightarrow$ per Article 10.7.3.8b, the "box" area should be used to compute the pile surface area

↓



$= 10.1 + 10.1 + 9.7 + 9.7$

$= 39.6" = 3.3'$

For 'D' in graphs in Figure 10.7.3.8.6b-1, use average of pile depth and flange width → 9.9" = 0.825'

Use top graph in Figure 10.7.3.8.6b-1. We expect a D_b value of 15'-20' (i.e., embedment into shale) based on increasing shale hardness

$20D = (20)(0.825') = 16.5' \longrightarrow$ use the $D_b = 20D$ line

$\alpha \approx 0.93$

$q_s = (0.93)(2.1\ ksf) = \underline{1.95\ ksf}$

The tip resistance will be determined from AASHTO LRFD Eq. 10.7.3.8.6e-1

$q_p = 9 S_u = (9)(2.1\ ksf) = \underline{18.9\ ksf}$

Layer 6 (pile depth of 15.5' to 18')

N-value → 78 blows / 10" of penetration

$N_{rate} = \frac{78\ blows}{10"} \rightarrow 94\ blows/ft$

JOB MOE-CR29-06.95
SHEET NO. 4 OF 12
CALCULATED BY JHolko DATE 8/19/21
CHECKED BY DATE
SCALE

sampler depth range of 24.5'-27'

$$(N_{rate})_{90} = \frac{94 \times 84 \times 1.0 \times 1.0 \times 0.95}{90} = 83.3 \text{ blows/ft}$$

$$q_u = 0.092 \times (N_{rate})_{90} = 0.092 \times 83.3 = 7.66 \text{ ksf} \longrightarrow S_u = 3.83 \text{ ksf}$$

To find α, use top graph in AASHTO LRFD Fig. 10.7.3.8.6b-1; $\alpha = 0.75$ for $S_u > \approx 3$

$$q_s = (0.75)(3.83) = 2.87 \text{ ksf}$$

$$q_p = (9)(3.83 \text{ ksf}) = 34.5 \text{ ksf}$$

Layer 7 (pile depth of 18' to 20')

$$N_{rate} = \frac{50 \text{ blows}}{6''} \longrightarrow \text{use } \frac{100 \text{ blows}}{1 \text{ ft}}$$

sampler depth range of 27'-29'

$$(N_{rate})_{90} = \frac{100 \times 84 \times 1.0 \times 1.0 \times 0.95}{90} = 88.67 \text{ blows/ft}$$

$$q_u = 0.092 \times 88.67 = 8.16 \text{ ksf} \longrightarrow S_u = 4.08 \text{ ksf} \longrightarrow \alpha = 0.75$$

$$q_s = (0.75)(4.08) = 3.06 \text{ ksf} \qquad q_p = (9)(4.08) = 36.72 \text{ ksf}$$

Layer 8 (pile depth of 20' to 25')

sampler depth range of 29'-34'

$$N_{rate} = \frac{50 \text{ blows}}{4''} \longrightarrow \text{use } \frac{150 \text{ blows}}{1 \text{ ft}} \longrightarrow (N_{rate})_{90} = \frac{150 \times 84 \times 1.0 \times 1.0 \times 0.99}{90} = 139 \text{ blows/ft}$$

$$q_u = 0.092 \times 139 = 12.79 \text{ ksf} \longrightarrow S_u = 6.39 \text{ ksf} \longrightarrow \alpha = 0.75$$

$$q_s = (0.75)(6.39 \text{ ksf}) = 4.79 \text{ ksf} \qquad q_p = (9)(4.79) = 43.1 \text{ ksf}$$

Layer 9 (pile depth of 25' to 27')

**assume that shale transitions from 50/4" material to 50/1" material gradually from 33.5' to 38.5'; hence, the addition of Layers 7 and 8

$$N_{rate} = \frac{50 \text{ blows}}{3''} \longrightarrow \text{use } \frac{200 \text{ blows}}{1 \text{ ft}}$$

$$(N_{rate})_{90} = \frac{200 \times 84 \times 1.0 \times 1.0 \times 1.0}{90} = 186.67 \text{ blows/ft}$$

$$q_u = 0.092 \times 186.67 = 17.17 \text{ ksf} \longrightarrow S_u = 8.59 \text{ ksf} \longrightarrow \alpha = 0.75$$

$$q_s = (0.75)(8.59 \text{ ksf}) = 6.44 \text{ ksf} \qquad q_p = (9)(8.59 \text{ ksf}) = 77.31 \text{ ksf}$$

JOB MOE-CR29-06.95

SHEET NO. 5 OF 12

CALCULATED BY JHolko DATE 8/19/21

CHECKED BY DATE

SCALE

Layer 10 (pile depth of 27' - 29')

$$N_{rate} = \frac{50\text{ blows}}{2''} \longrightarrow \text{use } \frac{300\text{ blows}}{1\text{ ft}}$$

$$(N_{rate})_{90} = \frac{300 \times 84 \times 1.0 \times 1.0 \times 1.0}{90} = 280\text{ blows/ft}$$

$$q_u = 0.092 \times 280 = 25.76\text{ ksf} \longrightarrow s_u = 12.88\text{ ksf}$$

$$\alpha = 0.75$$

$$q_s = (0.75)(12.88) = 9.66\text{ ksf}$$

$$q_p = (9)(12.88) = 115.92\text{ ksf}$$

Layer 11 (pile depth of 29' - 32.5')

$$N_{rate} = \frac{50\text{ blows}}{1''} \longrightarrow \text{use } \frac{600\text{ blows}}{1\text{ ft}}$$

$$(N_{rate})_{90} = \frac{600 \times 84 \times 1.0 \times 1.0 \times 1.0}{90} = 560\text{ blows/ft}$$

$$q_u = 0.092 \times 560 = 51.52\text{ ksf} \longrightarrow s_u = 25.76\text{ ksf}$$

$$\alpha = 0.75$$

$$q_s = (0.75)(25.76) = 19.32\text{ ksf}$$

$$q_p = (9)(25.76) = 231.84\text{ ksf}$$

Finally, we need to make sure that the pile will not be overstressed. Using AASHTO LRFD Eq. 10.7.8-1, the driving stresses cannot exceed:

$$\sigma_{dr} = 0.9\,\phi_{da}\,f_y = (0.9)(1.0)(50\text{ ksi}) = 45\text{ ksi}$$

↑ From AASHTO LRFD Article 6.5.4.2

The drivability graph shows a blow count of ≈ 240 blows/ft (20 blows/in) at about 29 feet. Per ODOT BDM 305.3.5.2, estimated length = 30 ft and Order Length = 35 ft.

Max compressive strength in pile is ≈ 35 ksi ⟶ < 45 ksi ✓

SHEET NO. 6 OF 12

PROJECT: MOE-CR29-06.95	DRILLING FIRM / OPERATOR: GCI / R. BRANDUM	DRILL RIG: CME 45B (RIG 9)	STATION / OFFSET: -
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: GCI / R. BRANDUM	HAMMER: CME AUTOMATIC	ALIGNMENT: -
PID: 111130 BR ID: -	DRILLING METHOD: 3.5" SSA	CALIBRATION DATE: 12/18/20	ELEVATION: - EOB: 41.5 ft.
START: 6/24/21 END: 6/24/21	SAMPLING METHOD: SPT	ENERGY RATIO (%): 84	LAT / LONG: 39.766355, -80.964914

EXPLORATION ID B-001-0-21

PAGE 1 OF 2

MATERIAL DESCRIPTION AND NOTES

- Topsoil
- MEDIUM STIFF, DARK BROWN, **SANDY SILT**, LITTLE SAND, LITTLE GRAVEL, FILL, DAMP
- MEDIUM STIFF, BROWN, **CLAY**, LITTLE SAND, TRACE GRAVEL, DAMP
- LOOSE TO MEDIUM DENSE, BROWN, **STONE FRAGMENTS WITH SAND, SILT, AND CLAY**, SOME SAND, LITTLE SILT, LITTLE CLAY, DAMP
- with zones of clay
- wet
- LOOSE, BROWN, **STONE FRAGMENTS WITH SAND**, LITTLE SAND, LITTLE SILT, WET
- MEDIUM DENSE, BROWN, **STONE FRAGMENTS**, LITTLE SAND, LITTLE SILT, WET
- MEDIUM DENSE, BROWN, **GRAVEL WITH SAND**, SOME SAND, LITTLE SILT, WET
- **SHALE**, BLACK, HIGHLY WEATHERED, FRIABLE
- **SHALE**, GRAY, HIGHLY WEATHERED, FRIABLE
- **SHALE**, GRAY, MODERATELY WEATHERED
- argillaceous zones

ELEV.

DEPTHS: ▽ 7; W 9

Layer 1 (9 – 10.5), 0 – 1.5

Layer 2 (10.5 – 15), 6

Layer 3 (15 – 17), 8

Layer 4 (17 – 20), TR 11

Layer 5 (20 – 24.5), 15.5

Layer 6 (24.5 – 27), 18

Layer 7 (27 – 29), 20

Layer 8

SPT/ RQD	N₆₀	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
2 2 3	7	11	SS-1	1.75	17	13	31	- 39 -		-	-	-	23	A-4a (V)	
3 3 3	8	78	SS-2	1.75	7	9	12	25	47	44	24	20	23	A-7-6 (12)	
3 4 4	11	83	SS-3		-	-	-	-	-	-	-	-	13	A-2-6 (V)	
3 3 3	8	56	SS-4		57	11	10	7	15	37	20	17	12	A-2-6 (0)	
3 5 5	14	33	SS-5		-	-	-	-	-	-	-	-	12	A-2-6 (V)	
1 1 1	3	17	SS-6		38	12	16	13	21	35	20	15	23	A-2-6 (1)	
1 1 2	4	17	SS-7		70	7	7	- 16 -		NP	NP	NP	29	A-1-b (0)	
1 2 2	6	0	SS-8		-	-	-	-	-	-	-	-	-	A-1-b (V)	
3 5 6	15	11	SS-9		79	5	5	- 11 -		NP	NP	NP	17	A-1-a (0)	
6 6 7	18	56	SS-10		43	26	15	- 16 -		NP	NP	NP	16	A-1-b (0)	
8 12 40	73	44	SS-11		-	-	-	-	-	-	-	-	-	Rock (V)	
10 19 32	71	33	SS-12		-	-	-	-	-	-	-	-	-	Rock (V)	
14 28 50/4"	-	50	SS-13		-	-	-	-	-	-	-	-	-	Rock (V)	
50	-	50	SS-14		-	-	-	-	-	-	-	-	-	Rock (V)	

GRADATION (%) = GR, CS, FS, SI, CL; ATTERBERG = LL, PL, PI

SHEET NO. 7 OF 12

PID: 111130 | BR ID: - | PROJECT: MOE-CR29-06.95 | STATION / OFFSET: - | START: 6/24/21 | END: 6/24/21 | PG 2 OF 2 | B-001-0-21

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N60	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI			
SHALE, GRAY, MODERATELY WEATHERED (continued)		31 32 33	50/4"	-	50	SS-15		-	-	-	-	-	-	-	-	-	Rock (V)	
Layer 8		25																
Layer 9		34 35 27	50/4"	-	0	SS-16		-	-	-	-	-	-	-	-	-	Rock (V)	
Layer 10		36 37 29																
Layer 11		38 39 40 41 32.5 EOB	50/1"	-	0	SS-17		-	-	-	-	-	-	-	-	-	Rock (V)	

Auger refusal at 41.5'

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH AUGER CUTTINGS

GCI Job No: 21-G-25284

JOB MOE-CR29-06.95
SHEET NO. 8 OF 12
CALCULATED BY JHolko DATE 8/19/21
CHECKED BY DATE
SCALE

e) Set up profile for boring B-002-0-21 in GRLWEAP (East Abutment)

- assume that bottom of pile cap is at 9' below road surface (also, top of boring)
- See attached boring log for separation of layers

Layer 1 → pile depth of 0'-1.5' → N = 7

Layer 2 → pile depth of 1.5'-8' → Avg N = 17.25 ≈ 17

Layer 3 → pile depth of 8'-11' → N = 28

Layer 4 → pile depth of 11'-13' → find q_u

$$(N_{rate})_{90} = \frac{59 \text{ blows/ft} \times 84 \times 1.0 \times 1.0 \times 0.95}{90} = 52.3 \text{ blows/ft}$$

$$q_u = 0.092 \times (N_{rate})_{90} = 4.81 \text{ ksf} \rightarrow s_u = 2.4 \text{ ksf}$$

The side friction and tip resistance will be determined using the same AASHTO LRFD articles, equations, and graphs as the west abutment.

$$q_s = \alpha s_u \longrightarrow \alpha \approx 0.86 \longrightarrow q_s = \underline{2.06 \text{ ksf}}$$

$$q_p = 9 s_u = (9)(2.4 \text{ ksf}) = \underline{21.6 \text{ ksf}}$$

Layer 5 → pile depth of 13' - 15' → Avg $N_{rate} = \frac{50}{6''} = \frac{100 \text{ blows}}{1 \text{ ft}}$

$$(N_{rate})_{90} = \frac{100 \times 84 \times 1.0 \times 1.0 \times 0.95}{90} = 88.67 \text{ blows/ft}$$

$$q_u = 0.092 \times 88.67 = 8.16 \text{ ksf} \longrightarrow s_u = 4.08 \text{ ksf}$$

$$q_s = \alpha s_u = (0.75)(4.08) = \underline{3.06 \text{ ksf}} \qquad q_p = 9 s_u = (9)(s_u) = (9)(4.08) = \underline{36.72} \text{ ksf}$$

Layer 6 → pile depth of 15'-16.5' → Use N_{rate} of $\frac{50 \text{ blows}}{5''} = 120$ blows/ft

$$(N_{rate})_{90} = \frac{120 \times 84 \times 1.0 \times 1.0 \times 0.95}{90} = 106 \text{ blows/ft}$$

$$q_u = 0.092 \times 106 = 9.75 \text{ ksf} \longrightarrow s_u = 4.88 \text{ ksf}$$

$$q_s = \alpha s_u = (0.75)(4.88) = 3.66 \text{ ksf} \qquad q_p = 9 s_u = (9)(4.88) = 43.9 \text{ ksf}$$

