FINAL REPORT
STRUCTURE FOUNDATION EXPLORATION
BRIDGE WOO-281-11.97 OVER CREPS DITCH
PID: 113085
WOOD COUNTY, OHIO

Prepared For:

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NEAS PROJECT 24-0049

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National Engineering & Architectural Services Inc.

EXECUTIVE SUMMARY

The bridge replacement project WOO-281-11.97 (PID# 113085) has been proposed to replace the existing bridge carrying SR-281 over Creps Ditch in Wood County, Ohio. This report provides a summary of the encountered surficial and subsurface conditions, as well as our recommendations for the bridge foundation's design and construction. National Engineering and Architectural Services Inc. (NEAS) was contracted to perform geotechnical engineering services for this project. The purpose of these services was to conduct geotechnical explorations within the project area to gather essential data on subsurface soil, rock, and groundwater conditions relevant to the bridge's design and construction requirements. As part of these explorations, NEAS advanced two project borings and performed laboratory testing to characterize the soil and rock for engineering purposes. This report presents a summary of these findings, along with recommendations for the bridge foundation's design and construction.

The subsurface profile at the site is generally consistent with the geological model for the project in terms of the materials encountered. The profile at the proposed bridge replacement site generally consists primarily of stiff to hard cohesive fine materials, with minor amounts of medium dense to very dense granular materials. Bedrock was encountered in both project borings, at depths ranging from 8.4 ft to 9.0 ft below ground surface, corresponding to elevations of approximately between 674.8 ft and 675 ft above mean sea level.

The bedrock at the site was classified as gray, unweathered to slightly weathered, strong to very strong, very thin to medium bedded, highly to moderately fractured, narrow to tight dolomite. Bedrock recovery ranged from 95 to 96 percent, with a Rock Quality Designation (RQD) value of 76 percent. The unconfined compressive strength of the bedrock ranges from 21,117 psi to 25,396 psi. Slake durability index of the bedrock ranges from 99.3% to 99.4%.

Based on our lab testing results, the equivalent D_{50} (mm) values of bedrock were estimated. At both rear and forward abutments, the equivalent D_{50} (mm) is estimated to be 5516.4 mm, based on borings B-001-0-24 and B-002-0-24. The Erodibility Index is estimated at 475 for both abutments. The bedrock can be considered as non-scour-resistant, starting at elevations of 674.7 ft at the rear abutment and 674.3 ft at the forward abutment.

Based on our subgrade analysis, unstable and high moisture content soils were encountered in more than 30% of the proposed subgrade surface. These soils exhibited an N_{60} value below 12 bpf and a moisture content exceeding the optimum level by more than 3 percent within 3 feet of subgrade. Therefore, NEAS recommends global stabilization using one of the following methods: Item 204 Excavate and Replace with Geotextile to a depth of 12 inches, or Item 206 Lime Stabilization to a depth of 14 inches. A cost analysis of the options should be conducted.

NEAS recommends that the bottom of footings be keyed at least 3-in into non-scour-resistant, competent bedrock. Based on the site plan, the bottom of footing is at the elevation of 673.50 ft for both abutments. The anticipated scour is expected to terminate within the soil zone, approximately 3.5 ft above the bottom of the rear abutment footing and 2.5 ft above the bottom of the forward abutment footing. Therefore, scour is not expected to affect the footing bearing elevation. The factored bearing resistance of the bedrock was estimated to be 1,621 ksf at the rear abutment and 1,348 ksf at the forward abutment using the RMR method in accordance with ODOT GDM Section 1303.3.3. Global stability is not expected to be a concern for either foundation system.

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

TABLE OF CONTENTS

LIST OF TABLES

LIST OF APPENDICES

1. INTRODUCTION

1.1. General

National Engineering and Architectural Services Inc. (NEAS) presents our Structure Foundation Exploration Report for the planned replacement of the existing bridge carrying SR-281 over Creps Ditch in Wood County, Ohio. The report presents a summary of the encountered surficial and subsurface conditions and our recommendations for bridge foundation design and construction. Our recommendations are in accordance with ODOT's 2024 LRFD Bridge Design Manual (BDM) (ODOT, 2024), ODOT's 2024 Geotechnical Design Manual (GDM) (ODOT, 2024).

The exploration was conducted in general accordance with NEAS, Inc.'s proposal to Colliers Engineering and Design dated April 1, 2024, and with the provisions of ODOT's *Specifications for Geotechnical Explorations* (SGE) (ODOT, 2024).

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

1.2. Proposed Construction

The existing WOO-281-11.97 bridge is a three-span continuous reinforced concrete slab bridge with a reinforced concrete substructure. The existing bridge is approximately 52 ft in length with an approximate roadway width of 32 ft.

It is our understanding that the proposed structure is a single span prestressed concrete box beam bridge with composite concrete deck and semi-integral abutments support on spread footings. The proposed bridge is 53'-0" (from center of bearing to center of bearing) long and the roadway of the proposed bridge is 32'-0" wide.

2. GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1. Geology and Physiography

The project site is located within the Maumee Lake Plain. This area ranges in elevation from 570 ft to 800 ft with very low relief (5 ft). It is characterized as 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. The geology is described as Pleistocene-age silt, clay, and wave-planed clayey till over Silurian and Devonian-age carbonate rocks and shales (ODGS, 1998).

Based on the Quaternary geology map of Ohio, the geology at the project site is mapped as clayey till (Hiram till), lake-planed moraine, very flat, planed by waves in glacial lakes, small patches of sand, silt or clay on the surface in many areas (Pavey, et al 1999).

Based on the Bedrock Geologic Units Map of Ohio (USGS & ODGS, 2006), bedrock within the project area consists of combination of Dolomite and Shale of the Tymochtee and Greenfield formation. The formation is comprised of Silurian-age. This formation is described as; Dolomite, olive-gray to yellowishbrown, thin to massive bedded, upper two-thirds commonly contains brownish-black to gray shale laminae; locally developed brecciated zones in lower one third.

The bedrock appears to follow the natural topography of the site. The bedrock is relatively level throughout the project (ODGS, 2003). Based on the ODNR bedrock topography map of Ohio, bedrock elevations at the project site can be expected to be from 660 to 680 ft amsl, putting bedrock at depths of between 5 and 20 ft below ground surface (bgs).

The soils at the project site have been mapped (Web Soil Survey) by the Natural Resources Conservation Service (USDA, 2024) as a Hoytville clay loam. Hoytville clay loam is comprised of fine-grained soils and classified as A-6, A-7-5 and A-7-6 type soils according to the AASHTO method of soil classification.

2.2. Hydrology/Hydrogeology

Groundwater at the project site is expected to be at an elevation consistent with the nearby Creps Ditch, which serves as the primary hydraulic influence within the project's boundaries. The water level in the Creps Ditch may generally represent the local groundwater table. However, it should be noted that perched groundwater systems may be existent in areas due to the presence of fine-grained soils making it difficult for groundwater to permeate to the phreatic surface.

The project site is located within a special flood hazard area (Zone A) based on available mapping by the Federal Emergency Management Agency's (FEMA) National Flood Hazard mapping program (FEMA, 2024).

2.3. Mining and Oil/Gas Production

No mines were noted on ODNR's Abandoned Underground Mine Locator in the vicinity of the project site. (ODNR [1], 2024).

No oil or gas wells were noted on ODNR's Oil and Gas Well Locator in the vicinity of the project site (ODNR [1], 2024).

2.4. Historical Records and Previous Phases of Project Exploration

The following report/plans were available for review and evaluation for this report:

- Site plan, site investigation and field soil boring log data for WOO-281-120, Dated September, 1954

Historical soil boring associated with the above projects were reviewed, however, they were not utilized for our analysis, and therefore, are not referenced or presented within this report.

2.5. Site Reconnaissance

A field reconnaissance visit for the proposed bridge was conducted on September 17th, 2024, within the project limits. Site conditions, including the existing land conditions and pavement conditions, were noted and photographed during the visit. Photographs of notable features and a summary of our observations are provided below.

1) Land Use and Cover

The land use of most of the project area consists of ODOT ROW (Right of Way), agricultural farmland, and woodland.

2) WOO-281-1197 bridge over Creps ditch

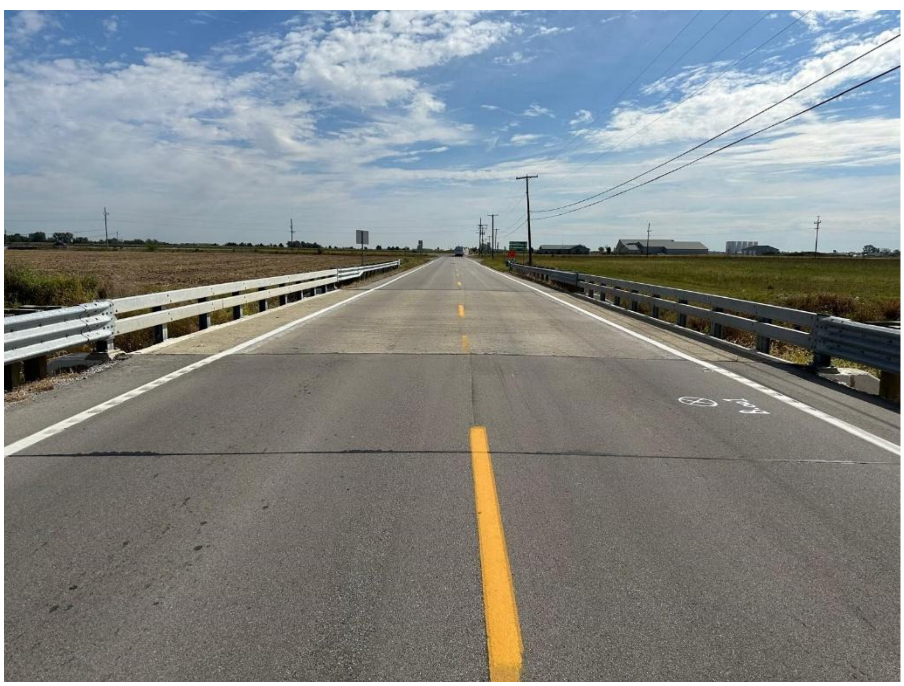
The pavement conditions in project area were observed to be in good condition at the time of reconnaissance, with some signs of weathering and surface wear. Low-severity raveling was observed (Photograph 1). The project area includes embankment slopes associated with the existing bridge, as well as pavement and drainage ditches along SR 281. The roadway embankment slopes at the site generally appeared stable, with no signs of instability observed during our site visit. The slopes were vegetated with small to large bushes and ranged from 2 Horizontal to 1 Vertical (2H:1V) to 3H:1V (Photograph 2). The bridge slopes southward, ultimately draining into the Creps Ditch (Photograph 3).

The existing bridge carrying SR 281 over Creps ditch is a two-lane, three-span concrete slab bridge sitting on concrete abutments. During the visit, low severity iron staining and efflorescence were observed on the bridge abutments (Photograph 4). No evidence of scour, erosion, or standing water was noted in the bridge area. Additionally, no signs of geotechnical instability were observed along the existing embankments at the time of our reconnaissance.

Photograph 1: Pavement conditions

Photograph 2: Embankment slopes

Photograph 3: Bridge Slope



Photograph 4: Bridge Abutment

3. GEOTECHNICAL EXPLORATION

3.1. Field Exploration Program

The project subsurface exploration was conducted by NEAS on October 9, 2024, and included 2 borings drilled to a depth of 19.1 ft below ground surface. The boring locations were selected by NEAS in general accordance with the guidelines contained in the SGE with the intent to evaluate subsurface soil and groundwater conditions. Borings were typically located within the planned project construction areas that were not restricted by underground utilities or dictated by terrain (e.g. steep embankment slopes). Project boring locations were surveyed in the field after drilling by project surveyor. Each individual project boring log (included within Appendix B) includes the recorded boring latitude and longitude location and the corresponding ground surface elevation (surveyed by the project surveyors). The boring locations are depicted on the Site Plan provided in Appendix A. Latitude/Longitude, elevations and stationing and offsets of the borings are shown on Table 1 below.

Table 1: Project Boring Summary

Boring Number	Location (Sta/offset)	Easting	Northing	Latitude	Longitude	Elevation (NAVD 88) (ft)	Depth (ft)	Structure
B-001-0-24	631+12, 9' RT.	163182.909	412786.861	41.282970	-83.653120	683.8	19.1	Rear Abutment
B-002-0-24	632+06, 9' LT.	163276.735	412803.565	41.283020	-83.652780	683.4	19.1	Forward Abutment

Notes:
1. Stationing and Offset are in reference to centerline of Proposed SR-281.

