REPORT OF GEOHAZARD EXPLORATION
WAR-350-8.30 (FINAL)

PID: 116664
Warren County, Ohio

January 15, 2026

Prepared for:

Ohio Department of Transportation, District 8
Lebanon, Ohio

Prepared by:
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Project Number: 173410733

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Executive Summary

Embankment erosion has occurred along a section of State Route (SR) 350 at Straight Line Mileage (SLM) 8.30 in Warren County, Ohio. The erosion is causing slope instability along approximately 100 feet of the western embankment of SR 350. This erosion has resulted in undermining of the existing pile and lagging wall with backfill material being lost below the bottom of the precast concrete lagging. A creek (an unnamed tributary of Todd Fork) runs adjacent to SR 350 and is likely the source of the erosion occurring at the toe of the roadway embankment. The project site is located approximately 1.4 miles west of Clarksville, Ohio. The Ohio Department of Transportation (ODOT) plans to protect the affected roadway from the erosion with gabions placed at the toe of the slope along the bank of the creek with backfill placed behind the gabion wall to restore the undermined sections of lagging. Stantec Consulting Services Inc. (Stantec) was contracted by ODOT to perform the design for this project and prepare construction plans for the wall repair.

S&ME advanced one boring along the southbound shoulder of SR 350 and one dynamic cone penetrometer (DCP) sounding at the toe of the retaining wall to obtain geotechnical data for the proposed repair. They were advanced in accordance with the ODOT Specifications for Geotechnical Explorations (SGE).

The surface materials encountered in the boring consisted of 6 inches of asphalt followed by 6 inches of granular base. Below the roadway materials, medium dense to dense granular soil classifying as gravel (A-1-a) were encountered to a depth of 4.0 feet. The remaining soils encountered were predominately fine-grained, classifying as sandy silt (A-4a) and silty clay (A-6a). Groundwater was observed at a depth of 5.5 feet during drilling. Bedrock was encountered at a depth of 15.0 feet in the boring. Bedrock was described as interbedded shale (90%) and limestone (10%). The shale was described as dark gray, highly to moderately weathered, very weak to weak, laminated to very thinly bedded, and calcareous. The limestone was described as bluish gray, slightly weathered, moderately strong, very thin to thin bedded, and fossiliferous. The DCP sounding reached refusal on bedrock at a depth of 6.7 feet.

A gabion wall is being designed to protect the slope from further erosion caused by the creek at the base of the embankment. It is recommended that the gabion wall begin at Station 32+03 and end at Station 33+17, resulting in a wall length of 114 feet. The gabion wall will follow the creek bank during for the alignment; therefore, the wall offset will vary from approximately 52 to 77 feet right of centerline. At this offset it is anticipated rock excavation will be required to meet the bearing depth for the entire length of the wall. The gabion baskets are recommended to be configured with a 9-foot base width, 3-foot top width, and a stepped back face. Either 3 or 4 rows of gabions will be required depending on the slope geometry. The gabion walls should be battered at an angle of 6 degrees away from the creek. Additional design recommendations for the gabion wall are included in this report.

Acronyms / Abbreviations

DCP	Dynamic Cone Penetrometer
ER	Energy Ratio
GDM	Geotechnical Design Manual
ODNR	Ohio Department of Natural Resources
ODOT	Ohio Department of Transportation
KSF	Kips per Square Foot
PCF	Pounds per Cubic Foot
PSF	Pounds per Square Foot
PSI	Pounds per Square Inch
RQD	Rock Quality Designation
SGE	Specifications for Geotechnical Exploration
SLM	Straight Line Mileage
SPT	Standard Penetration Test
SR	State Route
TIMS	Traffic Information Management System
UCR	Unconfined Compression Strength for Rock Core
USDA	United States Department of Agriculture

1 INTRODUCTION

Embankment erosion has occurred along a section of State Route (SR) 350 at Straight Line Mileage (SLM) 8.30 in Warren County, Ohio. The erosion is causing slope instability along approximately 100 feet of the western embankment of SR 350. This erosion has resulted in undermining of the existing pile and lagging wall with backfill material being lost below the bottom of the precast concrete lagging. A creek (an unnamed tributary of Todd Fork) runs adjacent to SR 350 and is likely the source of erosion occurring at the toe of the embankment. The project site is located approximately 1.4 miles west of Clarksville, Ohio.

The Ohio Department of Transportation (ODOT) plans to protect the affected roadway from the erosion with gabions placed along the creek bank with backfill placed behind the gabion wall to restore the undermined sections of lagging. Stantec Consulting Services Inc. (Stantec) was contracted by ODOT to perform the design for this project and prepare construction plans for the wall repair. Figure 1 shows the site vicinity.

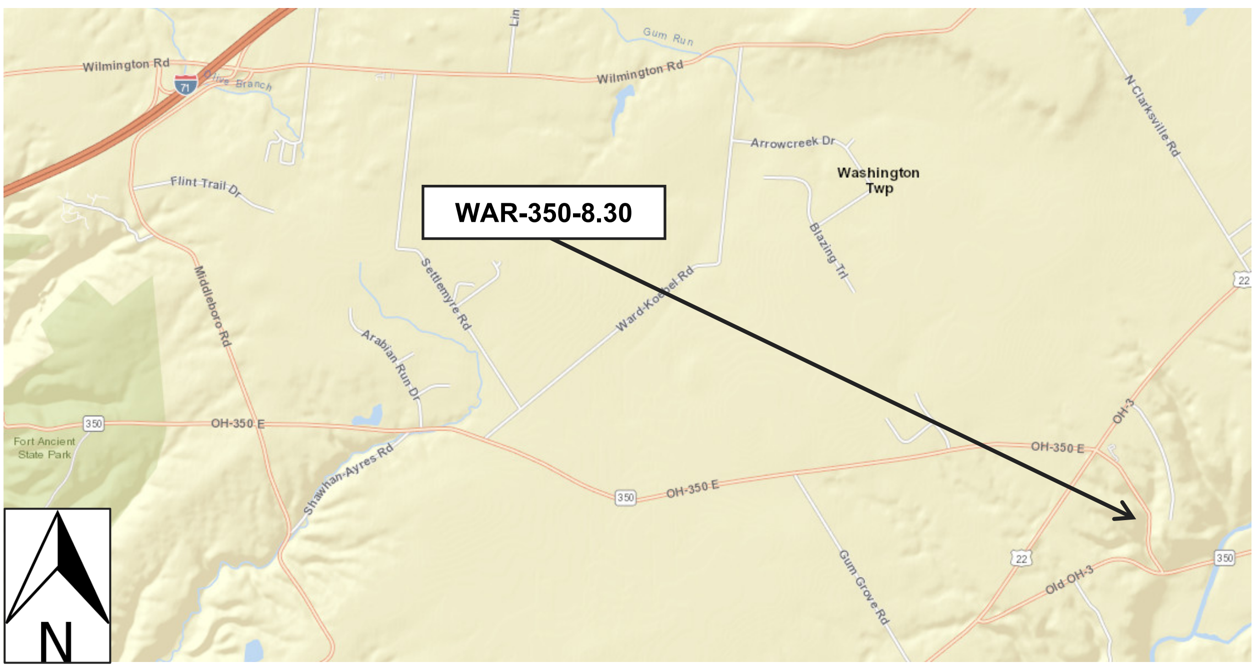


Figure 1: Site Vicinity
(ODOT Transportation Information Mapping System [TIMS] Interactive Mapping, 2024)

2 GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1 GENERAL

The *Physiographic Regions of Ohio Map* (Ohio Department of Natural Resources [ODNR], 1998) indicates that the project is located within the Illinoian Till Plain. This region is described as a rolling ground moraine of older till generally lacking ice-constructional features such as moraines, kames, and eskers. The geology of the Illinoian Till Plain contains silt-loam, high-lime, Illinoian-age till with a loess cap over Ordovician- and Silurian-age carbonate rocks and calcareous shales. Moderately low relief of approximately 50 feet can be observed in the region, with elevations ranging from 600 feet to 1,100 feet.

2.2 SOIL GEOLOGY

According to the *Ohio Geology Interactive Map* (ODNR, 2024), the project site is underlain by Illinoian-age ground moraine. The *Ohio Geology Interactive Map* also suggests a glacial drift thickness ranging between 15 and 50 feet. The soil survey (*Web Soil Survey of Warren County, Ohio*, United States Department of Agriculture [USDA], 2024) indicates that most surficial soils at the project site are from the Hickory silt loam complex. These soils primarily consist of silt loam, loam, clay loam, and gravelly loam of various thicknesses. These soils are typically well-drained with a moderately high to high capacity of transmitting water.

2.3 BEDROCK GEOLOGY

Bedrock mapping (*Ohio Geology Interactive Map*, ODNR, 2024) and *Descriptions of Geologic Map Units* (ODNR, 2011) indicates that the overburden soils at the project site are underlain primarily by sedimentary bedrock of the Waynesville Formation from the Ordovician group. Bedrock from the Waynesville Formation is comprised of interbedded shale (70%) and limestone (30%). The bedrock is described as shades of gray to bluish gray and weathering light gray, with unit thicknesses between 90 to 120 feet.

According to the *Ohio Mine Locator* (ODNR, 2024), there are two active surface gravel and sand mines within a 5-mile radius of the project footprint. A few surface aggregate mine affected areas are located approximately 5 miles southwest of the project. The *Karst Interactive Map* (ODNR, 2024) indicates there are over 20 field verified karst features within 1 mile of the immediate project vicinity. The closest feature, located approximately 0.4 miles southwest of the project location, is described as a culvert into a sinkhole. The *Ohio Oil and Gas Well Map* (ODNR, 2024) shows no wells within a 10-mile radius of the project.

2.4 HYDROLOGY AND HYDROGEOLOGY

An unnamed creek is located to the west of the project site and flows to the south into Todd Fork which flows southwest into the Little Miami River near Morrow, Ohio. The Little Miami River then flows south to the Ohio River near Fort Thomas, Kentucky.

The *Ohio Geology Interactive Map* shows that the site is underlain by the Fort Ancient Thin Upland Aquifer, which has a yield less than 5 gallons per minute and a thickness less than 25 feet. Bedrock wells in the area typically yield less than 5 gallons of water per minute with a thickness of over 100 feet. A search was performed using the ODNR *Ohio Water Wells Map* (2024) to determine if any water wells are located near the project site. According to the map, five water wells have been drilled within a 0.5-mile radius of the project footprint. The well logs indicate a bedrock depth ranging from 4 to 72 feet. The bedrock encountered at these wells were described as shale. The logs also indicate a considerable variation of the static water depth in the area surrounding the site, ranging from 2 to 47 feet.

2.5 SEISMIC

A review of the seismic data available in the project vicinity was completed using the ODNR *Ohio Earthquake Epicenters Map* (2024). Overall, Ohio has a relatively limited amount of seismic activity. Within a 10-mile radius of the project, there have been no recorded earthquake epicenters. The available data reviewed included events that occurred in Ohio from 1804 to present day.

2.6 SITE RECONNAISSANCE

ODOT and Stantec representatives visited the site on August 1, 2024. The land surrounding the project site can be described as rural and wooded with some residential areas located to the east. The pavement was observed to be generally in fair condition. The existing soldier pile lagging retaining wall was observed to be in good condition. The loss of wall backfill material from below the precast concrete lagging was observed over a length of about 100 feet. The loss of the backfill was causing settlement along the edge of the pavement. The backfill loss was likely due to embankment erosion caused by the creek undermining the lagging of the wall. Gabions along the creek bank with replaced backfill were discussed as the most likely repair method. The bank of the creek was well vegetated. Stantec and S&ME representative later visited the site on September 18, 2024 to mark boring locations and evaluate site access.

3 EXPLORATION

3.1 HISTORIC EXPLORATION PROGRAMS

Stantec was provided with the soil profile drawings completed prior to the construction of the retaining wall. Fourteen borings were performed in 2002 during the geotechnical exploration for the retaining wall. Cohesive soils were described as sandy silt (A-4a), silt and clay (A-6a), silty clay (A-6b), and clay (A-7-6).

Lesser amounts of granular soils were described as gravel (A-1-a) and coarse and fine sand (A-3a). Groundwater was encountered at a depth of 13.5 feet in one boring. Bedrock was generally encountered at depths ranging from 13.5 to 18.5 feet. Bedrock was described as interbedded shale (80%) and limestone (20%).

3.2 PROJECT EXPLORATION PROGRAM

S&ME advanced one conventional boring and one dynamic cone penetrometer (DCP) sounding near the center of the observed erosion and loss of wall backfill on September 25, 2024. The soil boring was advanced along the southbound shoulder of the road while the DCP boring was advanced at the toe of the retaining wall west of SR 350. A summary of these borings is shown in Table 1. Boring locations are shown on the geotechnical profile in Appendix A. The locations and elevations of the borings were surveyed by Stantec. Boring logs for the project were initially completed by S&ME and updated by Stantec for the geotechnical profile drawings (Appendix A). The S&ME Geotechnical Data Report is provided as Appendix B and includes the original S&ME boring logs.

Table 1. Boring Summary

Boring No.	Station (feet)	Offset (feet)	Ground Surface Elevation (feet)	Top of Bedrock Elevation (feet)	Bottom of Boring Elevation (feet)
B-001-0-24	32+50	11 Rt.	865.4	850.4	833.2
D-001-0-24	32+51	34 Rt.	849.8	843.5	843.5

The soil boring was advanced in accordance with the ODOT Specifications for Geotechnical Explorations (SGE). The boring was performed with a CME 550X track-mounted drill rig using 3¼-inch inside diameter (ID) hollow stem augers to advance through soil. Standard Penetration Test (SPT) sampling was performed continuously until the bedrock was encountered. The energy ratio (ER) of the drill rig automatic hammer and drill rod system were measured to be 85.9 percent on August 22, 2024. The depths and elevations of the SPTs with the corresponding N_{60}-values are shown on the boring log in Appendix A. One undisturbed Shelby tube (ST) sample was obtained at a depth from 7.0 to 9.0 feet.

Upon encountering fairly competent bedrock, approximately 15 feet of rock coring was performed using NQ2-size equipment. Recovery, core loss, and rock quality designation (RQD) values were recorded as percentages for each coring run. These values are shown on the boring log contained in Appendix A.

The materials encountered were logged by a Stantec engineer, with attention given to soil type, consistency, and moisture content. The boring was checked for the presence of groundwater during drilling and at its conclusion with the depth of water recorded. The boring was sealed with bentonite grout and capped with asphalt cold patch.

The soil samples obtained from the boring were returned to S&ME's geotechnical laboratory for visual classification and tested for water content. Engineering classification testing was performed on samples reflecting each of the main soil horizons. The engineering classification tests conducted on the samples were sieve and hydrometer analysis (ASTM D 422) and Atterberg limits (ASTM D 4318). The samples were classified according to the ODOT classification method. One loss on ignition test (ASTM D7348) was completed on a recovered soil sample. Results from classification and moisture content testing are shown on the boring log in Appendix A.

The Shelby tube was extruded in the laboratory and one unconfined compressive strength of soil (UCS) test (ASTM D2166) was completed on the extruded sample. Two rock core samples were subjected to unconfined compressive strength testing according to ASTM D7012, Method C. The results of available laboratory testing are included in Appendix A and Appendix B.

S&ME performed one DCP sounding using a Kessler K-100 series probe downslope of the retaining wall between the creek and the retaining wall. The DCP sounding (designated as D-001-1-24) was performed by raising a 17.6-pound weight and measuring the penetration obtained with 10 blows. The DCP sounding was advanced until refusal. The DCP sounding log is provided in Appendix A. The hole was backfilled with cuttings upon completion of testing.

4 FINDINGS

The surface materials encountered in the boring consisted of 6 inches of asphalt followed by 6 inches of granular base. Below the roadway materials, medium dense to dense granular soil classifying as gravel (A-1-a) were encountered to a depth of 4.0 feet. The remaining soils encountered were predominately fine-grained, classifying as sandy silt (A-4a) and silty clay (A-6a).The fine-grained soils were described as medium stiff to hard (N_{60} values ranging from 6 to 70 with an average of 28), brown to grayish brown, moist (natural moisture contents ranging from 7 to 21 percent with an average of 17 percent), and slightly to moderately plastic (plasticity indices ranging from 6 to 23 with an average of 14). One Shelby tube advanced in silty clay resulted in an unconfined compressive strength of 0.7 kips per square foot (ksf). Groundwater was observed at a depth of 5.5 feet during drilling.

Bedrock was encountered at a depth of 15.0 feet in the boring. Bedrock was described as interbedded shale (90%) and limestone (10%). The shale was described as dark gray, highly to moderately weathered, very weak to weak, laminated to very thinly bedded, and calcareous. The limestone was described as bluish gray, slightly weathered, moderately strong, very thin to thin bedded, and fossiliferous. Approximately 15.5 feet of bedrock was cored in the boring prior to termination. Core recoveries ranged from 98 to 100 percent and RQD values varied from 24 to 72. Unconfined compressive testing was completed on two rock core samples, resulting in compressive strengths of 529 and 1,706 pounds per square inch (psi). The DCP sounding reached refusal on bedrock at a depth of 6.7 feet.

Boring log, DCP test data, photographs of the rock core, and laboratory testing results are presented in Appendix A and Appendix B.

5 ANALYSIS AND RECOMMENDATIONS

5.1 GENERAL

The recommendations that follow are based on the information discussed in this report and the interpretation of the subsurface conditions encountered at the site during our fieldwork. If future design changes are made, Stantec should be notified so that such changes can be reviewed, and the recommendations amended as necessary.

These conclusions and recommendations are based on data and subsurface conditions from the boring and sounding advanced during this exploration using the degree of care and skill ordinarily exercised under similar circumstances by competent members of the engineering profession. No warranties can be made regarding the continuity of conditions.

5.2 SLOPE STABILITY BACK ANALYSIS

Embankment erosion has occurred along the toe of a section of SR 350 at SLM 8.30 in Warren County, Ohio. The erosion is causing slope instability along approximately 100 feet of the western embankment of SR 350. This erosion has resulted in undermining of the existing pile and lagging wall and backfill material spilling below the precast concrete lagging. A creek (unnamed tributary of Todd Fork) runs adjacent to SR 350 and is likely the source of the erosion occurring on the embankment.

To model existing slope conditions at the toe, a cross-section was developed based on field observations and information from the boring. A cross-section through B-001-0-24 and D-001-0-24 was selected as representative of the affected alignment. Parameters for each soil layer were chosen to represent existing conditions, including residual shear strengths in soils. The material parameters were chosen based upon correlation of the SPT N_{60}-values recorded on the boring log and published correlations where applicable as well as laboratory testing results. Calculations performed to obtain initial parameters are provided in Appendix C.

To determine the approximate shear strength along the failure surface of the erosion, a back-analysis was performed using conventional, limit equilibrium methods as implemented in GeoStudio SLOPE/W 2019 software. A simplified subsurface stratigraphy which included granular roadway fill, cohesive foundation soils, and bedrock was modeled based on the soil and rock encountered in the boring. It was assumed that the failure surface passes along the interface of cohesive soil and bedrock.

The foundation soils were initially given shear strength parameters based on the results of the field and laboratory testing and experience with similar materials on other projects. These strength parameters were then adjusted to achieve a safety factor of 1.0, simulating failure. A unit weight of 123 pounds per cubic foot (pcf) and an angle of internal friction of 33 degrees were estimated for the granular roadway fill. For the cohesive soils, a unit weight of 126 pcf, a drained cohesion of 70 pounds per square foot (psf), and an internal angle of friction of 25 degrees were estimated. Results of the back analysis are shown in Appendix C.

5.3 GABION WALL

A gabion wall is planned to protect the slope from further erosion caused by the creek at the base of the embankment. It is recommended that the gabion wall begin at Station 32+03 and end at Station 33+17, resulting in a wall length of 114 feet. The gabion wall will follow the creek bank during for the alignment; therefore, the wall offset will vary from approximately 52 to 77 feet right of centerline. At this offset it is anticipated rock excavation will be required to meet the bearing depth for the entire wall. The gabion baskets are recommended to be configured with a 9-foot base width, 3-foot top width, and a stepped back face. Either 3 or 4 rows of gabions (each 3 feet tall) stacked in 3 columns will be required depending on the slope geometry. The gabion walls should be battered at an angle of 6 degrees away from the creek. Below the gabions, a 6-inch leveling pad is recommended using Item 203 Granular Material, Type C according to the ODOT Geotechnical Design Manual (GDM). Figure 2 shows the general layouts for each gabion wall, including the leveling pad. Excavations into bedrock may be done at a vertical slope, but excavations in soil should be done at a one to one (horizontal to vertical) slope. Table 2 provides the resistance checks for the gabion wall at the top and bottom of the leveling pad, and detailed calculations are provided in Appendix D.