JOB MOE-CR29-06.95
SHEET NO. 9 OF 12
CALCULATED BY JHolka DATE 8/31/21
CHECKED BY DATE
SCALE

Layer 7 → pile depth of 16.5' - 17' → use $N_{rate} = \frac{50 \text{ blows}}{4''} = 150$ blows/ft

$(N_{rate})_{90} = \frac{150 \times 84 \times 1.0 \times 1.0 \times 0.95}{90} = 133$ blows/ft $\longrightarrow q_u = 0.092 \times 133 = 12.2$ ksf $\rightarrow S_u = 6.1$ ksf

$q_s = \alpha S_u = (0.75)(6.1) = 4.58$ ksf $\quad q_p = (9)(S_u) = (9)(6.1) = 54.9$ ksf

Layer 8 → pile depth of 17' - 17.5' → use $N_{rate} = \frac{50 \text{ blows}}{3''} = 200$ blows/ft

$(N_{rate})_{90} = \frac{200 \times 84 \times 1.0 \times 1.0 \times 0.95}{90} = 177$ blows/ft $\longrightarrow q_u = 0.092 \times 177 = 16.28$ ksf $\rightarrow S_u = 8.14$ ksf

$q_s = \alpha S_u = (0.75)(8.14) = 6.1$ ksf $\quad q_p = (9)(8.14) = 73.26$ ksf

Layer 9 → pile depth of 17.5' - 18' → use $N_{rate} = \frac{50 \text{ blows}}{2''} = 300$ blows/ft

$(N_{rate})_{90} = \frac{300 \times 84 \times 1.0 \times 1.0 \times 0.95}{96} = 266$ blows/ft $\longrightarrow q_u = 0.092 \times 266 = 24.47$ ksf

$S_u = 12.24$ ksf

$q_s = \alpha S_u = (0.75)(12.24 \text{ ksf}) = 9.18$ ksf $\quad q_p = (9)(12.24) = 110.16$ ksf

Layer 10 → pile depth of 18' - 18.5' → use $N_{rate} = \frac{50 \text{ blows}}{1.5''} = 400$ blows/ft

$(N_{rate})_{90} = \frac{400 \times 84 \times 1.0 \times 1.0 \times 0.95}{90} = 355$ blows/ft $\longrightarrow q_u = 0.092 \times 355 = 32.66$ ksf

$S_u = 16.33$ ksf

$q_s = \alpha S_u = (0.75)(16.33) = 12.25$ ksf $\quad q_p = (9)(16.33) = 147$ ksf

Layer 11 → pile depth of 18.5' - 27' → $N_{rate} = \frac{50 \text{ blows}}{1''} = 600$ blows/ft $\longrightarrow (N_{rate})_{90} = \frac{600 \times 84 \times 1.0 \times 1.0 \times 0.985}{90} = 552 \frac{\text{blows}}{\text{ft}}$

$q_u = 0.092 \times 552 = 50.78$ ksf $\quad S_u = 25.39$ ksf $\quad q_s = (0.75)(25.39) = 19.04$ ksf $\quad q_p = (9)(25.39) = 228.51$ ksf

Layer 12 → pile depth of 27' - 32.5 → use $N_{rate} = \frac{50 \text{ blows}}{0.5''} = 1{,}200$ blows/ft

$(N_{rate})_{90} = \frac{1{,}200 \times 84 \times 1.0 \times 1.0 \times 1.0}{90} = 1{,}120$ blows/ft $\longrightarrow q_u = 0.092 \times 1{,}120 = 103$ ksf $\longrightarrow S_u = 51.52$ ksf

$q_s = (0.75)(51.52) = 38.64$ ksf $\quad q_p = (9)(51.52) = 463.7$ ksf

JOB MOE-CR29-06.95

SHEET NO. 10 OF 12

CALCULATED BY JHolko DATE 8/19/21

CHECKED BY DATE

SCALE

The drivability graph shows blow count of ≈240 blows/ft (20 blows/in) at about 20 ft.
Per ODOT BDM 305.3.5.2, estimated length = 25 ft and Order Length = 30 ft.

Max compressive strength in pile is ≈ 38 ksi → < 45 ksi ✓

SHEET NO. 11 OF 12

PROJECT: MOE-CR29-06.95
TYPE: BRIDGE REPLACEMENT
PID: 111130 BR ID: -
START: 6/24/21 END: 6/24/21

DRILLING FIRM / OPERATOR: GCI / R. BRANDUM
SAMPLING FIRM / LOGGER: GCI / R. BRANDUM
DRILLING METHOD: 3.5" SSA
SAMPLING METHOD: SPT

DRILL RIG: CME 45B (RIG 9)
HAMMER: CME AUTOMATIC
CALIBRATION DATE: 12/18/20
ENERGY RATIO (%): 84

STATION / OFFSET: -
ALIGNMENT: -
ELEVATION: - EOB: 41.5 ft.
LAT / LONG: 39.766457, -80.964803

EXPLORATION ID
B-002-0-21

PAGE
1 OF 2

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
Topsoil STIFF, DARK GRAY, **SANDY SILT**, SOME SAND, SOME GRAVEL, FILL, DAMP		1	3 5 5	14	44	SS-1	2.5	26 -	16 -	18 -	24 -	16 -	37 -	27 -	10 -	15 10	A-4a (1) A-2-4 (V)	
MEDIUM DENSE, BROWN, **STONE FRAGMENTS WITH SAND AND SILT**, SOME SAND, LITTLE SILT, LITTLE CLAY, DAMP brown clay layers		2 3 4	5 6 7	18	50	SS-2		42	13	15	18	12	28	19	9	11	A-2-4 (0)	
LOOSE, BROWN, **STONE FRAGMENTS WITH SAND**, LITTLE TO SOME SAND, LITTLE TO SOME SILT, DAMP		5 6 7	3 3 3	8	50	SS-3		60	9	10	- 21 -		NP	NP	NP	9	A-1-b (0)	
with gray clay layers		▼ W 0 8	2 2 3	7	44	SS-4		62	9	13	- 16 -		NP	NP	NP	19	A-1-b (0)	
wet Layer 1		9 1.5 10	4 2 3	7	50	SS-5		57	14	11	- 18 -		NP	NP	NP	16	A-1-b (0)	
LOOSE TO MEDIUM DENSE, BROWN, **STONE FRAGMENTS**, LITTLE SAND, LITTLE SILT, WET		11	8 6 6	17	61	SS-6		69	10	9	- 12 -		NP	NP	NP	14	A-1-a (0)	
Layer 2		12 13	6 6 8	20	44	SS-7		65	9	13	- 13 -		NP	NP	NP	13	A-1-a (0)	
		14 15	5 4 3	10	44	SS-8		65	10	11	- 14 -		NP	NP	NP	15	A-1-a (0)	
		16 8 17	4 6 10	22	39	SS-9		57	14	14	- 15 -		NP	NP	NP	15	A-1-a (0)	
Layer 3		18 19 TR 11	6 9 11	28	56	SS-10		-	-	-	-	-	-	-	-	18	A-1-a (V)	
SHALE, DARK BROWN, HIGHLY WEATHERED, ARGILLACEOUS Layer 4		20 21 13 22	10 17 25	59	44	SS-11		-	-	-	-	-	-	-	-	-	Rock (V)	
SHALE, DARK GRAY, MODERATELY WEATHERED, FRIABLE Layer 5		23 15 24	50	-	67	SS-12		-	-	-	-	-	-	-	-	-	Rock (V)	
Layer 6 7 8 9 10		25 16.5 26 27 17 17.5 18 18.5	50/5"	-	60	SS-13		-	-	-	-	-	-	-	-	-	Rock (V)	
Layer 11		28 29	50/1"	-	50	SS-14		-	-	-	-	-	-	-	-	-	Rock (V)	

SHEET NO. 12 OF 12

PID: 111130 | BR ID: - | PROJECT: MOE-CR29-06.95 | STATION / OFFSET: - | START: 6/24/21 | END: 6/24/21 | PG 2 OF 2 | B-002-0-21

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N60	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
SHALE, DARK GRAY, MODERATELY WEATHERED, FRIABLE *(continued)* Layer 11		31 32 33	50/1"	-	50	SS-15		-	-	-	-	-	-	-	-	-	Rock (V)	
		27 34 35 36	50/1"	-	0	SS-16		-	-	-	-	-	-	-	-	-	Rock (V)	
Layer 12		37 38																
		32.5 EOB 39 40 41	50/0"	-		SS-17		-	-	-	-	-	-	-	-	-	Rock (V)	

Auger refusal at 41.5'

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH AUGER CUTTINGS AND ASPHALT PATCH

GCI Job No: 21-G-25284

MSPT Sheet No. 1

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MODIFIED STANDARD PENETRATION TEST–BASED DRILLED SHAFT DESIGN METHOD FOR WEAK ROCKS

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ICT PROJECT R27-145
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Illinois Center for Transportation

December 2017

MSPT Sheet No. 2

2.4 MODIFIED STANDARD PENETRATION TEST (MSPT)

The standard penetration test (SPT) has been used to estimate strength parameters for soils and weak rock when it is difficult to obtain high-quality/undisturbed samples for laboratory testing (Peck et al., 1974). SPTs require 18-in.-penetration of the split-spoon sampler, which can be difficult to impossible to obtain in weak rocks or shales. In Phase 1 of this study, the procedure for conducting and interpreting the standard penetration test was modified to provide results in penetration per 10 blows increments where the penetration is less than 18 in. in weak shales. This new procedure is termed the modified standard penetration test (MSPT) and utilizes the concept of the split-spoon sampler penetration rate (N_{Rate}), not the sum of the penetration blow counts, to estimate the undrained strength parameters of weak shales. The penetration rate is the inverse of the linear slope of the penetration depth versus cumulative blow count relationship. This proposed test and recommended test procedure are discussed in detail in Appendix Q.

During this phase of the study, 16 IDOT bridge sites where weak shales are present were investigated. Modified standard penetration tests were conducted, and penetration rates were determined at various depths in weak shales in accordance with the MSPT procedure and recommendations developed herein and outlined in Appendix Q. MSPT results from the 16 sites investigated herein are presented in Appendices A through P. The results of the MSPT penetration rates (NRate), together with the laboratory-measured unconfined compressive strength for weak shales tested during both phases of the study were used to develop a useable empirical correlation between N_{Rate} and UCS (see Section 2.5.1).

2.5 SPT HAMMER ENERGY MEASUREMENTS

The SPT hammer energy used to measure penetration rate can vary from 40 to 100% of the maximum theoretical energy of a 140-lb weight falling 30 in. The wide variation in the transferred energy can cause inconsistent measurements of the MSPT penetration rate, which can undermine the targeted correlation. This inconsistency can lead to inaccurate values of UCS. Therefore, an energy correction must be developed and applied to the MSPT penetration rate to improve the reliability of the correlation, as is done for blow counts in soils where they are corrected to 60% of the maximum theoretical energy. In general, a higher energy results in a lower MSPT penetration rate, a lower UCS, and thus a more conservative drilled shaft design. Thus, it was important that the energy used to measure penetration rate be measured and/or obtained for each drill rig used in this study, to develop this energy-based correlation between UCS and penetration rate so designers can enter the correlation with a similar magnitude of MSPT energy to obtain an accurate estimate of UCS.

The research team measured the SPT hammer energy for all IDOT drill rigs used in this study. The tests were performed using an instrumented AW-J rod and a dynamic pile analyzer. Dynamic measurements were obtained using pairs of strain transducers and accelerometers mounted about 1 ft from the top of the drill rod. Measurements from the gauges were

MSPT Sheet No. 3

processed using the pile-driving analyzer (PDA), manufactured by Pile Dynamics, Inc. Table 2.2 summarizes the SPT hammer energy efficiencies for all of the operational IDOT drill rigs, together with the reported energies of the private drilling companies' drill rigs used in this study. Detailed SPT hammer energy measurements and results for all of the IDOT drill rigs are presented in Appendix S.

Table 2.2 Summary of the SPT Hammer Energies for all Drill Rigs Used in this Study

IDOT District/Drilling Company	Drill Rig	Hammer Energy Efficiency (%)
District 3	CME-75	93.2
	CME-45c	85.8
District 5	CME-75	91.3
District 6	CME-75	96.4
	CME-550x	80.4
District 7	CME-55	97.5
Wang Engineering	Mobile B-57	100
	D-50 TMR	78
Bulldog Drilling	CME-550x	94
Geocon	D-120	77
TSi Engineering	CME-550x	92

The results from this study indicate that 75 to 100% of the theoretical maximum hammer energy was delivered to the drill rod by the automatic hammers used herein. Because automatic hammers are now being widely used, an energy ratio of 90% shall be used to correct N_{Rate} for all of the drill rigs used during this study. In short, all of the drill rigs used during this study utilized an automatic trip hammer that imparted an average of 90% of the theoretical maximum hammer energy. Thus, MSPT N_{Rate} values obtained using an automatic trip hammer, which is the hammer most commonly used by IDOT, do not require significant corrections, in comparison to the previously suggested energy correction factor for soils, i.e., 60% of the theoretical maximum hammer energy, which is primarily based on a rope-and-pulley system. A normalized penetration rate, $(N_{Rate})_{90}$, was developed herein and is defined as follows for hammers that deliver 90% of theoretical maximum energy:

$$(N_{rate})_{90} = \frac{N_{rate} \times E_M \times C_B \times C_S \times C_R}{90}$$

MSPT Sheet No. 4

where:

$(N_{Rate})_{90}$ = Nrate corrected for 90% of the theoretical energy and various field procedures

E_M = hammer efficiency, %

C_B = borehole diameter correction

C_S = sampler correction

C_R = rod length correction, and

N_{Rate} = measured penetration rate, bpf

Table Q.1 in appendix Q shows the recommended borehole diameter, rod length, and sampler correction factors from Skempton (1986). If the hammer does not yield 90% of the theoretical maximum hammer energy, the measured hammer energy should be inserted for E_M in the equation above to normalize the measured N_{Rate} to 90% of the theoretical maximum hammer energy. The sampler correction assumes that liners will be installed in the split-spoon sampler to be consistent with Skempton (1986) even though the practice now is to not use liners.