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

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

3.2. Laboratory Testing Program

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

3.2.1. Classification Testing

Representative soil samples were selected for index properties (Atterberg Limits) and gradation testing for classification purposes on 100% of the samples. At each boring location, samples were selected for testing with the intent of identification and classification of all significant soil units. Soils not selected for testing were compared to laboratory tested samples/strata and classified visually. Moisture content testing was conducted on all samples. The laboratory testing was performed in general accordance with applicable AASHTO specifications.

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

3.2.2. Standard Penetration Test Results

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

3.2.3. D_{50} Values for Scour Evaluation

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

Table 2: D_{50} Values for Scour Evaluation

Boring Number	Specimen ID	Specimen Elevation (ft)	ODOT (Modified AASHTO) ~ USCS Classification	D_{50} (mm)	Scour Critical Shear Stress, τ_c (psf)	$D_{50, equiv}$ (mm)	Erosion Category (EC)
B-001-0-24	SS-1	682.0' - 680.8'	A-2-4 ~ SILTY SAND with GRAVEL(SM)	1.160	0.024	1.160	2.277
	SS-2	680.8' - 679.3'	A-7-6 ~ SANDY LEAN CLAY(CL)	0.018	0.639	30.614	3.983
	SS-3	679.3' - 677.8'	A-7-6 ~ LEAN CLAY with SAND(CL)		0.688	32.925	3.867
	SS-4	677.8' - 676.3'	A-7-6 ~ LEAN CLAY with SAND(CL)	0.005	0.409	19.572	3.775
	SS-5	676.3' - 674.8'	A-2-4 ~ SILTY GRAVEL with SAND(GM)	1.069	0.022	1.069	2.235
B-002-0-24	SS-1	681.9' - 680.4'	A-7-6 ~ LEAN CLAY with SAND(CL)	0.007	0.348	16.668	3.823
	SS-2	680.4' - 678.9'	A-7-6 ~ LEAN CLAY with SAND(CL)	0.007	0.349	16.717	3.983
	SS-3	678.9' - 677.4'	A-7-6 ~ FAT CLAY with SAND(CH)	0.005	0.690	33.026	4.078
	SS-4	677.4' - 675.9'	A-7-6 ~ SANDY LEAN CLAY(CL)	0.016	0.240	11.495	3.775
	SS-5	675.9' - 675.0'	A-1-b ~ SILTY GRAVEL with SAND(GM)	4.289	0.090	4.289	2.959

Based on our lab testing results, the equivalent D_{50} (mm) values of the bedrock were estimated using the methods outlined in ODOT's BDM Section 305.2.1.2.b and ODOT's GDM Section 1302.1.3. At both rear and forward abutments, the equivalent D_{50} is estimated to be 5516.4 mm, derived from borings B-001-0-24 and B-002-0-24. The Erodibility Index is estimated at 475 for both abutments. Based on our assessment, the bedrock can be considered non-scour-resistant starting at elevations 674.7 ft and 674.3 ft for the rear and forward abutments, respectively. The lab testing results, Erodibility Index, and equivalent D_{50} (mm) calculation process are provided in Appendix B.

Colliers Engineering and Design conducted scour analyses, indicating a total scour of 3 feet at the rear abutment and 4 feet at the forward abutment. This information was provided via email on November 11, 2024. The estimated bottom of scour is at elevations 677 feet and 676 feet at the rear and forward abutment locations, respectively.

3.2.4. Unconfined Compressive Strength of Rock Core Results

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

Table 3: Unconfined Compressive Strength Test Results

Boring ID	Depth (ft)	Elevation (ft)	Unconfined Compressive Strength (psi)	Stain at Failure (%)
B-001-0-24	9.2 - 9.6	674.2 - 674.6	25,396	1.0
B-002-0-24	13.5 - 13.9	669.5 - 669.9	21,117	0.7

3.2.5. Slake Durability Index of Rock Core

The slake durability index test of rock core samples was conducted following ASTM D4644, the standard method for assessing the durability of shales and similar weak rocks. Tests on two samples from the exploration program showed indices of 99.4% for B-001-0-24 and 99.3% for B-002-0-24, indicating high durability of the rock cores.

4. GEOTECHNICAL FINDINGS

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

4.1. Subsurface Conditions

The subsurface profile at the site is generally consistent with the geological model for the project in terms of the materials encountered. The profile at the proposed bridge replacement site generally consists primarily of stiff to hard cohesive fine materials, with minor amounts of medium dense to very dense granular materials. Bedrock was encountered in both project borings, at depths ranging from 8.4 ft to 9.0 ft below ground surface (elevations between 674.8 ft and 675 ft above mean sea level).

4.1.1. Overburden Soil

At the proposed rear abutment, the subsurface soils encountered below the pavement generally consist of non-cohesive coarse-grained soils interbedded with cohesive fine-grained soils. The granular soil layer, classified as Stone Fragments with Sand and Silt (A-2-4), is found between elevations 682.0 ft to 680.8 ft amsl and from 676.3 ft to 674.8 ft amsl. The cohesive fine-grained soils, classified as Clay (A-7-6), are present from elevation 680.8 ft to 676.3 ft amsl. The non-cohesive soils at the rear abutment location are described as having a relative compactness of medium dense to very dense correlating to N_{60} values between 12 and 95. The natural moisture content of the non-cohesive soils ranged from 5 to 12 percent. The cohesive soils can be described as having a very stiff to hard consistency based on N_{60} values between 7 and 13 bpf and unconfined compressive strengths (estimated by means of hand penetrometer) between approximately 2.50 and 4.50 tsf. Natural moisture contents of the cohesive soils ranged from 19 to 25 percent. Based on the Atterberg Limit test, the liquid limit and plastic limit for those cohesive soil samples range from 43 to 46 percent and from 20 to 23 percent, respectively. The plastic index ranges from 21 to 26 percent.

At the proposed forward abutment, the subsurface soils encountered below the pavement generally consist primarily of cohesive fine-grained soils overlying a thin layer of non-cohesive coarse-grained soils above bedrock. The cohesive fine-grained soils, classified as Clay (A-7-6), are present from elevation 681.9 ft to 675.9 ft amsl. The non-cohesive coarse-grained soils, classified as Stone Fragments with Sand (A-1-b), are encountered from elevation 675.9 ft to 675.0 ft amsl. The cohesive soils can be described as having a stiff to very stiff consistency based on N_{60} values between 9 and 19 bpf and unconfined compressive strengths (estimated by means of hand penetrometer) between approximately 1.75 and 2.75 tsf. Natural moisture contents of the cohesive soils ranged from 23 to 26 percent. Based on the Atterberg Limit test, the liquid limit and plastic limit for those cohesive soil samples range from 41 to 50 percent and from 20 to 22 percent, respectively. The plastic index ranges from 21 to 29 percent. The non-cohesive soils at the forward abutment location are described as having a relative compactness of very dense correlating to N_{60} value of refusal. The natural moisture content of the non-cohesive soil was 8 percent.

4.1.2. Groundwater

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

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

4.1.3. Bedrock

Bedrock was encountered in both project borings at a terminating depth of 19.1 ft for the rear and forward abutments. At the rear abutment, bedrock was encountered at a depth of 9.0 feet below ground surface (674.8 ft above mean sea level), while at the forward abutment, it was found at 8.4 feet below ground surface (675.0 ft above mean sea level).

Based on the exploration and testing conducted, the bedrock at the project site was classified as gray, unweathered to slightly weathered, strong to very strong, very thin to medium bedded, highly fractured to moderately fractured, narrow to tight Dolomite. Recovery of the bedrock core performed ranged from 95 to 96 percent, with a Rock Quality Designation (RQD) value of 76 percent.

5. ANALYSES AND RECOMMENDATIONS

We understand that this project entails replacing the existing bridge with a three-span continuous reinforced concrete slab bridge on reinforced concrete substructure. The proposed structure is a single span prestressed concrete box beam bridge with composite concrete deck and semi-integral abutments support on spread footings. The summary and results of our evaluation as well as recommendations presented in subsequent sections.

5.1. Soil Profile for Analysis

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

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

WOO-281-11.97 Bridge over Creps Ditch: Rear Abutment B-001-0-24							
Soil Description	Unit Weight(1) (pcf)	Moist Unit Weight(1) (pcf)	Saturated Unit Weight(1) (pcf)	Undrained Shear Strength(2) (psf)	Effective Cohesion(3) (psf)	Effective Friction Angle(3) (degrees)	Setup Factor (f_{su})
Gravel with Sand and Silt (A-2-4) Depth (683.8 ft - 680.8 ft)	112	112	122	-	-	34	1.20
Clay (A-7-6) Depth (680.8 ft - 676.3 ft)	110	110	120	1250	150	22	2.00
Gravel with Sand and Silt (A-2-4) Depth (676.3 ft - 674.8 ft)	130	130	140	-	-	40	1.20

Notes:
1. *Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405.*
2. *Values calculated from Terzaghi and Peck (1967) if N160<52, else Stroud and Butler (1975) was used.*
3. *Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.*

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

WOO-281-11.97 Bridge over Creps Ditch: Forward Abutment B-002-0-24							
Soil Description	Unit Weight(1) (pcf)	Moist Unit Weight(1) (pcf)	Saturated Unit Weight(1) (pcf)	Undrained Shear Strength(2) (psf)	Effective Cohesion(3) (psf)	Effective Friction Angle(3) (degrees)	Setup Factor (f_{su})
Clay (A-7-6) Depth (683.4 ft - 677.4 ft)	108	108	118	1150	100	21	2.00
Clay (A-7-6) Depth (677.4 ft - 675.9 ft)	112	112	122	2350	200	24	2.00
Gravel with Sand and Silt (A-2-4) Depth (675.9 ft - 675 ft)	130	130	140	-	-	40	1.20

Notes:
1. *Values interpreted from ODOT Geotechnical Design Manual (GDM) Section 405.*
2. *Values calculated from Terzaghi and Peck (1967) if N160<52, else Stroud and Butler (1975) was used.*
3. *Values interpreted from LRFD BDS Table 10.4.6.2.4-1 and ODOT GDM Table 400-3.*

5.2. Subgrade Design and Recommendations

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

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

5.2.1. Pavement Design Recommendations

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

Table 6: Pavement Design Values

Segment	Maximum N_{60L}	Minimum N_{60L}	Average N_{60L}	Average PI Value	Design CBR
SR-281	9	7	8	24	5

5.2.2. Unsuitable/Unstable Subgrade

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

5.2.2.1. Rock

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

5.2.2.2. Prohibited Soils

Prohibited soil types, per the GDM, include A-4b, A-2-5, A-5, A-7-5, A-8a, A-8b, and soils with liquid limits greater than 65. No prohibited soils were encountered within 3 ft of the subgrade of the referenced project roadway.

5.2.2.3. Weak Soils

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

A summary of the boring locations where unstable soils were encountered and determined to have a potential impact on subgrade performance are shown in Table 7 below, per the roadway segment for which they were encountered.

Table 7: Unstable Soil Location Summary

Boring ID	N_{60L}	Subgrade Depth (ft)
B-001-0-24	7	2.0 - 6.5
B-002-0-24	9	0.8 - 5.3

It should be noted that *Figure B - Subgrade Stabilization* does not apply to soil types A-1-a, A-1-b, A-3, or A-3a, nor to soils with N_{60L} values of 15 or more. Per GB1 guidance, *these soils should be reworked to stabilize the subgrade.*

5.2.2.4. High Moisture Content Soils

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

Table 8: High Moisture Content Soils Summary

Boring ID	Soil Type	Moisture Content (%)	Optimum Moisture Content (%)	Depth Below Subgrade (ft)
B-001-0-24	A-7-6	23	20	3.5 - 5.0
		25	19	5.0 - 6.5
B-002-0-24	A-7-6	26	19	0.8 - 2.3
		23	18	2.3 - 3.8
		23	18	3.8 - 5.3
		24	18	5.3 - 6.8

5.2.3. Stabilization Recommendations

According to our subgrade analysis, unstable and high moisture content soils were encountered in 100% of the proposed subgrade surface. These soils exhibited an N60 value below 12 bpf and a moisture content exceeding the optimum level by more than 3 percent within 3 feet of subgrade. According to ODOT GDM Section 605, "for all other roadways, if it is determined that 30 percent or more of the subgrade area must be stabilized, consider stabilizing the entire project (global stabilization)". Therefore, NEAS recommends global stabilization using one of the following methods: Item 204 Excavate and Replace with Geotextile to a depth of 12 inches, or Item 206 Lime Stabilization to a depth of 14 inches, as presented in Table 9 below. Generally, chemical stabilization is more economical when stabilizing large areas (approximately greater than 1mile of roadway), however, a cost analysis of the options should be conducted.