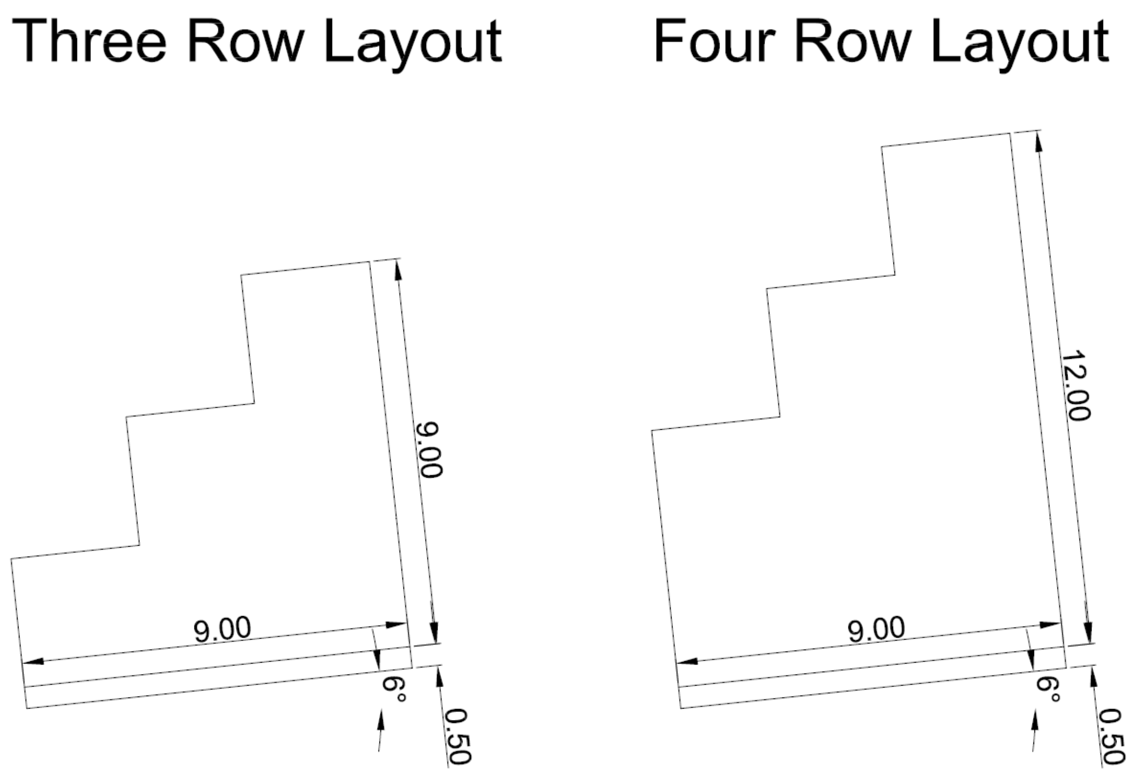


Figure 2: Gabion Wall Layouts

Table 2. Gabion Wall Resistance Checks

Analysis Location	Required Bearing Resistance (ksf)	Factored Bearing Resistance (ksf)	Sliding Force (ksf)	Sliding Resistance (ksf)	Maximum Design Eccentricity (ft)	Maximum Allowable Eccentricity (ft)
Three Rows: Top of Leveling Pad	1.5	7.5	3.4	5.2	1.16	2.26
Three Rows: Bottom of Leveling Pad	1.7	5.9	3.9	4.9	1.32	2.26
Four Rows: Top of Leveling Pad	2.0	7.6	4.1	6.9	1.10	2.26
Four Rows: Bottom of Leveling Pad	2.2	6.0	4.6	6.6	1.25	2.26

According to Section 1503.4 of the ODOT GDM, the back face of the gabion wall is to be covered with Geotextile Fabric, Type A to act as a filter fabric between the gabion wall and retained soil. It is recommended that concrete be used as backfill within the rock excavation. A two to one (horizontal to vertical) final slope of embankment fill is recommended from top of the gabion wall to the front of the existing retaining wall to prevent further undermining of the roadway. Embankment compaction requirements should confirm to Item 203 of the ODOT Construction and Material Specifications.

5.4 GLOBAL STABILITY

A global stability analysis was performed with the gabion wall and backfilling described above in place. Concrete backfill was given minimum properties found in research, and fill material was given properties of an unknown borrow source provided in Table 500-2 of the ODOT GDM. The resulting factor of safety for a failure between the existing retaining wall and gabion wall was 1.7. A second global stability analysis was completed assuming the failure surface can pass through the gabion wall, resulting in a factor of safety of 1.6. The analysis cross section is shown in Appendix E. Typically, a minimum factor of safety of 1.3 is suitable for roadway applications according to Section 502 of the ODOT GDM. It was assumed that a failure surface will not pass through the existing retaining wall in both global stability analyses.

APPENDIX A
GEOTECHNICAL PROFILE DRAWINGS

PROJECT DESCRIPTION

THIS PROJECT, WAR-350-8.30, IS THE EXPLORATION FOR THE UNDERMINING OF A RETAINING WALL LOCATED ALONG SR 350 IN WARREN COUNTY.

HISTORIC RECORDS

STANTEC WAS PROVIDED THE SOIL PROFILE DRAWINGS COMPLETED DURING THE GEOTECHNICAL EXPLORATION COMPLETED PRIOR TO THE CONSTRUCTION OF THE RETAINING WALL. FOURTEEN BORINGS WERE COMPLETED IN 2002 FOR THE RETAINING WALL. COHESIVE SOILS WERE DESCRIBED AS SANDY SILT (A-4A), SILT AND CLAY (A-6A), SILTY CLAY (A-6B), AND CLAY (A-7-6). LESSER AMOUNTS OF GRANULAR SOILS WERE DESCRIBED AS GRAVEL (A-1-A) AND COARSE AND FINE SAND (A-3A). GROUNDWATER WAS ENCOUNTERED AT A DEPTH OF 13.5 FEET IN ONE BORING. BEDROCK WAS GENERALLY ENCOUNTERED AT DEPTHS RANGING FROM 13.5 TO 18.5 FEET. BEDROCK WAS DESCRIBED AS INTERBEDDED LAYERS OF SHALE (80%) AND LIMESTONE (20%).

GEOLOGY

THE PROJECT SITE IS LOCATED WITHIN THE ILLINOIAN TILL PLAIN PHYSIOGRAPHIC REGION. THIS REGION IS DESCRIBED AS A ROLLING GROUND MORAINE OF OLDER TILL GENERALLY LACKING ICE-CONSTRUCTIONAL FEATURES SUCH AS MORAINES, KAMES, AND ESKERS. THE GEOLOGY OF THE ILLINOIAN TILL PLAIN CONTAINS SILT-LOAM, HIGH-LIME, ILLINOIAN-AGE TILL WITH LOESS CAP OVER ORDOVICIAN- AND SILURIAN-AGE CARBONATE ROCKS AND CALCAREOUS SHALES. MODERATELY LOW RELIEF OF APPROXIMATELY 50 FEET CAN BE OBSERVED IN THE REGIONS, WITH ELEVATIONS RANGING FROM 600 FEET TO 1,100 FEET. OVERBURDEN SOILS AT THE PROJECT SITE ARE UNDERLAIN PRIMARILY BY SEDIMENTARY BEDROCK FROM THE WAYNESVILLE FORMATION FROM THE ORDOVICIAN GROUP. BEDROCK FROM THE WAYNESVILLE FORMATION IS COMPRISED OF INTERBEDDED SHALE (70%) AND LIMESTONE (30%). THE BEDROCK IS DESCRIBED AS SHADES OF GRAY TO BLUISH GRAY AND WEATHERING LIGHT GRAY, WITH UNIT THICKNESSES BETWEEN 90 TO 120 FEET.

RECONNAISSANCE

ODOT AND STANTEC REPRESENTATIVES VISITED THE SITE ON AUGUST 1, 2024. THE LAND SURROUNDING THE PROJECT SITE CAN BE DESCRIBED AS RESIDENTIAL. THE PAVEMENT WAS OBSERVED TO BE GENERALLY IN FAIR CONDITION. THE RETAINING WALL AT THE SITE WAS OBSERVED TO BE IN GOOD CONDITION. AN UNNAMED TRIBUTARY TO TODD FORK WAS OBSERVED AT THE TOE OF SLOPE BELOW THE WALL.THE LOSS OF WALL BACKFILL MATERIAL FROM UNDERNEATH THE WALL LAGGING WAS OBSERVED OVER ABOUT 100 FEET. GABIONS ALONG THE TRIBUTARY BANK WITH BACKFILL TO REPLACE ERODED MATERIAL WERE DISCUSSED AS THE MOST LIKELY REPAIR METHOD. THE BANK OF THE CREEK WAS WELL VEGETATED. STANTEC AND S&ME REPRESENTATIVES VISITED THE SITE ON SEPTEMBER 18, 2024 TO MARK BORING LOCATIONS AND EVALUATE SITE ACCESS.

SUBSURFACE EXPLORATION

S&ME ADVANCED ONE SOIL BORING ALONG THE PROJECT ALIGNMENT ON SEPTEMBER 25, 2024. THE SOIL BORING WAS ADVANCED ALONG THE SOUTHBOUND SHOULDER OF SR.350. THE BORING WAS DRILLED WITH A CME 550X TRACK MOUNTED DRILL RIG USING 3.25-INCH I.D. HOLLOW-STEM AUGERS. DISTURBED SOIL SAMPLES WERE OBTAINED IN ACCORDANCE WITH THE STANDARD PENETRATION TEST (AASHTO T206) AT CONTINUOUS OR 2.5-FOOT INTERVALS. THE AUTOMATIC SAMPLING EQUIPMENT WAS CALIBRATED ON AUGUST 22, 2024 AND HAS A DRILL ROD ENERGY RATIO (ER) OF 85.9 PERCENT. ROCK CORING WAS PERFORMED IN THE BORING USING NQ2-SIZE CORING EQUIPMENT.

S&ME PERFORMED ONE DYNAMIC CONE PENETROMETER (DCP) TEST USING A KESSLER K-100 SERIES PROBE DOWNSLOPE OF THE RETAINING WALL BETWEEN THE CREEK AND THE RETAINING WALL. THE DCP SOUNDING (DESIGNATED AS D-001-1-24) WAS PERFORMED BY RAISING A 17.6-POUND WEIGHT AND MEASURING THE PENETRATION OBTAINED WITH 10 BLOWS. THE DCP WAS ADVANCED UNTIL REFUSAL.

EXPLORATION FINDINGS

THE SURFACE MATERIALS ENCOUNTERED IN THE BORING CONSISTED OF 6 INCHES OF ASPHALT FOLLOWED BY 6 INCHES OF BASE. BELOW THE ROADWAY MATERIALS, MEDIUM DENSE TO DENSE GRANULAR SOIL CLASSIFYING AS GRAVEL (A-1-A) WERE ENCOUNTERED TO A DEPTH OF 4.0 FEET. THE REMAINING SOILS ENCOUNTERED WERE PREDOMINATELY FINE-GRAINED, CLASSIFYING AS SANDY SILT (A-4A) AND SILTY CLAY (A-6A). THE FINE-GRAINED SOILS WERE DESCRIBED AS MEDIUM STIFF TO HARD, BROWN TO GRAYISH BROWN TOWARDS BEDROCK, MOIST, AND SLIGHTLY TO MODERATELY PLASTIC. GROUNDWATER WAS OBSERVED AT A DEPTH OF 5.5 FEET DURING DRILLING.

BEDROCK WAS ENCOUNTERED AT A DEPTH OF 15.0 FEET IN THE BORING. BEDROCK WAS DESCRIBED AS INTERBEDDED SHALE (90%) AND LIMESTONE (10%). THE SHALE WAS DESCRIBED AS DARK GRAY, HIGHLY TO MODERATELY WEATHERED, VERY WEAK TO WEAK, LAMINATED TO VERY THINLY BEDDED, AND CALCAREOUS. THE LIMESTONE WAS DESCRIBED AS BLUISH GRAY, SLIGHTLY WEATHERED, MODERATELY STRONG, VERY THIN TO THIN BEDDED, AND FOSSILIFEROUS.

SPECIFICATIONS

THIS GEOTECHNICAL EXPLORATION WAS PERFORMED IN ACCORDANCE WITH THE STATE OF OHIO, DEPARTMENT OF TRANSPORTATION, OFFICE OF GEOTECHNICAL ENGINEERING, SPECIFICATIONS FOR GEOTECHNICAL EXPLORATIONS, DATED JULY 2024.

AVAILABLE INFORMATION

THE SOIL, BEDROCK, AND GROUNDWATER INFORMATION COLLECTED FOR THIS SUBSURFACE EXPLORATION THAT CAN BE CONVENIENTLY DISPLAYED ON THE SOIL PROFILE SHEETS HAS BEEN PRESENTED. GEOTECHNICAL REPORTS, IF PREPARED, ARE AVAILABLE FOR REVIEW ON THE OFFICE OF CONTRACT SALES WEBSITE.

LEGEND

DESCRIPTION	ODOT CLASS	CLASSIFIED MECH.	CLASSIFIED VISUAL
GRAVEL	A-3a	1	1
SANDY SILT	A-4a	1	1
SILTY CLAY	A-6b	3	2
	TOTAL	5	4
SHALE	VISUAL		
INTERBEDDED SHALE AND LIMESTONE	VISUAL		
PAVEMENT OR BASE = X = APPROXIMATE THICKNESS	VISUAL		

PROJECT BORING LOCATION - PLAN VIEW.

DDCP SOUNDING PLOTTED TO VERTICAL SCALE ONLY.

DDCP BORING LOCATION - PLAN VIEW.

DRIVE SAMPLE AND/OR ROCK CORE BORING PLOTTED TO VERTICAL SCALE ONLY. HORIZONTAL BAR INDICATES A CHANGE IN STRATIGRAPHY.

WC INDICATES WATER CONTENT IN PERCENT.

N_{60} INDICATES STANDARD PENETRATION RESISTANCE NORMALIZED TO 60% DRILL ROD ENERGY RATIO.

X/Y/Z NUMBER OF BLOWS FOR STANDARD PENETRATION TEST (SPT):
X= NUMBER OF BLOWS FOR FIRST 6 INCHES.
Y= NUMBER OF BLOWS FOR SECOND 6 INCHES.
Z= NUMBER OF BLOWS FOR THIRD 6 INCHES.

W INDICATES FREE WATER ELEVATION.

TR INDICATES TOP OF ROCK.

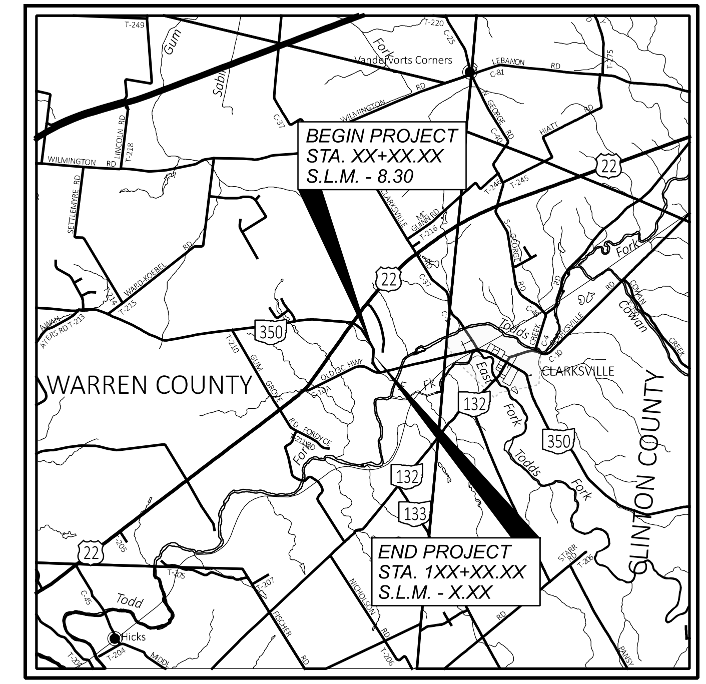
SS INDICATES A SPLIT SPOON SAMPLE.

ST INDICATES A SHELBY TUBE SAMPLE.

UCS UNCONFINED COMPRESSIVE STRENGTH (SOIL) SHOWN IN (KSF).

Qu INDICATES UNCONFINED COMPRESSIVE TEST (ROCK), ASTM D7012.

LOI UNCONFINED ORGANIC CONTENT BY LOSS ON IGNITION, AASHTO T267.



LOCATION MAP

SCALE IN MILES

0 1 2 3 4

N

PARTICLE SIZE DEFINITIONS

12"		3"	2.0 mm	0.42 mm	0.074 mm	0.005 mm
BOULDERS	COBBLES	GRAVEL	COARSE SAND	FINE SAND	SILT	CLAY
		No. 10 SIEVE	No. 40 SIEVE	No. 200 SIEVE		

RECON. - EK & MS/ JS & SK 08/01/2024/ 09/18/2024

DRILLING - BK & JS 09/25/2024

DRAWN - MJ 12/2024 -01/2025

REVIEWED - EMK 01/27/2025

BEDROCK TEST SUMMARY				
EXPLOR. ID	SAMPLE ELEV. (FT.)	SAMPLE DEPTH (FT.)	QU (PSI)	LITHOLOGY
B-001-0-24	842.5'-842.1' 838.7'-838.3'	23.5'-23.9' 27.3'-27.7'	529 1,706	INTERBEDDED SHALE/LIMESTONE

ORGANIC CONTENT BY LOSS ON IGNITION TEST				
EXPLOR. ID	SAMPLE ID	SAMPLE ELEVATION	SAMPLE DEPTH	LOI (%)
B-001-0-24	SS-3	862.0'-860.5'	4.0'-5.5'	0.5

GEOTECHNICAL PROFILE - LANDSLIDE
WAR-350-8.30

DESIGN AGENCY
Stantec
10200 Alliance Road, Suite 300
Cincinnati, OH 45242
(513) 842-8200

DESIGNER	MSJ
REVIEWER	EMK 12-19-25
PROJECT ID	116664
SUBSET / TOTAL	0 / 0
SHEET / TOTAL	P.22 / 28

WAR-350-8.30

WAR-350-8.30

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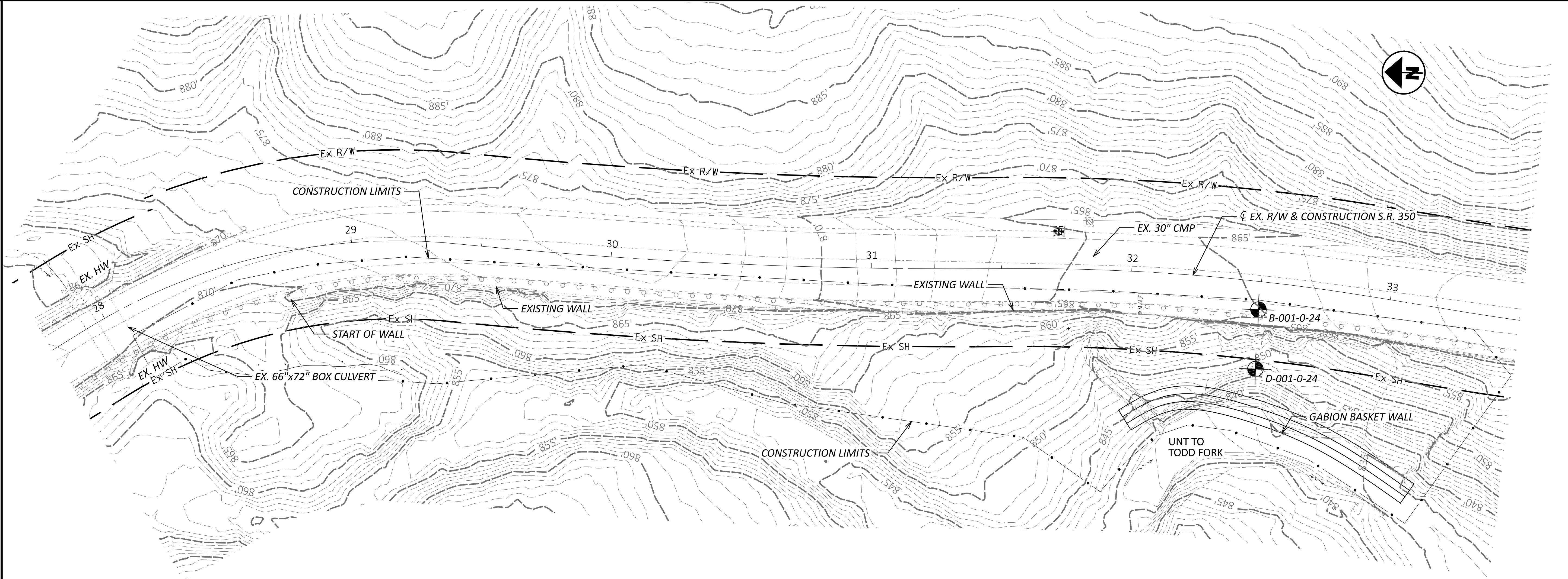