2.5.1 *Proposed Correlation*

The MSPT provides a convenient means for estimating the in situ strength properties of weak, fine-grained rocks, e.g., weak shales. Figure 2.4 presented the refined and calibrated correlation of MSPT penetration rate, corrected for 90% of the theoretical energy and various field procedures $(N_{Rate})_{90}$, and UCS of the weak shales tested herein. Figure 2.4 shows a linear relationship between $(N_{Rate})_{90}$ and the UCS of weak shales that can be used for future drilled shaft design. This correlation for estimating the UCS of weak rocks reduces or eliminates the need for rock coring and subsequent laboratory testing that may be expensive, time-consuming, and problematic because of the fractured nature of weak rocks or shales.

Figure 2.4 shows the current line of best fit of the MSPT penetration rate and UCS data for the of Illinois weak shales tested herein. The following equation is recommended to estimate the UCS of weak shales, using the normalized MSPT penetration rate:

$$\text{UCS (ksf)} = 0.092 * (N_{rate})_{90} \tag{2.2}$$

where

UCS = Unconfined compressive strength, ksf

$(N_{Rate})_{90}$ = MSPT penetration rate corrected for 90% of the theoretical energy and various field procedures, bpf. (see appendix Q)

MSPT Sheet No. 5

Figure 2.4 also presents upper and lower bounds of the empirical correlation, which can be used to investigate the range of UCS and thus drilled shaft design. For less critical structures, it may be possible to use the upper bound; while for vital structures, the lower bound may be relevant. This correlation should only be used to estimate the UCS values for geomaterials that have a UCS of 10 to 100 ksf. For fine-grained soils with UCS values lower than 10 ksf, previously published correlations (e.g. Stroud 1974) should be used. Differences in the compressive strength of the geomaterials and the procedures used to measure the blow count or penetration rate (N_{spt} and N_{rate}) are the reasons for the significant difference between previous correlations (e.g., Stroud 1974) and the correlation presented herein to estimate the UCS.

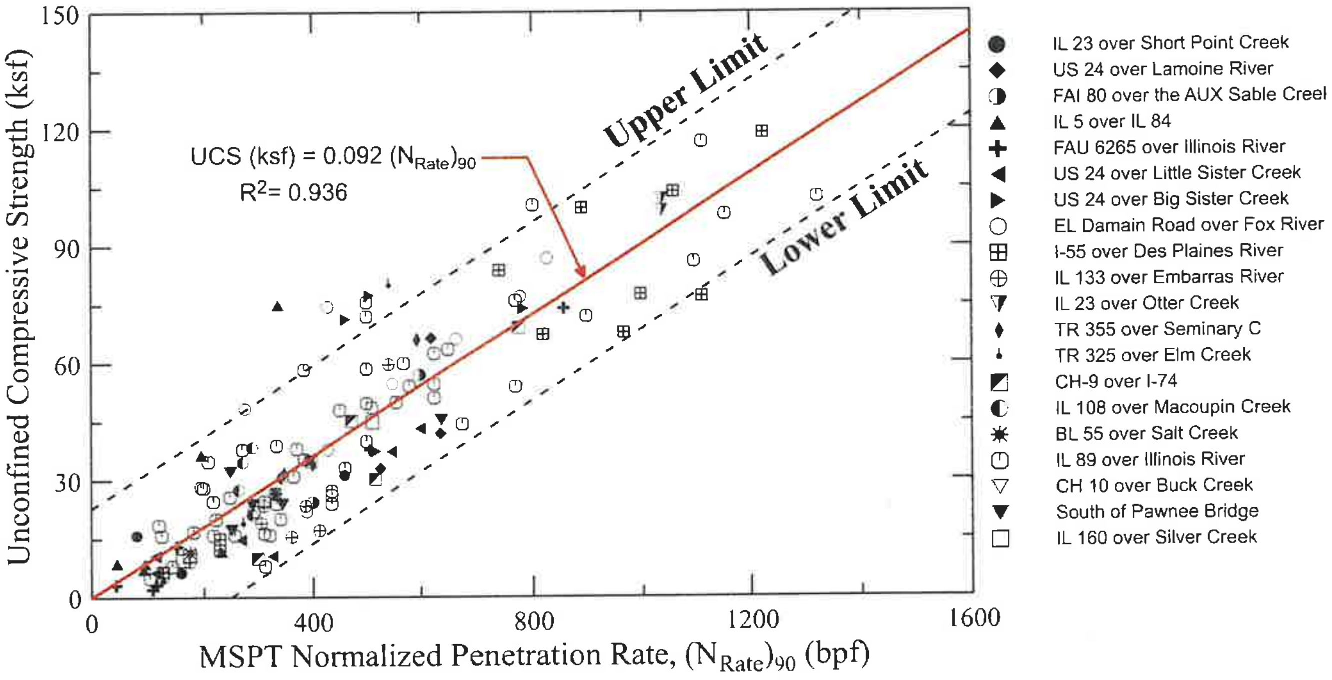

Figure 2.4. Relationship between UCS and $(N_{Rate})_{90}$ from MSPTs at 21 IDOT bridge sites.

2.6 SUMMARY

Field exploration was conducted at 16 additional IDOT bridge sites where weak shales are present. The main objective of this exploration was to develop and validate the MSPT penetration rate versus the unconfined compressive strength of weak shales relationship proposed in Phase 1 of this study and to investigate the strength and compressibility properties of weak shale in Illinois. The following is a summary of the major findings:

- Undrained Young's modulus was correlated with the in situ water content and the unconfined compressive strength of weak shales. These correlations can be used for estimating the modulus of shales for preliminary settlement analysis of bridge piers when site-specific data are not available or to evaluate site-specific data and laboratory testing.

MSPT Sheet No. 6

- SPT hammer energy measurements for all operational IDOT drill rigs and the ones used for MSPT penetration rate measurements imparted an average of 90% of the theoretical maximum hammer energy. As a result, a normalized penetration rate, $(N_{Rate})_{90}$, was developed herein to improve the reliability and consistency of the proposed correlation between unconfined compressive strength and MSPT penetration rates.
- An energy-based correlation between unconfined compressive strength and normalized MSPT penetration rate was developed and validated herein for Illinois weak shales. This correlation can be used with MSPT penetration rates for drilled shaft design, especially when obtaining high-quality shale samples for triaxial compression testing is difficult or impossible. The use of MSPT penetration rates for drilled shaft design should reduce the design time and costs by reducing or eliminating shale coring and laboratory triaxial compression testing by IDOT.

MSPT Sheet No. 7

of 90% of the theoretical maximum hammer energy. Thus, MSPT N_{rate} values obtained using an automatic trip hammer, which is the most commonly used hammer by IDOT, do not require significant corrections in comparison to the previously suggested energy correction factor for soils, i.e., 60% of the theoretical maximum hammer energy. A normalized penetration rate, $(N_{rate})_{90}$, was developed herein and is defined as follows for hammers that deliver 90% of theoretical maximum energy:

$$(N_{rate})_{90} = \frac{N_{rate} \times E_M \times C_B \times C_S \times C_R}{90}$$

where:

$(N_{rate})_{90}$ = N_{rate} corrected for 90% of the theoretical energy and various field procedures

E_M = hammer efficiency (i.e. average energy transfer ratio), %

C_B = borehole diameter correction

C_S = sampler correction

C_R = rod length correction, and

N_{rate} = measured penetration rate, bpf

Table Q.1 shows the recommended borehole diameter, rod length, and sampler correction factors from Skempton (1986). If the hammer does not yield 90% of the theoretical maximum hammer energy, the measured hammer energy should be inserted for E_M in the equation above to normalize the measured N_{rate} to 90% of the theoretical maximum hammer energy. The sampler correction assumes that liners will be installed in the split-spoon sampler to be consistent with Skempton (1986) even though the practice now is to not use liners.

MSPT Sheet No. 8

Table Q.1: N_{rate} Correction factors after Skempton (1986)

Effect	Variable	Term	Value
Borehole diameter	2.5 – 4.5 inches 6 inches 8 inches	C_B	1.00 1.05 1.15
Sampling Spoon	Smooth sampler (or with liners) Sampler without liners	C_S	1.0 1.2
Rod Length	30 – 100 ft 20 – 30 ft 13 – 20 ft 10 – 13 ft	C_R	1.0 0.95 0.85 0.75

MSPT Data Sheets

Drilling information and MSPT data obtained at each borehole shall be recorded in the field and include the following:

1. Date,
2. Name of the Drilling Crew,
3. Type and Make of the drill rig,
4. SPT Hammer Efficiency,
5. Project/Bridge Location,
6. Boring Number and location (station and coordinates),
7. Ground Surface Elevation,
8. Ground water surface Elevation,
9. MSPT elevations and depths,
10. Description of recovered weak rock or shale, and
11. Measured penetration depth every 10 blows to the nearest 0.1 inches (2.5 mm).

The top of competent bedrock will roughly correspond with the depth at which auger refusal is reached, and at which further bedrock sampling must be done by diamond-tipped core bit. This rock will typically have a relative strength of slightly strong to moderately strong, with an unconfined compressive strength in the range of 1500 psi to 7500 psi. Competent bedrock is often slightly to moderately weathered.

Strong bedrock may be slow and difficult to core, and is important to note for constructability reasons. This rock will typically have a relative strength of strong to extremely strong, with an unconfined compressive strength greater than 7500 psi. This rock is usually unweathered to slightly weathered.

2. Estimate Soil Engineering Properties

Estimate the engineering properties of the soil strata in order to model the subsurface profile for stability analyses. Interpret these values directly from the results of undisturbed soil testing, or provide estimates through engineering judgment and experience using the results of soil classification testing and SPT blow counts.

Table 1 provides estimates for the unit weights of cohesive and granular (cohesionless) soils based on SPT blow count and depth of the soil sample. The values in Table 1 are based on the engineering experience of the author, and are useful as a first approximation for unit weight to be used in stability analyses, where unit weight testing of the soil has not been performed.

TABLE 1 – Typical Unit Weight Relationships for Various Soils

All unit weights in this table are expressed in pounds per cubic foot (pcf).

Properties for Cohesive Soils		Unconfined Compressive Strength qu		Dry Unit Weight / Wet Unit Weight at Depth				
Consistency	Blow Counts N_{60}	tsf	psf	0-5 ft	5-10 ft	10-20 ft	20-40 ft	>40 ft
Very Soft	< 2	< 0.25	< 500	85/105	85/105	90/110	95/110	100/120
Soft	2 - 4	0.25 - 0.5	500 - 1000	90/105	90/110	95/115	100/120	105/125
Medium Stiff	4 - 8	0.5 - 1	1000 - 2000	95/110	95/120	95/120	105/125	115/130
Stiff	8 - 15	1 - 2	2000 - 4000	100/120	105/125	110/125	115/130	120/135
Very Stiff	15 - 30	2 - 4	4000 - 8000	105/125	110/125	115/130	120/135	125/140
Hard	> 30	> 4	> 8000	115/125	120/130	125/135	130/140	135/145

Properties for Granular Soils		Unconfined Compressive Strength qu*		Dry Unit Weight / Wet Unit Weight at Depth				
Density	Blow Counts N_{60}	tsf	psf	0-5 ft	5-10 ft	10-20 ft	20-40 ft	>40 ft
Very Loose	0 - 4			90/115	95/115	100/120	105/125	105/125
Loose	4 - 10			95/115	100/120	105/125	110/130	110/130
Medium Dense	10 - 30			100/120	105/125	110/130	115/135	115/140
Dense	30 - 50			110/125	115/130	120/135	120/140	120/140
Very Dense	> 50			115/130	120/135	125/140	125/140	130/150

*** Granular (cohesionless) soils cannot, by definition, exhibit a meaningful value for unconfined compressive strength.**

Estimate the angle of internal friction (ϕ) and cohesion (c) of the soils as appropriate for a long-term (drained) stability analysis. Similarly to Table 1 for the unit weight, Table 2 provides estimates for the drained internal friction angle (ϕ′) and cohesion (c′) of cohesive and granular (cohesionless) soils based on SPT blow count, consistency, and density. The values given in Table 1 and Table 2 are approximations, derived from SPT blow counts. It should be noted that the Standard Penetration Test yields highly variable results, and gives a poor approximation of the strength of cohesive soils, or soils which have a large amount of gravel or larger particles. These values provide a fair first estimate of the soil engineering properties; adjust these as necessary to fit the observed existing conditions and the results of stability analyses.

TABLE 2 – Typical Strength Values for Various Soils

Properties for Cohesive Soils		"Typical" Long-Term Strength Values	
Consistency	**Blow Counts N_{60}**	**Friction Angle (ϕ′)**	**Cohesion (c′)**
Very Soft	< 2	12-18°	0-25 psf
Soft	2 - 4	18-20°	25-50 psf
Medium Stiff	4 - 8	20-22°	50-100 psf
Stiff	8 - 15	22-24°	100-150 psf
Very Stiff	15 - 30	24-26°	150-200 psf
Hard	> 30	26-28°	200-250 psf

Properties for Granular Soils		"Typical" Long-Term Strength Values	
Density	**Blow Counts N_{60}**	**Friction Angle (ϕ′)**	**Cohesion (c′) (psf)**
Very Loose	0 - 4	26-28°	
Loose	4 - 10	28-30°	
Medium Dense	10 - 30	30-34°	
Dense	30 - 50	34-36°	
Very Dense	> 50	38-40°	

3. Locate Ground Water Surface

Determine the ground water surface in the subsurface profile for representation in the stability model. In some instances, complex hydrogeologic conditions may exist, such that there is not one single ground water table with dry or moist soils above and saturated soils below. However, in most cases, a single ground water surface may be approximated. In the subsurface, the ground water surface may be located fairly accurately at single points through long-term observations with ground water monitoring wells. Short-term observations (made during drilling) are often inaccurate, due to low permeability limiting the rate of water level recharge in the open boring hole, caving of soils from the walls of the open boring hole displacing free water, and the use of drilling fluids. However, short term observations may give a clue about the range of depths at which the ground water surface lies, and sometimes, fairly accurate observations of the depth at which water was "first encountered" will be made. Water contents of the soil samples may also provide data to estimate the depth to the ground water surface.