Table 9: Subgrade Stabilization Recommendation

Segment	Remediation Depth (inches)		
	Excavate and Replace (Item 204 w/ Geotextile)	Excavate and Replace (Item 204 w/ Geogrid - SS 861)	Chemical Stabilization (Item 206)
SR-281	12	N/A	14
Note: N/A, Not Applicable based on GB1- Figure B - Subgrade Stabilization			

Stabilization limits should extend 18-inches beyond the edge of the proposed paved roadway, shoulder or median and it is recommended removing any topsoil, existing pavement materials or abandoned structure foundation materials. The mix design should be conducted in accordance with ODOT's CMS Supplement 1120 (Mixture Design for Chemically Stabilized Soils). For design purposes it may be assumed that the lime addition will be 5% using the following formula.

Lime: $C = 0.75 \times T \times 115 \times 0.05$

Where:

C = amount of chemical in pounds / square yard and

T = thickness of the treatment zone in inches

A dry density of 115-pounds per cubic foot (pcf) is assumed.

NEAS's opinion that the subgrade soils will provide adequate pavement support assuming it is designed and constructed in accordance with the recommendations provided within this report, as well as all applicable ODOT standards and specifications.

5.3. Bridge Foundation Analysis and Recommendations

A foundation review was conducted for the replacement bridge, based on the following design information: 1) the site plan for the bridge provided by Colliers Engineering and Design; 2) subsurface exploration data; 3) subsequent conversations with Colliers Engineering and Design; and 4) lab testing results.

5.3.1. Shallow Foundation

At the proposed bridge site, the bedrock was classified as strong to very strong dolomite. The unconfined compressive strength ranged from 21,117 psi to 25,396 psi, and the slake durability index ranged from 99.3 percent to 99.4 percent. However, the Geological Strength Index (GSI) falls within the range of 40 to 50, in accordance with LRFD Figure 10.4.6.4-1, due to the slicken-sided shear discontinuities observed at depths of approximately 17 ft in boring B-001 and 10 ft in boring B-002. The rock mass rating (RMR) was determined to be 68 in accordance with FHWA GEC 5, Section 6.2.3. This bedrock does not meet all the requirements for scour-resistant rock in accordance with BDM Section 305.2.1.2.b. Therefore, based on our assessment, the bedrock was considered non-scour-resistant.

NEAS recommends that the bottom of footing be keyed a minimum of 3 inches into non-scour-resistant bedrock. Based on the site plan, the bottom of footing elevation is 673.50 ft at both abutments. The anticipated scour is expected to terminate within the soil zone, approximately 3.5 ft above the bottom of the rear abutment footing and 2.5 ft above the bottom of the forward abutment footing. Therefore, scour is not expected to affect the footing bearing elevation.

The factored bearing resistance of the bedrock was estimated to be 1,621 ksf at the rear abutment and 1,348 ksf at the forward abutment using the RMR method in accordance with ODOT GDM Section 1303.3.3. The bedrock bearing resistance analysis is included in Appendix D.

5.4. Global Stability Analysis

Global stability is not expected to be a concern for the bridge with footing keyed into the bedrock.

5.5. Seismic Site Class

Based on the results of the subsurface exploration, laboratory test data, and the AASHTO Site Class Definitions indicated in Table 3.10.3.1-1 of the *LRFD Bridge Design Specifications, 9th Edition* (AASHTO LRFD, 2020), the average Standard Penetration Test blow count $\bar{N}$ for B-001-0-24 and B-002-0-24 is 12 blows/ft and 11 blows/ft, respectively. Therefore, the project site is classified as Site Class of E – Soft Soil, with $\bar{N}$ <15 blows/ft.

6. QUALIFICATIONS

This investigation was performed in accordance with accepted geotechnical engineering practice for the purpose of characterizing the subsurface conditions at the site of the proposed project WOO-281-11.97 (PID# 113085) in Wood County, Ohio. This report has been prepared for Colliers Engineering and Design, ODOT and their design consultants to be used solely in evaluating the soils underlying the indicated structures and presenting geotechnical engineering recommendations specific to this project. The assessment of general site environmental conditions or the presence of pollutants in the soil, rock and groundwater of the site was beyond the scope of this geotechnical exploration. Our recommendations are based on the results of our field explorations, laboratory test results from representative soil samples, geotechnical engineering analyses and historical information. The results of the field explorations and laboratory tests, which form the basis of our recommendations, are presented in the appendices as noted. This report does not reflect any variations that may occur between the borings or elsewhere on the site, or variations whose nature and extent may not become evident until a later stage of construction. In the event that any changes occur in the nature, design or location of the proposed structural work, the conclusions and recommendations contained in this report should not be considered valid until they are reviewed and have been modified or verified in writing by a geotechnical engineer.

It has been a pleasure to be of service to Colliers Engineering and Design in performing this geotechnical exploration for the WOO-281-11.97 (PID# 113085) bridge replacement project. Please call if there are any questions, or if we can be of further service.

Respectfully Submitted,

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

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

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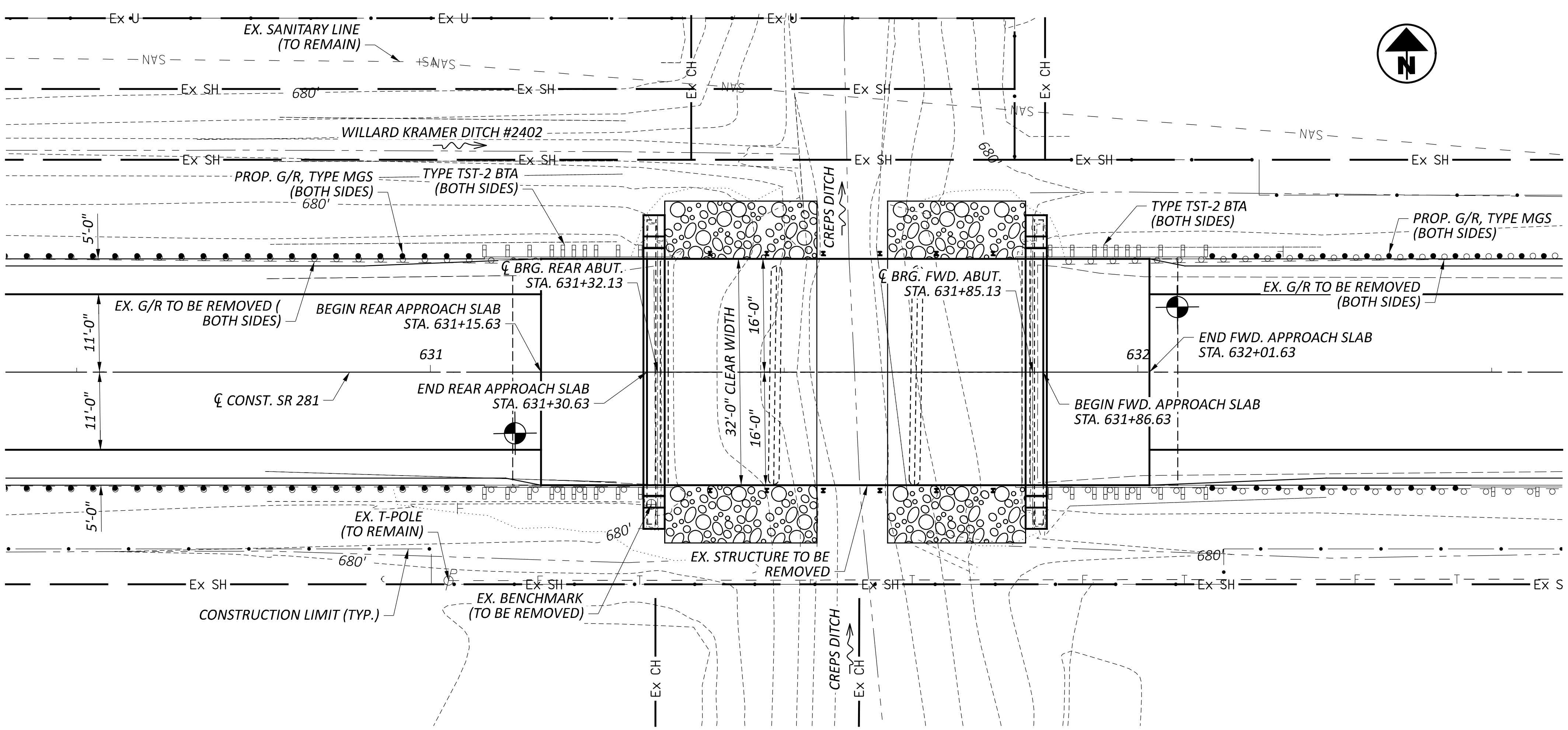
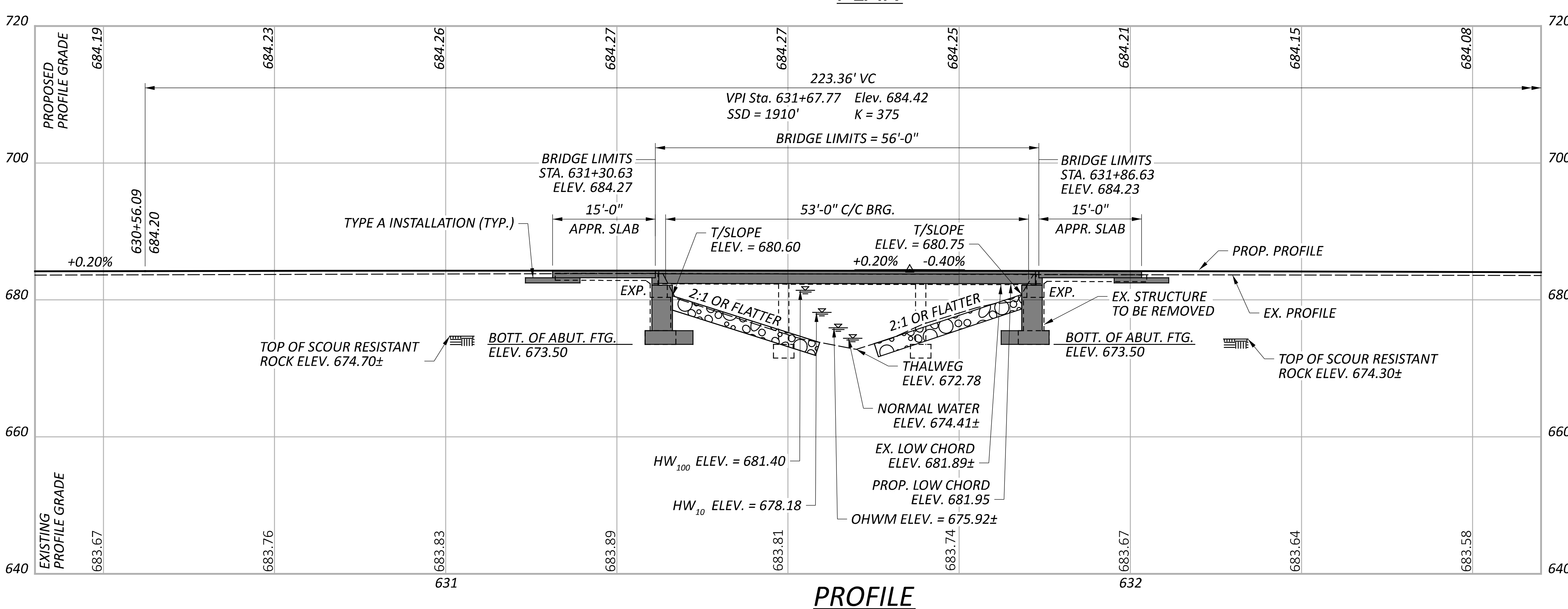
APPENDIX A

SITE PLAN & BORING LOCATION PLAN



PLAN



PROFILE

ALONG ℄ CONST. SR281

BENCHMARK DATA

BM 301 STA. 628+56.20	ELEV. 683.01	28.4' OFFSET,	RIGHT
BM 300 STA. 635+76.40	ELEV. 683.10	29.2' OFFSET,	RIGHT
BM 105 STA. 638+21.70	ELEV. 684.99	498.3' OFFSET,	LEFT

FOR ADDITIONAL BENCHMARK INFORMATION. SEE ROADWAY PLAN SHEET XX.