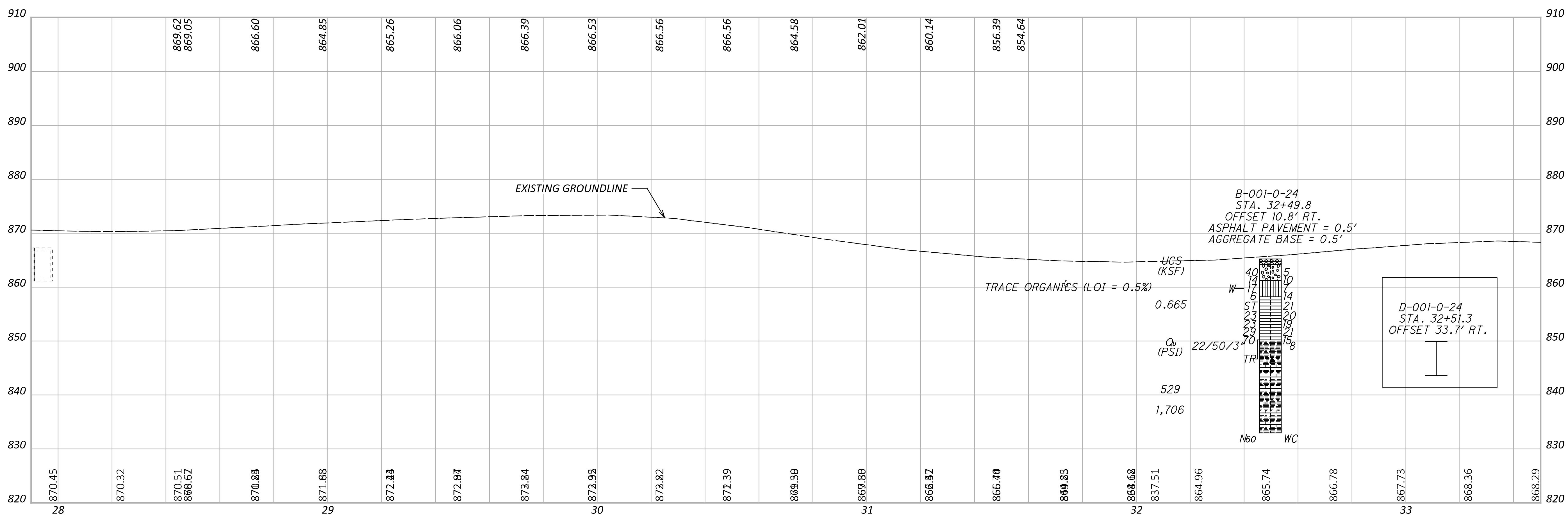

HORIZONTAL SCALE IN FEET

0 10 20 40

GEOTECHNICAL PROFILE - LANDSLIDE

S.R-350 - STA. 27+75.00 - STA. 33+50.00

DESIGN AGENCY

Stantec

10200 Alliance Road, Suite 300
Cincinnati, OH 45242
(513) 842-8200

DESIGNER	MSJ
REVIEWER	EMK 12-19-25
PROJECT ID	116664
SUBSET / TOTAL	0 / 0
SHEET / TOTAL	P.23 / 28

WAR-350-8.30

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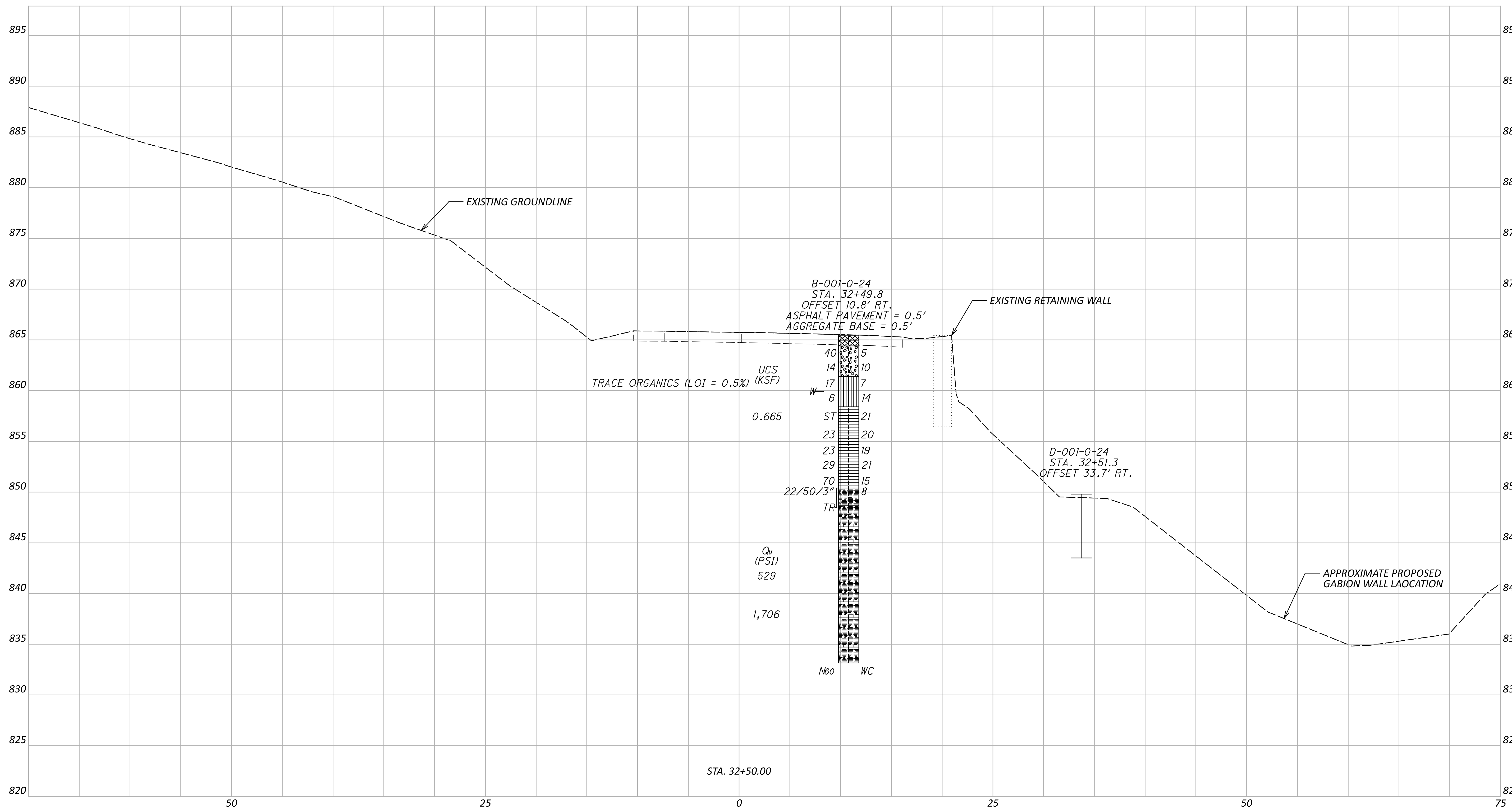

HORIZONTAL SCALE IN FEET
0 2.5 5 10

CROSS SECTION - S.R. 350
S.R. 350 - STA. 32+50.00

DESIGN AGENCY
Stantec
10200 Alliance Road, Suite 300
Cincinnati, OH 45242
(513) 842-8200

DESIGNER
MSJ

REVIEWER
EMK 12-19-25

PROJECT ID
116664

SUBSET	TOTAL
0	0

SHEET	TOTAL
P.24	28

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PROJECT: WAR 350 8.30 | TYPE: ROADWAY | PID: 116664 SFN: N/A | START: 9/25/24 END: 9/25/24

DRILLING FIRM / OPERATOR: S&ME / B. KENYON | SAMPLING FIRM / LOGGER: S&ME / J. SAMPLES | DRILLING METHOD: 3.25" HSA / NQ2 | SAMPLING METHOD: SPT / ST / NQ2

DRILL RIG: S&ME CME 550X (R50) | HAMMER: CME AUTOMATIC | CALIBRATION DATE: 8/22/24 | ENERGY RATIO (%): 85.9

STATION / OFFSET: 32+49.8, 10.8' RT. | ALIGNMENT: SR350 | ELEVATION: 866.0 (MSL) EOB: 32.25 ft. | LAT / LONG: 39.401474, -84.006700

EXPLORATION ID B-001-0-24 | PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 866.0	DEPTHS
BLACK, **PAVEMENT AND BASE**, DRY Asphalt - 6 inches, Aggregate Base - 6 inches	865.0	
MEDIUM DENSE TO DENSE, LIGHT BROWN TO GRAY, **GRAVEL AND/OR STONE FRAGMENTS**, SOME SAND, FILL, MOIST	862.0	
STIFF TO VERY STIFF, DARK GRAY TO BROWN, **SANDY SILT**, TRACE TO SOME CLAY, LITTLE GRAVEL, MOIST FROM 4.0 FT. TO 5.5 FT., ORGANIC CONTENT = 0.5% MEDIUM STIFF @ SS-4	859.0	W 860.5
VERY STIFF TO HARD, LIGHT BROWN TO GRAY, **SILTY CLAY**, TRACE SAND, TRACE TO LITTLE LIMESTONE FRAGMENTS, MOIST FROM 7.0 FT. TO 9.0 FT., UCS = 0.665 KSF	851.0	TR
HARD, BROWN AND GRAY, HIGHLY WEATHERED SHALE WITH LIMESTONE FRAGMENTS, MOIST	849.3	
INTERBEDDED SHALE (90%) AND LIMESTONE (10%), RQD 50%, REC. 99.5%; **SHALE**, DARK GRAY, HIGHLY TO MODERATELY WEATHERED, VERY WEAK TO WEAK, LAMINATED TO VERY THIN BEDDED, CALCAREOUS; **LIMESTONE**, BLUISH-GRAY, SLIGHTLY WEATHERED, MODERATELY STRONG, VERY THIN TO THIN BEDDED, ARGILLACEOUS, FOSSILIFEROUS. FROM 23.5 FT. TO 23.9 FT., UCR = 529 PSI FROM 27.3 FT. TO 27.7 FT., UCR = 1,706 PSI	833.8	EOB

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
6 16 12	40	50	SS-1	-	-	-	-	-	-	-	-	-	5	A-1-a (V)	
3 4 6	14	22	SS-2	-	66	24	9	-	1	20	14	6	10	A-1-a (0)	
3 5 7	17	61	SS-3	-	-	-	-	-	-	-	-	-	7	A-4a (V)	
1 2 2	6	56	SS-4	0.25	19	22	18	17	24	24	14	10	14	A-4a (1)	
		67	ST-1	-	4	7	18	21	50	40	17	23	21	A-6b (12)	
6 7 9	23	67	SS-5	1.00	-	-	-	-	-	-	-	-	20	A-6b (V)	
7 7 9	23	61	SS-6	4.50	5	4	3	22	66	39	23	16	19	A-6b (10)	
5 7 13	29	89	SS-7	4.50	-	-	-	-	-	-	-	-	21	A-6b (V)	
18 21 28	70	56	SS-8	4.00	16	5	2	17	60	39	22	17	15	A-6b (11)	
22 50/3"	-	89	SS-9	-	-	-	-	-	-	-	-	-	8	Rock (V)	
37		98	NQ2-1											CORE	
72		100	NQ2-2											CORE	
48		100	NQ2-3											CORE	
24		100	NQ2-4											CORE	

NOTES: GROUNDWATER ENCOUNTERED AT 5.5 FEET DURING DRILLING, LOCATION COORDINATES AND ELEVATIONS WERE OBTAINED FROM AVAILABLE MAPPING.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: SURFACE CAPPED WITH ASPHALT PATCH; TREMIED BENTONITE AND CEMENT GROUT

Date: 9/27/2024

Photographer: Suman Marahatta

1		
	Location / Orientation	B-001-0-24, 16.75 feet to 25.167 feet
	Remarks	Rock Core Photo Log

Date: 9/27/2024

Photographer: Suman Marahatta

2		
	Location / Orientation	B-001-0-24, 25.167 feet to 32.25 feet
	Remarks	Rock Core Photo Log

GEOTECHNICAL PROFILE - LANDSLIDE
BORING LOG B-001-0-24 & ROCK CORE PHOTOS

DESIGN AGENCY: Stantec, 10200 Alliance Road, Suite 300, Cincinnati, OH 45242, (513) 842-8200
DESIGNER: MSJ
REVIEWER: EMK 12-19-25
PROJECT ID: 116664
SUBSET 0 TOTAL 0

DCP TEST DATA

STA. 32+51.3, OFFSET 33.7' RT.

Project: *24780134 WAR350 8.30 PID 116664* **Date:** 25-Sep-24
Location: Clarksville, OH **Soil Type(s):** Silty Clay (A-6b)

Hammer
- ○ 10.1 lbs.
- ● 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ● All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	0	1
10	630	1
10	944	1
10	1186	1
10	1387	1
10	1548	1
10	1651	1
10	1737	1
10	1834	1
10	1922	1
10	1972	1
10	1975	1
10	1979	1
10	1991	1
10	2010	1
		1
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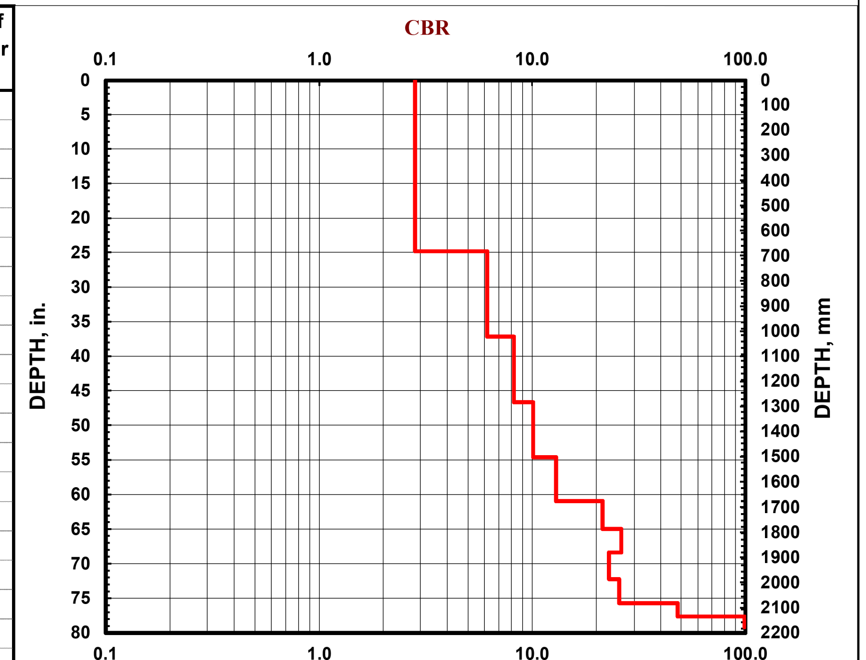
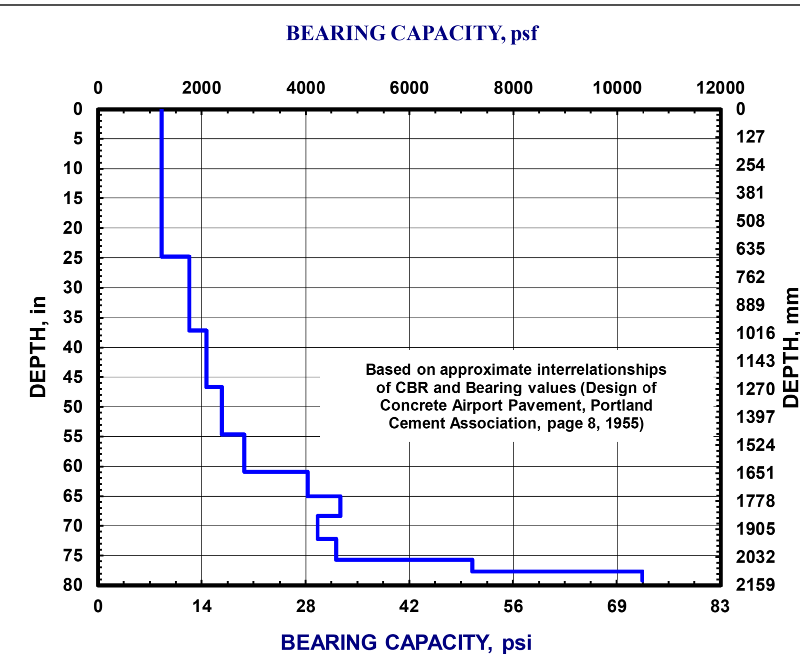




GEOTECHNICAL PROFILE - LANDSLIDE
DCP BORING D-001-0-24

DESIGN AGENCY
Stantec
10200 Alliance Road, Suite 300 Cincinnati, OH 45242 (513) 842-8200

DESIGNER MSJ
REVIEWER EMK 12-19-25
PROJECT ID 116664
SUBSET 0 TOTAL 0
SHEET P.26 TOTAL 28

WAR-350-8.30

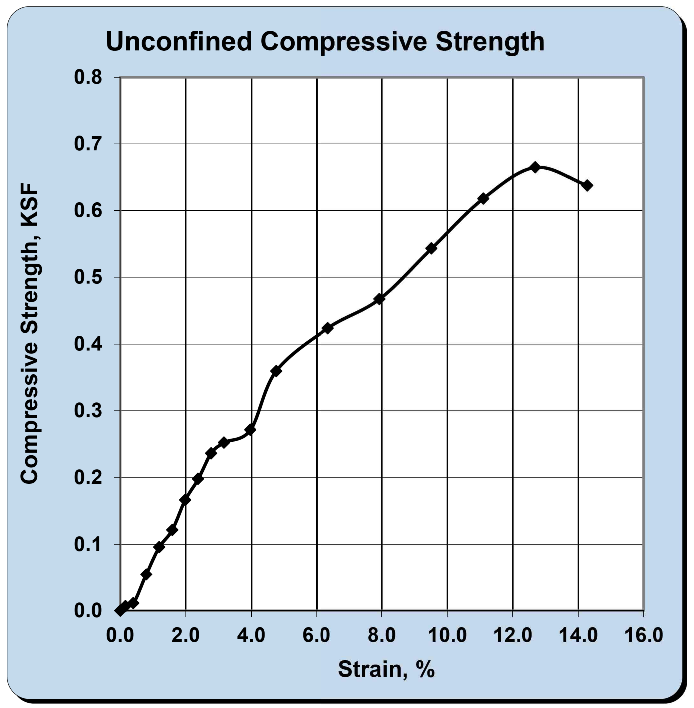
Form No. TR-D2166-01
Revision No. : 1
Revision Date: 08/16/17

UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS

ASTM D2166

S&ME, Inc. Cincinnati: 862 East Crescentville Road, West Chester, OH 45246

Project No.:	24780134	Report Date:	10/18/2024
Project Name:	WAR 350 8.30 PID 116664	Test Date(s):	9/30/2024
Client Name:	Stantec Consulting Services, Inc.		
Client Address:	10200 Alliance Road, Siute 300, Cincinnati, OH 45242		
Location:	B-001 Sample No. ST-1	Sample Date:	9/25/2024
		Depth:	7.0-9.0
Sample Description:	Silty Clay (A-6b)		



Failed Specimen

Type of Sample: Intact
Source of Moisture Sample: Test Specimen

Liquid Limit: 40
Plasticity Index: 23
Height to Diameter Ratio: 2.2
Rate of Strain (%/min.): 0.0086
Strain at Failure: 14.3

Initial Dry Unit Weight: 106.1 pcf
Initial Water Content: 18.7%
Unconfined Compressive Strength, q_u: **0.665** KSF
Undrained Shear Strength, s_u: **0.332** KSF

References / Comments / Deviations:

K. Cannady	*(signature)*	QAS	10/18/2024
Technical Responsibility	*Signature*	*Position*	*Date*

Form No. TR-D7012C-01
Revision No. : 1LexC
Revision Date: 12/12/23

UNIAXIAL COMPRESSIVE STRENGTH OF ROCK

ASTM D7012 Method C

Quality Assurance

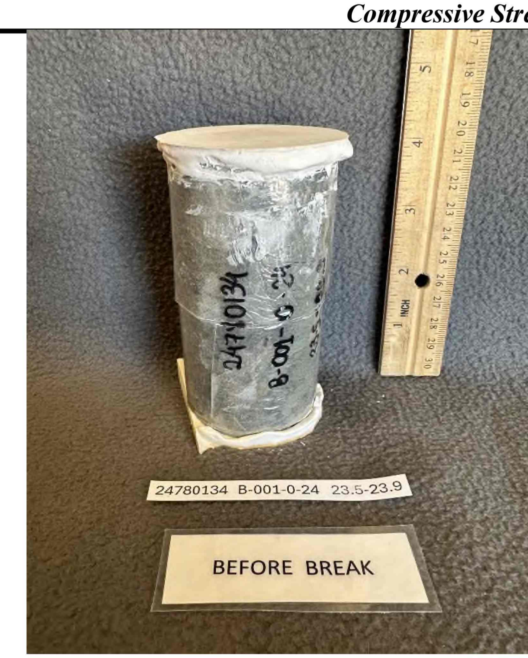
S&ME, Inc. - Lexington: 2020 Liberty Road, Suite 105, Lexington, KY 40505

Project No.:	24780134	Report Date:	10/14/24
Project Name:	WAR 350 8.30 PID 116664	Test Date(s):	10/11/24
Client Name:	Stantec Consulting Services Inc.		
Client Address:	10200 Alliance Road, Suite 300, Cincinnati, OH 45242	Received Date:	09/25/24
Location:	B-001-0-24	Depth, ft:	23.5-23.9
Sample Description:	Shale		

Angle of load relative to lithology: Approximately perpendicular

Test Results

Moisture Content ***6.6 %*** ***Dry Unit Weight*** ***141.9 pcf***
Compressive Strength ***529 psi***



Before Test

After Test

Strain rate: 0.015 in/min.

Notes / Deviations / References: **Per ASTM D4543 7.6, for specimens which have physical characteristics which prevent planing the sample to within the flatness requirement it is permissible to cap them with gypsum. This specimen was too fragile for planing to within the flatness requirement and was capped with gypsum.**

J. Folsom	*Jacob Folsom*	Lab Services Manager	10/14/2024
Technical Responsibility	*Signature*	*Position*	*Date*

GEOTECHNICAL PROFILE - LANDSLIDE
LAB DATA SHEET

DESIGN AGENCY
Stantec
10200 Alliance Road, Suite 300
Cincinnati, OH 45242
(513) 842-8200

DESIGNER: MSJ
REVIEWER: EMK 12-19-25
PROJECT ID: 116664
SUBSET: 0 TOTAL: 0
SHEET: P.27 TOTAL: 28

WAR-350-8.30

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Form No. TR-D7012C-01
Revision No. : 1LexC
Revision Date: 12/12/23

UNIAXIAL COMPRESSIVE STRENGTH OF ROCK

ASTM D7012 Method C

Quality Assurance

S&ME, Inc. - Lexington: 2020 Liberty Road, Suite 105, Lexington, KY 40505

Project No.:	24780134	Report Date:	10/14/24
Project Name:	WAR 350 8.30 PID 116664	Test Date(s):	10/11/24
Client Name:	Stantec Consulting Services Inc.		
Client Address:	10200 Alliance Road, Suite 300, Cincinnati, OH 45242	Received Date:	09/25/24
Location:	B-001-0-24	Depth, ft:	27.3-27.7
Sample Description:	Shale		

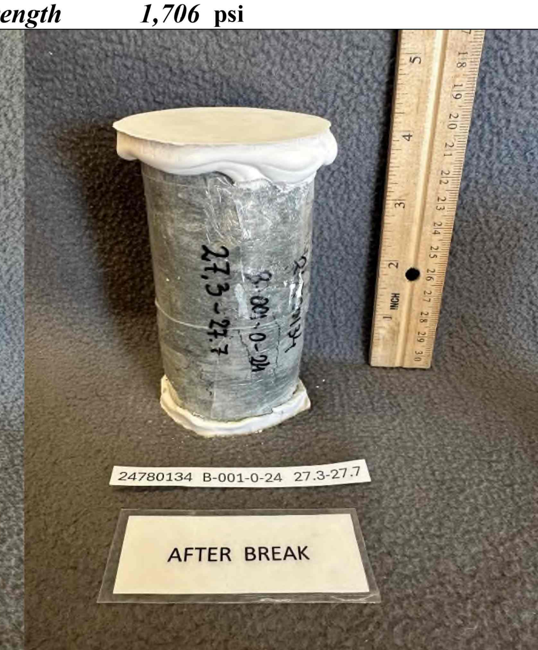
Angle of load relative to lithology: Approximately perpendicular

Test Results

Moisture Content *4.9* % ***Dry Unit Weight*** *148.1* pcf

Compressive Strength *1,706* psi

Before Test



After Test

Strain rate: 0.015 in/min.