Utilize knowledge of hydrogeology and subsurface flow to connect the ground water surface between known points. The ground water surface should intersect with free water at the ground surface, and should slope downwards with a realistic potentiometric surface, generally following the lay of the land. If bedrock is shallow, the ground water surface often coincides with the top of bedrock. Figure 5 shows the ground water surface in the subsurface

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Federal Highway Administration

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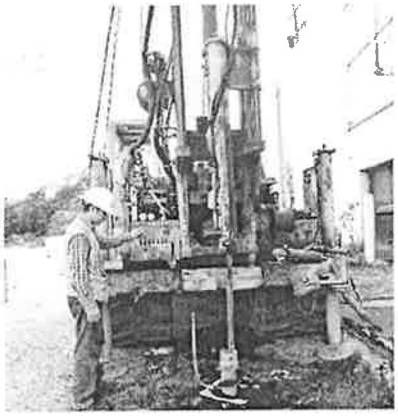
July 2016

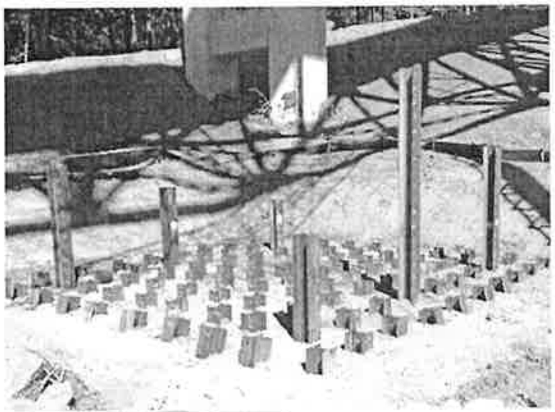
NHI Courses No. 132021 and 132022

Design and Construction of Driven Pile Foundations – Volume I

Developed following:
AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014,with 2015 Interim.

AASHTO LRFD Bridge Construction Specifications, 3rd Edition, 2010, with '11, '12, '13, '14, and '15 Interims.

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cohesive soils. The undrained shear strength is one half of the unconfined compressive strength ($s_u = q_u / 2$). Correlation of N values to the undrained shear strength of clays is crude and unreliable for final design and as stated, should only be used for preliminary estimating purposes.

Table 5-9 Empirical Values for Unconfined Compressive Strength, q_u, and Consistency of Cohesive Soils Based on Uncorrected N- Value (after Bowles 1977)

Consistency	Very Soft	Soft	Medium	Stiff	Very Stiff	Hard
q_u, ksf	0-0.5	0.5-1.0	1.0-2.0	2.0-4.0	4.0-8.0	8.0+
Standard Penetration N value	0-2	2-4	4-8	8-16	16-32	32+
γ (saturated), lb/ft^3	100-120	100-120	110-130	120-140	120-140	120-140

The Vane Shear Test (VST) can be used for soft to medium clays and produces a correlation for s_u from the input torque, T_v, and vane diameter, d_v. During the VST both peak and residual shear strengths are measured, thus the sensitivity can be calculated (Equation 5-5). When the vane height to diameter ratio is equal to two, $h_v/d_v = 2$, a widely used relationship found in GEC-5 by Sabatini et al. (2002) is shown in Equation 5-20. Furthermore, Bjerrum (1972) developed a correction based on static equilibrium theory as shown in Equation 5-21.

$$s_u = \frac{6T_v}{7\pi(d_v)^3} \qquad \text{For } \frac{h_v}{d_v} = 2 \qquad \text{Eq. 5-20}$$

Where:

s_u = undrained shear strength.
T_v = input torque during shear.
d_v = vane diameter.
h_v = vane height.

$$\mu = 2.5(PI)^{-0.3} \leq 1.1 \qquad \text{Eq. 5-21}$$

Where:

μ = correction factor.
PI = plasticity index.

Table 7-16 Soil Setup Factors (after Rausche et al. 1996)

Predominant Soil Type Along Pile Shaft	Range in Soil Setup Factor	Recommended Soil Setup Factors*	Number of Sites and (Percentage of Database)
Clay	1.2-5.5	2.0	7 (15%)
Silt - Clay	1.0-2.0	1.0	10 (22%)
Silt	1.5-5.0	1.5	2 (4%)
Sand - Clay	1.0-6.0	1.5	13 (28%)
Sand - Silt	1.2-2.0	1.2	8 (18%)
Fine Sand	1.2-2.0	1.2	2 (4%)
Sand	0.8-2.0	1.0	3 (7%)
Sand - Gravel	1.2-2.0	1.0	1 (2%)

* Confirmation with Local Experience Recommended.

7.2.4.2.1 Estimation of Pore Pressures During Driving

According to Lo and Stermac (1965), the maximum pore pressure induced from pile driving may be estimated from the following equation.

$$\Delta_{um} = \left[(1 - K_o) + \left(\frac{\Delta u}{\sigma'_v} \right)_m \right] \sigma'_{vi} \qquad \text{Eq. 7-47}$$

Where:

Δ_{um} = maximum excess pore pressure (ksf).

K_o = at rest earth pressure coefficient.

σ'_{vi} = initial vertical effective stress prior to pile driving (ksf).

$(\Delta u/\sigma'_v)_m$ = maximum value of the pore pressure ratio, $\Delta u/\sigma'_{vo}$, measured in a CU triaxial test with pore pressure measurements.

Ismael and Klym (1979) presented a case history where the above procedure was used. They reported good agreement between measured excess pore pressures with estimates from the Lo and Stermac procedure.

Poulos and Davis (1980) summarized measurements of excess pore pressures due to pile driving from several case histories. In this compilation, the reported excess pore pressure measurements divided by the vertical effective stress were plotted versus the radial distance from the pile surface divided by the pile radius. These

When estimating pile length for friction piles, use static analysis methods to determine the depth of pile penetration necessary to develop the required Ultimate Bearing Value as described in BDM Section 305.3.2.

Calculate the following pile lengths:

A. Estimated Length = Pile Cutoff Elevation - Pile Tip Elevation

   Round Estimated Length up to the nearest 5-ft. Provide the Estimated Length on the Site Plan.

B. Order Length = Estimated Length + 5-ft

   Provide the Order Length for each pile in the Structure General Notes.

C. Furnished Length = Order length x No. of Piles

   Include Furnished Length in the Estimated Quantities.

D. Driven Length = Estimated Length x No. of Piles

   Include Driven Length in the Estimated Quantities.

305.3.5.3 CORROSION AND PROTECTION

If the subsurface exploration identifies soil or site conditions considered indicative of potential pile deterioration or corrosion from environmental conditions according to *LRFD 10.7.5*, verify conditions with laboratory testing of soil samples. Consider soils with an organic content of 4 percent or more as "high organic content".

For soils that are not indicative of a potential pile corrosion problem, ignore corrosion for steel not exposed to atmospheric conditions over the design life of the structure. Provide pile encasement for portions of piles exposed to atmospheric conditions. The pile encasement shall extend a minimum of 3-ft below the ground line/stream bottom.

For soils that are indicative of a potential pile corrosion problem, determine the appropriate corrosion loss rate for carbon steel per Eurocode 3, Part 5, Section 4.4 for the specific environmental conditions at the site. Apply the appropriate corrosion loss rate to all surfaces of the piles in the respective exposure area.

Design the steel pile section to retain the required factored structural resistance after discounting corrosion loss and provide a plan note that addresses the amount of additional pile section specified to account for the corrosion loss. Alternately, provide corrosion protection for the piles.

The estimated length may need to be adjusted during detail design as the design loads for the Service, Strength and Extreme Event Limit States are refined.

Note that pile cutoff elevation includes the embedment into the pile cap per BDM Section 305.3.5.1 and free-standing length for capped pile piers. If rounding up to the nearest 5-ft for Estimated Length adds less than a foot, increase to the next 5-foot interval.

C305.3.5.3

A form of pile encasement is detailed on Standard Bridge Drawing CPP-1-08. The top of the encasement shall be located no more than 1-ft from the bottom of the pile cap and the concrete fill shall be sloped to drain.

The following maximum center-to-center pile spacings by structure type may be used as a guide:

A. In capped pile piers, 7.5-ft.

B. In capped pile abutments, 8-ft.

C. In stub abutments, front row, 8-ft.

D. In wall type abutments and retaining walls, front row, 7-ft.

Cap and column piers shall have at least 4 piles per individual footing.

For minimum center-to-center spacing of the piles, refer to *LRFD 10.7.1.2*.

Reinforce the pile cap to resist bending and shear based on the proposed center-to-center spacing of the piles.

Piles supporting capped pile piers shall be embedded 1.5-ft into the concrete cap. For other substructure units on a single row of piles, the piles shall be embedded 2-ft into the concrete. A 1-ft embedment depth into the concrete footing is required for all other cases. Perform a punching shear analysis to determine the necessary concrete thickness over the top of pile. In every case, there shall be at least 1.5-ft cover over top of pile.

The distance from the edge of a footing to the center of a pile shall be not less than 1.5-ft. The distance from the edge of a concrete pier cap to the side of a pile shall be not less than 9-in.

305.3.5.2 ESTIMATED PILE LENGTH

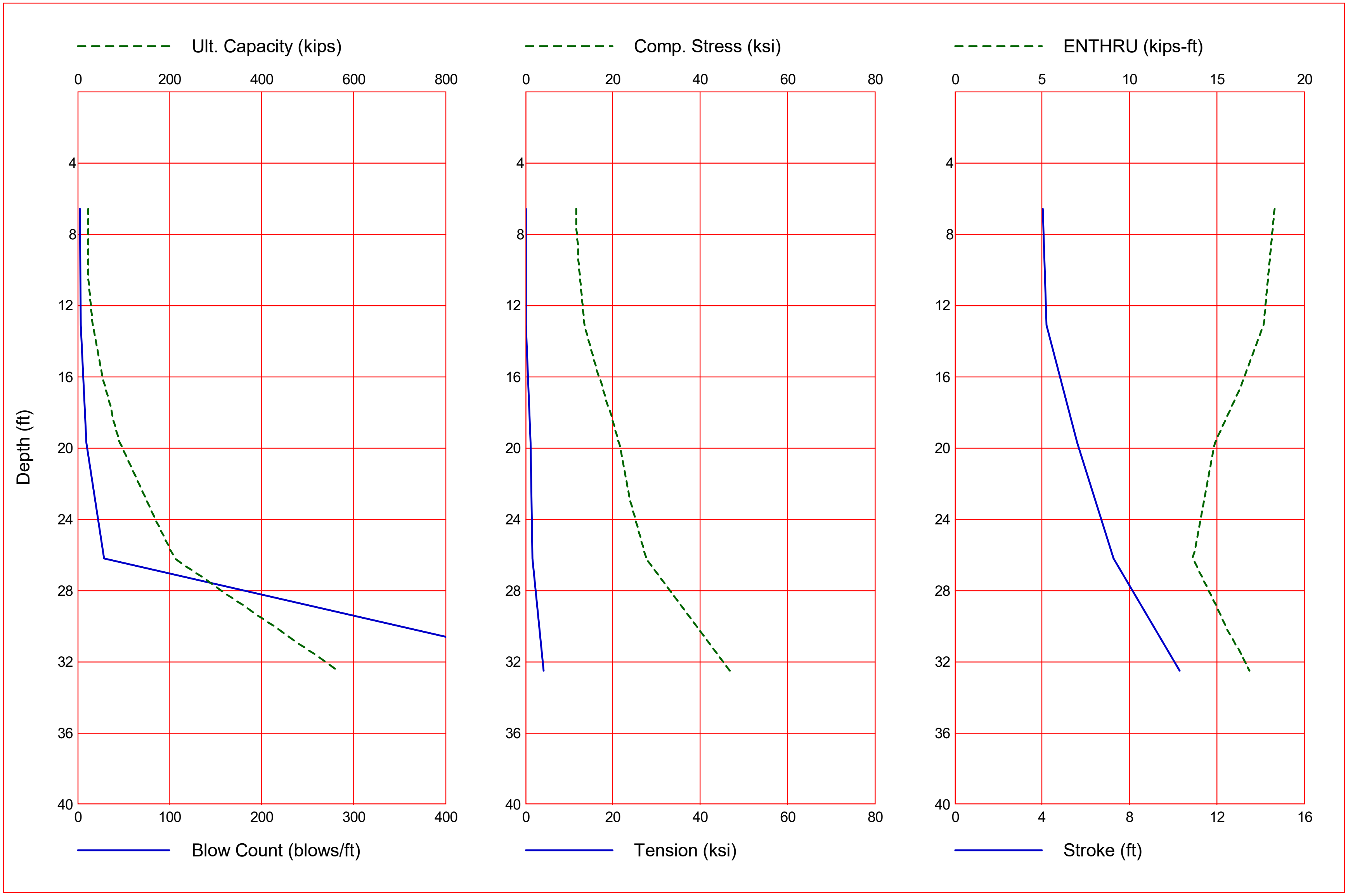
When estimating pile length for point bearing piles on bedrock, except as noted, assume the pile tip elevation as the elevation on the nearest soil boring where either bedrock coring begins or where SPT refusal blow count occurs with a recovered sample visually classified as bedrock. If using piles placed in prebored holes in bedrock, use the bottom of the prebored hole elevation as the pile tip elevation. If sufficient boring data is available, analyze the dip and strike of the bedrock in all three dimensions. If exploration reveals that the rock dips more than 8H:1V across the width of the unit, specify more than one pile length for the unit in divisions of 5-foot lengths. If there are no borings within 50 feet of the substructure unit, locate the top of rock at the substructure unit by interpolation between borings located to either side of the unit; provide a discussion to this effect in the Analyses and Recommendations section of the Foundation Report.