NOTES

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

DESIGN TRAFFIC:

2027 ADT = 1500 2027 ADTT = 345

2047 ADT = 1900 2047 ADTT = 437

DIRECTIONAL DISTRIBUTION = 0.52

LEGEND

BORING LOCATION

ROCK CHANNEL PROTECTION, TYPE C WITH GEOTEXTILE FABRIC

HYDRAULIC DATA

DRAINAGE AREA = 18.6 SQ. MILES

Q (10) = 423 CFS V (10) = 5.14 FT/S

Q (100) = 1090 CFS V (100) = 5.38 FT/S

STRUCTURE CLEARS THE 10 YEAR

DESIGN HW BY 3.77 FEET.

EXISTING STRUCTURE

TYPE: CONTINUOUS REINFORCED CONCRETE SLAB ON REINFORCED CONCRETE SUBSTRUCTURE

SPANS: 16'-0"±, 20'-0"±, 16'-0"± C/C BEARINGS

ROADWAY: 32'-0"± F/F GUARDRAIL

LOADING: CF = 30 (51)

SKEW: NONE

WEARING SURFACE: 1¾" MICRO SILICA MODIFIED CONCRETE OVERLAY

APPROACH SLABS: AS-1-54 (15' LONG)

ALIGNMENT: TANGENT

CROWN: ¼" / FT.

STRUCTURE FILE NUMBER: 8706158

DATE BUILT: 1955, REHABILITATED 2006

DISPOSITION: TO BE REPLACED

PROPOSED STRUCTURE

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

SPANS: 53'-0" C/C BEARINGS

ROADWAY: 32'-0" F/F RAILING

LOADING: HL-93 (PLUS FUTURE WEARING SURFACE = 60 PSF)

SKEW: NONE

WEARING SURFACE: 1" MONOLITHIC CONCRETE

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

ALIGNMENT: TANGENT

CROWN: 0.016 FT/FT

DECK AREA: 1,792 SF

COORDINATES: LATITUDE N 41°16'58"

LONGITUDE W 83°39'11"

HORIZONTAL SCALE IN FEET 0 5 10 20

PRESTRESSED BOX BEAMS WITH TST RAILING ALTERNATIVE 1 SR281 OVER CREPS DITCH

SFN 8706158

DESIGN AGENCY: Colliers Engineering & Design

DESIGNER	CHECKER
CED	CED

REVIEWER: CED 11-15-24

PROJECT ID: 113085

SUBSET	TOTAL
XX	XX

SHEET	TOTAL
XX	XX

WOO-281-11.97

MODEL: CLP_SR281 - Plan 1-1 [Sheet] PAPERSIZE: 17x11 (in.) DATE: 11/15/2024 TIME: 11:34:24 AM USER: cdrexler
P:\Projects\ODOT\24003454G\4.0_Dwgs\400-Engineering\Structures\SFN_8706158\Sheets\113085_SFN_8706158_SP001.dgn

APPENDIX B

BORING LOGS & LABORATORY TEST RESULTS

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 1/31/25 12:48 - X:\1ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\WOO-281-11.97\GINT FILES\WOO-281-11.97_101024.GPJ

PROJECT: WOO-281-11.97	DRILLING FIRM / OPERATOR: NEAS INC. / JL	DRILL RIG: CME 55TB	STATION / OFFSET: 631+12, 9' RT.
TYPE: BRIDGE	SAMPLING FIRM / LOGGER: NEAS INC. / JL	HAMMER: CME AUTOMATIC	ALIGNMENT: SR-281
PID: 113085 SFN:	DRILLING METHOD: 3.25" HSA / NQ2	CALIBRATION DATE: 7/30/24	ELEVATION: 683.8 (MSL) EOB: 19.1 ft.
START: 10/9/24 END: 10/9/24	SAMPLING METHOD: SPT / NQ2	ENERGY RATIO (%): 89	LAT / LONG: 41.282970, -83.653120

EXPLORATION ID B-001-0-24

PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 683.8	DEPTHS
8.0" ASPHALT AND 14.0" BASE (DRILLERS DESCRIPTION)	682.0	
MEDIUM DENSE, BROWN, **GRAVEL WITH SAND AND SILT**, LITTLE CLAY, DRY	680.8	
VERY STIFF, GRAY AND BROWN, **CLAY**, SOME SAND, SOME SILT, LITTLE GRAVEL, IRON STAINING, DAMP	679.3	
VERY STIFF TO HARD, GRAY MOTTLED WITH ORANGISH BROWN, **CLAY**, SOME SILT, LITTLE SAND, TRACE GRAVEL, IRON STAINING, DAMP TO MOIST	676.3	
VERY DENSE, BROWN, **STONE FRAGMENTS WITH SAND AND SILT**, LITTLE CLAY, DAMP	674.8	TR
DOLOMITE, GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, STRONG TO VERY STRONG, VERY THIN TO MEDIUM BEDDED, STYLOLITIC, CONTAINS SLICKENSIDES FROM 17.1'-17.4', BEDDING DISCONTINUITIES: LOW ANGLE, SHEAR DISCONTINUITIES FROM 17.1'-17.4', HIGHLY FRACTURED TO MODERATELY FRACTURED, NARROW TO TIGHT, SLIGHTLY ROUGH TO SLICKENSIDED, BLOCKY, FAIR TO POOR SURFACE CONDITION; RQD 76%, REC 96%. @9.2'-9.6': Qu = 25396 PSI @ 1.0%	664.7	EOB

SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
5 4 4	12	28	SS-1	-	45	16	11	17	11	NP	NP	NP	5	A-2-4 (0)	
3 3 4	10	33	SS-2	4.00	13	14	12	23	38	46	20	26	19	A-7-6 (12)	
3 4 5	13	44	SS-3	4.50	2	5	12	27	54	46	23	23	23	A-7-6 (14)	
3 2 3	7	50	SS-4	2.50	1	5	13	32	49	43	22	21	25	A-7-6 (13)	
3 30 34	95	67	SS-5	-	46	9	11	22	12	NP	NP	NP	12	A-2-4 (0)	
50/1"	-	100	SS-6	-	-	-	-	-	-	-	-	-	7	Rock (V)	
76		96	NQ2-1											CORE	

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. BORING OFFSET 10.0' WEST.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 1 BAG HOLE PLUG; SHOVELED SOIL CUTTINGS

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 1/31/25 12:48 - X:\1ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\WOO-281-11.97\GINT FILES\WOO-281-11.97_101024.GPJ

PROJECT: WOO-281-11.97	DRILLING FIRM / OPERATOR: NEAS / JL	DRILL RIG: CME 55TB	STATION / OFFSET: 632+06, 9' LT.	EXPLORATION ID B-002-0-24
TYPE: BRIDGE	SAMPLING FIRM / LOGGER: NEAS / JL	HAMMER: CME AUTOMATIC	ALIGNMENT: SR-281	
PID: 113085 SFN:	DRILLING METHOD: 3.25" HSA / NQ2	CALIBRATION DATE: 7/30/24	ELEVATION: 683.4 (MSL) EOB: 19.1 ft.	PAGE 1 OF 1
START: 10/9/24 END: 10/9/24	SAMPLING METHOD: SPT / NQ2	ENERGY RATIO (%): 89	LAT / LONG: 41.283020, -83.652780	

MATERIAL DESCRIPTION AND NOTES	ELEV. 683.4	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
8.0" ASPHALT AND 10.0" BASE (DRILLERS DESCRIPTION)	681.9	1																
VERY STIFF, DARK GRAY, **CLAY**, SOME SILT, SOME SAND, TRACE GRAVEL, SLIGHTLY ORGANIC, MOIST	680.4	2	5 3 3	9	39	SS-1	2.75	2	4	20	28	46	44	22	22	26	A-7-6 (13)	
STIFF TO VERY STIFF, GRAY MOTTLED WITH ORANGISH BROWN, **CLAY**, SOME SILT, SOME SAND, TRACE TO LITTLE GRAVEL, IRON STAINING, SS-2 CONTAINS NO INTACT SOIL FOR HP READINGS, MOIST		3–4	3 3 3	9	44	SS-2	-	1	4	23	27	45	47	21	26	23	A-7-6 (15)	
		5	3 3 4	10	44	SS-3	2.75	1	3	19	27	50	50	21	29	23	A-7-6 (18)	
	675.9	6–7	2 2 11	19	78	SS-4	1.75	15	6	15	27	37	41	20	21	24	A-7-6 (10)	
VERY DENSE, GRAY, **STONE FRAGMENTS WITH SAND**, LITTLE SILT, TRACE CLAY, MOIST	675.0	8 TR	45 50/5"	-	82	SS-5	-	59	14	10	13	4	NP	NP	NP	8	A-1-b (0)	
DOLOMITE, GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, STRONG TO VERY STRONG, VERY THIN TO MEDIUM BEDDED, STYLOLITIC, CONTAINS SLICKENSIDES FROM 10.1'-10.3', BEDDING DISCONTINUITIES: LOW ANGLE, SHEAR DISCONTINUITIES FROM 10.1'-10.3', HIGHLY FRACTURED TO MODERATELY FRACTURED, NARROW TO TIGHT, SLIGHTLY ROUGH TO SLICKENSIDED, BLOCKY, FAIR TO POOR SURFACE CONDITION; RQD 76%, REC 95%. SS-6 CONTAINS NO RECOVERY @13.5'-13.9': Qu = 21117 PSI @ 0.7%		9	50/1"	-	0	SS-6	-	-	-	-	-	-	-	-	-	-		
	664.3	10–19 EOB	76		95	NQ2-1											CORE	

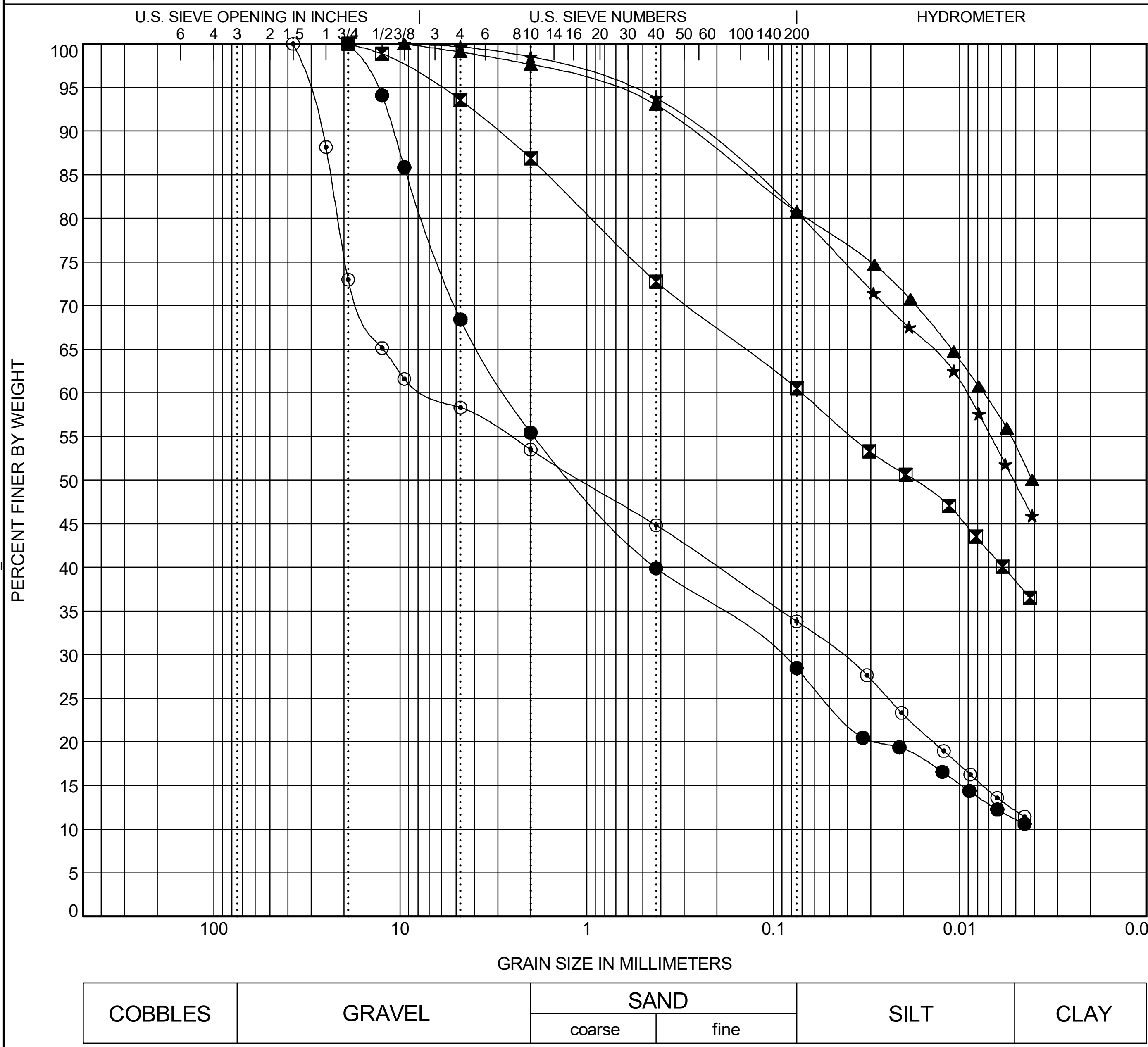
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE. DRILLED AS STAKED.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; POURED 1 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT WOO-281-11.97 **PID** 113085
OGE NUMBER 0 **PROJECT TYPE**