Notes / Deviations / References: **Per ASTM D4543 7.6, for specimens which have physical characteristics which prevent planing the sample to within the flatness requirement it is permissible to cap them with gypsum. This specimen was too fragile for planing to within the flatness requirement and was capped with gypsum.**

J. Folsom	Jacob Folsom	Lab Services Manager	10/14/2024
Technical Responsibility	*Signature*	*Position*	*Date*

APPENDIX B
S&ME GEOTECHNICAL DATA REPORT

Geohazard Exploration Data Report
WAR-350-8.30 Landslide (PID: 116664)
Clarksville, Ohio
S&ME Project No. 24780134

PREPARED FOR:

Stantec Consulting Services, Inc.
10200 Alliance Road, Suite 300
Cincinnati, OH 45242

PREPARED BY:

S&ME, Inc.
862 East Crescentville Road
Cincinnati, OH 45246

November 8, 2024

November 8, 2024

Stantec Consulting Services, Inc.
10200 Alliance Road, Suite 300
Cincinnati, OH 45242

Attention: Mr. Eric M. Kistner
E: eric.kistner@stantec.com

Reference: **Geohazard Exploration Data Report**
WAR-350-8.30 Landslide (PID: 116664)
Clarksville, Ohio
S&ME Project No. 24780134

Dear Mr. Kistner:

S&ME, Inc. (S&ME) has completed the geohazard exploration and Dynamic Cone Penetration (DCP) testing at the location of a failing retaining wall on SR-350 in Clarksville, Warren County, Ohio. The work was performed in general accordance with S&ME Proposal No. 24780134, dated August 15, 2024. The purpose of our exploration was to explore subsurface conditions and to provide site information regarding surface and subsurface conditions, ground water observation, provide the laboratory test result and prepare a field log of the boring.

We appreciate having been given the opportunity to be of service on this project. If you require additional assistance or have any questions, please feel free to contact our office at 513-771-8471.

Sincerely,

S&ME, Inc.

Sudip B. Khadka, P.E.
Project Manager/Team Lead

Benjamin C. Dusina, P.E.
Senior Reviewer/Principal Engineer

Table of Contents

List of Figures

List of Tables

Appendices

1.0 Introduction

S&ME, Inc. (S&ME) has completed the geohazard exploration for structure boring and Dynamic Cone Penetrometer (DCP) testing at the location of a failing retaining wall on SR-350 in Clarksville, Warren County, Ohio. The work was performed in general accordance with S&ME Proposal No. 24780143, dated August 15, 2024. The purpose of our exploration was to explore subsurface conditions and to provide site information regarding surface and subsurface conditions, ground water observation, provide the laboratory test result and prepare a field log of the boring. This report describes our understanding of the project, presents the results of field exploration and laboratory testing.

Project information was provided to S&ME by Stantec on August 8, 2024. Based on the project information, S&ME understands that Ohio Department of Transportation District 08 (ODOT-D08) is planning to repair the slope and restore the pavement along the northern portion of the existing soldier pile and lagging wall. The outside bend of the creek has eroded the toe of the embankment below the wall causing the wall to backfill to pipe out underneath the wall. Stantec retained S&ME to perform one (1) boring and one (1) DCP test at the site and provide a data report for the project. Figure 1-1 below shows the Site Map.

The project information and considerations detailed above should be reviewed and confirmed by the appropriate team members.

Figure 1-1 – Site Map

2.0 Geology and Observations of the Project

2.1 Regional Geology

Based on the Web Soil Survey, the primary soil type is Hickory silt loam of 25 to 35% slopes derived from a thin layer of Loess and the underlying till. A review of Ohio Geology Interactive Map maintained by Ohio Department of Natural Resources (ODNR), indicates that the site consists of Ground Moraine soil from the Illinoian age. Bedrock geology indicates that the site consists of layers of gray to bluish gray to weathered light gray interbedded Shale and Limestone of Waynesville Formation from Ordovician age, with a composition of 70% Shale and 30% Limestone. The overburden soil above the bedrock is 7 to 29 feet thick. We encountered bedrock at a drilled depth of 15 feet during SPT and 6.25 feet during DCP testing and is consistent with published geology.

2.2 Available Information

Our review of the available mines record maintained by ODNR indicates that there are no mining activities within the immediate vicinity of the site. The nearest surface affected area with the Permit ID IM-0679 is located almost 4.92 miles Southwest of the site.

Based on the review of Karst Mapping maintained by the ODNR, there are no mapped Karst features within the immediate vicinity of the site. The nearest field verified Karst is located within 0.38 miles southwest of the site. However, there are 26 other fields verified Karst located within 0.8 miles west and 14 more field verified Karsts located within 0.82 miles Northeast of the site. Refer to Figure 2-1 on the following page.

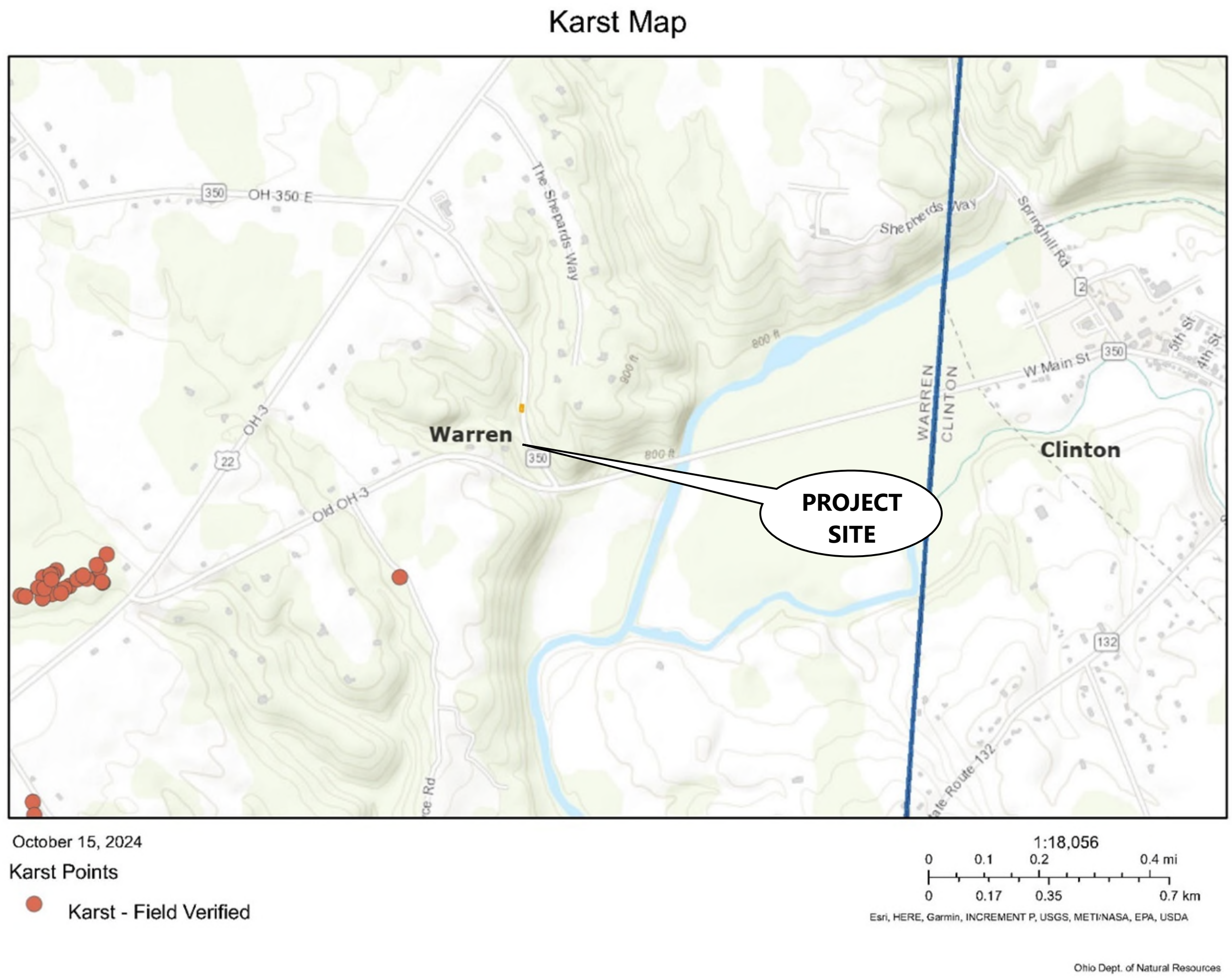


Figure 2-1 – Karst Map

2.3 Reconnaissance

Based on our site visit on September 18, 2024 and aerial maps, the project site is along SR-350 in Clarksville, Ohio, about 900 feet north of the SR-350 and Old CCC Road intersection. A small seasonal creek flows west of the site, and thick trees line both sides of the road. The land slopes down, with hills to the east and gentle slopes to the west. A few single-family homes are located within a few hundred feet on both sides of the site. Figure 2-2 on the following page illustrates the site outlined in red.

Figure 2-2 – Site Location Map

3.0 Exploration

3.1 Field Exploration

For this geohazard exploration, one (1) boring and one (1) DCP test were performed on site on September 25, 2024. The boring was extended to 32.3 feet below the existing pavement and 6.3 feet for DCP Test. The boring location was staked in the field prior to our field exploration by Stantec and S&ME personnel.

The boring was performed using CME-550X All-Terrain Vehicle (ATV) drill rig using 3¼-inch hollow stem augers with an 85.9% efficiency. Soil samples were obtained using a split-barrel sampler (SPT) driven by an automatic hammer system in general accordance with ASTM D1586. Split-barrel soil samples were placed in air-tight glass jars and retained for visual classification and subsequent laboratory testing. The augers hit refusal at 16.8 feet and rock coring began at 16.8 feet and continued down to 32.25 feet. One (1) Undisturbed samples (Shelby tube) were obtained for this project. A general description of our field procedures, a test boring log legend, and boring logs are provided in Appendix II of this report. The stratification lines shown on the Boring Log represent the approximate boundaries between soil types. The actual transitions may be more gradual than shown.

Boring coordinates and elevations are summarized in Table 3-1, below. The approximate locations of boring and DCP test are shown on the Boring Location Plan (Figure 2) in Appendix I.

Table 3-1 – Boring Coordinate Summary

Boring No.	Latitude (°)	Longitude (°)	Surface Elevation (ft)	Asphalt and Base (Inch)	Termination Depth (ft)
B-001-0-24	39.401474	-84.006700	866	12	32.3
D-001-1-24	39.401476	-84.006762	856	--	6.3

Note:

1. Latitude and Longitude were obtained using a handheld GPS with sub-meter horizontal accuracy.
2. Surface Elevations were obtained using available topographic information.

3.1.1 Dynamic Cone Penetration (DCP)

S&ME performed one (1) DCP test using a Kessler K-100 series probe downslope of the retaining wall between the creek and the retaining wall. The DCP sounding (designated as D-001-1-24) was performed by raising a 17.6-pound weight and measuring the penetration obtained with 10 blows. The technicians advanced the DCP until they ran out of tooling or could not advance any deeper with successive blows (refusal). The DCP was advanced to approximately 80 inches below the top of grade. The Log of Kessler DCP probe is attached in Appendix II. The hole was backfilled with cuttings upon completion of testing.

3.2 Laboratory Testing

Following retrieval, the Stantec and S&ME staff on-site preserved the recovered soil samples in air-tight glass jars. The recovered samples were returned to our laboratory where applicable laboratory tests were assigned. These tests are used to assess the engineering properties of the soil. The soil samples were visually classified by a geotechnical engineer. S&ME conducted the following laboratory test for the obtained split spoon, Shelby tube and rock core sample.

- Moisture Content Determinations
- Atterberg Limits
- Minus 200 Wash
- Hydrometer Analyses
- Loss of Ignition (LOI)
- Unconfined Compression Test (Soil)
- Unconfined Compression Test (Rock)

The results of the laboratory index tests are recorded numerically on individual boring logs and the results of the strength tests are presented in Appendix III. Based upon the results of the laboratory testing program, the field logs were modified, as necessary, and copies of the laboratory corrected boring logs are presented in Appendix II. Shown on these logs are: descriptions of the soil stratigraphy encountered; depths from which samples were preserved; sampling efforts (blow-counts) required to obtain the specimens in the borings; laboratory testing results; seepage and groundwater observations made at the time of drilling; and, values of hand-penetrometer measurements made in soil samples exhibiting cohesion. For your reference, hand-penetrometer values are roughly equivalent to the unconfined compressive strength of the cohesive fraction of the soil sample.

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

4.0 Findings

The following is a generalized description of the subsurface conditions. The stratification of the soil, as shown on the Boring Logs in Appendix II, represents the conditions at the actual boring locations. Lines of demarcation represent the approximate boundary between the soil types, but the transition may be gradual or not clearly defined.

4.1 General Subsurface Profile

The Boring was drilled through the 6-inches of asphalt over 6-inches of aggregate base. The subsurface conditions were consistent with the published geological mapping information. Table 4-1 on the following page provides the generalized subsurface conditions of the soils encountered.

Table 4-1 – Boring Location Summary

Depth (ft.)	Group	Description [1]	Range of SPT N-Values (blows per foot)
0-0.5	N/A (Surface cover)	6-inches of Asphalt	--
0.5-1.0	N/A (Surface cover	6-inches of Base	--
1.0-5.5	A	Fill: Gravel and/or Stone Fragments with Sand (A-1-b), Gravel and/or Stone Fragments (A-1-a), loose to medium dense, black to brown, dry to moist	10-28
5.5-15.0	B	Residuum: Sandy Silt (A-4a), Silty Clay (A-6b), soft to hard, light brown, brown and gray, moist	4-49
15.0-16.8	C	Weathered Bedrock: Shale, with limestone fragments highly weathered, brown and gray	20-50/3”
16.8-32.3	D	Bedrock: Shale (90%), interbedded with limestone (10%), highly to moderately weathered, very weak to moderately strong, dark gray and bluish-gray	--

Note:
1. This is a generalized summary and may not accurately depict the actual conditions encountered in the test borings. These groups may not be encountered at each boring location.

4.2 Groundwater

Seepage and groundwater observations were made during drilling operations. Water level was encountered at a depth of 5.5 feet at the time of drilling. It wasn’t possible to accurately measure the water level at the end of drilling due to water use during rock coring. Due to safety concerns, the borings were backfilled immediately upon completion; therefore, extended water level measurements were not obtained.

Fluctuations in the level of the groundwater table (or saturated soils/perched water levels) will occur due to seasonal variances in rainfall, drainage, types of soils present and other factors. We caution that groundwater can be perched at various elevations above the general static groundwater level after periods of rainfall, especially in the lower elevations and natural drainage paths of the site.

For further details of subsurface conditions encountered, refer to the individual Boring Logs presented in Appendix II. The stratum lines shown on the boring logs should be considered approximate.

5.0 Limitations of Report

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

If the project information described in this report is not accurate, or if it changes during project development, we should be notified of the changes so that we can modify our data report based on this additional information if necessary.

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

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

Appendices

Appendix I – Additional Figures

Important Information About Geotechnical Report
Vicinity Map
Boring Location Plan

Important Information About Your
Geotechnical Engineering Report

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

Geotechnical Findings Are Professional Opinions

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

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

Scope of Geotechnical Services

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

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

Services Are Performed for Specific Projects

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

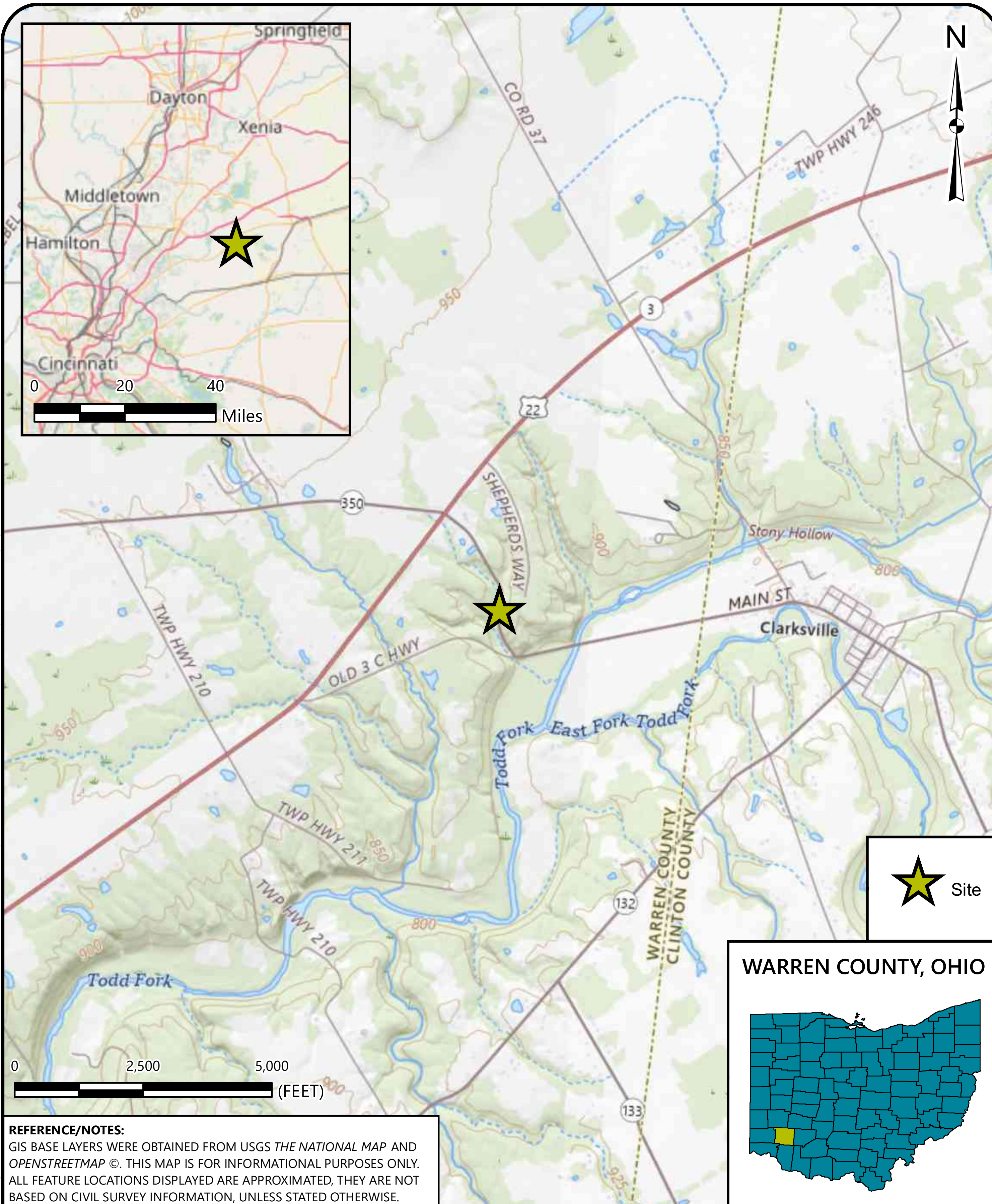
Subsurface information and recommendations may not be adequate for other uses. Changes in a proposed structure location, foundation loads, grades, schedule, etc. may require additional geotechnical exploration, analyses, and consultation. The geotechnical engineer should be consulted to determine if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

Geo-Environmental Issues

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

Geotechnical Recommendations Are Not Final

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



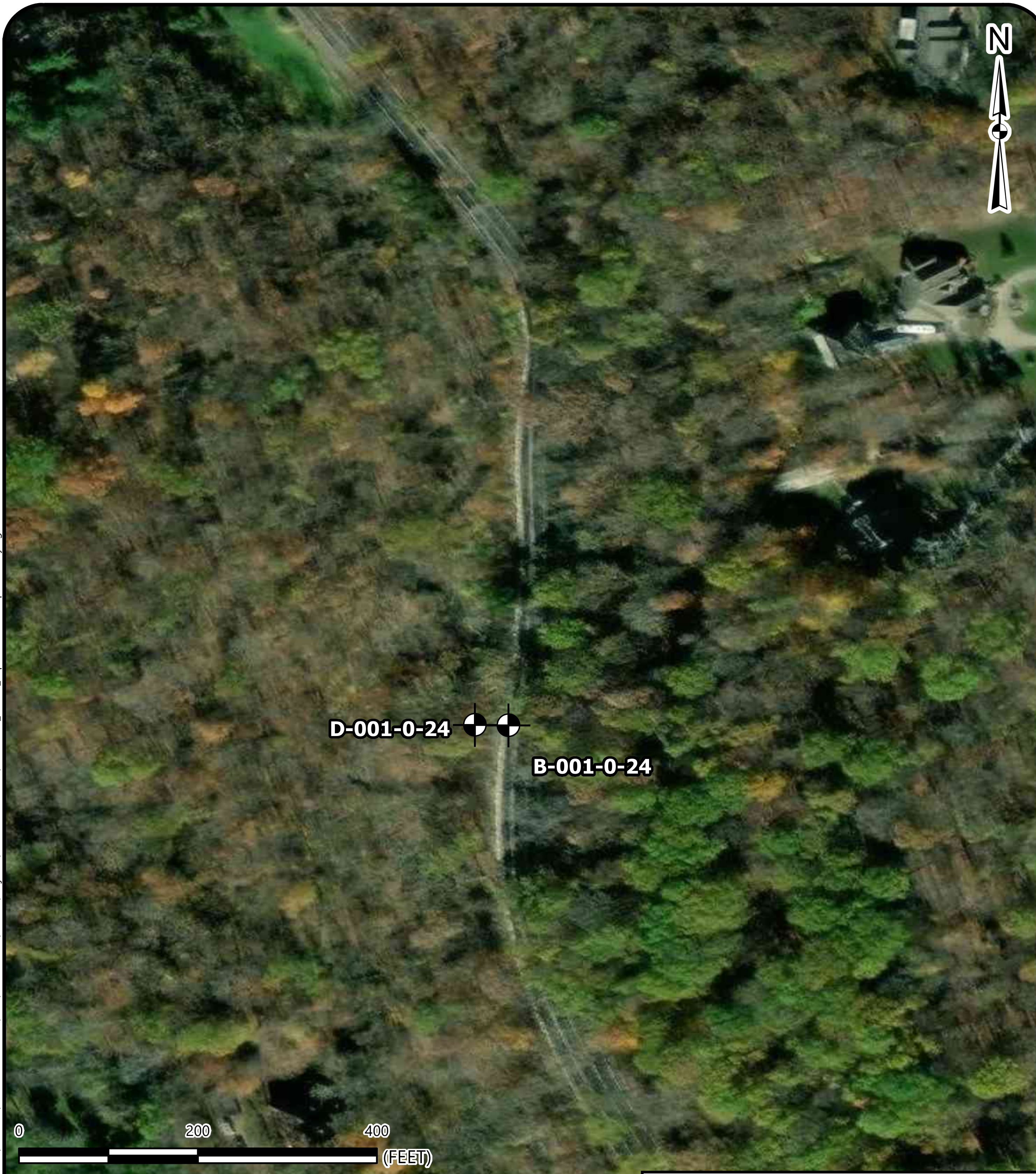
REFERENCE/NOTES:
GIS BASE LAYERS WERE OBTAINED FROM USGS *THE NATIONAL MAP* AND *OPENSTREETMAP* ©. THIS MAP IS FOR INFORMATIONAL PURPOSES ONLY. ALL FEATURE LOCATIONS DISPLAYED ARE APPROXIMATED, THEY ARE NOT BASED ON CIVIL SURVEY INFORMATION, UNLESS STATED OTHERWISE.