C305.3.5.2

Geotechnical Consultants, Inc.
MOE-CR29 - W Abutment - B-001 - HP10x42

Aug 31 2021
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000



Gain/Loss 1 at Shaft and Toe 0.833 / 1.000

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
6.6	24.0	2.7	21.3	2.6	11.666	0.000	4.03	18.3
13.1	33.3	20.5	12.9	3.1	13.610	0.000	4.19	17.7
19.7	93.0	68.0	25.0	10.1	21.513	-1.160	5.63	14.9
26.2	211.3	158.7	52.6	29.2	27.578	-1.625	7.26	13.6
32.5	568.6	410.9	157.7	561.3	46.752	-4.176	10.32	16.9

Total Continuous Driving Time 54.00 minutes; Total Number of Blows 2045

GRLWEAP - Version 2010
WAVE EQUATION ANALYSIS OF PILE FOUNDATIONS

written by GRL Engineers, Inc. (formerly Goble Rausche Likins
and Associates, Inc.) with cooperation from Pile Dynamics, Inc.

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 non-axial stresses and prestress effects 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 building and other factors.

```
 Input  File: S:\ENGINEERING\ENGFOLDER\2021 FOLDERS\25284 - MOE-CR29-06.95 BRIDGE REPLACEMENT
\GRLWEAP\WEST ABUTMENT (B-001-0-21)\HP10X42\B-001-0-21.GWW
 Hammer File: C:\ProgramData\PDI\GRLWEAP\2010\Resource\HAMMER2003.GW
 Hammer File Version:  2003 (2/3/2012)

                        Input File Contents
     MOE-CR29 - W Abutment - B-001 - HP10x42
 OUT OSG HAM STR FUL PEL  N SPL N-U P-D %SK ISM  0 PHI RSA ITR H-D MXT      DEx
-100   0  39   0   1   0  0   0   0   0   0   1  0   0   0   0   0   0   0.000
    Pile g  Hammer g  Toe Area Pile Size           Pile Type
    32.185    32.185    97.950    10.070              H Pile
      W Cp      A Cp      E Cp      T Cp       CoR      ROut      StCp
     1.900   227.000     530.0     2.000     0.800     0.010       0.0
      A Cu      E Cu      T Cu       CoR      ROut      StCu
     0.000       0.0     0.000     0.000     0.000       0.0
      LPle      APle      EPle      WPle      Peri        CI       CoR      ROut
    32.500     12.40   30457.9   493.356     3.299         0     0.850     0.010
 Manufac  Hmr Name  HmrType No Seg-s
 DELMAG   D 14-42         1        4
    Ram Wt     Ram L   Ram Dia   MaxStrk   RtdStrk    Efficy
      3.09    113.80     11.81     11.81     11.18      0.80
    IB. Wt     IB. L    IB.Dia    IB CoR     IB RO
      0.62     24.50     11.81     0.900     0.010
  CompStrk A Chamber V Chamber   C Delay  C Duratn Exp Coeff VolCStart  Vol CEnd
     14.00    109.50    108.40     0.002     0.002     1.250      0.00      0.00
     P atm        P1        P2        P3        P4        P5
     14.70   1695.00   1526.00   1373.00   1235.00      0.00
    Stroke    Effic.  Pressure  R-Weight   T-Delay Exp-Coeff   Eps-Str  Total-AW
   11.1800    0.8000 1695.0000    0.0000    0.0000    0.0000    0.0100    0.0000
        Qs        Qt        Js        Jt        Qx        Jx      Rati      Dept
     0.100     0.100     0.050     0.150     0.000     0.000     0.000     0.000
  Research Soil Model: Atoe, Plug,  Gap, Q-fac
     0.000     0.000     0.000     0.000
  Research Soil Model: RD-skn: m, d, toe: m, d
     0.000     0.000     0.000     0.000
  Res. Distribution
```

```
  Dpth    Rskn      Rtoe      Qs      Qt      Js      Jt   SU F    LimD   SU T
  0.00    0.00     19.89    0.10    0.17    0.05    0.15   1.20    6.56    1.0
  1.50    0.05     19.89    0.10    0.17    0.05    0.15   1.20    6.56    1.0
  1.50    0.07      5.68    0.10    0.17    0.05    0.15   1.20    6.56    1.0
  6.00    0.27      5.68    0.10    0.17    0.05    0.15   1.20    6.56    1.0
  6.00    0.32     21.31    0.10    0.17    0.05    0.15   1.20    6.56    1.0
  8.00    0.44     21.31    0.10    0.17    0.05    0.15   1.20    6.56    1.0
  8.00    0.41     25.57    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 11.00    0.54     25.57    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 11.00    2.10     12.86    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 15.50    2.10     12.86    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 15.50    2.87     23.47    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 18.00    2.87     23.47    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 18.00    3.06     24.98    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 20.00    3.06     24.98    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 20.00    4.79     29.32    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 25.00    4.79     29.32    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 25.00    6.44     52.59    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 27.00    6.44     52.59    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 27.00    9.66     78.85    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 29.00    9.66     78.85    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 29.00   19.32    157.70    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 32.50   19.32    157.70    0.10    0.17    0.05    0.15   1.20    6.56    1.0
Gain/Loss factors: shaft and toe
 0.83300   0.00000   0.00000   0.00000   0.00000
 1.00000   0.00000   0.00000   0.00000   0.00000
    Dpth         L      Wait      Strk      Pmx%      Eff.      Stff       CoR
    6.56      0.00      0.00     0.000     0.000     0.000     0.000     0.000
   13.12      0.00      0.00     0.000     0.000     0.000     0.000     0.000
   19.68      0.00      0.00     0.000     0.000     0.000     0.000     0.000
   26.25      0.00      0.00     0.000     0.000     0.000     0.000     0.000
   32.50      0.00      0.00     0.000     0.000     0.000     0.000     0.000
    0.00      0.00      0.00     0.000     0.000     0.000     0.000     0.000
         1           0   11.18000       11.81000
```

```
          GRLWEAP: WAVE EQUATION ANALYSIS OF PILE FOUNDATIONS
                              Version 2010
                             English Units

                  MOE-CR29 - W Abutment - B-001 - HP10x42
                  ________________________________________

           Hammer Model:   D 14-42              Made by:      DELMAG

           No.    Weight    Stiffn       CoR     C-Slk     Dampg
                    kips    k/inch                  ft    k/ft/s
             1     0.771
             2     0.771  111662.0     1.000    0.0100
             3     0.771  111662.0     1.000    0.0100
             4     0.771  111662.0     1.000    0.0100
     Imp Block     0.617   59995.9     0.900    0.0100
     Helmet        1.900   60155.0     0.800    0.0098       5.3
     Combined Pile Top      9684.0

HAMMER OPTIONS:
  Hammer File ID No.                 39  Hammer Type                 OE Diesel
  Stroke Option               FxdP-VarS  Stroke Convergence Crit.        0.010
  Fuel Pump Setting             Maximum

HAMMER DATA:
  Ram Weight          (kips)       3.09  Ram Length          (inch)    113.80
  Maximum Stroke        (ft)      11.81
  Rated Stroke          (ft)      11.18  Efficiency                     0.800

  Maximum Pressure     (psi)    1695.00  Actual Pressure      (psi)   1695.00
  Compression Exponent            1.350  Expansion Exponent             1.250
  Ram Diameter        (inch)      11.81
  Combustion Delay       (s)    0.00200  Ignition Duration      (s)   0.00200

         The Hammer Data Includes Estimated (NON-MEASURED) Quantities
         ____________________________________________________________
```

```
HAMMER CUSHION                                 PILE CUSHION
Cross Sect. Area     (in2)    227.00  Cross Sect. Area     (in2)       0.00
Elastic-Modulus      (ksi)     530.0  Elastic-Modulus      (ksi)        0.0
Thickness           (inch)      2.00  Thickness           (inch)       0.00
Coeff of Restitution             0.8  Coeff of Restitution              1.0
RoundOut              (ft)       0.0  RoundOut              (ft)        0.0
Stiffness        (kips/in)   60155.0  Stiffness        (kips/in)        0.0
```

```
 Depth                 (ft)       6.6
 Shaft Gain/Loss Factor         0.833  Toe Gain/Loss Factor           1.000

 PILE PROFILE:
 Toe Area             (in2)    97.950  Pile Type                     H Pile
 Pile Size           (inch)    10.070

  L b Top     Area    E-Mod  Spec Wt    Perim  C Index  Wave Sp      EA/c
       ft      in2      ksi   lb/ft3       ft               ft/s    k/ft/s
      0.0    12.40   30458.    493.4      3.3        0    16911.     22.3
     32.5    12.40   30458.    493.4      3.3        0    16911.     22.3

 Wave Travel Time 2L/c (ms)     3.844

        Pile and Soil Model            Total Capacity Rut  (kips)      24.0
No. Weight   Stiffn C-Slk T-Slk  CoR  Soil-S  Soil-D Quake  LbTop Perim    Area
      kips     k/in    ft    ft         kips    s/ft  inch     ft    ft     in2
  1  0.138     9684 0.010 0.000 0.85     0.0   0.000 0.100   3.25   3.3    12.4
  2  0.138     9684 0.000 0.000 1.00     0.0   0.000 0.100   6.50   3.3    12.4
  8  0.138     9684 0.000 0.000 1.00     0.0   0.050 0.100  26.00   3.3    12.4
  9  0.138     9684 0.000 0.000 1.00     0.6   0.050 0.100  29.25   3.3    12.4
 10  0.138     9684 0.000 0.000 1.00     2.1   0.050 0.100  32.50   3.3    12.4
Toe                                     21.3   0.150 0.165

     1.381 kips total unreduced pile weight (g= 32.17 ft/s2)
     1.381 kips total reduced pile weight   (g= 32.19 ft/s2)

 PILE, SOIL, ANALYSIS OPTIONS:
 Uniform pile                          Pile Segments: Automatic
 No. of Slacks/Splices              0  Pile Damping           (%)         1
                                       Pile Damping Fact.(k/ft/s)     0.447
 Driveability Analysis
 Soil Damping Option            Smith
```

```
Max No Analysis Iterations         0  Time Increment/Critical          160
Output Time Interval               1  Analysis Time-Input   (ms)         0
Output Level: Normal
Gravity Mass, Pile, Hammer:   32.170    32.185    32.185
Output Segment Generation: Automatic

   Depth    Stroke  Pressure    Efficy
      ft        ft     Ratio
    6.56     11.18      1.00     0.800
```

```
    Rut    Bl Ct    Stroke (ft) Ten Str   i    t Comp Str   i    t ENTHRU   Bl Rt
   kips     b/ft    down      up     ksi              ksi         kip-ft   b/min
   24.0      2.6    4.03    4.06    0.00   1    0    11.67   1    2   18.3    58.9
         1            0   11.18000        11.81000
```

```
MOE-CR29 - W Abutment - B-001 - HP10x42                            08/31/2021
Geotechnical Consultants, Inc.                           GRLWEAP Version 2010

 Depth                 (ft)      13.1
 Shaft Gain/Loss Factor         0.833  Toe Gain/Loss Factor           1.000

 PILE PROFILE:
 Toe Area             (in2)    97.950  Pile Type                     H Pile
 Pile Size           (inch)    10.070

  L b Top     Area    E-Mod  Spec Wt    Perim  C Index  Wave Sp      EA/c
      ft      in2      ksi   lb/ft3       ft              ft/s    k/ft/s
     0.0    12.40   30458.    493.4      3.3        0    16911.     22.3
    32.5    12.40   30458.    493.4      3.3        0    16911.     22.3

 Wave Travel Time 2L/c (ms)     3.844

       Pile and Soil Model           Total Capacity Rut  (kips)      33.3
No. Weight   Stiffn C-Slk T-Slk  CoR Soil-S  Soil-D Quake  LbTop Perim    Area
      kips     k/in    ft    ft        kips    s/ft  inch     ft    ft     in2
  1  0.138     9684 0.010 0.000 0.85    0.0   0.000 0.100   3.25   3.3    12.4
  2  0.138     9684 0.000 0.000 1.00    0.0   0.000 0.100   6.50   3.3    12.4
  6  0.138     9684 0.000 0.000 1.00    0.0   0.050 0.100  19.50   3.3    12.4
  7  0.138     9684 0.000 0.000 1.00    0.7   0.050 0.100  22.75   3.3    12.4
  8  0.138     9684 0.000 0.000 1.00    2.1   0.050 0.100  26.00   3.3    12.4
  9  0.138     9684 0.000 0.000 1.00    3.8   0.050 0.100  29.25   3.3    12.4
 10  0.138     9684 0.000 0.000 1.00   13.8   0.050 0.100  32.50   3.3    12.4
Toe                                    12.9   0.150 0.165

     1.381 kips total unreduced pile weight (g= 32.17 ft/s2)
     1.381 kips total reduced pile weight   (g= 32.19 ft/s2)


    Depth    Stroke  Pressure    Efficy
       ft        ft     Ratio
    13.12     11.18      1.00     0.800
```

```
    Rut   Bl Ct   Stroke (ft) Ten Str   i   t Comp Str   i   t ENTHRU   Bl Rt
   kips    b/ft   down     up     ksi             ksi          kip-ft   b/min
   33.3     3.1   4.19   4.23    0.00   1   0    13.61   1   2   17.7    57.8
         1           0   11.18000        11.81000
```

```
 Depth                 (ft)      19.7
 Shaft Gain/Loss Factor         0.833  Toe Gain/Loss Factor           1.000

 PILE PROFILE:
 Toe Area             (in2)    97.950  Pile Type                     H Pile
 Pile Size           (inch)    10.070

  L b Top     Area    E-Mod  Spec Wt    Perim  C Index  Wave Sp      EA/c
       ft      in2      ksi   lb/ft3       ft             ft/s    k/ft/s
      0.0    12.40   30458.    493.4      3.3        0    16911.     22.3
     32.5    12.40   30458.    493.4      3.3        0    16911.     22.3

 Wave Travel Time 2L/c (ms)     3.844

        Pile and Soil Model           Total Capacity Rut  (kips)      93.0
No. Weight   Stiffn C-Slk T-Slk  CoR  Soil-S  Soil-D Quake  LbTop Perim    Area
      kips     k/in    ft    ft         kips    s/ft  inch     ft    ft     in2
  1  0.138     9684 0.010 0.000 0.85     0.0   0.000 0.100   3.25   3.3    12.4
  2  0.138     9684 0.000 0.000 1.00     0.0   0.000 0.100   6.50   3.3    12.4
  4  0.138     9684 0.000 0.000 1.00     0.0   0.050 0.100  13.00   3.3    12.4
  5  0.138     9684 0.000 0.000 1.00     0.7   0.050 0.100  16.25   3.3    12.4
  6  0.138     9684 0.000 0.000 1.00     2.2   0.050 0.100  19.50   3.3    12.4
  7  0.138     9684 0.000 0.000 1.00     3.9   0.050 0.100  22.75   3.3    12.4
  8  0.138     9684 0.000 0.000 1.00    14.1   0.050 0.100  26.00   3.3    12.4
  9  0.138     9684 0.000 0.000 1.00    20.7   0.050 0.100  29.25   3.3    12.4
 10  0.138     9684 0.000 0.000 1.00    26.5   0.050 0.100  32.50   3.3    12.4
Toe                                     25.0   0.150 0.165

     1.381 kips total unreduced pile weight (g= 32.17 ft/s2)
     1.381 kips total reduced pile weight   (g= 32.19 ft/s2)


    Depth    Stroke  Pressure    Efficy
```

```
      ft          ft      Ratio
   19.68       11.18       1.00      0.800
```

```
    Rut   Bl Ct   Stroke (ft) Ten Str   i   t Comp Str   i   t ENTHRU   Bl Rt
   kips    b/ft   down     up     ksi              ksi           kip-ft   b/min
   93.0    10.1   5.63   5.59   -1.16   7  34    21.51   6   3   14.9    49.9
          1           0   11.18000       11.81000
```

```
MOE-CR29 - W Abutment - B-001 - HP10x42                          08/31/2021
Geotechnical Consultants, Inc.                           GRLWEAP Version 2010

 Depth                 (ft)      26.2
 Shaft Gain/Loss Factor         0.833  Toe Gain/Loss Factor           1.000

 PILE PROFILE:
 Toe Area            (in2)    97.950  Pile Type                     H Pile
 Pile Size          (inch)    10.070

  L b Top     Area    E-Mod  Spec Wt    Perim  C Index  Wave Sp      EA/c
      ft      in2      ksi   lb/ft3       ft              ft/s    k/ft/s
     0.0    12.40   30458.    493.4      3.3        0    16911.     22.3
    32.5    12.40   30458.    493.4      3.3        0    16911.     22.3

 Wave Travel Time 2L/c (ms)     3.844

       Pile and Soil Model           Total Capacity Rut  (kips)     211.3
No. Weight   Stiffn C-Slk T-Slk  CoR Soil-S  Soil-D Quake  LbTop Perim    Area
      kips     k/in    ft    ft        kips    s/ft  inch     ft    ft     in2
  1  0.138     9684 0.010 0.000 0.85    0.0   0.000 0.100   3.25   3.3    12.4
  2  0.138     9684 0.000 0.000 1.00    0.0   0.050 0.100   6.50   3.3    12.4
  3  0.138     9684 0.000 0.000 1.00    0.7   0.050 0.100   9.75   3.3    12.4
  4  0.138     9684 0.000 0.000 1.00    2.2   0.050 0.100  13.00   3.3    12.4
  5  0.138     9684 0.000 0.000 1.00    3.9   0.050 0.100  16.25   3.3    12.4
  6  0.138     9684 0.000 0.000 1.00   14.4   0.050 0.100  19.50   3.3    12.4
  7  0.138     9684 0.000 0.000 1.00   20.9   0.050 0.100  22.75   3.3    12.4
  8  0.138     9684 0.000 0.000 1.00   26.5   0.050 0.100  26.00   3.3    12.4
  9  0.138     9684 0.000 0.000 1.00   41.6   0.050 0.100  29.25   3.3    12.4
 10  0.138     9684 0.000 0.000 1.00   48.4   0.050 0.100  32.50   3.3    12.4
Toe                                    52.6   0.150 0.165

    1.381 kips total unreduced pile weight (g= 32.17 ft/s2)
    1.381 kips total reduced pile weight   (g= 32.19 ft/s2)
```

```
Depth     Stroke  Pressure    Efficy
   ft         ft     Ratio
26.25      11.18      1.00     0.800
```

```
MOE-CR29 - W Abutment - B-001 - HP10x42                            08/31/2021
Geotechnical Consultants, Inc.                           GRLWEAP Version 2010

    Rut   Bl Ct   Stroke (ft) Ten Str   i   t Comp Str   i   t ENTHRU   Bl Rt
   kips    b/ft   down      up     ksi              ksi         kip-ft   b/min
  211.3    29.2   7.26   7.20   -1.62   4  43    27.58   6   3   13.6    44.0
          1           0   11.18000       11.81000
```

```
 Depth                 (ft)      32.5
 Shaft Gain/Loss Factor         0.833  Toe Gain/Loss Factor           1.000

 PILE PROFILE:
 Toe Area             (in2)    97.950  Pile Type                     H Pile
 Pile Size           (inch)    10.070

  L b Top     Area     E-Mod  Spec Wt    Perim  C Index  Wave Sp      EA/c
       ft      in2       ksi   lb/ft3       ft              ft/s    k/ft/s
      0.0    12.40    30458.    493.4      3.3        0    16911.     22.3
     32.5    12.40    30458.    493.4      3.3        0    16911.     22.3

 Wave Travel Time 2L/c (ms)     3.844

        Pile and Soil Model            Total Capacity Rut  (kips)     568.6
No. Weight   Stiffn C-Slk T-Slk  CoR  Soil-S  Soil-D Quake  LbTop Perim    Area
      kips     k/in    ft    ft         kips    s/ft  inch     ft    ft     in2
  1  0.138     9684 0.010 0.000 0.85     0.6   0.050 0.100   3.25   3.3    12.4
  2  0.138     9684 0.000 0.000 1.00     2.1   0.050 0.100   6.50   3.3    12.4
  3  0.138     9684 0.000 0.000 1.00     3.8   0.050 0.100   9.75   3.3    12.4
  4  0.138     9684 0.000 0.000 1.00    13.3   0.050 0.100  13.00   3.3    12.4
  5  0.138     9684 0.000 0.000 1.00    20.3   0.050 0.100  16.25   3.3    12.4
  6  0.138     9684 0.000 0.000 1.00    26.4   0.050 0.100  19.50   3.3    12.4
  7  0.138     9684 0.000 0.000 1.00    40.4   0.050 0.100  22.75   3.3    12.4
  8  0.138     9684 0.000 0.000 1.00    47.3   0.050 0.100  26.00   3.3    12.4
  9  0.138     9684 0.000 0.000 1.00    84.1   0.050 0.100  29.25   3.3    12.4
 10  0.138     9684 0.000 0.000 1.00   172.6   0.050 0.100  32.50   3.3    12.4
Toe                                    157.7   0.150 0.165

     1.381 kips total unreduced pile weight (g= 32.17 ft/s2)
     1.381 kips total reduced pile weight   (g= 32.19 ft/s2)
```

```
Depth    Stroke  Pressure    Efficy
   ft        ft     Ratio
32.50     11.18      1.00     0.800
```

```
    Rut   Bl Ct   Stroke (ft) Ten Str   i   t Comp Str   i   t ENTHRU   Bl Rt
   kips    b/ft   down     up     ksi              ksi         kip-ft   b/min
  568.6   561.3  10.32  10.30   -4.18   4  13    46.75   1   6   16.9    37.0
```

```
                             SUMMARY OVER DEPTHS

                        G/L at Shaft and Toe:  0.833  1.000
   Depth      Rut   Frictn   End Bg   Bl Ct Com Str Ten Str  Stroke   ENTHRU
      ft     kips     kips     kips   bl/ft     ksi     ksi      ft   kip-ft
     6.6     24.0      2.7     21.3     2.6  11.666   0.000    4.03     18.3
    13.1     33.3     20.5     12.9     3.1  13.610   0.000    4.19     17.7
    19.7     93.0     68.0     25.0    10.1  21.513  -1.160    5.63     14.9
    26.2    211.3    158.7     52.6    29.2  27.578  -1.625    7.26     13.6
    32.5    568.6    410.9    157.7   561.3  46.752  -4.176   10.32     16.9

Total Driving Time      54 minutes;           Total No. of Blows    2045
```