	Specimen Identification		ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
●	**B-001-0-24**	**1.5**	**A-2-4 ~ SILTY SAND with GRAVEL(SM)**	**NP**	**NP**	**NP**
⊠	**B-001-0-24**	**3.0**	**A-7-6 ~ SANDY LEAN CLAY(CL)**	**46**	**20**	**26**
▲	**B-001-0-24**	**4.5**	**A-7-6 ~ LEAN CLAY with SAND(CL)**	**46**	**23**	**23**
★	**B-001-0-24**	**6.0**	**A-7-6 ~ LEAN CLAY with SAND(CL)**	**43**	**22**	**21**
⊙	**B-001-0-24**	**7.5**	**A-2-4 ~ SILTY GRAVEL with SAND(GM)**	**NP**	**NP**	**NP**

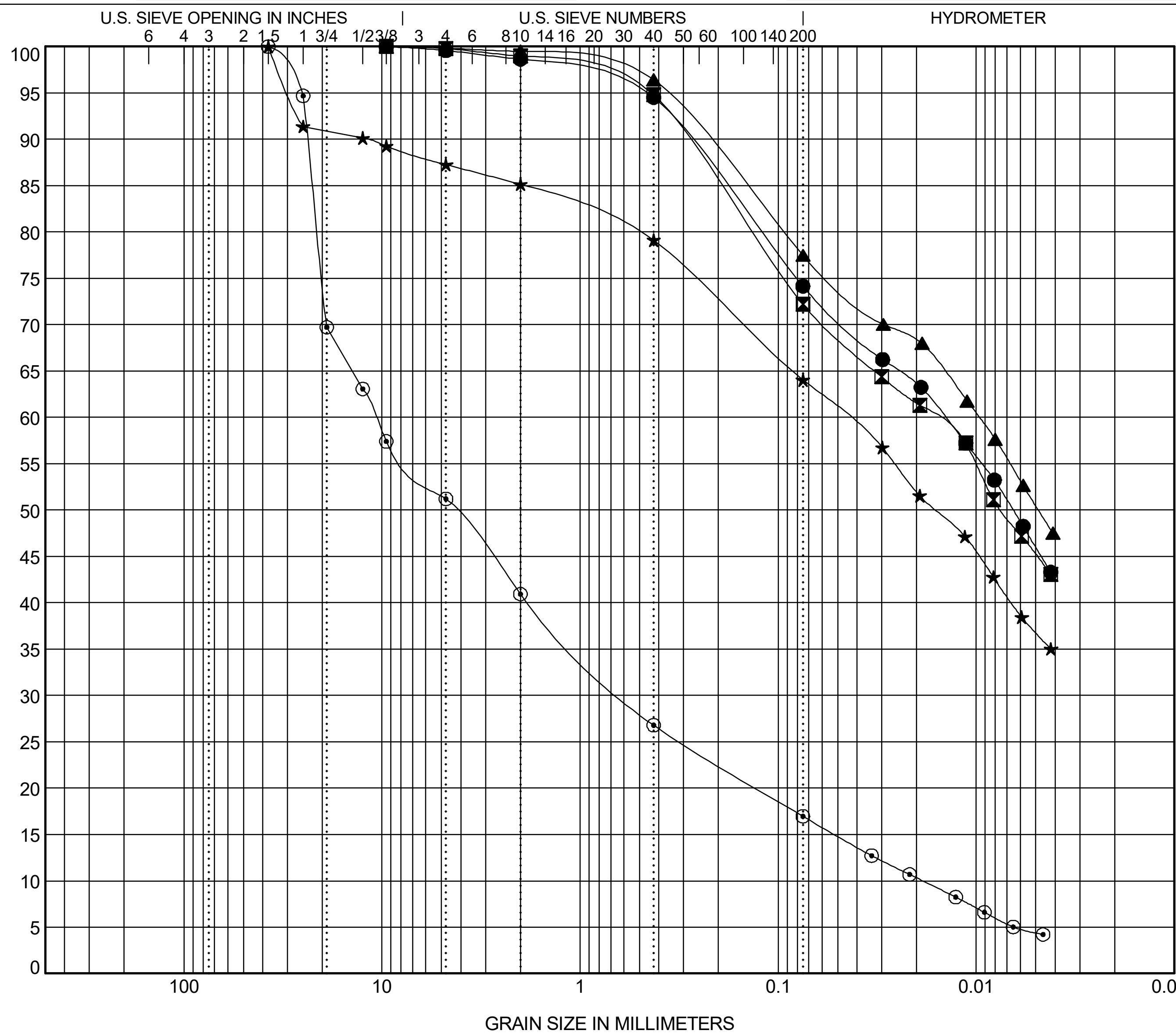
	Specimen Identification		D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
●	**B-001-0-24**	**1.5**	**10.903**	**1.16**	**0.094**		**45**	**16**	**11**	**17**	**11**		
⊠	**B-001-0-24**	**3.0**	**3.001**	**0.018**			**13**	**14**	**12**	**23**	**38**		
▲	**B-001-0-24**	**4.5**	**0.277**				**2**	**5**	**12**	**27**	**54**		
★	**B-001-0-24**	**6.0**	**0.257**	**0.005**			**1**	**5**	**13**	**32**	**49**		
⊙	**B-001-0-24**	**7.5**	**26.615**	**1.069**	**0.044**		**46**	**9**	**11**	**22**	**12**		

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT WOO-281-11.97 **PID** 113085

OGE NUMBER 0 **PROJECT TYPE**



Specimen Identification		ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
● B-002-0-24	1.5	**A-7-6 ~ LEAN CLAY with SAND(CL)**	44	22	22
⧗ B-002-0-24	3.0	**A-7-6 ~ LEAN CLAY with SAND(CL)**	47	21	26
▲ B-002-0-24	4.5	**A-7-6 ~ FAT CLAY with SAND(CH)**	50	21	29
★ B-002-0-24	6.0	**A-7-6 ~ SANDY LEAN CLAY(CL)**	41	20	21
⊙ B-002-0-24	7.5	**A-1-b ~ SILTY GRAVEL with SAND(GM)**	NP	NP	NP

Specimen Identification		D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
● B-002-0-24	1.5	0.289	0.007			2	4	20	28	46		
⧗ B-002-0-24	3.0	0.294	0.007			1	4	23	27	45		
▲ B-002-0-24	4.5	0.236	0.005			1	3	19	27	50		
★ B-002-0-24	6.0	11.973	0.016			15	6	15	27	37		
⊙ B-002-0-24	7.5	23.744	4.289	0.604	0.019	59	14	10	13	4	1.82	577.45

NEAS
National Engineering & Architectural Services Inc.
5710 Westbourne Avenue
Columbus, OH 43213
614-892-0162

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

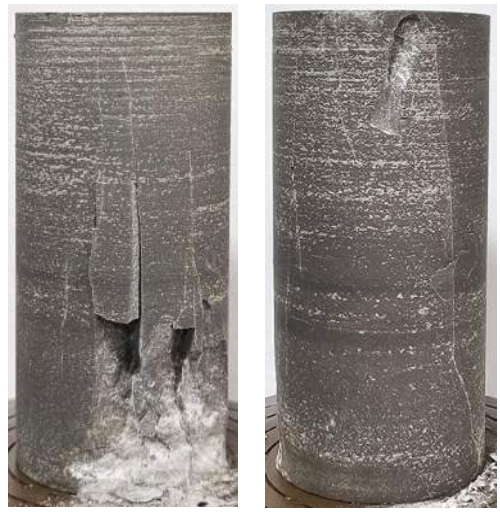
(Project: WOO-281-11.97, Boring Location: B-001-0-24, NQ2-1, Depth: 9.2 - 9.6ft)

Tested Date: 10/15/2024

Specimen Properties

Average Dia., D_{avg} (in):	1.98
Average Height, H_{avg} (in):	4.45
Length to Diameter Ratio:	2.25
Area, A (in^2):	3.08
Volume, V (in^3):	13.71
Wet Mass of Specimen (lb):	1.4
Moisture Content (%):	0.8
Dry Mass of Specimen (lb):	1.4
Wet Unit Weight, γ (lb/ft^3):	172.4
Dry Unit Weight, γ_d (lb/ft^3):	171.0

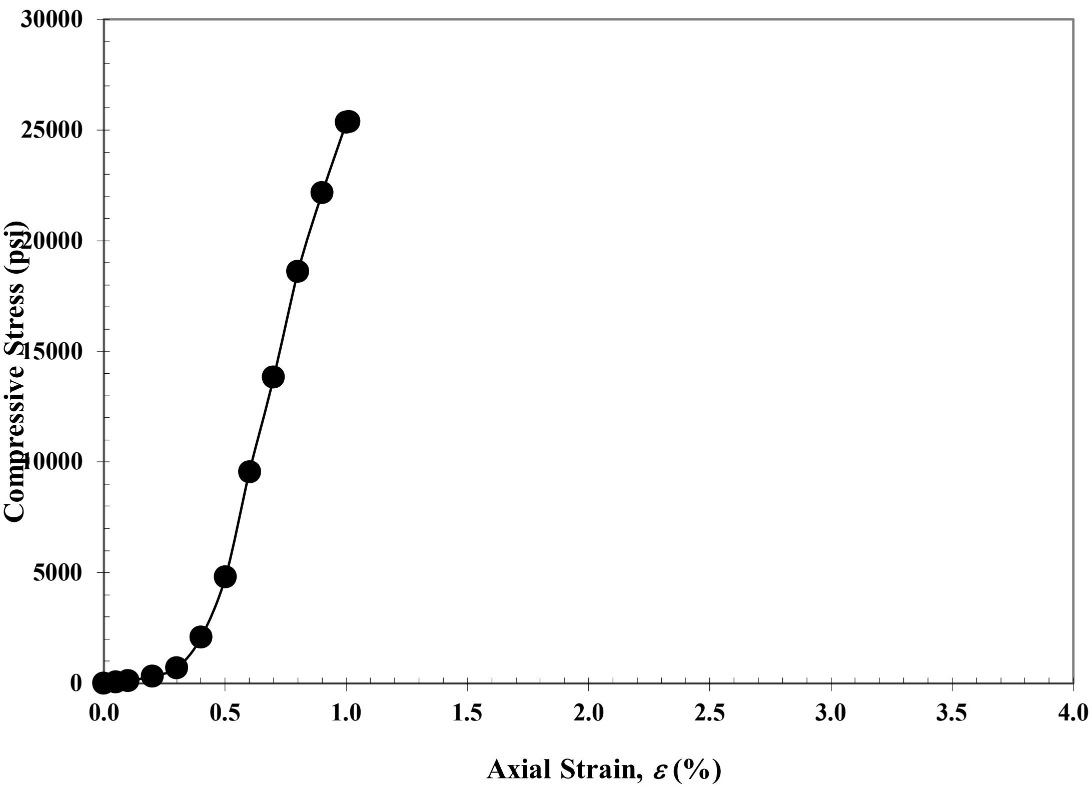
Final Specimen Figure



Results

Unconfined Compressive Strength (psi): **25396** **175** (MPa)

Strain (%): **1.0**



Notes: Dolomite, gray, unweathered, very strong.