Drawing Path: C:\Users\AGostic\OneDrive - S&ME, Inc\Documents\ArcGIS\Projects\24780134 SR350\24780134_SR350_vicinity.mxd plotted by agostic 10-15-2024

VICINITY MAP

STATE ROUTE 350 LANDSLIDE
CLARSKVILLE, WARREN COUNTY, OHIO

SCALE: 1 " = 2,500 '	FIGURE NO. 1
DATE: 11-8-24	
PROJECT NUMBER 24780134	



REFERENCE: ESRI
GIS BASE LAYERS WERE OBTAINED FROM ESRI. THIS MAP IS FOR INFORMATIONAL PURPOSES ONLY. ALL FEATURE LOCATIONS DISPLAYED ARE APPROXIMATED. THEY ARE NOT BASED ON CIVIL SURVEY INFORMATION, UNLESS STATED OTHERWISE.

APPROXIMATE BORING LOCATION

BORING LOCATION PLAN

STATE ROUTE 350 LANDSLIDE
CLARSKVILLE, WARREN COUNTY, OHIO

SCALE: 1 " = 150 '

DATE: 11-8-24

PROJECT NUMBER 24780134

FIGURE NO. 2

Appendix II – Field Exploration

Soil Log Legend
Rock Core Log Legend
Test Boring Logs
DCP Log
Rock Core Picture
Summary of Field Procedures

ODOT SOIL LOG LEGEND

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

2 3 4

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

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

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

ODOT SOIL CLASSIFICATION DESCRIPTION AND SYMBOL

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

SOIL LOG SYMBOLS

SS - Split-Spoon Sample

ST - Shelby Tube Sample

TR - Top of Rock

REC - Sample Recovery, %

HP - Hand Penetrometer Value, tsf

LOI - Loss on Ignition Test, %

Qu - Unconfined Compressive Strength

γ_d - Dry Unit Weight, pcf

γ_m - Moist Unit Weight, pcf

GR - Gravel Content, %

CS - Coarse Sand Content, %

FS - Fine Sand Content, %

SI - Silt Content, %

CL - Clay Content, %

LL - Liquid Limit

PL - Plastic Limit

PI - Plasticity Index

WC - Natural Water Content, %

NOTE: Particle size contents are expressed % by weight.

PARTICLE SIZE

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

FINE-GRAINED SOIL
(Relative Consistency)

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

COARSE-GRAINED SOIL
(Relative Density)

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

MINOR CONSTITUENTS
(% By Weight)

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

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

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

RELATIVE MOISTURE CONDITION

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

—W **At Time of Drilling**	Free water (seepage or groundwater) observation made anytime during the drilling process. Depending on time of reading and drilling methodologies, this value may be influenced by the drilling process.
▽ **At end of Drilling**	Free water measurement soon after the drilling processes are complete, and the borehole is at final depth. Drilling fluids, if introduced during drilling, may influence this measurement.
▼ **24 hrs After Drilling**	Free water measurements made in a borehole hours to days after drilling is complete including the time elapsed (i.e., "24 hrs" as shown at left). Depending on subsurface conditions, elapsed time, drilling process, etc. this observation may reflect a stabilized level.

REFERENCES:

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

ODOT ROCK CORE LOG LEGEND

DESCRIPTIVE ORDER FOR ROCK STRATA

Bedrock type, color, weathering, strength, texture, bedding, other descriptors, type and condition of discontinuities, unit RQD, unit recovery.

When alternating layers occur between two distinct rock types, describe the material as "Interbedded" with the major rock type first, with estimated percentage, and the secondary rock type second, with estimated percentage. Provide the unit RQD and unit recovery, then describe each rock type in detail.

For spread footings founded on or into bedrock, describe discontinuities using the modified Rock Mass Rating (RMR) system (degree of fracturing, aperture width and surface roughness). For drilled shafts extending into bedrock, describe discontinuities using the Geologic Strength Index (GSI) system (discontinuity structure and surface condition). For rock cut slopes, describe discontinuities using both the modified RMR and GSI systems.

COMMON OHIO BEDROCK TYPES AND SYMBOLS

SHALE	SILTSTONE	LIMESTONE	COAL
CLAYSTONE/ MUDSTONE	SANDSTONE	DOLOMITE	UNDERCLAY/ FIRECLAY

WEATHERING

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

STRENGTH

		APPROX. UNCONFINED COMPRESSIVE STRENGTH (PSI)
Extremely Strong	Cannot be scratched by a knife or sharp pick. Chipping off hand specimens requires hard repeated blows of a geologist's hammer.	> 30,000
Very Strong	Cannot be scratched by a knife or sharp pick. Breaking off hand specimens requires hard repeated blows of a geologist's hammer.	30,000 - 15,000
Strong	Can be scratched with a knife or pick with difficulty. Requires hard hammer blows to detach hand specimen.	15,000 - 7,500
Moderately Strong	Can be scratched with a knife or pick. Gouges ¼" deep can be excavated by a pick. Requires moderate hammer blows to detach specimen.	7,500 - 3,600
Slightly Strong	Can be gouged 0.05 inch deep by firm pressure with a knife or pick point. Can excavate small pieces (1-inch) by hard blows with a pick.	3,600 - 1,500
Weak	Can be gouged readily by a knife or pick or excavated in small fragments by moderate blows of a pick. Small, thin pieces can be broken by hand.	1,500 - 750
Very Weak	Can be carved with a knife and excavated readily with a pick. Pieces 1 inch or more thick can be broken by hand. Can be scratched by fingernail.	750 - 40

TEXTURE

Boulder	> 12 in.
Cobble	12 - 3 in.
Gravel	3 - 0.08 in.
Coarse Sand	0.08 - 0.02 in.
Medium Sand	0.02 - 0.01 in.
Fine Sand	0.01 - 0.005 in.
Very Fine Sand	0.005 - 0.003 in.

BEDDING

Very Thick Bedded	> 36 in.
Thick Bedded	36 in. - 18 in.
Medium Bedded	18 in. - 10 in.
Thin Bedded	10 in. - 2 in.
Very Thin Bedded	2 in. - 0.4 in.
Laminated	0.4 in. - 0.1 in.
Thinly Laminated	< 0.1 in.

REFERENCES: OHIO DEPARTMENT OF TRANSPORTATION (ODOT), SPECIFICATIONS FOR GEOTECHNICAL EXPLORATIONS (SGE).

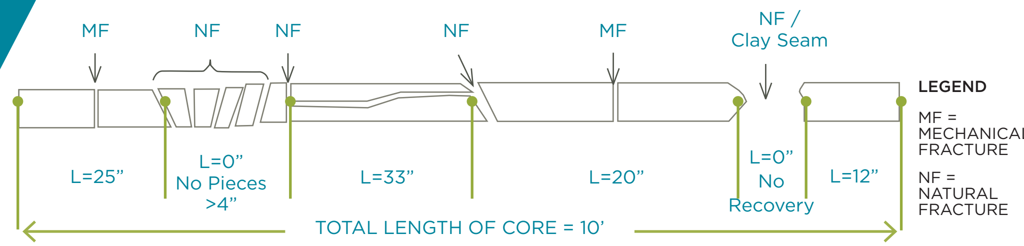
ODOT ROCK CORE LOG LEGEND

ROCK CORE RECOVERY

Recovery to be determined by core run and by rock unit (layer).

$$\text{REC} \,(\text{Recovery}) = \frac{\text{Length of Rock Core Recovered}}{\text{Length of Core Run}} \times 100$$

ROCK QUALITY DESIGNATION (RQD)



$$\text{RQD} \,(\text{Equation}) = \left(\frac{\Sigma \text{ Core with Length (L)} \geq 4''}{\text{Core Run or Interval Total Length}} \right) \times 100$$

$$\text{RQD} \,(\text{Example}) = \left(\frac{25'' + 33'' + 20'' + 12''}{120''} \right) \times 100 = 75\%$$

DESCRIPTORS

Arenaceous - Sandy

Argillaceous - Clayey

Brecciated - Contains angular gravel

Calcareous - Contains calcium carbonate

Carbonaceous - Contains carbon

Cherty - Contains chert

Conglomeritic - Contains rounded gravel

Crystalline - Contains crystalline structure

Dolomitic - Contains Ca/Mg carbonate

Ferriferous - Contains iron

Fissile - Thin planar partings

Fossiliferous - Contains fossils

Friable - Easily broken down

Micaceous - Contains mica

Pyritic - Contains pyrite

Siliceous - Contains silica

Stylolitic - Contains stylotites

Vuggy - Contains openings

DISCONTINUITIES IN BEDROCK

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

MODIFIED RMR DISCONTINUITY TERMS

DEGREE OF FRACTURING

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

APERTURE WIDTH

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

SURFACE ROUGHNESS

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

GSI DISCONTINUITY TERMS

ROCK MASS STRUCTURE

Intact or Massive	Intact rock with few widely spaced discontinuities
Blocky	Well interlocked undisturbed rock mass, formed by 3 intersecting discontinuity sets
Very Blocky	Interlocked, partially disturbed mass formed by 4 or more joint sets
Blocky/ Disturbed/Seamy	Angular blocks formed by many intersecting discontinuity sets, bedding planes
Disintegrated	Poorly interlocked, heavily broken rock mass
Laminated/ Sheared	Lack of blockiness due to close spacing of weak shear planes

SURFACE CONDITION

Very Good	Very rough, fresh unweathered surfaces
Good	Rough, slightly weathered, iron stained surfaces
Fair	Smooth, moderately weathered and altered surfaces
Poor	Slickensided, high weathered surface with compact coatings
Very Poor	Slickensided, highly weathered surface with soft clay coatings

PROJECT: WAR-350-08.30	DRILLING FIRM / OPERATOR: S&ME / B. KENYON	DRILL RIG: S&ME CME 550X (R50)	STATION / OFFSET: 32+50, 11' RT.	EXPLORATION ID B-001-0-24
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: S&ME / J. SAMPLES	HAMMER: CME AUTOMATIC	ALIGNMENT: SR350	
PID: 116664 SFN: N/A	DRILLING METHOD: 3.25" HSA / NQ2	CALIBRATION DATE: 8/22/24	ELEVATION: 865.4 (MSL) EOB: 32.25 ft.	PAGE 1 OF 2
START: 9/25/24 END: 9/25/24	SAMPLING METHOD: SPT / ST / NQ2	ENERGY RATIO (%): 85.9	LAT / LONG: 39.401720, 84.006640	

MATERIAL DESCRIPTION AND NOTES	ELEV. 865.4	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
BLACK, **PAVEMENT AND BASE**, DRY Asphalt - 6 inches, Aggregate Base - 6 inches	864.4																	
MEDIUM DENSE TO DENSE, LIGHT BROWN TO GRAY, **GRAVEL AND/OR STONE FRAGMENTS**, SOME SAND, FILL, MOIST			6 16 12	40	50	SS-1	-	-	-	-	-	-	-	-	-	5	A-1-a (V)	
	861.4		3 4 6	14	22	SS-2	-	66	24	9	- 1 -		20	14	6	10	A-1-a (0)	
STIFF TO VERY STIFF, DARK GRAY TO BROWN, **SANDY SILT**, TRACE TO SOME CLAY, LITTLE GRAVEL, MOIST FROM 4.0 FT. TO 5.5 FT., ORGANIC CONTENT = 0.5%		W 859.9	3 5 7	17	61	SS-3	-	-	-	-	-	-	-	-	-	7	A-4a (V)	
MEDIUM STIFF @ SS-4	858.4		1 2 2	6	56	SS-4	0.25	19	22	18	17	24	24	14	10	14	A-4a (1)	
VERY STIFF TO HARD, LIGHT BROWN TO GRAY, **SILTY CLAY**, TRACE SAND, TRACE TO LITTLE LIMESTONE FRAGMENTS, MOIST FROM 7.0 FT. TO 9.0 FT., UCS = 0.665 KSF					67	ST-1	-	4	7	18	21	50	40	17	23	21	A-6b (12)	
			6 7 9	23	67	SS-5	1.00	-	-	-	-	-	-	-	-	20	A-6b (V)	
			7 7 9	23	61	SS-6	4.50	5	4	3	22	66	39	23	16	19	A-6b (10)	
			5 7 13	29	89	SS-7	4.50	-	-	-	-	-	-	-	-	21	A-6b (V)	
	850.4	TR	18 21 28	70	56	SS-8	4.00	16	5	2	17	60	39	22	17	15	A-6b (11)	
HARD, BROWN AND GRAY, HIGHLY WEATHERED SHALE WITH LIMESTONE FRAGMENTS, MOIST	848.7		22 50/3"	-	89	SS-9	-	-	-	-	-	-	-	-	-	8	Rock (V)	
INTERBEDDED SHALE (90%) AND LIMESTONE (10%), RQD 50%, REC. 99.5%; **SHALE**, DARK GRAY, HIGHLY TO MODERATELY WEATHERED, VERY WEAK TO WEAK, LAMINATED TO VERY THIN BEDDED, CALCAREOUS; **LIMESTONE**, BLUISH-GRAY, SLIGHTLY WEATHERED, MODERATELY STRONG, VERY THIN TO THIN BEDDED, ARGILLACEOUS, FOSSILIFEROUS.			37		98	NQ2-1											CORE	
FROM 23.5 FT. TO 23.9 FT., UCR = 529 PSI			72		100	NQ2-2											CORE	

Depth scale (ft): 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 12/26/24 15:44 - U:\17341073\TECHNICAL_PRODUCTION\FIELD_DATA\WAR-350\24780134.GPJ

PID: 116664	SFN: N/A	PROJECT: WAR-350-08.30	STATION / OFFSET: 32+50, 11' RT.	START: 9/25/24	END: 9/25/24	PG 2 OF 2	B-001-0-24

MATERIAL DESCRIPTION AND NOTES	ELEV. 840.4	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
INTERBEDDED SHALE (90%) AND LIMESTONE (10%), RQD 50%, REC. 99.5%; **SHALE**, DARK GRAY, HIGHLY TO MODERATELY WEATHERED, VERY WEAK TO WEAK, LAMINATED TO VERY THIN BEDDED, CALCAREOUS; **LIMESTONE**, BLUISH-GRAY, SLIGHTLY WEATHERED, MODERATELY STRONG, VERY THIN TO THIN BEDDED, ARGILLACEOUS, FOSSILIFEROUS. *(continued)* FROM 27.3 FT. TO 27.7 FT., UCR = 1,706 PSI		26 – 30	48		100	NQ2-3											CORE	
	833.2	30 – 32 EOB	24		100	NQ2-4											CORE	

NOTES: GROUNDWATER ENCOUNTERED AT 5.5 FEET DURING DRILLING, LOCATION COORDINATES AND ELEVATIONS WERE UPDATED WITH STANTEC SURVEY.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: SURFACE CAPPED WITH ASPHALT PATCH; TREMIED BENTONITE AND CEMENT GROUT

DCP TEST DATA

Project: *24780134 WAR350 8.30 PID 116664*
Date: 25-Sep-24
Location: Clarksville, OH
Soil Type(s): Silty Clay (A-6b)

Hammer
- ○ 10.1 lbs.
- ● 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ● All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	0	1
10	630	1
10	944	1
10	1186	1
10	1387	1
10	1548	1
10	1651	1
10	1737	1
10	1834	1
10	1922	1
10	1972	1
10	1975	1
10	1979	1
10	1991	1
10	2010	1
		1
		1
		1
		1
		1
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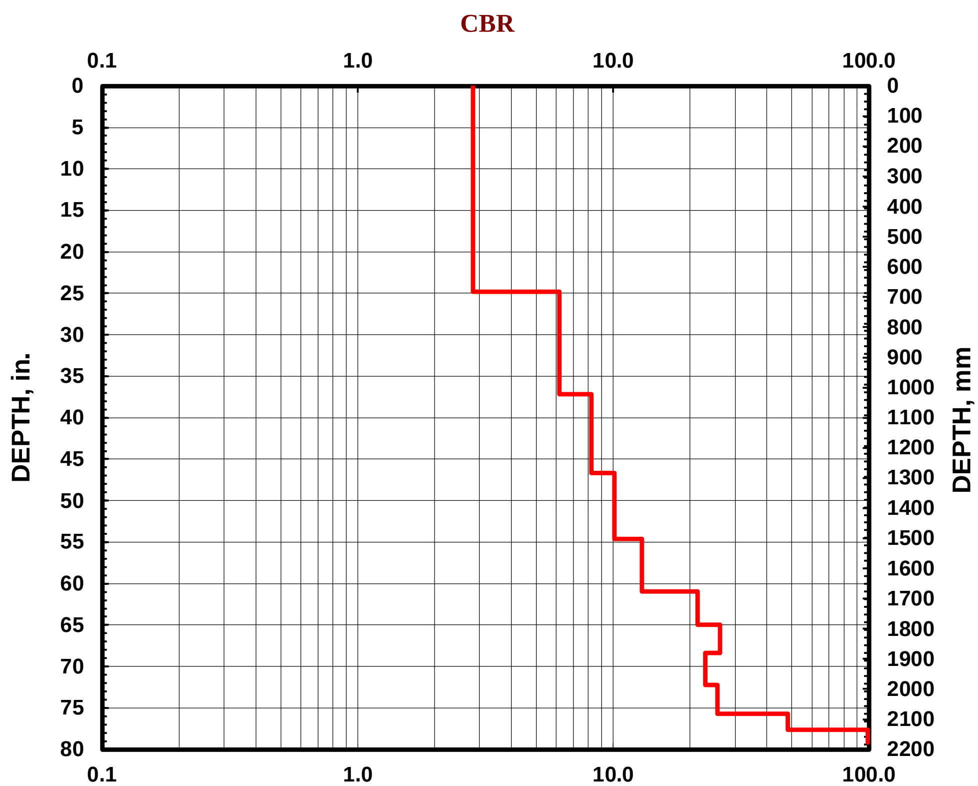
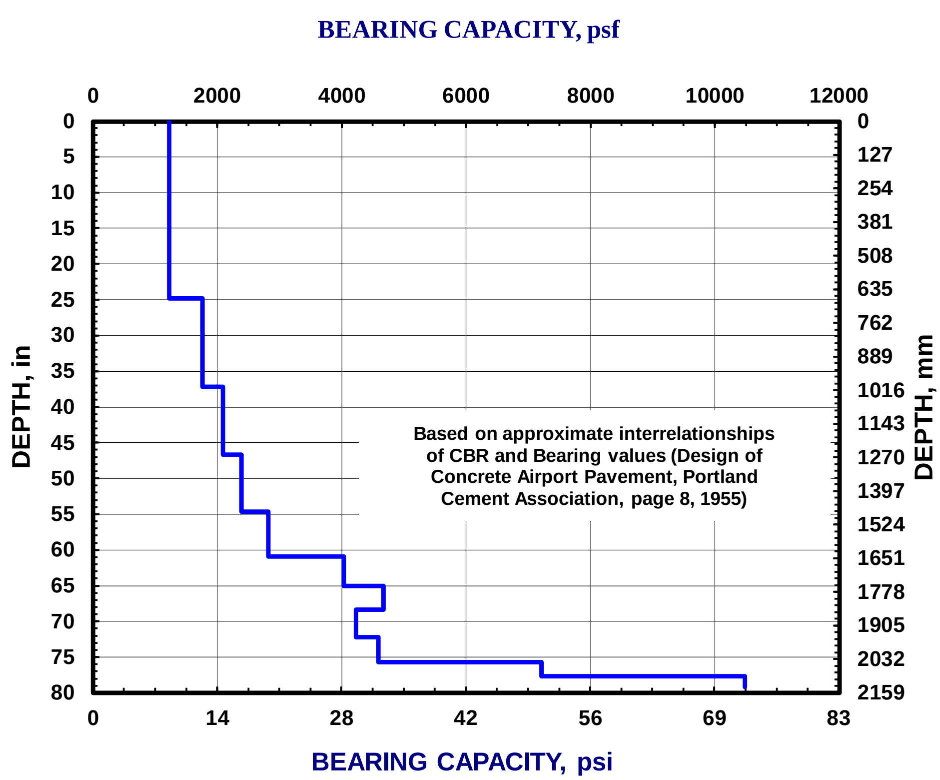




Date: 9/27/2024
Photographer: Suman Marahatta

1	**Location / Orientation**	B-001-0-24, 16.75 feet to 25.167 feet
	Remarks	Rock Core Photo Log

Date: 9/27/2024
Photographer: Suman Marahatta

2	**Location / Orientation**	B-001-0-24, 25.167 feet to 32.25 feet
	Remarks	Rock Core Photo Log

Summary of Field Procedures

♦ Boring and Sampling

Soil Test Boring with Flight Auger

Soil sampling and penetration testing were performed in general accordance with ASTM D1586, *Standard Test Method for Penetration Test and Split Barrel Sampling of Soils*. Borings were made by mechanically twisting a continuous steel flight auger into the soil. At regular intervals, soil samples were obtained with a standard 1.4-inch I. D., 2-inch O. D., split barrel sampler. The sampler was first seated six inches to penetrate any loose cuttings, then driven an additional 12 inches with blows of a 140-pound hammer falling 30 inches. The number of hammer blows required to drive the sampler through the two final six inch increments was recorded as the penetration resistance (SPT N) value. The N-value, when properly interpreted by qualified professional staff, is an index of the soil strength and foundation support capability.