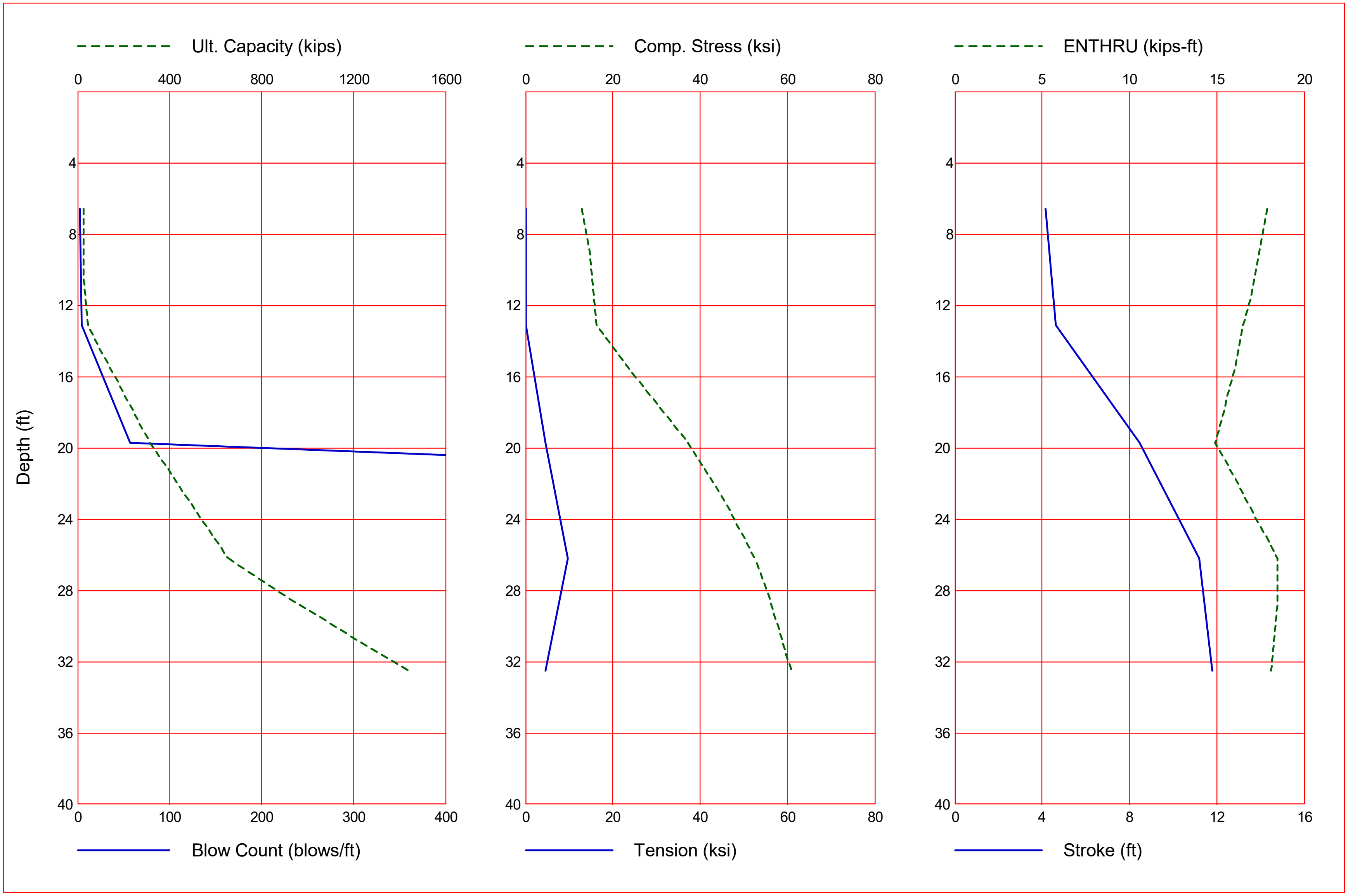
```
           Table of Depths Analyzed with Driving System Modifiers

           Temp.      Wait Equivalent Pressure            Stiffn.   Cushion
  Depth   Length      Time    Stroke     Ratio   Efficy.   Factor       CoR
     ft       ft        hr        ft
   6.56    32.50      0.00     11.18      1.00      0.80     1.00      1.00
  13.12    32.50      0.00     11.18      1.00      0.80     1.00      1.00
  19.68    32.50      0.00     11.18      1.00      0.80     1.00      1.00
  26.25    32.50      0.00     11.18      1.00      0.80     1.00      1.00
  32.50    32.50      0.00     11.18      1.00      0.80     1.00      1.00


                   Soil Layer Resistance Values
        Shaft      End  Shaft     Toe   Shaft     Toe    Soil   Limit   Setup
 Depth   Res. Bearing   Quake   Quake Damping Damping   Setup Distance   Time
    ft  k/ft2    kips    inch    inch    s/ft    s/ft Normlzd      ft     hrs
  0.00   0.00   19.89   0.100   0.165   0.050   0.150   1.000   6.560   1.000
  1.50   0.05   19.89   0.100   0.165   0.050   0.150   1.000   6.560   1.000
  1.50   0.07    5.68   0.100   0.165   0.050   0.150   1.000   6.560   1.000
  6.00   0.27    5.68   0.100   0.165   0.050   0.150   1.000   6.560   1.000
  6.00   0.32   21.31   0.100   0.165   0.050   0.150   1.000   6.560   1.000
  8.00   0.44   21.31   0.100   0.165   0.050   0.150   1.000   6.560   1.000
  8.00   0.41   25.57   0.100   0.165   0.050   0.150   1.000   6.560   1.000
 11.00   0.54   25.57   0.100   0.165   0.050   0.150   1.000   6.560   1.000
 11.00   2.10   12.86   0.100   0.165   0.050   0.150   1.000   6.560   1.000
 15.50   2.10   12.86   0.100   0.165   0.050   0.150   1.000   6.560   1.000
 15.50   2.87   23.47   0.100   0.165   0.050   0.150   1.000   6.560   1.000
 18.00   2.87   23.47   0.100   0.165   0.050   0.150   1.000   6.560   1.000
 18.00   3.06   24.98   0.100   0.165   0.050   0.150   1.000   6.560   1.000
 20.00   3.06   24.98   0.100   0.165   0.050   0.150   1.000   6.560   1.000
 20.00   4.79   29.32   0.100   0.165   0.050   0.150   1.000   6.560   1.000
 25.00   4.79   29.32   0.100   0.165   0.050   0.150   1.000   6.560   1.000
```

```
 25.00    6.44   52.59   0.100   0.165   0.050   0.150   1.000   6.560   1.000
 27.00    6.44   52.59   0.100   0.165   0.050   0.150   1.000   6.560   1.000
 27.00    9.66   78.85   0.100   0.165   0.050   0.150   1.000   6.560   1.000
 29.00    9.66   78.85   0.100   0.165   0.050   0.150   1.000   6.560   1.000
 29.00   19.32  157.70   0.100   0.165   0.050   0.150   1.000   6.560   1.000
 32.50   19.32  157.70   0.100   0.165   0.050   0.150   1.000   6.560   1.000
```