5710 Westbourne Avenue
Columbus, OH 43213
614-892-0162

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

(Project: WOO-281-11.97, Boring Location: B-002-0-24, NQ2-1, Depth: 13.5 - 13.9ft)

Tested Date: 10/15/2024

Specimen Properties

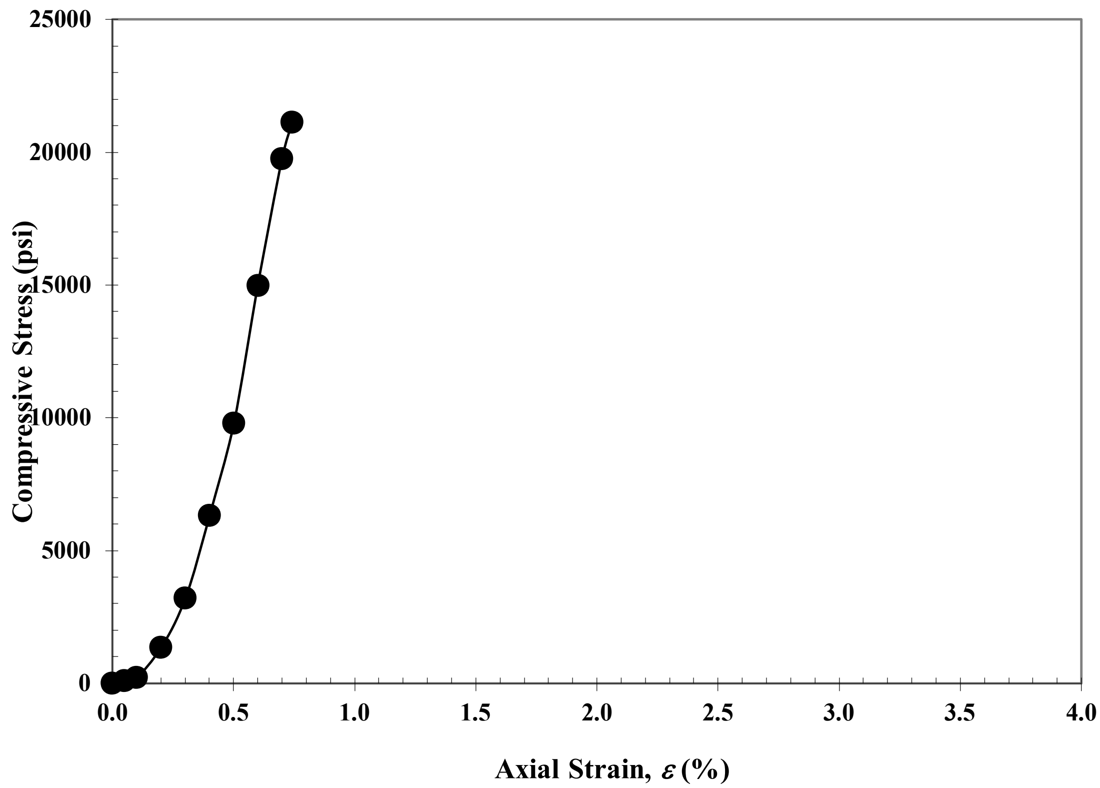
Average Dia., D_{avg} (in):	1.99
Average Height, H_{avg} (in):	4.46
Length to Diameter Ratio:	2.24
Area, A (in^2):	3.11
Volume, V (in^3):	13.87
Wet Mass of Specimen (lb):	1.3
Moisture Content (%):	1.9
Dry Mass of Specimen (lb):	1.3
Wet Unit Weight, γ (lb/ft^3):	168.1
Dry Unit Weight, γ_d (lb/ft^3):	164.9

Final Specimen Figure

Results

Unconfined Compressive Strength (psi): **21117** **146** (MPa)

Strain (%): **0.7**



Notes: Dolomite, gray, unweathered, very strong.

SLAKE DURABILITY TEST
ASTM D4644

National Engineering & Architectural Services Inc.

5710 Westbourne Avenue
Columbus, Ohio 43213
614-892-0162

Tech	PJ	Checked	LR	Report Date:	4/28/2026
County	WOO	Route	281	Section	11.97
Boring Number	B-001-0-24	District	2	PID	113085
Station	63112	Offset	9	Offsest Direction	Right
Latitude	41.28297	Longitude	-83.65312	Ground Elev. (Ft)	683.8
Sample Number	NQ2-1	Top Depth	9.1'	Bottom Depth	12.1'

Description	Dolomite, gray, slightly weathered, very strong, stylolitic, crystalline, contains thin shale laminations.

NATURAL MOISTURE DETERMINATION

Pan ID	Sample Weight (g)	Tare Weight (g)		IN: 4/27/26	OUT: 4/28/26	Moisture Content (%)
shale	505.30	116.06	Time	1:40p	8:10a	
			Mass	621.37	620.55	0.16%

Start Time (mil):	End Time (mil):		First Cycle (I_{d1})					
9:15	9:25		Drum ID	Tare Weight (g)		IN: 4/28/26	OUT: 4/28/26	Final Dry Mass (g)
Start Temp (ºC):	End Temp (ºC):	Avg. Temp (ºC)	A	792.38	Time	9:30a	12:00p	
21.5	21.5	21.5			Mass	1305.59	1294.77	502.39

Start Time (mil):	End Time (mil):		Second Cycle (Id2)					
1:00	1:10		Drum ID	Tare Weight (g)		IN: 4/28/26	OUT: 4/28/26	Final Dry Mass (g)
Start Temp (ºC):	End Temp (ºC):	Avg. Temp (ºC)	A	792.38	Time	1:13p	3:23p	
22.1	22.1	22.1			Mass	1304.31	1293.67	501.29

Before First Cycle

After Second Cycle

Slake Durability Index
$I_{d2}=\{(W_F-C)/(B-C)\}*100$

I_{d2} = **99.4%**

Retained Material
Type: **T1**
(Reference Below)

WF = Drum mass + oven dried specimen after second cycle; B = Drum mass + specimen prior to test; C = Drum mass

From ASTM D4644

T 1 — Retained pieces remain virtually unchanged

T 2 — Retained material consists of large and small pieces

T 3 — Retained material is exclusively small pieces

SLAKE DURABILITY TEST
ASTM D4644

National Engineering & Architectural Services Inc.

5710 Westbourne Avenue
Columbus, Ohio 43213
614-892-0162

Tech	PJ	Checked	LR	Report Date:	4/28/2026
County	WOO	Route	281	Section	11.97
Boring Number	B-002-0-24	District	2	PID	113085
Station	63206	Offset	9	Offsest Direction	Left
Latitude	41.28302	Longitude	-83.65278	Ground Elev. (Ft)	683.4
Sample Number	NQ2-1	Top Depth	9.1'	Bottom Depth	12.3'

Description	Dolomite, gray, slightly weathered, very strong, stylolitic, crystalline, contains thin shale laminations.

NATURAL MOISTURE DETERMINATION

Pan ID	Sample Weight (g)	Tare Weight (g)		IN: 4/27/26	OUT: 4/28/26	Moisture Content (%)
Parker	514.14	243.90	Time	1:55p	8:10a	
			Mass	758.04	757.44	0.12%

Start Time (mil):	End Time (mil):		First Cycle (I_{d1})					
9:15	9:25		Drum ID	Tare Weight (g)		IN: 4/28/26	OUT: 4/28/26	Final Dry Mass (g)
Start Temp (ºC):	End Temp (ºC):	Avg. Temp (ºC)	B	797.28	Time	9:30p	12:00p	
21.5	21.6	21.55			Mass	1321.94	1308.23	510.95

Start Time (mil):	End Time (mil):		Second Cycle (Id2)					
1:00p	1:10p		Drum ID	Tare Weight (g)		IN: 4/28/26	OUT: 4/28/26	Final Dry Mass (g)
Start Temp (ºC):	End Temp (ºC):	Avg. Temp (ºC)	B	797.28	Time	1:13p	3:24p	
22.1	22.3	22.175			Mass	1319.08	1307.05	509.77

Before First Cycle

After Second Cycle

Slake Durability Index
$I_{d2}=\{(W_F-C)/(B-C)\}*100$

I_{d2} = **99.3%**

Retained Material
Type: **T1**
(Reference Below)

WF = Drum mass + oven dried specimen after second cycle; B = Drum mass + specimen prior to test; C = Drum mass

From ASTM D4644

T 1	Retained pieces remain virtually unchanged	T 2	Retained material consists of large and small pieces	T 3	Retained material is exclusively small pieces

Ohio Department of Transportation
Division of Engineering

Office of Geotechnical Engineering

B-001-0-24

Run #:	Depth		Recovery		RQD	
NQ2-1	9.1'	19.1'	115"	96%	91.5"	76%

WOO-281-11.97 PID 113085

Ohio Department of Transportation
Division of Engineering

Office of Geotechnical Engineering

B-002-0-24

Run #:	Depth		Recovery		RQD	
NQ2-1	9.1’	19.1’	114”	95%	91”	76%

WOO-281-11.97 PID 113085

Rock Mass Rating

Job Name: WOO-281-11.97 By: LR Date: 10/15/2024

Boring	Sample	Depth		Cored	Recovery	RQD
B-001-0-24	NQ2-1	9.1'-19.1'		120"	115"	91.5"
					96%	76%

Erodability Index (K) **K=(Ms)(RQD/Jn)(Jr/Ja)(Js)**	Strength (Ms) = 70.0
	Joint Set # (Jn) = 2.24
	Joint Roughness (Jr) = 0.5
	Joint Alteration (Ja) = 1.0
	Joint Orientation (Js) = 0.4
RMR Rating	45 - Class III Fair Rock

Rock Mass Rating

Job Name: WOO-281-11.97 By: LR Date: 10/15/2024

Boring	Sample	Depth		Cored	Recovery	RQD
B-002-0-24	NQ2-1	9.1'-19.1'		120"	114"	91"
					95%	76%
Erodability Index (K) **K=(Ms)(RQD/Jn)(Jr/Ja)(Js)**	Strength (Ms) = 70.0					
	Joint Set # (Jn) = 2.24					
	Joint Roughness (Jr) = 0.5					
	Joint Alteration (Ja) = 1.0					
	Joint Orientation (Js) = 0.4					
RMR Rating	45 - Class III Fair Rock					

Objective: **To estimate depth of rock scour for foundations (shallow foundations/drilled shafts) in rock per direction of ODOT.**

Method: **In accordance with FHWA Publication No. FHWA-HIF-12-003, Hydraulic Engineering Circular No. 18 (HEC-18) and ODOT's BDM Section 305.2.1.2.b**

Erodibility Index (K):

Givens:

$q_u := 25396\ psi$ $\quad q_u = 175.1\ MPa$ Uniaxial Compressive Strength of Rock Core

$RQD := 76$ Rock Quality Designation, Unit: Percentage

$J_n := 2.24$ Rock Joint Set Number (Boring Logs, HEC-18 Table 4.23)

Per ODOT BDM: If Jn, cannot be determined from observation or bore hole data, then assume Jn = 5.

$J_r := 0.5$ Joint Roughness Number (Boring Logs, HEC-18 Table 4.24)

Per ODOT BDM: If Jr, cannot be determined from observation or bore hole data, then assume Jn = 1.

$J_a := 1.0$ Joint Alteration Number (Boring Logs, HEC-18 Table 4.25)

Per ODOT BDM: If Ja, cannot be determined from observation or bore hole data, then assume Jn = 5.

$J_s := 0.4$ Relative Joint Orientation Parameter (Boring Logs, HEC-18 Table 4.26)

Per ODOT BDM: If Js, cannot be determined from observation or bore hole data, then assume Jn = 0.4.