Soil Test Boring with Hollow-Stem Auger

Soil sampling and penetration testing were performed in general accordance with ASTM D1586, *Standard Test Method for Penetration Test and Split Barrel Sampling of Soils*. Borings were made by mechanically twisting a continuous steel hollow stem auger into the soil. At regular intervals, soil samples were obtained with a standard 1.4-inch I. D., 2-inch O. D., split barrel sampler. The sampler was first seated six inches to penetrate any loose cuttings, then driven an additional 12 inches with blows of a 140-pound hammer falling 30 inches. The number of hammer blows required to drive the sampler through the two final six inch increments was recorded as the penetration resistance (SPT N) value. The N-value, when properly interpreted by qualified professional staff, is an index of the soil strength and foundation support capability.

Auger Borings

Auger borings were advanced mechanically by a drill rig using a flight auger or hollow stem auger in general accordance with ASTM D1452, *Standard Practice for Soil Investigation and Sampling by Auger Borings*. The soils encountered were identified in the field by examining the cuttings brought to the surface. Soil consistency was qualitatively estimated by the relative difficulty of advancing the augers.

Hand Auger Borings

Auger borings were advanced using hand operated augers. The soils encountered were identified in the field by cuttings brought to the surface. Representative samples of the cuttings were placed in glass jars and later transported to the laboratory. Soil consistency was qualitatively estimated by the relative difficulty of advancing the augers. At selected intervals, the augers were withdrawn and soil consistency measured with a dynamic cone penetrometer. The conical point of the penetrometer was first seated 1¾ inches to penetrate any loose cuttings in the boring, then driven two additional 1¾-inch increments by a 15 pound hammer falling 20 inches. The number of hammer blows required to achieve this penetration was recorded. When properly evaluated by qualified professional staff, the blow count is an index to the soil strength and ability to support foundations.

Kessler Dynamic Cone Penetrometer (DCP)

The DCP test involves driving a rod with a tip that is a 0.79-inch outside diameter, 60° cone into the ground using a 10.1-pound hammer, free-falling through a drop of 22.6 inches. The average depth (inch) driven by each hammer blow over each approximately 2-inch sampling interval is defined as the DCP Index. Softer/looser soils will yield a higher DCP Index than more stiff/dense soils. The DCP indices can be correlated to California Bearing Ratio (CBR) values based on extensive US Army Corps of Engineers research data.

Undisturbed (UD) Sampling

Split spoon or split barrel sampling provide samples suitable for visual examination and classification tests but not sufficiently intact for quantitative laboratory testing. To provide samples for quantitative tests, relatively undisturbed samples were obtained by pushing sections of 3-inch O.D., 16-guage, steel tubing (Shelby tube) into the soil at the desired sampling intervals. The procedures used generally followed those described in ASTM D1587, *Standard Practice for Thin-Walled Tube Geotechnical Sampling of Soils*. Each tube, together with the encased soil, was carefully removed from the ground and the length of the recovered soil measured. Locations and depths of undisturbed samples were recorded on each field test boring record.

Refusal to Drilling

Refusal to the soil drilling methods used at this site may result from encountering hard cemented soil, soft weathered rock, coarse gravel, cobbles or boulders, thin rock seams, or the upper surface of sound continuous rock. Core drilling would be required to determine the character and continuity of materials below refusal of the soil auger in natural soils. Where fills are present, refusal to drilling may also result from encountering buried debris, building materials, or objects. Backhoe test pits would be required to expose and identify buried materials below refusal levels in filled areas.

Rock Core Drilling in Uncased Borehole

In selected borings where refusal to the drilling tools had been encountered, materials below refusal level were cored using a diamond studded bit fastened to the end of hollow double tube core barrel. Hollow stem augers were left in place to stabilize the borehole and the core barrel inserted through the annulus of the drilling rod. Coring was conducted in general accordance with the procedures described in ASTM D2113, *Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigation*. In this case an NX size core barrel was used to produce cylindrical cores 1-7/8 inches in diameter. Core rod RPM and advance rate were closely monitored during each run to prevent plugging the bit or core blockage or damage. Water without additives was trucked to the site and circulated through the boring to flush cuttings and cool the drill bit during the coring process. Circulating water was released on the surface after completion of coring.

Borehole Closure

Boreholes in areas subject to foot traffic or farm animals were closed immediately after drilling. Boreholes were filled by slowly pouring auger cuttings into the open hole such that minimal “bridging” of the material occurred in the hole. Backfilling of the upper two feet of each hole was tamped as heavily as possible with a shovel handle or other hand held equipment, and the backfill crowned to direct rainfall away on the surface. Where boreholes exceeded five feet in depth, a plastic hole plug was firmly tamped into place within the backfill at a depth of about two feet.

Patching of Asphalt Surface

Penetrations of asphalt surfaces made during the drilling process were patched using compacted asphalt cold patch material. Cold patch asphalt was placed to provide a surface flush with existing pavement adjacent to the boring. Cold patch asphalt was compacted by tamping it into the boring with a shovel handle or similar hand held equipment.

Preservation and Transporting of Soil Samples with Control of Field Moisture

Procedures for preserving soil samples obtained in the field and transportation of samples to the laboratory generally followed those given in ASTM D4220, *Standard Practice for Preserving and Transporting Soil Samples* for Group B samples as defined in Section 4. Group B samples are those samples not suspected of being contaminated and for which only water content and classification, proctor, relative density, or profile logging will be performed. Group B samples also include bulk samples that are intended to be remolded in the laboratory for compaction, swell pressure, percent swell, consolidation, permeability, CBR, or shear testing. Representative samples of the cuttings or split spoon samples, or representative bulk samples, were placed in suitably identified, sealed glass jars or plastic containers and transported to the laboratory. Sample identification numbers on the containers corresponded to sample numbers recorded on field boring records or test pit records. Thin-walled tube samples were sealed at the ends with paraffin and capped with plastic end caps.

Preservation and Transporting of Intact Soil Samples

Procedures for preserving certain selected soil samples obtained in the field and transportation of those samples to the laboratory generally followed procedures given in ASTM D4220, *Standard Practice for Preserving and Transporting Soil Samples* for Group C samples as defined in Section 4. Group C samples are intact, naturally formed or field fabricated, samples for density determination, swell pressure, percent swell, permeability testing or shear testing with or without stress-strain plots or volume change measurement, including dynamic and cyclic testing. Representative thin walled tube samples were protected against vibration or shock, or extreme heat or cold, during transport to the laboratory. Sample identification numbers on the containers corresponded to sample numbers recorded on field boring records or test pit records. Thin-walled tube samples were sealed at the ends with paraffin and capped with plastic end caps. Samples were transported in the upright position in containers providing complete encasement in cushioning or insulation for individual samples.

Preservation and Transport of Rock Core Requiring Routine Care

Procedures for preserving recovered rock core specimens followed those given for routine care of non-sensitive, non-fragile samples for which only general visual examination will be performed. Steps for routine care are described in ASTM D5079, *Standard Practices for Preserving and Transporting Rock Core Samples*, section 7.5.1. Rock cored in 5 to 10 foot runs were placed in sleeves or channels in specially constructed wood or cardboard core boxes. Empty portions of sleeves or channels were packed with wood or paper to prevent slippage of the core during transport. Boxes were transported flat and secured to prevent sliding or vibration. A preliminary field log of each core indicating recovery and general visual description was prepared prior to packing of the core.

♦ Field Tests of Earth Materials

The subsurface conditions encountered during drilling were reported on a field test boring record by the chief driller. The record contains information about the drilling method, samples attempted and sample recovery, indications of materials in the borings such as coarse gravel, cobbles, etc., and indications of materials encountered between sample intervals. Representative soil samples were placed in glass jars and transported to the laboratory along with the field boring records. Recovered samples not expended in laboratory tests are commonly retained in our laboratory for 60 days following completion of drilling. Field boring records are retained at our office.

Measurement of Static Water Levels

Water level readings were made in the open boreholes immediately after completing drilling and withdrawal of the tools. Where feasible, measurements were repeated after an elapsed period of 24 hours to gauge the stabilized water level. Procedures for measurement of liquid levels in open boreholes are described in ASTM D4750, *Standard Test Method for Determining Subsurface Liquid Levels in a Borehole or Monitoring Well (Observation Well)*. A weighted measuring tape was slowly lowered into each borehole until the liquid surface was penetrated by the weighted end. The reading on the tape was recorded at a reference point on the surface and compared to the reading at the demarcation of the wetted and unwetted portions of the tape. The difference between the two readings was recorded as the depth of the liquid surface below the reference point. Measurements made by this method were then repeated until approximately consistent values were obtained.

Appendix III– Laboratory Testing

SUMMARY OF LABORATORY TEST RESULTS

BORING	SAMPLE (#)	Top Depth (ft)	MC %	LL %	PL %	PI %	AGGREGATE %	COARSE SAND %	FINE SAND %	SILT %	CLAY %	SILT/CLAY %	D / 50 mm	D / 95 mm	HRB ODOT CLASSIFICATION
B-01-0-24		1.00	5												
B-01-0-24		2.50	10	20	14	6	66	24	9			1	4.6481	22.7705	A-1-a (0)
B-01-0-24		4.00	7												
B-01-0-24		5.50	14	24	14	10	19	22	18	17	24		0.2080	10.9440	A-4a (1)
B-01-0-24		7.00	21	40	17	23	4	8	17	21	50		0.0048	1.6148	A-6b (12)
B-01-0-24		9.00	20												
B-01-0-24		10.50	19	39	23	16	5	4	3	22	66			2.2023	A-6b (10)
B-01-0-24		12.00	21												
B-01-0-24		13.50	15	39	22	17	16	5	2	17	60			14.7075	A-6b (11)
B-01-0-24		15.00	8												
B-01-0-24		16.75													
B-01-0-24		20.17													
B-01-0-24		25.17													
B-01-0-24		30.17													

PROJECT 116664
LOCATION WAR 350 8.30
JOB NO. 24780134
DATE 10/18/24

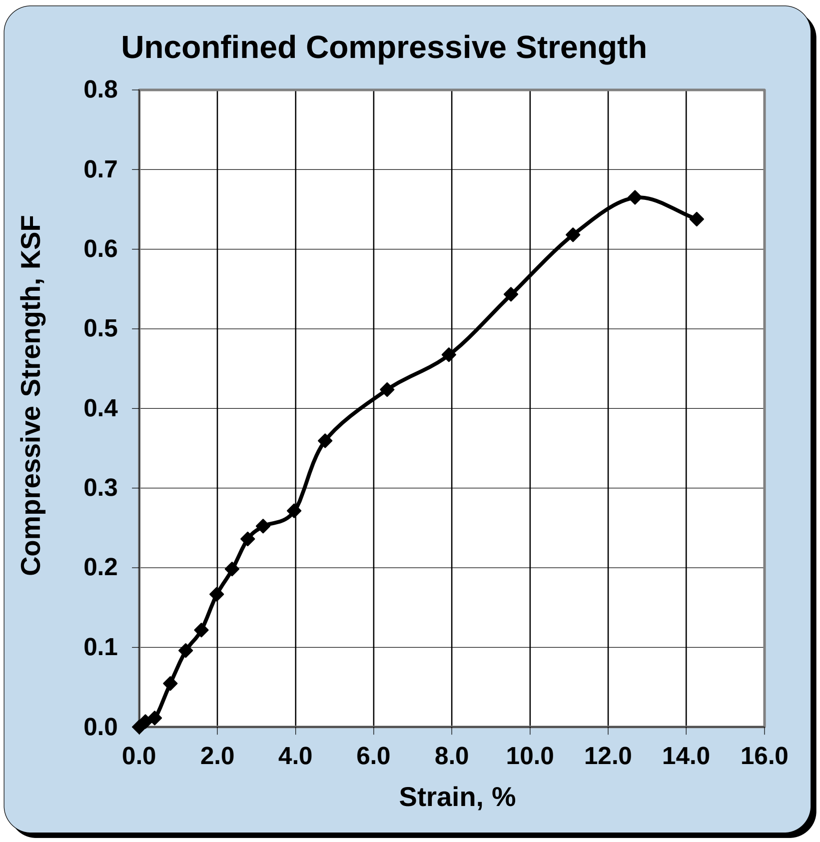
Form No. TR-D2166-01
Revision No. : 1
Revision Date: 08/16/17

UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS

ASTM D2166

S&ME, Inc. Cincinnati: 862 East Crescentville Road, West Chester, OH 45246

Project No.:	24780134	Report Date:	10/18/2024
Project Name:	WAR 350 8.30 PID 116664	Test Date(s):	9/30/2024
Client Name:	Stantec Consulting Services, Inc.		
Client Address:	10200 Alliance Road, Siute 300, Cincinnati, OH 45242		
Location:	B-001 Sample No. ST-1	Sample Date:	9/25/2024
		Depth:	7.0-9.0
Sample Description:	Silty Clay (A-6b)		



Failed Specimen

Type of Sample: Intact
Source of Moisture Sample: Test Specimen

Liquid Limit: 40
Plasticity Index: 23
Height to Diameter Ratio: 2.2
Rate of Strain (%/min.): 0.0086
Strain at Failure: 14.3

Initial Dry Unit Weight: 106.1 pcf   Initial Water Content: 18.7%
Unconfined Compressive Strength, q_u: **0.665** KSF
Undrained Shear Strength, s_u: **0.332** KSF

References / Comments / Deviations:

K. Cannady	[signature]	QAS	10/18/2024
Technical Responsibility	*Signature*	*Position*	*Date*

Form No. TR-D7012C-01
Revision No. : 1LexC
Revision Date: 12/12/23

UNIAXIAL COMPRESSIVE STRENGTH OF ROCK

ASTM D7012 Method C

Quality Assurance

S&ME, Inc. - Lexington: 2020 Liberty Road, Suite 105, Lexington, KY 40505

Project No.:	24780134	Report Date:	10/14/24
Project Name:	WAR 350 8.30 PID 116664	Test Date(s):	10/11/24
Client Name:	Stantec Consulting Services Inc.		
Client Address:	10200 Alliance Road, Suite 300, Cincinnati, OH 45242	Received Date:	09/25/24
Location:	B-001-0-24	Depth, ft:	23.5-23.9
Sample Description:	Shale		

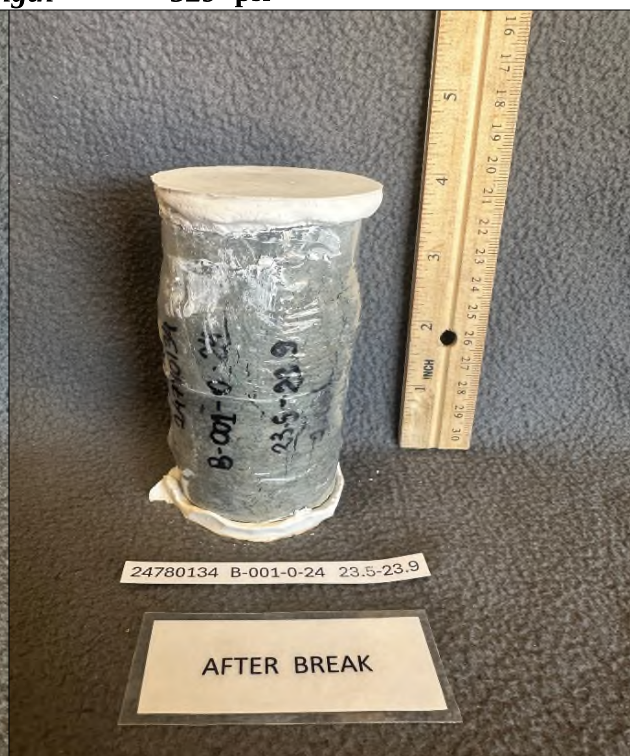
Angle of load relative to lithology: Approximately perpendicular

Test Results

Moisture Content ***6.6*** **%** ***Dry Unit Weight*** ***141.9*** **pcf**

Compressive Strength ***529*** **psi**

Before Test



After Test

Strain rate: 0.015 in/min.

Notes / Deviations / References: **Per ASTM D4543 7.6, for specimens which have physical characteristics which prevent planing the sample to within the flatness requirement it is permissible to cap them with gypsum. This specimen was too fragile for planing to within the flatness requirement and was capped with gypsum.**

J. Folsom	Jacob Folsom	Lab Services Manager	10/14/2024
Technical Responsibility	*Signature*	*Position*	*Date*

Form No. TR-D7012C-01
Revision No. 0
Devision Date: 06/25/15

UNIAXIAL COMPRESSIVE STRENGTH OF ROCK

ASTM D 7012 Method C

S&ME, Inc. - Lexington: 2020 Liberty Road, Suite 105, Lexington, KY 40505

Project Name: WAR 350 8.30 PID 116664 Location: B-001-0-24 Depth, feet: 23.5-23.9

Summary of Specimen Tolerances

Length/diameter target:	**MET**	Perpendicularity target:	**MET**
Side straightness target:	**MET**	Planeness target:	**CAPPED**
Parallelism target:	**CAPPED**		

*ASTM D4543-08 Standard Practice for Preparing Rock Core as Cylindrical Test Specimens and Verifying Conformance to Dimensional and Shape Tolerance, Section 1.2 - "Rock is a complex engineering material that can vary greatly as a function of lithology, stress history, weathering, moisture content, chemistry, and other natural geologic processes. As such, it is not always possible to obtain or prepare rock core specimens that satisfy the desirable tolerances given in this practice. Most commonly, this situation presents itself with weaker, more porous, and poorly cemented rock types and rock types containing significant or weak (or both) structural features. For these and other rock types which are difficult to prepare, all reasonable efforts shall be made to prepare a specimen in accordance with this practice and for the intended test procedure. However, when it has been determined by trial that this is not possible, the rock specimen will be prepared to the closest tolerance practicable and be considered the best effort and report it as such. If allowable or necessary for the intended test, capping the ends of the specimen as discussed in ASTM D7012 is permitted."

Length to Diameter Ratio

Length, inches: 4.25 Diameter, inches: 1.988
Ratio: 2.14 length to 1 diameter
Target tolerance: L:D ratio between 2 to 1 and 2.5 to 1

Side Straightness

Maximum gap between side of core and reference plate, inches: < .02
Target tolerance: Maximum gap less than .02 inches

Planeness

Dial gauge reading, inches

Not Applicable - Capped

Distance along diameter, inches

Maximum point-line deviation, inches: NA
Target Tolerance: No individually measured point should deviate from the best fit line by more than .001 inches.

Perpendicularity

Slope of End 1, Diameter 1, degrees:	NA
Slope of End 2, Diameter 1, degrees:	NA
Slope of End 1, Diameter 2, degrees:	NA
Slope of End 2, Diameter 2, degrees:	NA

Target Tolerance: Each diameter perpendicular to the long axis to within 0.25º

Parallelism

Slope difference, Diameter 1, degrees: NA-
Slope difference, Diameter 2, degrees: CAPPED
Target Tolerance: Difference between slopes on each end less than 0.25º

Test Information

Strain rate, in/min:	0.015
OR	
Stress rate, lbs/sec:	
Time to failure, min:	8.88
Temperature:	room temperature

Form No. TR-D7012C-01
Revision No. : 1LexC
Revision Date: 12/12/23

UNIAXIAL COMPRESSIVE STRENGTH OF ROCK

ASTM D7012 Method C

Quality Assurance

S&ME, Inc. - Lexington: 2020 Liberty Road, Suite 105, Lexington, KY 40505

Project No.:	24780134	Report Date:	10/14/24
Project Name:	WAR 350 8.30 PID 116664	Test Date(s):	10/11/24
Client Name:	Stantec Consulting Services Inc.		
Client Address:	10200 Alliance Road, Suite 300, Cincinnati, OH 45242	Received Date:	09/25/24
Location:	B-001-0-24	Depth, ft:	27.3-27.7
Sample Description:	Shale		

Angle of load relative to lithology: Approximately perpendicular

Test Results

Moisture Content ***4.9*** **%** ***Dry Unit Weight*** ***148.1*** **pcf**

Compressive Strength ***1,706*** **psi**

Before Test

After Test

Strain rate: 0.015 in/min.