Geotechnical Consultants, Inc.
MOE-CR29 - E Abutment - B-002 - HP10x42

Aug 31 2021
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000



Geotechnical Consultants, Inc. Aug 31 2021
MOE-CR29 - E Abutment - B-002 - HP10x42 GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.833 / 1.000

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
6.6	27.0	2.8	24.1	3.0	12.867	0.000	4.16	17.9
13.1	44.9	20.0	25.0	4.8	16.376	0.000	4.63	16.5
19.7	312.1	156.7	155.4	56.9	37.204	-4.651	8.46	14.9
26.2	655.9	500.5	155.4	3308.4	52.554	-9.695	11.18	18.5
32.5	1439.2	1123.7	315.4	9999.0	60.855	-4.682	11.81	18.1

Refusal occurred; no driving time output possible

GRLWEAP - Version 2010
WAVE EQUATION ANALYSIS OF PILE FOUNDATIONS

written by GRL Engineers, Inc. (formerly Goble Rausche Likins
and Associates, Inc.) with cooperation from Pile Dynamics, Inc.

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 non-axial stresses and prestress effects 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 building and other factors.

```
 Input  File: S:\ENGINEERING\ENGFOLDER\2021 FOLDERS\25284 - MOE-CR29-06.95 BRIDGE REPLACEMENT
\GRLWEAP\EAST ABUTMENT (B-002-0-21)\HP10X42\B-002-0-21.GWW
 Hammer File: C:\ProgramData\PDI\GRLWEAP\2010\Resource\HAMMER2003.GW
 Hammer File Version:  2003 (2/3/2012)

                        Input File Contents
     MOE-CR29 - E Abutment - B-002 - HP10x42
 OUT OSG HAM STR FUL PEL  N SPL N-U P-D %SK ISM  0 PHI RSA ITR H-D MXT      DEx
-100   0  39   0   1   0  0   0   0   0   0   1  0   0   0   0   0   0    0.000
    Pile g  Hammer g  Toe Area Pile Size           Pile Type
    32.185    32.185    97.950    10.070              H Pile
      W Cp      A Cp      E Cp      T Cp       CoR      ROut      StCp
     1.900   227.000     530.0     2.000     0.800     0.010       0.0
      A Cu      E Cu      T Cu       CoR      ROut      StCu
     0.000       0.0     0.000     0.000     0.000       0.0
      LPle      APle      EPle      WPle      Peri        CI       CoR      ROut
    32.500     12.40   30457.9   493.356     3.299         0     0.850     0.010
 Manufac  Hmr Name  HmrType No Seg-s
 DELMAG   D 14-42         1        4
    Ram Wt     Ram L   Ram Dia   MaxStrk   RtdStrk    Efficy
      3.09    113.80     11.81     11.81     11.18      0.80
    IB. Wt     IB. L    IB.Dia    IB CoR     IB RO
      0.62     24.50     11.81     0.900     0.010
  CompStrk A Chamber V Chamber   C Delay  C Duratn Exp Coeff VolCStart  Vol CEnd
     14.00    109.50    108.40     0.002     0.002     1.250      0.00      0.00
     P atm        P1        P2        P3        P4        P5
     14.70   1695.00   1526.00   1373.00   1235.00      0.00
    Stroke    Effic.  Pressure  R-Weight   T-Delay Exp-Coeff   Eps-Str  Total-AW
   11.1800    0.8000 1695.0000    0.0000    0.0000    0.0000    0.0100    0.0000
        Qs        Qt        Js        Jt        Qx        Jx      Rati      Dept
     0.100     0.100     0.050     0.150     0.000     0.000     0.000     0.000
  Research Soil Model: Atoe, Plug,  Gap, Q-fac
     0.000     0.000     0.000     0.000
  Research Soil Model: RD-skn: m, d, toe: m, d
     0.000     0.000     0.000     0.000
  Res. Distribution
```

```
  Dpth    Rskn      Rtoe      Qs      Qt      Js      Jt   SU F    LimD   SU T
  0.00    0.00      9.94    0.10    0.17    0.05    0.15   1.20    6.56    1.0
  1.50    0.05      9.94    0.10    0.17    0.05    0.15   1.20    6.56    1.0
  1.50    0.08     24.15    0.10    0.17    0.05    0.15   1.20    6.56    1.0
  8.00    0.38     24.15    0.10    0.17    0.05    0.15   1.20    6.56    1.0
  8.00    0.34     39.78    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 11.00    0.49     39.78    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 11.00    2.06     14.69    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 13.00    2.06     14.69    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 13.00    3.06     24.98    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 15.00    3.06     24.98    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 15.00    3.66     29.86    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 16.50    3.66     29.86    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 16.50    4.58     37.34    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 17.00    4.58     37.34    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 17.00    6.10     49.83    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 17.50    6.10     49.83    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 17.50    9.18     74.93    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 18.00    9.18     74.93    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 18.00   12.25     99.99    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 18.50   12.25     99.99    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 18.50   19.04    155.43    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 27.00   19.04    155.43    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 27.00   38.64    315.41    0.10    0.17    0.05    0.15   1.20    6.56    1.0
 32.50   38.64    315.41    0.10    0.17    0.05    0.15   1.20    6.56    1.0
Gain/Loss factors: shaft and toe
 0.83300   0.00000   0.00000   0.00000   0.00000
 1.00000   0.00000   0.00000   0.00000   0.00000
    Dpth         L      Wait      Strk      Pmx%      Eff.      Stff      CoR
    6.56      0.00      0.00     0.000     0.000     0.000     0.000     0.000
   13.12      0.00      0.00     0.000     0.000     0.000     0.000     0.000
   19.68      0.00      0.00     0.000     0.000     0.000     0.000     0.000
   26.25      0.00      0.00     0.000     0.000     0.000     0.000     0.000
   32.50      0.00      0.00     0.000     0.000     0.000     0.000     0.000
    0.00      0.00      0.00     0.000     0.000     0.000     0.000     0.000
         1           0   11.18000       11.81000
```

```
           GRLWEAP: WAVE EQUATION ANALYSIS OF PILE FOUNDATIONS
                                  Version 2010
                                 English Units

                   MOE-CR29 - E Abutment - B-002 - HP10x42
                   ________________________________________

            Hammer Model:   D 14-42              Made by:      DELMAG

            No.    Weight    Stiffn       CoR     C-Slk     Dampg
                     kips    k/inch                  ft    k/ft/s
              1     0.771
              2     0.771  111662.0     1.000    0.0100
              3     0.771  111662.0     1.000    0.0100
              4     0.771  111662.0     1.000    0.0100
      Imp Block     0.617   59995.9     0.900    0.0100
      Helmet        1.900   60155.0     0.800    0.0098       5.3
      Combined Pile Top      9684.0

HAMMER OPTIONS:
  Hammer File ID No.                39  Hammer Type                OE Diesel
  Stroke Option              FxdP-VarS  Stroke Convergence Crit.       0.010
  Fuel Pump Setting            Maximum

HAMMER DATA:
  Ram Weight          (kips)      3.09  Ram Length          (inch)    113.80
  Maximum Stroke        (ft)     11.81
  Rated Stroke          (ft)     11.18  Efficiency                     0.800

  Maximum Pressure     (psi)   1695.00  Actual Pressure      (psi)   1695.00
  Compression Exponent           1.350  Expansion Exponent             1.250
  Ram Diameter        (inch)     11.81
  Combustion Delay       (s)   0.00200  Ignition Duration      (s)   0.00200

          The Hammer Data Includes Estimated (NON-MEASURED) Quantities
          ____________________________________________________________
```

```
HAMMER CUSHION                        PILE CUSHION
Cross Sect. Area     (in2)    227.00  Cross Sect. Area     (in2)       0.00
Elastic-Modulus      (ksi)     530.0  Elastic-Modulus      (ksi)        0.0
Thickness           (inch)      2.00  Thickness           (inch)      0.00
Coeff of Restitution             0.8  Coeff of Restitution             1.0
RoundOut              (ft)       0.0  RoundOut              (ft)       0.0
Stiffness        (kips/in)   60155.0  Stiffness        (kips/in)       0.0
```

```
 Depth                 (ft)       6.6
 Shaft Gain/Loss Factor         0.833  Toe Gain/Loss Factor           1.000

 PILE PROFILE:
 Toe Area             (in2)    97.950  Pile Type                     H Pile
 Pile Size           (inch)    10.070

  L b Top     Area    E-Mod  Spec Wt    Perim  C Index  Wave Sp      EA/c
      ft      in2      ksi   lb/ft3       ft              ft/s    k/ft/s
     0.0    12.40   30458.    493.4      3.3        0    16911.     22.3
    32.5    12.40   30458.    493.4      3.3        0    16911.     22.3

 Wave Travel Time 2L/c (ms)     3.844

       Pile and Soil Model           Total Capacity Rut  (kips)      27.0
No. Weight   Stiffn C-Slk T-Slk  CoR Soil-S  Soil-D Quake  LbTop Perim    Area
      kips     k/in    ft    ft        kips    s/ft  inch     ft    ft     in2
  1  0.138     9684 0.010 0.000 0.85     0.0   0.000 0.100   3.25   3.3    12.4
  2  0.138     9684 0.000 0.000 1.00     0.0   0.000 0.100   6.50   3.3    12.4
  8  0.138     9684 0.000 0.000 1.00     0.0   0.050 0.100  26.00   3.3    12.4
  9  0.138     9684 0.000 0.000 1.00     0.7   0.050 0.100  29.25   3.3    12.4
 10  0.138     9684 0.000 0.000 1.00     2.1   0.050 0.100  32.50   3.3    12.4
Toe                                     24.1   0.150 0.165

    1.381 kips total unreduced pile weight (g= 32.17 ft/s2)
    1.381 kips total reduced pile weight   (g= 32.19 ft/s2)

 PILE, SOIL, ANALYSIS OPTIONS:
 Uniform pile                          Pile Segments: Automatic
 No. of Slacks/Splices             0   Pile Damping          (%)          1
                                       Pile Damping Fact.(k/ft/s)     0.447
 Driveability Analysis
 Soil Damping Option            Smith
```

```
Max No Analysis Iterations         0  Time Increment/Critical          160
Output Time Interval               1  Analysis Time-Input   (ms)         0
Output Level: Normal
Gravity Mass, Pile, Hammer:   32.170    32.185    32.185
Output Segment Generation: Automatic

   Depth    Stroke  Pressure    Efficy
      ft        ft     Ratio
    6.56     11.18      1.00     0.800
```

```
    Rut   Bl Ct   Stroke (ft) Ten Str   i   t Comp Str   i   t ENTHRU   Bl Rt
   kips    b/ft   down      up     ksi              ksi           kip-ft   b/min
   27.0     3.0   4.16   4.19    0.00   1   0    12.87   1   2   17.9    58.0
          1           0   11.18000       11.81000
```

```
 Depth                 (ft)      13.1
 Shaft Gain/Loss Factor         0.833  Toe Gain/Loss Factor           1.000

 PILE PROFILE:
 Toe Area             (in2)    97.950  Pile Type                     H Pile
 Pile Size           (inch)    10.070

  L b Top     Area    E-Mod  Spec Wt    Perim  C Index  Wave Sp      EA/c
       ft      in2      ksi   lb/ft3       ft              ft/s    k/ft/s
      0.0    12.40   30458.    493.4      3.3        0    16911.     22.3
     32.5    12.40   30458.    493.4      3.3        0    16911.     22.3

 Wave Travel Time 2L/c (ms)     3.844

        Pile and Soil Model            Total Capacity Rut  (kips)      44.9
No. Weight   Stiffn C-Slk T-Slk  CoR  Soil-S  Soil-D Quake  LbTop Perim    Area
      kips     k/in    ft    ft        kips    s/ft  inch     ft    ft     in2
  1  0.138     9684 0.010 0.000 0.85     0.0   0.000 0.100   3.25   3.3    12.4
  2  0.138     9684 0.000 0.000 1.00     0.0   0.000 0.100   6.50   3.3    12.4