$M_s := 70$ Intact Rock Mass Strength Parameter (ODOT BDM, Sect. 305.2.1.2.b.B.6.b)

Analysis:

$K_b := \frac{RQD}{J_n} = 33.93$ Block Size Parameter (HEC-18, Eq. 4.18)

$K_d := \frac{J_r}{J_a} = 0.5$ Shear Strength Parameter (HEC-18, Eq. 4.19)

$K := M_s \cdot K_b \cdot K_d \cdot J_s = 475$ Erodibility Index (HEC-18, Eq. 4.17)

Approach Flow Stream Power (Pa):

Givens:

$\rho := 1000$ Mass Density of Water (kg/m^3)

Analysis:

$\tau c\_Pa := \rho \cdot \left(\frac{1000 \cdot K^{0.75}}{7.853 \cdot \rho} \right)^{\frac{2}{3}}$ $\tau c\_Pa = 5516.4$ Critical shear stress (Pa)

$\tau c\_psf := \tau c\_Pa \cdot \frac{1}{47.88} \, psf$ $\tau c\_psf = 115.2 \, psf$ Critical shear stress (Psf)

$D_{50\_equivalent} := \tau c\_Pa$ $D_{50\_equivalent} = 5516.4$ Equivalent D50 (mm)

Objective: **To estimate depth of rock scour for foundations (shallow foundations/drilled shafts) in rock per direction of ODOT.**

Method: **In accordance with FHWA Publication No. FHWA-HIF-12-003, Hydraulic Engineering Circular No. 18 (HEC-18) and ODOT's BDM Section 305.2.1.2.b**

Erodibility Index (K):

Givens:

$q_u := 21117\ psi$ $q_u = 145.6\ MPa$ Uniaxial Compressive Strength of Rock Core

$RQD := 76$ Rock Quality Designation, Unit: Percentage

$J_n := 2.24$ Rock Joint Set Number (Boring Logs, HEC-18 Table 4.23)

Per ODOT BDM: If Jn, cannot be determined from observation or bore hole data, then assume Jn = 5.

$J_r := 0.5$ Joint Roughness Number (Boring Logs, HEC-18 Table 4.24)

Per ODOT BDM: If Jr, cannot be determined from observation or bore hole data, then assume Jn = 1.

$J_a := 1.0$ Joint Alteration Number (Boring Logs, HEC-18 Table 4.25)

Per ODOT BDM: If Ja, cannot be determined from observation or bore hole data, then assume Jn = 5.

$J_s := 0.4$ Relative Joint Orientation Parameter (Boring Logs, HEC-18 Table 4.26)

Per ODOT BDM: If Js, cannot be determined from observation or bore hole data, then assume Jn = 0.4.

$M_s := 70$ Intact Rock Mass Strength Parameter (ODOT BDM, Sect. 305.2.1.2.b.B.6.b)

Analysis:

$K_b := \frac{RQD}{J_n} = 33.93$ Block Size Parameter (HEC-18, Eq. 4.18)

$K_d := \frac{J_r}{J_a} = 0.5$ Shear Strength Parameter (HEC-18, Eq. 4.19)

$K := M_s \cdot K_b \cdot K_d \cdot J_s = 475$ Erodibility Index (HEC-18, Eq. 4.17)

Approach Flow Stream Power (Pa):

Givens:

$\rho := 1000$ — Mass Density of Water (kg/m^3)

Analysis:

$\tau c\_Pa := \rho \cdot \left(\frac{1000 \cdot K^{0.75}}{7.853 \cdot \rho} \right)^{\frac{2}{3}}$ — $\tau c\_Pa = 5516.4$ — Critical shear stress (Pa)

$\tau c\_psf := \tau c\_Pa \cdot \frac{1}{47.88}\ psf$ — $\tau c\_psf = 115.2\ psf$ — Critical shear stress (Psf)

$D_{50\_equivalent} := \tau c\_Pa$ — $D_{50\_equivalent} = 5516.4$ — Equivalent D50 (mm)

APPENDIX C

ROADWAY SUNGRADE ANALYSIS

OHIO DEPARTMENT OF TRANSPORTATION

OFFICE OF GEOTECHNICAL ENGINEERING

PLAN SUBGRADES
Geotechnical Design Manual Section 600

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

WOO-281-11.97
PID: 113085

Replacement of the existing bridge carrying SR-281 over Creps Ditch

NEAS,Inc.

Prepared By: Zhao Mankoci
Date prepared: Friday, January 31, 2025

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

NO. OF BORINGS: 2

#	Boring ID	Alignment	Station	Offset	Dir	Drill Rig	ER	Boring EL.	Proposed Subgrade EL	Cut Fill
1	B-001-0-24	SR-281	631+12	9	RT	CME 55TB	89	683.8	682.8	1.0 C
2	B-002-0-24	SR-282	632+06	9	LT	CME 55TB	89	683.4	682.7	0.7 C

Ohio Department of Transportation

Subgrade Analysis

V. 14.7 11/6/2024

#	Boring	Sample	Sample Depth From	Sample Depth To	Subgrade Depth From	Subgrade Depth To	Standard Penetration N_{60}	Standard Penetration N_{60L}	HP (tsf)	LL	PL	PI	% Silt	% Clay	P200	M_C	M_{OPT}	Ohio DOT Class	GI	Sulfate Content (ppm)	Problem Unsuitable	Problem Unstable	Excavate and Replace (Item 204) Unsuitable	Excavate and Replace (Item 204) Unstable	Recommendation (Enter depth in inches)
1	B	SS-1	1.5	3.0	0.5	2.0	12			NP	NP	NP	17	11	28	5	10	A-2-4	0						
	001-0	SS-2	3.0	4.5	2.0	3.5	10		4	46	20	26	23	38	61	19	18	A-7-6	12			N_{60}			
	24	SS-3	4.5	6.0	3.5	5.0	13		4.5	46	23	23	27	54	81	23	20	A-7-6	14						
		SS-4	6.0	7.5	5.0	6.5	7	7	2.5	43	22	21	32	49	81	25	19	A-7-6	13						
2	B	SS-1	1.5	3.0	0.8	2.3	9		2.75	44	22	22	28	46	74	26	19	A-7-6	13			N_{60} & Mc		**12"**	
	002-0	SS-2	3.0	4.5	2.3	3.8	9			47	21	26	27	45	72	23	18	A-7-6	15						
	24	SS-3	4.5	6.0	3.8	5.3	10		2.75	50	21	29	27	50	77	23	18	A-7-6	18						
		SS-4	6.0	7.5	5.3	6.8	19	9	1.75	41	20	21	27	37	64	24	18	A-7-6							

PID:	PID: 113085
County-Route-Section:	WOO-281-11.97
No. of Borings:	2
Geotechnical Consultant:	NEAS,Inc.
Prepared By:	Zhao Mankoci
Date prepared:	1/31/2025

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

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

Design CBR	5

% Samples within 3 feet of subgrade			
$N_{60} \leq 5$	0%	HP ≤ 0.5	0%
$N_{60} < 12$	38%	0.5 < HP ≤ 1	0%
$12 \leq N_{60} < 15$	13%	1 < HP ≤ 2	0%
$N_{60} \geq 20$	0%	HP > 2	25%
M+	13%		
Rock	0%		
Unsuitable Soil	0%		

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

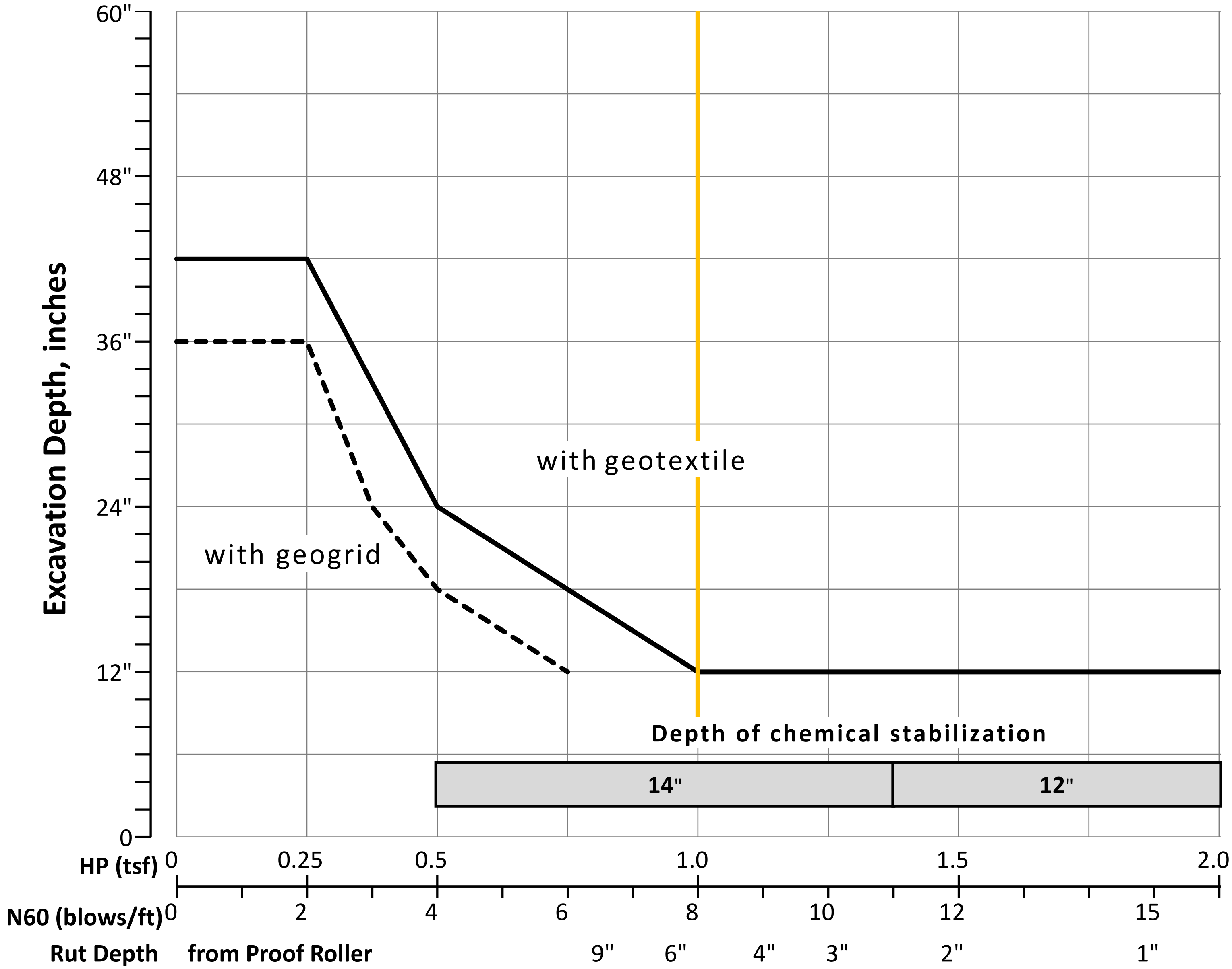
% Proposed Subgrade Surface	
Unstable & Unsuitable	50%
Unstable	50%
Unsuitable (Soil & Rock)	0%

	N_{60}	N_{60L}	HP	LL	PL	PI	Silt	Clay	P 200	M_C	M_{OPT}	GI
Average	11	8	3.04	45	21	24	26	41	67	21	18	12
Maximum	19	9	4.50	50	23	29	32	54	81	26	20	18
Minimum	7	7	1.75	41	20	21	17	11	28	5	10	0

Classification Counts by Sample

ODOT Class	UCF	Rock	A-1-a	A-1-b	A-2-4	A-2-5	A-2-6	A-2-7	A-3	A-3a	A-4a	A-4b	A-5	A-6a	A-6b	A-7-5	A-7-6	A-8a	A-8b	Totals
Count	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	7	0	0	8
Percent	0%	0%	0%	0%	13%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	88%	0%	0%	100%
% Rock|Granular|Cohesive	0%	0%	13%									88%								100%
Surface Class Count	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	3	0	0	4
Surface Class Percent	0%	0%	0%	0%	25%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	75%	0%	0%	100%

Fig. 600-1 – Subgrade Stabilization



OVERRIDE TABLE

Calculated Average	New Values	Check to Override
3.04	0.50	☐ HP
8.00	6.00	☐ N60L

Average HP

Average N_{60L}

APPENDIX D

BEARING RESISTANCE ANALYSIS

Objective: **To evaluate the bearing resistance of shallow foundation on bedrock with RMR system.**
Method: **In accordance with LRFD Bridge Design Specifications, 10th Ed., 2024, [Sect. 10.6.3.2.1 and Sect. 10.6.3.1.2] and LRFD Bridge Design Specifications, 6th Ed., 2012 [Sect. 10.4.6.4 and Sect. 10.8.3.5.4c]. Also, utilizing RMR and soil strength parameters correlations presented in Bieniawski, 1989.**

Estimate RMR:

	PARAMETER		RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>8 MPa	4 to 8 Mpa	2 to 4 MPa	1 to 2 MPa	For this low range – uniaxial compressive test is preferred		
		Uniaxial compressive strength	>200 MPa	100 to 200 MPa	50 to 100 MPa	25 to 50 MPa	10 to 25 MPa	3 to 10 MPa	1 to 3 MPa
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>3 m	1 to 3m	0.3 to 1 m	50 to 300 mm	<50mm		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1mm Hard joint wall rock	Slightly rough surfaces Separation <1mm Soft joint wall rock	Slickensided surfaces or Gouge <5 mm thick or Joints open 1 to 5 mm Continuous joints	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Ground water	Inflow per 10 m tunnel length	None		<25 liters/min	25 to 125 liters/min	>125 liters/min		
		Ratio= joint water pressure/major principal stress	OR 0		OR 0.0 to 0.2	OR 0.2 to 0.5	OR >0.5		
		General Conditions	OR Completely Dry		OR Moist only (interstitial water)	OR Water under moderate pressure	OR Severe water problems		
	Relative Rating		10		7	4	0		

Given:

Rock Strength Parameters (Average Below Footing):

$RMR := 12 + 17 + 20 + 12 + 7 = 68$

Rock mass rating (RMR) determined in accordance with **FHWA GEC 5 Sect. 6.2.3 (See table above).**
Do NOT apply adjustment for joint orientation

$m_i := 7$

Rock strength constant using **GDM Section 1303.3.3.**
Sandstone,breccia, and conglomerate: m_i=15; Claystone (mudstone), shale, clay-shale, and siltstone: m_i=10; Dolomite and limestone: m_i=7; Coal: m_i=1

$q_u := 25396\ psi$

Unconfined compressive strength of rock core.