Notes / Deviations / References: **Per ASTM D4543 7.6, for specimens which have physical characteristics which prevent planing the sample to within the flatness requirement it is permissible to cap them with gypsum. This specimen was too fragile for planing to within the flatness requirement and was capped with gypsum.**

J. Folsom	Jacob Folsom	Lab Services Manager	10/14/2024
Technical Responsibility	*Signature*	*Position*	*Date*

Form No. TR-D7012C-01
Revision No. 0
Devision Date: 06/25/15

UNIAXIAL COMPRESSIVE STRENGTH OF ROCK

ASTM D 7012 Method C

S&ME, Inc. - Lexington: 2020 Liberty Road, Suite 105, Lexington, KY 40505

Project Name: WAR 350 8.30 PID 116664 Location: B-001-0-24 Depth, feet: 27.3-27.7

Summary of Specimen Tolerances

Length/diameter target:	**MET**	Perpendicularity target:	**MET**
Side straightness target:	**MET**	Planeness target:	**CAPPED**
Parallelism target:	**CAPPED**		

*ASTM D4543-08 Standard Practice for Preparing Rock Core as Cylindrical Test Specimens and Verifying Conformance to Dimensional and Shape Tolerance, Section 1.2 - "Rock is a complex engineering material that can vary greatly as a function of lithology, stress history, weathering, moisture content, chemistry, and other natural geologic processes. As such, it is not always possible to obtain or prepare rock core specimens that satisfy the desirable tolerances given in this practice. Most commonly, this situation presents itself with weaker, more porous, and poorly cemented rock types and rock types containing significant or weak (or both) structural features. For these and other rock types which are difficult to prepare, all reasonable efforts shall be made to prepare a specimen in accordance with this practice and for the intended test procedure. However, when it has been determined by trial that this is not possible, the rock specimen will be prepared to the closest tolerance practicable and be considered the best effort and report it as such. If allowable or necessary for the intended test, capping the ends of the specimen as discussed in ASTM D7012 is permitted."

Length to Diameter Ratio

Length, inches: 4.36 Diameter, inches: 1.979
Ratio: 2.20 length to 1 diameter
Target tolerance: L:D ratio between 2 to 1 and 2.5 to 1

Side Straightness

Maximum gap between side of core and reference plate, inches: < .02
Target tolerance: Maximum gap less than .02 inches

Planeness

Dial gauge reading, inches

Not Applicable - Capped

Distance along diameter, inches

Maximum point-line deviation, inches: NA
Target Tolerance: No individually measured point should deviate from the best fit line by more than .001 inches.

Perpendicularity

Slope of End 1, Diameter 1, degrees:	NA
Slope of End 2, Diameter 1, degrees:	NA
Slope of End 1, Diameter 2, degrees:	NA
Slope of End 2, Diameter 2, degrees:	NA

Target Tolerance: Each diameter perpendicular to the long axis to within 0.25º

Parallelism

Slope difference, Diameter 1, degrees: NA-
Slope difference, Diameter 2, degrees: CAPPED
Target Tolerance: Difference between slopes on each end less than 0.25º

Test Information

Strain rate, in/min:	0.015
OR	
Stress rate, lbs/sec:	
Time to failure, min:	9.57
Temperature:	room temperature

Form No: TR-D2974-1
Revision No. 1
Revision Date: 08/26/17

MOISTURE, ASH, AND ORGANIC MATTER

ASTM D-2974

S&ME, Inc. Cincinnati: 862 East Crescentville Road, West Chester, OH 45246

Project #:	24780134	Report Date:	10/18/24
Project Name:	WAR 350 8.30 PID 116664	Test Date(s):	09/27/24
Client Name:	Stantec Consulting Services Inc,		
Client Address:	10200 Alliance Road, Suite 300, Cincinnati, OH 45242		

Boring No.	B-001	Sample No.	SS-3	Sample Date:	9/25/24
Location:		Sampled by:		Depth (ft.)	4.0-5.5
Sample Description:					

Equipment: *Balance: 0.01 g readability*

Balance: *S&ME ID #:* *26700* *Cal. Date:* *1/24/24* *Due:* *01/24/25*

Method A: Moisture Content Determination *Required Oven Temperature :110 $\pm$ 5 ^{o} C*

Oven Temperature: 110 °C		*Tare #*	36
t	Tare Weight *(Dish plus Aluminum Foil Cover)*	*grams*	29.76
a	Mass of *As-Received* Specimen + Tare Wt.	*grams*	101.13
b	Mass of Oven Dry Specimen + Tare Wt.	*grams*	96.42
w	Water Weight	*(a-b)*	4.71
A	Mass of As-Received Specimen	*(a-t)*	71.37
B	Mass of Oven Dry Specimen	*(b-t)*	66.66
	% Moisture Content as a % of As Received or Total Mass	*(w/A)*100*	**6.6%**
	% Moisture Content as a % of Oven-dried Mass	*(w/B)*100*	**7.1%**

Oven *S&ME ID #:* *27008* *Cal. Date:* *12/15/23* *Due:* *12/15/24*

Method C (440 ^{o} C) or D (750 ^{o} C): Ash Content and Organic Matter Determination

Muffle Furnace: 440 °C		*Tare #*	B
t	Tare Weight *(Dish plus Aluminum Foil Cover)*	*grams*	89.71
b	Mass of Oven Dry Specimen + Tare Wt.	*grams*	156.38
c	Ash Weight + Tare Wt.	*grams*	156.02
C	Ash Weight	*c-t*	66.31
B	Mass of Oven Dry Specimen	*(b-t)*	66.67
D	***% Ash Content***	*(C/B)*100*	**99.5%**
	% Organic Matter	*100-D*	**0.5%**

Muffle Furnace: *S&ME ID #:* *24239* *Cal. Date:* *Due:*

Notes / Deviations / References: ASTM D2974: Moisture, Ash, and Organic Matter of Peat and Other Organic Soils

B. Roberts	9/27/2024	on file	T4
Technician Name	*Date*	*Signature*	*Level/Certification*
K.Cannady	Kusu Cannady	QAS	10/18/2024
Technical Responsibility	*Signature*	*Position*	*Date*

Summary of Laboratory Procedures

Recovered disturbed and undisturbed samples and the drillers' field logs were transported to the laboratory where they were examined by the geotechnical engineer. Selected samples representative of certain groups of soils were subjected to simple classification tests by hand or other simple means.

Recovered disturbed and undisturbed samples and the drillers' field logs were transported to the laboratory where they were examined by the geotechnical engineer. Selected samples representative of certain groups of soils were subjected to simple classification tests by hand or other simple means. Other samples were tested in the laboratory to determine their strength or consolidation properties.

♦ Laboratory Tests of Soil

Examination of Split Spoon Soil Samples

Soil and rock samples and field boring records were reviewed in the laboratory by the geotechnical engineer. Soils were classified in general accordance with the visual-manual method described in ASTM D 2488, *Standard Practice for Description and Identification of Soils (Visual-Manual Method)*. The geotechnical engineer also prepared the final boring records enclosed with this report.

Examination of Split Spoon Soil Samples

Soil and rock samples and field boring records were reviewed in the laboratory by the geotechnical engineer. Representative soil samples were selected for classification testing to provide grain size and plasticity data to allow classification of the samples in general accordance with the AASHTO Classification method described in ASTM D3282, *Standard Practice for Classification of Soils and Soil Aggregate Mixtures for Highway Construction Purposes*. The geotechnical engineer also prepared the final boring records enclosed with this report.

Extrusion and Examination of Group C Undisturbed Samples

Undisturbed samples were stored in the vertical position in the laboratory. Samples were extruded from the thin-walled sampler, using a specially constructed extruder, in the same direction of travel as the sample entered the tube during sampling. In certain cases it was necessary to cut the tube into short sections to facilitate removal of the soil without compressing or disturbing the sample. Specimens were trimmed using a wire saw or steel straightedge. Where removal of pebbles or crumbling resulting from trimming caused voids on the surface of the specimens selected for quantitative laboratory testing, they were filled with remolded soil obtained from the trimmed portion of the sample.

Moisture Content Testing of Soil Samples by Oven Drying

Moisture content was determined in general conformance with the methods outlined in ASTM D2216, "Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil or Rock by Mass." This method is limited in scope to Group B, C, or D samples of earth materials which do not contain appreciable amounts of organic material, soluble solids such as salt or reactive solids such as cement. This method is also limited to samples which do not contain contamination.

A representative portion of the soil was divided from the sample using one of the methods described in Section 9 of ASTM D2216. The split portion was then placed in a drying oven and heated to approximately 110 degrees C overnight or until a constant mass was achieved after repetitive weighing. The moisture content of the soil was then computed as the mass of water removed from the sample by drying, divided by the mass of the sample dry, times 100 percent. No attempt was made to exclude any particular particle size from the portion split from the sample.

Liquid and Plastic Limits Testing

Atterberg limits of the soils was determined generally following the methods described by ASTM D4318, *Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils*. Albert Atterberg originally defined "limits of consistency" of fine grained soils in terms of their relative ease of deformation at various moisture contents. In current engineering usage, the liquid limit of a soil is defined as the moisture content, in percent, marking the upper limit of viscous flow and the boundary with a semi-liquid state. The plastic limit defines the lower limit of plastic behavior, above which a soil behaves plastically below which it retains its shape upon drying. The plasticity index (PI) is the range of water content over which a soil behaves plastically. Numerically, the PI is the difference between liquid limit and plastic limit values.

Representative portions of fine grained Group A, B, C, or D samples were prepared using the wet method described in Section 10.1 of ASTM D4318. The liquid limit of each sample was determined using the multipoint method (Method A) described in Section 11. The liquid limit is by definition the moisture content where 25 drops of a hand operated liquid limit device are required to close a standard width groove cut in a soil sample placed in the device. After each test, the moisture content of the sample was adjusted and the sample replaced in the device. The test was repeated to provide a minimum of three widely spaced combinations of N versus moisture content. When plotted on semilog paper, the liquid limit moisture content was determined by straight line interpolation between the data points at N equals 25 blows.

The plastic limit was determined using the procedure described in Section 17 of ASTM D4318. A selected portion of the soil used in the liquid limit test was kneaded and rolled by hand until it could no longer be rolled to a 3.2 mm thread on a glass plate. This procedure was repeated until at least 6 grams of material was accumulated, at which point the moisture content was determined using the methods described in ASTM D2216.

Grain Size Analysis of Samples

The distribution of particle sizes greater than 75 µm was determined in general accordance with the procedures described by ASTM D421, *Standard Practice for Dry Preparation of Soil Samples for Particle-Size Analysis and Determination of Soil Constants*, and D422, *Standard Test Method for Particle Size Analysis of Soils*. During preparation samples were divided into two portions. The material coarser than the No. 30 U.S. sieve size fraction was dry sieved through a nest of standard sieves as described in Article 6. Material passing the No. 30 sieve was independently passed through a nest of sieves down to the No. 200 size.

Grain Size Analysis of Samples with Hydrometer

The distribution of particle sizes was determined in general accordance with the procedures described by ASTM D421, *Standard Practice for Dry Preparation of Soil Samples for Particle-Size Analysis and Determination of Soil Constants*, and D422, *Standard Test Method for Particle Size Analysis of Soils*. During preparation samples were divided into two portions. The material coarser than the No. 10 U.S. sieve size fraction was dry sieved through a nest of standard sieves as described in Article 6. Material passing the No. 10 sieve was soaked in demineralized water and a dispersing agent, then the soil-water slurry placed in a glass sedimentation chamber and the specific gravity of the slurry recorded at various time intervals. The grain size distribution was calculated from the time rate of sedimentation of the various size particles. After the final hydrometer reading was obtained, the suspension was washed through the No. 200 sieve. The remaining material retained on the No. 200 sieve was oven dried, and then passed through a standard nest of sieves.

Percent Fines Determination of Samples

A selected specimen of soils was washed over a No. 200 sieve after being thoroughly mixed and dried. This test was conducted in general accordance with ASTM D1140, *Standard Test Method for Amount of Material Finer Than the No. 200 Sieve*. Method A, using water to wash the sample through the sieve without soaking the sample for a prescribed period of time, was used and the percentage by weight of material washing through the sieve was deemed the “percent fines” or percent clay and silt fraction.

Percent Organics (Organic Loss on Ignition)

The content of relatively undecayed or undecomposed vegetative matter in the soils is determined for representative samples of topsoil or stained subsoils using the procedures described by AASHTO T-267, *Determination of Organic Content in Soils by Loss on Ignition*. Representative samples of the minus No. 10 sieve size are dried at 105 C, then heated in a muffle furnace at 455 C for six hours. The resulting dry weight of the sample after ignition is then compared to the pre-ignition dry weight to estimate the organic content.

Unconfined Compressive Strength Tests of Intact Cohesive Samples

The unconfined compressive strength of intact cohesive soils was determined generally following the procedures described by ASTM D2166, *Standard Test Method for Unconfined Compressive Strength of Cohesive Soil*. Specimens were extracted from Shelby tube and trimmed on ends as necessary to provide a surface perpendicular to the specimen’s long axis, but the ends were not capped.

The prepared sample was placed in a compressive testing machine and the specimen compressed in the platen at a rate of 1 to 2 percent strain per minute. Deformation and loading of the sample were recorded at regular intervals until the load values began to decrease with increasing axial strain, or a total strain of 15 percent of the original sample length was attained. Sample stress was corrected at each load increment for the change in cross sectional area produced by deformation of the sample using the formulae in sections 8.2 and 8.3 of ASTM D2166.

♦ Laboratory Tests of Rock

Examination of Rock Core Specimens

Rock core samples returned to the laboratory were examined by the geotechnical engineer or geologist and the percentage recovery and rock quality designation (RQD) estimated for each run. A core run is defined either as 1) a drill run defined by the length of the core barrel; 2) a change in formation or rock type could constitute the end of a core run; or 3) a core run can be a selected zone of concern. Core run lengths are indicated on the attached boring records.

The "recovery" is the ratio of the sample length recovered in the core barrel to the total length of the core run, expressed as a percent. Rock Quality Designation is described by ASTM D6032, *Standard Test Method for Determining Rock Quality Designation (RQD) of Rock Core*. The RQD is the percentage of the core run consisting of moderately hard or harder NX-sized rock core recovered in segments 4 inches long or longer. When properly interpreted by a qualified professional, the RQD value provides a basis for preliminary design decisions involving foundations or excavation in rock.

Only those pieces or rock formed by natural joints, bedding planes, shear zones, or cleavage planes that result in surfaces of separation were considered for RQD purposes. Pieces formed by breaks in the core due to drilling or handling were not considered. Pieces were considered intact when they appeared to have been bonded together prior to coring and broken surfaces consisted of fresh rock. Where a surface could not be determined as either a natural or mechanical break, it was considered a natural break.

Rock core specimens were classified based on the following characteristics:

Hardness	Description of Core
Soft Rock	May be broken with fingers
Moderately Soft	May be scratched by a nail, corners and edges may be broken with fingers
Moderately Hard	Light blow of hammer required to break sample
Hard Rock	Hard blow of hammer required to break sample
Very Hard	Rock core rings when struck by hammer

Continuity	Core Recovery in Percent
Incompetent	Less than 40 percent
Competent	40 – 70 percent
Fairly Continuous	70 – 90 percent
Continuous	90 – 100 percent

Rock Quality	Rock Quality Designation
Very Poor	0 - 25 percent
Poor	25 – 50 percent
Fair	50 – 75 percent
Good	75 – 90 percent
Excellent	90 – 100 percent

Weathering	Description
Fresh	Rock fresh, crystals bright, some joints may show slight staining
Very Slight	Joints stained, some joints may show thin clay coatings
Slight	Joints stained and rock discolored up to 1 inch from joint surfaces

Moderate	Significant discoloration and weathering effects
Moderately Severe	All rock except quartz discolored and stained
Severe	Rock severely discolored and stained, few intact pieces remain
Very Severe	Rock fabric remains but reduced in strength to strong soil

Detailed rock descriptions, percent recovery, RQD values and the core barrel or bit size used are shown on the appropriate boring records in the Appendix.

Unconfined Compressive Strength Tests of Intact Rock Core

The unconfined compressive strength of intact rock core specimens will be determined generally following the procedures described in ASTM D7012, *Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures*. Selected recovered samples of intact rock core representative of each run will be cut to length and the ends machined flat. Specimens will then be placed in a loading frame and axial load continuously applied until peak load and failure are obtained. Specimens selected for testing will meet shape and L/D proportions outlined in ASTM D4543, *Standard Practice for Preparing Rock Core Specimens and Determining Dimensional and Shape Tolerances*. The specimen minimum dimension should be at least six to ten times the maximum particle or mineral dimension, and the L/D ratio at least 2 to 2.5. Samples will be soaked prior to testing.

APPENDIX C
SLOPE STABILITY BACK ANALYSIS

Material Property Estimation Using N_{60} Values

B-001-0-24

Top Depth	Bot Depth	Avg SPT Depth	Blows 1st	Blows 2nd	Blows 3rd	N-value	Hammer Eff. (%)	N_{60}	ODOT Class.	σ'_v (ksf)	Unit Weight (pcf)	C_N	$N1_{60}$	ϕ (AASHTO)	ODOT Correction	ϕ (ODOT)	S_u (psf)
1	2.5	2	6	16	12	28	85.9	40	A-1-a	0.226	123	1.73	69	40.5	2.5	43	N/A
2.5	4	3.5	3	4	6	10	85.9	14	A-1-a	0.396	123	1.54	22	37	2.5	39.5	N/A
4	5.5	5	3	5	7	12	85.9	17	A-4a	0.565	123	1.42	24	37.75	-2.5	35.25	N/A
5.5	7	6.5	1	2	2	4	85.9	6	A-4a	0.626	123	1.39	8	31.5	-2.5	29	N/A
9	10.5	10	6	7	9	16	85.9	23	A-6b	1.381	126	1.13	26	38.5	#N/A	#N/A	2863.33333
10.5	12	11.5	7	7	9	16	85.9	23	A-6b	1.477	126	1.10	25	38.125	#N/A	#N/A	2863.33333
12	13.5	13	5	7	13	20	85.9	29	A-6b	1.572	126	1.08	31	40.025	#N/A	#N/A	3579.16667
13.5	15	14.5	18	21	28	49	85.9	70	A-6b	1.668	126	1.06	75	40.5	#N/A	#N/A	N/A

ϕ (ODOT) Average 36.6875

B-001			
	Water Depth	5.5	ft
	Unit Weight Water	62.4	pcf

Unit Weight Estimation using N_{60} Values

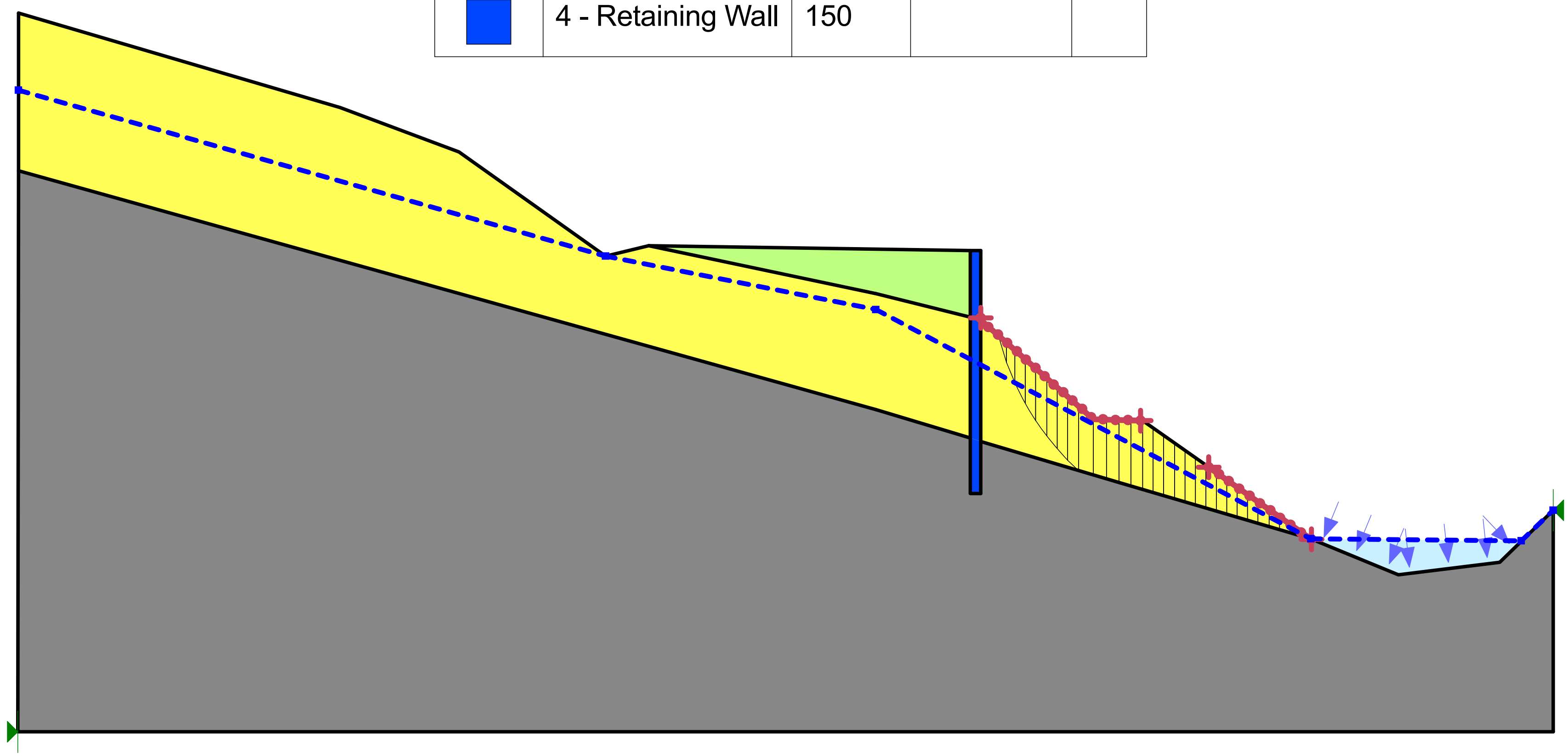
B-001-0-23

N60	ODOT Class.	Unit Wt	Type	Layer Average
40	A-1-a	130	g	
14	A-1-a	122	g	
17	A-4a	125	g	
6	A-4a	115	g	123
23	A-6b	125	c	
23	A-6b	125	c	
29	A-6b	128	c	126
70	A-6b	140	c	

Table 400-4: Soil Unit Weight Estimated from N_{60}

Cohesive N_{60}	Granular N_{60}	γ_{tot} (pcf)
0	-	100
1	-	105
2	-	108
3	0	110
4	-	112
5-6	1-2	115
7-9	3-5	118
10-13	6-8	120
14-19	9-14	122
20-27	15-24	125
28-35	25-34	128
36-39	35-44	130
40-43	45-54	132
44-51	55-64	135
52+	65+	140

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	1 - Granular Fill	123	0	33
	2 - Cohesive Soil	126	70	25
	3 - Bedrock			
	4 - Retaining Wall	150		



Slope Stability	
WAR-350_back analysis.gsz	
01/31/2025	1:200

Performed by: James Samples 1/31/2025
Reviewed by: Robert Lopina 1/31/2025

APPENDIX D
GABION WALL CALCULATIONS

Retaining Wall Analysis Using AASHTO LRFD Bridge Design Specifications

Stantec Consulting Services Inc.