  6  0.138     9684 0.000 0.000 1.00     0.0   0.050 0.100  19.50   3.3    12.4
  7  0.138     9684 0.000 0.000 1.00     0.7   0.050 0.100  22.75   3.3    12.4
  8  0.138     9684 0.000 0.000 1.00     2.2   0.050 0.100  26.00   3.3    12.4
  9  0.138     9684 0.000 0.000 1.00     3.3   0.050 0.100  29.25   3.3    12.4
 10  0.138     9684 0.000 0.000 1.00    13.8   0.050 0.100  32.50   3.3    12.4
Toe                                     25.0   0.150 0.165

     1.381 kips total unreduced pile weight (g= 32.17 ft/s2)
     1.381 kips total reduced pile weight   (g= 32.19 ft/s2)


    Depth    Stroke  Pressure    Efficy
       ft        ft     Ratio
    13.12     11.18      1.00     0.800
```

```
MOE-CR29 - E Abutment - B-002 - HP10x42                            08/31/2021
Geotechnical Consultants, Inc.                           GRLWEAP Version 2010

    Rut   Bl Ct   Stroke (ft) Ten Str   i   t Comp Str   i   t ENTHRU   Bl Rt
   kips    b/ft   down      up     ksi              ksi         kip-ft   b/min
   44.9     4.8   4.63   4.66    0.00   1   0    16.38   1   2   16.5    55.0
          1           0   11.18000       11.81000
```

```
 Depth                 (ft)      19.7
 Shaft Gain/Loss Factor         0.833  Toe Gain/Loss Factor           1.000

 PILE PROFILE:
 Toe Area             (in2)    97.950  Pile Type                     H Pile
 Pile Size           (inch)    10.070

  L b Top     Area    E-Mod  Spec Wt    Perim  C Index  Wave Sp      EA/c
      ft      in2      ksi   lb/ft3       ft              ft/s    k/ft/s
     0.0    12.40   30458.    493.4      3.3        0    16911.     22.3
    32.5    12.40   30458.    493.4      3.3        0    16911.     22.3

 Wave Travel Time 2L/c (ms)     3.844

        Pile and Soil Model            Total Capacity Rut  (kips)     312.1
No. Weight   Stiffn C-Slk T-Slk  CoR  Soil-S  Soil-D Quake  LbTop Perim    Area
      kips     k/in    ft    ft        kips    s/ft  inch     ft    ft     in2
  1  0.138     9684 0.010 0.000 0.85     0.0   0.000 0.100   3.25   3.3    12.4
  2  0.138     9684 0.000 0.000 1.00     0.0   0.000 0.100   6.50   3.3    12.4
  4  0.138     9684 0.000 0.000 1.00     0.0   0.050 0.100  13.00   3.3    12.4
  5  0.138     9684 0.000 0.000 1.00     0.8   0.050 0.100  16.25   3.3    12.4
  6  0.138     9684 0.000 0.000 1.00     2.2   0.050 0.100  19.50   3.3    12.4
  7  0.138     9684 0.000 0.000 1.00     3.3   0.050 0.100  22.75   3.3    12.4
  8  0.138     9684 0.000 0.000 1.00    14.2   0.050 0.100  26.00   3.3    12.4
  9  0.138     9684 0.000 0.000 1.00    29.7   0.050 0.100  29.25   3.3    12.4
 10  0.138     9684 0.000 0.000 1.00   106.6   0.050 0.100  32.50   3.3    12.4
Toe                                    155.4   0.150 0.165

    1.381 kips total unreduced pile weight (g= 32.17 ft/s2)
    1.381 kips total reduced pile weight   (g= 32.19 ft/s2)


   Depth    Stroke  Pressure    Efficy
```

```
        ft          ft     Ratio
     19.68       11.18      1.00     0.800
```

```
   Rut   Bl Ct   Stroke (ft) Ten Str   i   t Comp Str   i   t ENTHRU   Bl Rt
  kips    b/ft   down     up     ksi            ksi         kip-ft   b/min
 312.1    56.9   8.46   8.45   -4.65   8  17    37.20   1   6   14.9    40.8
         1           0   11.18000       11.81000
```

```
 Depth                 (ft)      26.2
 Shaft Gain/Loss Factor         0.833  Toe Gain/Loss Factor           1.000

 PILE PROFILE:
 Toe Area             (in2)    97.950  Pile Type                     H Pile
 Pile Size           (inch)    10.070

  L b Top     Area    E-Mod  Spec Wt    Perim  C Index  Wave Sp      EA/c
      ft      in2      ksi   lb/ft3       ft              ft/s    k/ft/s
     0.0    12.40   30458.    493.4      3.3        0    16911.     22.3
    32.5    12.40   30458.    493.4      3.3        0    16911.     22.3

 Wave Travel Time 2L/c (ms)     3.844

       Pile and Soil Model           Total Capacity Rut  (kips)     655.9
No. Weight   Stiffn C-Slk T-Slk  CoR Soil-S  Soil-D Quake  LbTop Perim    Area
      kips     k/in    ft    ft        kips    s/ft  inch     ft    ft     in2
  1  0.138     9684 0.010 0.000 0.85    0.0   0.000 0.100   3.25   3.3    12.4
  2  0.138     9684 0.000 0.000 1.00    0.0   0.050 0.100   6.50   3.3    12.4
  3  0.138     9684 0.000 0.000 1.00    0.8   0.050 0.100   9.75   3.3    12.4
  4  0.138     9684 0.000 0.000 1.00    2.2   0.050 0.100  13.00   3.3    12.4
  5  0.138     9684 0.000 0.000 1.00    3.3   0.050 0.100  16.25   3.3    12.4
  6  0.138     9684 0.000 0.000 1.00   14.7   0.050 0.100  19.50   3.3    12.4
  7  0.138     9684 0.000 0.000 1.00   29.8   0.050 0.100  22.75   3.3    12.4
  8  0.138     9684 0.000 0.000 1.00  109.5   0.050 0.100  26.00   3.3    12.4
  9  0.138     9684 0.000 0.000 1.00  170.1   0.050 0.100  29.25   3.3    12.4
 10  0.138     9684 0.000 0.000 1.00  170.1   0.050 0.100  32.50   3.3    12.4
Toe                                   155.4   0.150 0.165

    1.381 kips total unreduced pile weight (g= 32.17 ft/s2)
    1.381 kips total reduced pile weight   (g= 32.19 ft/s2)
```

```
Depth    Stroke  Pressure    Efficy
   ft        ft     Ratio
26.25     11.18      1.00     0.800
```

```
MOE-CR29 - E Abutment - B-002 - HP10x42                            08/31/2021
Geotechnical Consultants, Inc.                           GRLWEAP Version 2010

    Rut   Bl Ct   Stroke (ft) Ten Str   i   t Comp Str   i   t ENTHRU   Bl Rt
   kips    b/ft   down      up     ksi              ksi         kip-ft   b/min
  655.9  3308.4  11.18  11.24   -9.70   7  13    52.55   1   6   18.5    35.5
          1           0   11.18000       11.81000
```

```
MOE-CR29 - E Abutment - B-002 - HP10x42                            08/31/2021
Geotechnical Consultants, Inc.                           GRLWEAP Version 2010

 Depth                 (ft)      32.5
 Shaft Gain/Loss Factor         0.833  Toe Gain/Loss Factor           1.000

 PILE PROFILE:
 Toe Area             (in2)    97.950  Pile Type                     H Pile
 Pile Size           (inch)    10.070

  L b Top     Area    E-Mod  Spec Wt    Perim  C Index  Wave Sp      EA/c
       ft      in2      ksi   lb/ft3      ft              ft/s    k/ft/s
      0.0    12.40   30458.    493.4      3.3        0    16911.     22.3
     32.5    12.40   30458.    493.4      3.3        0    16911.     22.3

 Wave Travel Time 2L/c (ms)     3.844

        Pile and Soil Model            Total Capacity Rut  (kips)    1439.2
No. Weight   Stiffn C-Slk T-Slk  CoR Soil-S  Soil-D Quake  LbTop Perim    Area
      kips     k/in    ft    ft         kips    s/ft  inch     ft    ft     in2
  1  0.138     9684 0.010 0.000 0.85     0.7   0.050 0.100   3.25   3.3    12.4
  2  0.138     9684 0.000 0.000 1.00     2.1   0.050 0.100   6.50   3.3    12.4
  3  0.138     9684 0.000 0.000 1.00     3.3   0.050 0.100   9.75   3.3    12.4
  4  0.138     9684 0.000 0.000 1.00    12.9   0.050 0.100  13.00   3.3    12.4
  5  0.138     9684 0.000 0.000 1.00    29.4   0.050 0.100  16.25   3.3    12.4
  6  0.138     9684 0.000 0.000 1.00    99.0   0.050 0.100  19.50   3.3    12.4
  7  0.138     9684 0.000 0.000 1.00   170.1   0.050 0.100  22.75   3.3    12.4
  9  0.138     9684 0.000 0.000 1.00   291.2   0.050 0.100  29.25   3.3    12.4
 10  0.138     9684 0.000 0.000 1.00   345.1   0.050 0.100  32.50   3.3    12.4
Toe                                    315.4   0.150 0.165

     1.381 kips total unreduced pile weight (g= 32.17 ft/s2)
     1.381 kips total reduced pile weight   (g= 32.19 ft/s2)


    Depth    Stroke  Pressure    Efficy
```

```
       ft         ft      Ratio
    32.50      11.18       1.00      0.800
*** CAUTION: RAM MIGHT BLOW OUT; Combustion pressure was reduced ***
```

```
    Rut   Bl Ct   Stroke (ft) Ten Str   i   t Comp Str   i   t ENTHRU   Bl Rt
   kips    b/ft   down     up     ksi              ksi         kip-ft   b/min
 1439.2  9999.0  11.81  11.85   -4.68   4  16    60.85   4   5   18.1    34.6
```

```
                             SUMMARY OVER DEPTHS

                    G/L at Shaft and Toe:  0.833  1.000
   Depth       Rut   Frictn   End Bg   Bl Ct Com Str Ten Str  Stroke   ENTHRU
      ft      kips     kips     kips   bl/ft     ksi     ksi      ft   kip-ft
     6.6      27.0      2.8     24.1     3.0  12.867   0.000    4.16     17.9
    13.1      44.9     20.0     25.0     4.8  16.376   0.000    4.63     16.5
    19.7     312.1    156.7    155.4    56.9  37.204  -4.651    8.46     14.9
    26.2     655.9    500.5    155.4  3308.4  52.554  -9.695   11.18     18.5
    32.5    1439.2   1123.7    315.4  9999.0  60.855  -4.682   11.81     18.1

Refusal occurred; no driving time output possible
```

```
              Table of Depths Analyzed with Driving System Modifiers

              Temp.      Wait Equivalent Pressure             Stiffn.   Cushion
    Depth    Length      Time    Stroke     Ratio   Efficy.    Factor       CoR
       ft        ft        hr        ft
     6.56     32.50      0.00     11.18      1.00      0.80      1.00      1.00
    13.12     32.50      0.00     11.18      1.00      0.80      1.00      1.00
    19.68     32.50      0.00     11.18      1.00      0.80      1.00      1.00
    26.25     32.50      0.00     11.18      1.00      0.80      1.00      1.00
    32.50     32.50      0.00     11.18      1.00      0.80      1.00      1.00


                        Soil Layer Resistance Values
          Shaft     End   Shaft     Toe   Shaft     Toe    Soil   Limit   Setup
  Depth    Res. Bearing   Quake   Quake Damping Damping   Setup Distance    Time
     ft   k/ft2    kips    inch    inch    s/ft    s/ft Normlzd      ft     hrs
   0.00    0.00    9.94   0.100   0.165   0.050   0.150   1.000   6.560   1.000
   1.50    0.05    9.94   0.100   0.165   0.050   0.150   1.000   6.560   1.000
   1.50    0.08   24.15   0.100   0.165   0.050   0.150   1.000   6.560   1.000
   8.00    0.38   24.15   0.100   0.165   0.050   0.150   1.000   6.560   1.000
   8.00    0.34   39.78   0.100   0.165   0.050   0.150   1.000   6.560   1.000
  11.00    0.49   39.78   0.100   0.165   0.050   0.150   1.000   6.560   1.000
  11.00    2.06   14.69   0.100   0.165   0.050   0.150   1.000   6.560   1.000
  13.00    2.06   14.69   0.100   0.165   0.050   0.150   1.000   6.560   1.000
  13.00    3.06   24.98   0.100   0.165   0.050   0.150   1.000   6.560   1.000
  15.00    3.06   24.98   0.100   0.165   0.050   0.150   1.000   6.560   1.000
  15.00    3.66   29.86   0.100   0.165   0.050   0.150   1.000   6.560   1.000
  16.50    3.66   29.86   0.100   0.165   0.050   0.150   1.000   6.560   1.000
  16.50    4.58   37.34   0.100   0.165   0.050   0.150   1.000   6.560   1.000
  17.00    4.58   37.34   0.100   0.165   0.050   0.150   1.000   6.560   1.000
  17.00    6.10   49.83   0.100   0.165   0.050   0.150   1.000   6.560   1.000
  17.50    6.10   49.83   0.100   0.165   0.050   0.150   1.000   6.560   1.000
```

```
17.50    9.18   74.93   0.100   0.165   0.050   0.150   1.000   6.560   1.000
18.00    9.18   74.93   0.100   0.165   0.050   0.150   1.000   6.560   1.000
18.00   12.25   99.99   0.100   0.165   0.050   0.150   1.000   6.560   1.000
18.50   12.25   99.99   0.100   0.165   0.050   0.150   1.000   6.560   1.000
18.50   19.04  155.43   0.100   0.165   0.050   0.150   1.000   6.560   1.000
27.00   19.04  155.43   0.100   0.165   0.050   0.150   1.000   6.560   1.000
27.00   38.64  315.41   0.100   0.165   0.050   0.150   1.000   6.560   1.000
32.50   38.64  315.41   0.100   0.165   0.050   0.150   1.000   6.560   1.000
```