$\gamma_q := 120\ \frac{lbf}{ft^3}$

Unit weight of soil above bearing depth (Used in Bearing Resistance Calculation **LRFD [Eq 10.6.3.1.2a-1]**

$\gamma_{fd} := 172.4\ \frac{lbf}{ft^3}$

Unit weight of bedrock

Empiracle parameters (undisturbed rock), s and m **(Hoek and Brown, 1988)**:

$s := e^{\left(\frac{RMR - 100}{9}\right)}$ $s = 0.0285655008$

$m := m_i \cdot e^{\left(\frac{RMR - 100}{28}\right)}$ $m = 2.2323459013$

Empiracle rock strength parameters, ϕ and c **(Bieniawski, 1989)**:

$\phi'_{fd} := \left(5 + \frac{RMR}{2}\right) deg$ $\quad$ $c'_{fd} := (0.104 \cdot RMR)\ ksf$

$\phi'_{fd} = 39\ deg$ $\quad$ $c'_{fd} = 7072\ psf$

Footing Geometry:

$D_f := 6\ ft$ — Depth to bottom of footing below proposed ground surface

$B := 8\ ft$ — Footing width

$B' := B$ — Effective base width (assume to match actual base width)

$L' := 56\ ft$ — Effective length of footing (assume to match actual length)

$d_w := 0\ ft$ — Depth of groundwater below proposed ground surface

Bearing Resistance for Weak and Flat Bedded Rock:

Drained Conditions (Effective Stress):

$$N_q := \text{if}\left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan\left(45\ deg + \frac{\phi'_{fd}}{2}\right)^2, 1.0\right) \qquad N_q = 55.96$$

$$N_c := \text{if}\left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14\right) \qquad N_c = 67.87$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd}) \qquad N_\gamma = 92.2$$

Compute shape correction factors per **LRFD [Table 10.6.3.1.2a-3]**:

$$s_c := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L'}\right)\right) \qquad s_c = 1.118$$

$$s_q := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \cdot \tan(\phi'_{fd})\right), 1\right) \qquad s_q = 1.116$$

$$s_\gamma := \text{if}\left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L'}\right), 1\right) \qquad s_\gamma = 0.943$$

Load inclination factors:

$i_q := 1$

$i_\gamma := 1$

$i_c := 1$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**. "Most geotechnical engineers do not used the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

Compute groundwater depth correction factors per **LRFD [Table 10.6.3.1.2a-2]**:

$$C_{wq} := \text{if}(d_w \geq D_f, 1.0, 0.5) \qquad C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w \geq (1.5 \cdot B) + D_f, 1.0, 0.5) \qquad C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \text{atan}\left(\frac{D_f}{B'}\right)$$

$$d_q = 1.14$$

Compute modified bearing capacity factors **LRFD [Equation 10.6.3.1.2a-2 to 10.6.3.1.2a-4]:**

$N_{cm} := N_c \cdot s_c \cdot i_c$ $\quad N_{cm} = 75.861$

$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q$ $\quad N_{qm} = 71.371$

$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma$ $\quad N_{\gamma m} = 86.975$

Compute nominal bearing resistance, **LRFD [Eq 10.6.3.1.2a-1]:**

$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$ $\quad q_{nd} = 592169.7 \, \frac{lbf}{ft^2}$

Bearing Resistance For Strong or 2H:1V Slope Rock:

Compute nominal bearing resistance, **LRFD Article 10.6.3.2.2:**

$q_n := q_u \cdot \left(\sqrt{s} + \sqrt{m \cdot \sqrt{s} + s}\right)$ $\quad q_n = 20471 \, psi$

Compute factored bearing resistance:

$\phi_b := 0.55$

Bearing resistance factor on rock for Retaining Wall and Abutment Footings **LRFD Table 11.5.7-1.**

$$q_R := \text{if}\left(q_u > 7500 \, psi, q_n \cdot \phi_b, \text{if}\left(RMR > 70, q_n \cdot \phi_b, q_{nd} \cdot \phi_b\right)\right)$$

$q_R = 1621.3 \, ksf$

Factored bearing resistance

Objective: **To evaluate the bearing resistance of shallow foundation on bedrock with RMR system.**
Method: **In accordance with LRFD Bridge Design Specifications, 10th Ed., 2024, [Sect. 10.6.3.2.1 and Sect. 10.6.3.1.2] and LRFD Bridge Design Specifications, 6th Ed., 2012 [Sect. 10.4.6.4 and Sect. 10.8.3.5.4c]. Also, utilizing RMR and soil strength parameters correlations presented in Bieniawski, 1989.**

Estimate RMR:

	PARAMETER		RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>8 MPa	4 to 8 Mpa	2 to 4 MPa	1 to 2 MPa	For this low range – uniaxial compressive test is preferred		
		Uniaxial compressive strength	>200 MPa	100 to 200 MPa	50 to 100 MPa	25 to 50 MPa	10 to 25 MPa	3 to 10 MPa	1 to 3 MPa
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>3 m	1 to 3m	0.3 to 1 m	50 to 300 mm	<50mm		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1mm Hard joint wall rock	Slightly rough surfaces Separation <1mm Soft joint wall rock	Slickensided surfaces or Gouge <5 mm thick or Joints open 1 to 5 mm Continuous joints	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Ground water	Inflow per 10 m tunnel length	None	<25 liters/min	25 to 125 liters/min	>125 liters/min			
		Ratio= joint water pressure/major principal stress	OR 0	OR 0.0 to 0.2	OR 0.2 to 0.5	OR >0.5			
		General Conditions	OR Completely Dry	OR Moist only (interstitial water)	OR Water under moderate pressure	OR Severe water problems			
	Relative Rating		10	7	4	0			

Given:

Rock Strength Parameters (Average Below Footing):

$RMR := 12 + 17 + 20 + 12 + 7 = 68$

Rock mass rating (RMR) determined in accordance with **FHWA GEC 5 Sect. 6.2.3 (See table above).**
Do NOT apply adjustment for joint orientation

$m_i := 7$

Rock strength constant using **GDM Section 1303.3.3.**
Sandstone,breccia, and conglomerate: m_i=15; Claystone (mudstone), shale, clay-shale, and siltstone: m_i=10; Dolomite and limestone: m_i=7; Coal: m_i=1

$q_u := 21117\ psi$

Unconfined compressive strength of rock core.

$\gamma_q := 120\ \frac{lbf}{ft^3}$

Unit weight of soil above bearing depth (Used in Bearing Resistance Calculation **LRFD [Eq 10.6.3.1.2a-1]**

$\gamma_{fd} := 168.1\ \frac{lbf}{ft^3}$

Unit weight of bedrock

Empiracle parameters (undisturbed rock), s and m **(Hoek and Brown, 1988)**:

$s := e^{\left(\frac{RMR - 100}{9}\right)}$ $s = 0.0285655008$

$m := m_i \cdot e^{\left(\frac{RMR - 100}{28}\right)}$ $m = 2.2323459013$

Empiracle rock strength parameters, ϕ and c **(Bieniawski, 1989)**:

$$\phi'_{fd} := \left(5 + \frac{RMR}{2}\right) deg \qquad c'_{fd} := (0.104 \cdot RMR)\ ksf$$

$$\phi'_{fd} = 39\ deg \qquad c'_{fd} = 7072\ psf$$

Footing Geometry:

$D_f := 6\ ft$ — Depth to bottom of footing below proposed ground surface

$B := 8\ ft$ — Footing width

$B' := B$ — Effective base width (assume to match actual base width)

$L' := 56\ ft$ — Effective length of footing (assume to match actual length)

$d_w := 0\ ft$ — Depth of groundwater below proposed ground surface

Bearing Resistance for Weak and Flat Bedded Rock:

Drained Conditions (Effective Stress):

$$N_q := \text{if}\left(\phi'_{fd} > 0, e^{\pi \cdot \tan(\phi'_{fd})} \cdot \tan\left(45\ deg + \frac{\phi'_{fd}}{2}\right)^2, 1.0\right) \qquad N_q = 55.96$$

$$N_c := \text{if}\left(\phi'_{fd} > 0, \frac{N_q - 1}{\tan(\phi'_{fd})}, 5.14\right) \qquad N_c = 67.87$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_{fd}) \qquad N_\gamma = 92.2$$

Compute shape correction factors per **LRFD [Table 10.6.3.1.2a-3]**:

$$s_c := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B'}{5 \cdot L'}\right)\right) \qquad s_c = 1.118$$

$$s_q := \text{if}\left(\phi'_{fd} > 0, 1 + \left(\frac{B'}{L'} \cdot \tan(\phi'_{fd})\right), 1\right) \qquad s_q = 1.116$$

$$s_\gamma := \text{if}\left(\phi'_{fd} > 0, 1 - 0.4 \cdot \left(\frac{B'}{L'}\right), 1\right) \qquad s_\gamma = 0.943$$

Load inclination factors:

$i_q := 1$

$i_\gamma := 1$

$i_c := 1$

Assumed to be 1.0, see **LRFD BDS C10.6.3.1.2a**. "Most geotechnical engineers do not used the load inclination factors". If desired, use LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9].

Compute groundwater depth correction factors per **LRFD [Table 10.6.3.1.2a-2]**:

$$C_{wq} := \text{if}(d_w \geq D_f, 1.0, 0.5) \qquad C_{wq} = 0.5$$

$$C_{w\gamma} := \text{if}(d_w \geq (1.5 \cdot B) + D_f, 1.0, 0.5) \qquad C_{w\gamma} = 0.5$$

Depth Correction Factor per Hanson (1970):

$$d_q := 1 + 2 \cdot \tan(\phi'_{fd}) \cdot (1 - \sin(\phi'_{fd}))^2 \cdot \operatorname{atan}\left(\frac{D_f}{B'}\right)$$

$$d_q = 1.14$$

Compute modified bearing capacity factors **LRFD [Equation 10.6.3.1.2a-2 to 10.6.3.1.2a-4]:**

$N_{cm} := N_c \cdot s_c \cdot i_c$ — $N_{cm} = 75.861$

$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q$ — $N_{qm} = 71.371$

$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma$ — $N_{\gamma m} = 86.975$

Compute nominal bearing resistance, **LRFD [Eq 10.6.3.1.2a-1]:**

$q_{nd} := c'_{fd} \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{fd} \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$ — $q_{nd} = 591421.7 \ \frac{lbf}{ft^2}$

Bearing Resistance For Strong or 2H:1V Slope Rock:

Compute nominal bearing resistance, **LRFD Article 10.6.3.2.2:**

$q_n := q_u \cdot \left(\sqrt{s} + \sqrt{\left(m \cdot \sqrt{s} + s\right)}\right)$ — $q_n = 17022 \ psi$

Compute factored bearing resistance:

$\phi_b := 0.55$ — Bearing resistance factor on rock for Retaining Wall and Abutment Footings **LRFD Table 11.5.7-1.**

$q_R := \text{if}\left(q_u > 7500 \ psi, q_n \cdot \phi_b, \text{if}\left(RMR > 70, q_n \cdot \phi_b, q_{nd} \cdot \phi_b\right)\right)$

$q_R = 1348.2 \ ksf$ — Factored bearing resistance