Stantec

EXTERNAL STABILITY ANALYSIS
Stantec Project: **173401733**
Analysis Location: **WAR-350-8.30 Gabion Wall (3 baskets, top of leveling pad)**

Height of Wall above Footing (H1) =	9.0	feet
Thickness of Footing (T) =	0.0	feet
Height of Wall (H) =	9.0	feet
Unit Weight of Retained Soil Mass (γs) =	60.6	pcf
Unit Weight of Wall Mass (γm) =	100.0	pcf
Unit Weight of Water (γw) =	62.4	pcf
Live Load Surcharge (LS) =	0.0	psf
Friction Angle of Retained Soil (ϕf)=	30.0	degrees
Coefficient of Lateral Earth Pressure (Ka) =	0.45	
Length of Heel (L1) =	0.0	feet
Length of Toe (L2) =	0.0	feet
Width of Wall (W1) =	9.0	feet
Width of Footing (B) =	9.0	feet
Water Table Height Above Bottom of Footing (Wt) =	9.0	feet
Interface Friction Angle (δ) =	20.0	degrees
Angle of Wall Backface (θ) =	96.0	degrees
Horizontal Abutment Live Load, P(ALh) =	0.0	K/ft
Distance of P(ALh) Above Top of Wall (d)=	0.0	feet
Slope of Backfill (X:1) =	2.0	
Angle of Backfill (β) =	26.6	degrees
Backfill Height (H')=	9.0	feet

Buoyant weight used due to water table at top of wall
(see Table 3.5.1-1 on page 3-14)

AASHTO Recommended Load Factors for Retaining Wall Analyses
(see Table 3.4.1-1 & -2)

	Limit State	EV	EH	LS	DC	WA
(Eccentricity, Sliding)	Strength 1-a	1.00	0.90	1.75	0.90	1.00
(Bearing Capacity)	Strength 1-b	1.35	1.50	1.75	1.25	1.00
	Service 1	1.00	1.00	1.00	1.00	1.00

Permanent Loads:
DC = Dead Load of Structural Components
EH = Horizontal Earth Pressure
ES = Earth Surcharge Load
EV = Vertical Earth Pressure

Temporary Loads:
LL = Vehicular Live Load
LS = Live Load Surcharge
WA = Water Load

Vertical Loads and Moments (Unfactored)

Load	VL (K/ft)	Moment Arm X(0) About Toe (ft)	Mv (K-ft/ft)
P(EV)	0.0	9.02	0.0
P(EHv)	0.4	9.02	3.6
P(LSv)	0.0	9.02	0.0
P(DC)	8.1	4.51	36.5
P(WA)	0.0	9.02	0.0

Horizontal Loads and Moments (Unfactored)

Load	HL (K/ft)	Moment Arm X(0) About Toe (ft)	Mh (K-ft/ft)
P(EHh)	1.0	3.00	3.0
P(LSh)	0.0	4.50	0.0
P(ALh)	0.0	9.00	0.0
P(WA)	2.5	3.00	7.6

Factored Vertical Loads and Moments

Limit State	P(EV) (K/ft)	P(EH) (K/ft)	P(LSv) (K/ft)	P(DC) (K/ft)	P(WA) (K/ft)	Pv(TOT) (K/ft)	Mv (K-ft/ft)
Strength 1-a	0.0	0.4	0.0	7.3	0.0	7.7	36.1
Strength 1-b	0.0	0.6	0.0	10.1	0.0	10.7	51.0
Service 1	0.0	0.4	0.0	8.1	0.0	8.5	40.1

Factored Horizontal Loads and Moments

Limit State	P(EH) (K/ft)	P(LSh) (K/ft)	P(ALh) (K/ft)	P(WA) (K/ft)	Ph(TOT) (K/ft)	Mh (K-ft/ft)
Strength 1-a	0.9	0.0	0.0	2.5	3.4	10.3
Strength 1-b	1.5	0.0	0.0	2.5	4.0	12.1
Service 1	1.0	0.0	0.0	2.5	3.5	11.6

Eccentricity

Limit State	Pv(TOT) (K/ft)	Mv (K-ft/ft)	Ph(TOT) (K/ft)	Mh (K-ft/ft)	X(0) (ft)	e (B) (ft)	q (applied) (ksf)
Strength 1-a	7.7	36.1	3.4	10.3	3.35	1.16	1.1
Strength 1-b	10.7	51.0	4.0	12.1	3.64	0.87	1.5
Service 1	8.5	40.1	3.5	11.6	3.35	1.16	1.3

where:
X(0) = [Mv - Mh] / Pv(TOT)
e(B) = B/2 - X(0)
q(applied) = P(EV) / 2X(0)

Eccentricity Check:		
Maximum Design Eccentricity (e) =	1.16 (feet)	
Maximum Allowable Eccentricity =	2.26 (feet)	OK

Performed by: James Samples 1/28/2025
Checked by: Eric Kistner 1/29/2025

Sliding

Foundation Soil Type =	Granular	
Strength of Foundation Soil (Su) =	N/A	ksf
Resistance Factor for Sliding (Φs) =	1	(see Table 10.5.5.2.2-1)
Friction Angle of Foundation Soil (φf) =	34.0	deg.

Sliding Check:		
Sliding Force =	3.4 (ksf)	
Factored Sliding Resistance (Q_R) =	5.2 (ksf)	OK

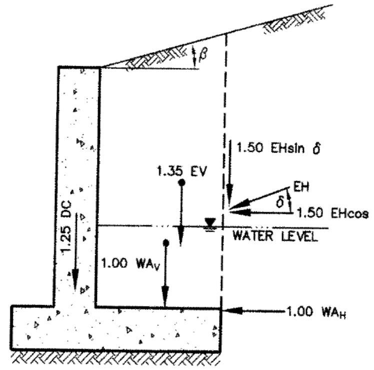


Figure C11.5.5-1 Typical Application of Load Factors for Bearing Resistance.

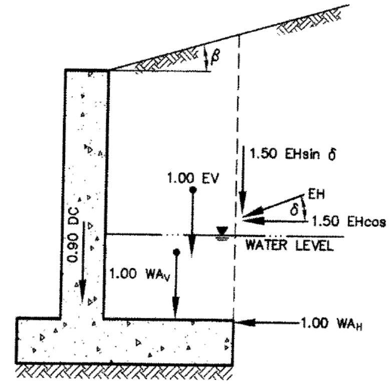


Figure C11.5.5-2 Typical Application of Load Factors for Sliding and Eccentricity.

Bearing Capacity

Foundation Soil Type =	Granular	
Resistance Factor for Bearing (Φb)=	0.55	(see Table 10.5.5.2.2-1)
Effective Footing Width (B') =	6.7	feet

	Cohesive Foundation Soil		Granular Foundation Soil		
Cohesion (~Su) of foundation soil (c) =	1.5	ksf	n/a	ksf	
Friction Angle of Foundation Soil (φf) =	0.0	degrees	34.0	degrees	
Bearing capacity factor (Nc) =	5.14		n/a		
Bearing capacity factor (Ngamma) =	0.0		41.0		(see Table 10.6.3.1.2c-2 on page 10-39)
Bearing capacity factor (Nq) =	0.0		29.0		(see Table 10.6.3.1.2c-2 on page 10-39)
Nominal Bearing Resistance (q ult)=	7.7	ksf	13.6	ksf	
Factored Unit Bearing Resistance (q R) =	4.2	ksf	7.5	ksf	

Group	Pv(TOT) (K/ft)	Mv (K-ft/ft)	Ph(TOT) (K/ft)	Mh (K-ft/ft)	X(0) (ft)	e (B)' (ft)	q (applied) (ksf)
Strength 1-a	7.7	36.1	3.4	10.3	3.4	1.1	1.1
Strength 1-b	10.7	51.0	4.0	12.1	3.6	0.9	1.5
Service 1	8.5	40.1	3.5	11.6	3.4	1.1	1.3

where:
X(0) = [Mv(TOT) - Mh(TOT)] / V(TOT)
e(B) = B/2 - X(0)
q(applied) = V(TOT) / 2X(0)

Bearing Check:		
Required Nominal Bearing Resistance =	1.5 (ksf)	
Factored Bearing Resistance (q R) =	7.5 (ksf)	OK

Bearing Capacity Factors

phi angle	N(gamma)	Nq
28.0	17	15
30.0	22	18
32.0	30	23
34.0	41	29
36.0	58	38
38.0	78	49
40.0	110	64
42.0	155	85
44.0	225	115
46.0	330	160

** Factors from Table 10.6.3.1.2c-2 (page 10-39) in the LRFD Bridge Design Specifications (2004)

Notes:
Modify cells in yellow only! Use necessary tables when appropriate.
Horizontal earth pressure is calculated using Coulomb's Active Earth Pressure Theory.
Embedment depth in granular bearing capacity calculations is assumed to be 3 feet.
The live load surcharge should only be used in this sheet when the backfill is horizontal.
Use cohesive/granular dropdown menus to select the appropriate equations for analyses.

Performed by: James Samples 1/28/2025
Checked by: Eric Kistner 1/29/2025

Retaining Wall Analysis Using AASHTO LRFD Bridge Design Specifications

Stantec Consulting Services Inc.

EXTERNAL STABILITY ANALYSIS
Stantec Project: 173401733
Analysis Location: WAR-350-8.30 Gabion Wall (3 baskets, bottom of leveling pad)

Height of Wall above Footing (H1) =	9.5	feet
Thickness of Footing (T) =	0.0	feet
Height of Wall (H) =	9.5	feet
Unit Weight of Retained Soil Mass (γs) =	60.6	pcf
Unit Weight of Wall Mass (γm) =	100.0	pcf
Unit Weight of Water (γw) =	62.4	pcf
Live Load Surcharge (LS) =	0.0	psf
Friction Angle of Retained Soil (ϕf)=	30.0	degrees
Coefficient of Lateral Earth Pressure (Ka) =	0.45	
Length of Heel (L1) =	0.0	feet
Length of Toe (L2) =	0.0	feet
Width of Wall (W1) =	9.0	feet
Width of Footing (B) =	9.0	feet
Water Table Height Above Bottom of Footing (Wt) =	9.5	feet
Interface Friction Angle (δ) =	20.0	degrees
Angle of Wall Backface (θ) =	96.0	degrees
Horizontal Abutment Live Load, P(ALh) =	0.0	K/ft
Distance of P(ALh) Above Top of Wall (d)=	0.0	feet
Slope of Backfill (X:1) =	2.0	
Angle of Backfill (β) =	26.6	degrees
Backfill Height (H')=	9.5	feet

Buoyant weight used due to water table at top of wall
(see Table 3.5.1-1 on page 3-14)

AASHTO Recommended Load Factors for Retaining Wall Analyses
(see Table 3.4.1-1 & -2 on page 3-12)

	Limit State	EV	EH	LS	DC	WA
(Eccentricity, Sliding)	Strength 1-a	1.00	0.90	1.75	0.90	1.00
(Bearing Capacity)	Strength 1-b	1.35	1.50	1.75	1.25	1.00
	Service 1	1.00	1.00	1.00	1.00	1.00

Permanent Loads:
DC = Dead Load of Structural Components
EH = Horizontal Earth Pressure
ES = Earth Surcharge Load
EV = Vertical Earth Pressure

Temporary Loads:
LL = Vehicular Live Load
LS = Live Load Surcharge
WA = Water Load

Vertical Loads and Moments (Unfactored)

Load	VL (K/ft)	Moment Arm X(0) About Toe (ft)	Mv (K-ft/ft)
P(EV)	0.0	9.02	0.0
P(EHv)	0.4	9.02	3.6
P(LSv)	0.0	9.02	0.0
P(DC)	8.6	4.51	38.8
P(WA)	0.0	9.02	0.0

Horizontal Loads and Moments (Unfactored)

Load	HL (K/ft)	Moment Arm X(0) About Toe (ft)	Mh (K-ft/ft)
P(EHh)	1.2	3.17	3.8
P(LSh)	0.0	4.75	0.0
P(ALh)	0.0	9.50	0.0
P(WA)	2.8	3.17	8.9

Factored Vertical Loads and Moments

Limit State	P(EV) (K/ft)	P(EH) (K/ft)	P(LSv) (K/ft)	P(DC) (K/ft)	P(WA) (K/ft)	Pv(TOT) (K/ft)	Mv (K-ft/ft)
Strength 1-a	0.0	0.4	0.0	7.7	0.0	8.1	38.2
Strength 1-b	0.0	0.6	0.0	10.8	0.0	11.4	53.9
Service 1	0.0	0.4	0.0	8.6	0.0	9.0	42.4

Factored Horizontal Loads and Moments

Limit State	P(EH) (K/ft)	P(LSh) (K/ft)	P(ALh) (K/ft)	P(WA) (K/ft)	Ph(TOT) (K/ft)	Mh (K-ft/ft)
Strength 1-a	1.1	0.0	0.0	2.8	3.9	12.3
Strength 1-b	1.8	0.0	0.0	2.8	4.6	14.6
Service 1	1.2	0.0	0.0	2.8	4.0	13.7

Eccentricity

Limit State	Pv(TOT) (K/ft)	Mv (K-ft/ft)	Ph(TOT) (K/ft)	Mh (K-ft/ft)	X(0) (ft)	e (B) (ft)	q (applied) (ksf)
Strength 1-a	8.1	38.2	3.9	12.3	3.20	1.31	1.3
Strength 1-b	11.4	53.9	4.6	14.6	3.45	1.06	1.7
Service 1	9.0	42.4	4.0	13.7	3.19	1.32	1.4

where:
X(0) = [Mv - Mh] / Pv(TOT)
e(B) = B/2 - X(0)
q(applied) = P(EV) / 2X(0)

Eccentricity Check:		
Maximum Design Eccentricity (e) =	1.32 (feet)	
Maximum Allowable Eccentricity =	2.26 (feet)	OK

Performed by: James Samples 1/28/2025
Checked by: Eric Kistner 1/29/2025

Sliding

Foundation Soil Type =	Granular	
Strength of Foundation Soil (Su) =	N/A	ksf
Resistance Factor for Sliding (Φs) =	0.9	(see Table 10.5.5.2.2-1)
Friction Angle of Foundation Soil (ϕf) =	34.0	deg.

Sliding Check:

Sliding Force =	3.9 (ksf)	
Factored Sliding Resistance (Q_R) =	4.9 (ksf)	OK

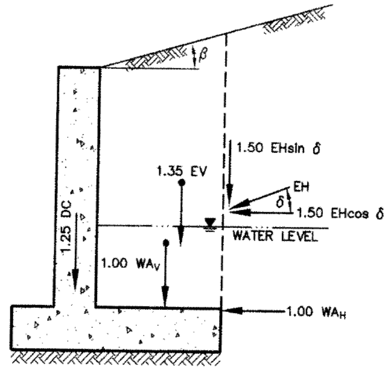


Figure C11.5.5-1 Typical Application of Load Factors for Bearing Resistance.

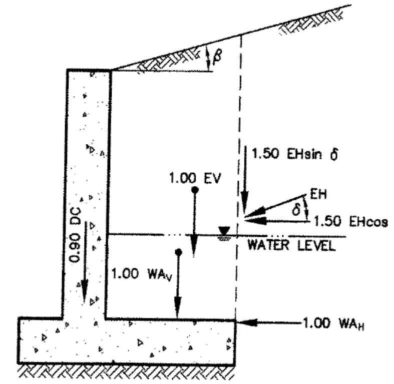


Figure C11.5.5-2 Typical Application of Load Factors for Sliding and Eccentricity.

Bearing Capacity

Foundation Soil Type =	Granular	
Resistance Factor for Bearing (Φb)=	0.45	(see Table 10.5.5.2.2-1)
Effective Footing Width (B') =	6.4	feet

	Cohesive Foundation Soil		Granular Foundation Soil	
Cohesion (~Su) of foundation soil (c) =	1.5	ksf	n/a	ksf
Friction Angle of Foundation Soil (ϕf) =	0.0	degrees	34.0	degrees
Bearing capacity factor (Nc) =	5.14		n/a	
Bearing capacity factor (Ngamma) =	0.0		41.0	(see Table 10.6.3.1.2c-2 on page 10-39)
Bearing capacity factor (Nq) =	0.0		29.0	(see Table 10.6.3.1.2c-2 on page 10-39)
Nominal Bearing Resistance (q ult)=	7.7	ksf	13.2	ksf
Factored Unit Bearing Resistance (q R) =	3.5	ksf	5.9	ksf

Group	Pv(TOT) (K/ft)	Mv (K-ft/ft)	Ph(TOT) (K/ft)	Mh (K-ft/ft)	X(0) (ft)	e (B)' (ft)	q (applied) (ksf)
Strength 1-a	8.1	38.2	3.9	12.3	3.2	1.3	1.3
Strength 1-b	11.4	53.9	4.6	14.6	3.4	1.1	1.7
Service 1	9.0	42.4	4.0	13.7	3.2	1.3	1.4

where:
X(0) = [Mv(TOT) - Mh(TOT)] / V(TOT)
e(B) = B/2 - X(0)
q(applied) = V(TOT) / 2X(0)

Bearing Check:

Required Nominal Bearing Resistance =	1.7 (ksf)	
Factored Bearing Resistance (q R) =	5.9 (ksf)	OK

Bearing Capacity Factors

phi angle	N(gamma)	Nq
28.0	17	15
30.0	22	18
32.0	30	23
34.0	41	29
36.0	58	38
38.0	78	49
40.0	110	64
42.0	155	85
44.0	225	115
46.0	330	160

** Factors from Table 10.6.3.1.2c-2 (page 10-39) in the LRFD Bridge Design Specifications (2004)

Notes: Modify cells in yellow only! Use necessary tables when appropriate.
Horizontal earth pressure is calculated using Coulomb's Active Earth Pressure Theory.
Embedment depth in granular bearing capacity calculations is assumed to be 3 feet.
The live load surcharge should only be used in this sheet when the backfill is horizontal.
Use cohesive/granular dropdown menus to select the appropriate equations for analyses.

Performed by: James Samples 1/28/2025
Checked by: Eric Kistner 1/29/2025

Retaining Wall Analysis Using AASHTO LRFD Bridge Design Specifications

Stantec Consulting Services Inc.

Stantec

EXTERNAL STABILITY ANALYSIS
Stantec Project: **173401733**
Analysis Location: **WAR-350-8.30 Gabion Wall (4 baskets, top of leveling pad)**

Height of Wall above Footing (H1) =	12.0	feet
Thickness of Footing (T) =	0.0	feet
Height of Wall (H) =	12.0	feet
Unit Weight of Retained Soil Mass (γs) =	60.6	pcf
Unit Weight of Wall Mass (γm) =	100.0	pcf
Unit Weight of Water (γw) =	62.4	pcf
Live Load Surcharge (LS) =	0.0	psf
Friction Angle of Retained Soil (φf)=	30.0	degrees
Coefficient of Lateral Earth Pressure (Ka) =	0.45	
Length of Heel (L1) =	0.0	feet
Length of Toe (L2) =	0.0	feet
Width of Wall (W1) =	9.0	feet
Width of Footing (B) =	9.0	feet
Water Table Height Above Bottom of Footing (Wt) =	9.0	feet
Interface Friction Angle (δ) =	20.0	degrees
Angle of Wall Backface (θ) =	96.0	degrees
Horizontal Abutment Live Load, P(ALh) =	0.0	K/ft
Distance of P(ALh) Above Top of Wall (d)=	0.0	feet
Slope of Backfill (X:1) =	2.0	
Angle of Backfill (β) =	26.6	degrees
Backfill Height (H')=	12.0	feet

Buoyant weight used due to water table at top of wall
(see Table 3.5.1-1 on page 3-14)

AASHTO Recommended Load Factors for Retaining Wall Analyses
(see Table 3.4.1-1 & -2)

	Limit State	EV	EH	LS	DC	WA
(Eccentricity, Sliding)	Strength 1-a	1.00	0.90	1.75	0.90	1.00
(Bearing Capacity)	Strength 1-b	1.35	1.50	1.75	1.25	1.00
	Service 1	1.00	1.00	1.00	1.00	1.00

Permanent Loads:
DC = Dead Load of Structural Components
EH = Horizontal Earth Pressure
ES = Earth Surcharge Load
EV = Vertical Earth Pressure

Temporary Loads:
LL = Vehicular Live Load
LS = Live Load Surcharge
WA = Water Load

Vertical Loads and Moments (Unfactored)

Load	VL (K/ft)	Moment Arm X(0) About Toe (ft)	Mv (K-ft/ft)
P(EV)	0.0	9.02	0.0
P(EHv)	0.7	9.02	6.3
P(LSv)	0.0	9.02	0.0
P(DC)	10.8	4.51	48.7
P(WA)	0.0	9.02	0.0

Horizontal Loads and Moments (Unfactored)

Load	HL (K/ft)	Moment Arm X(0) About Toe (ft)	Mh (K-ft/ft)
P(EHh)	1.8	4.00	7.2
P(LSh)	0.0	6.00	0.0
P(ALh)	0.0	12.00	0.0
P(WA)	2.5	3.00	7.6

Factored Vertical Loads and Moments

Limit State	P(EV) (K/ft)	P(EH) (K/ft)	P(LSv) (K/ft)	P(DC) (K/ft)	P(WA) (K/ft)	Pv(TOT) (K/ft)	Mv (K-ft/ft)
Strength 1-a	0.0	0.6	0.0	9.7	0.0	10.3	49.5
Strength 1-b	0.0	1.1	0.0	13.5	0.0	14.6	70.3
Service 1	0.0	0.7	0.0	10.8	0.0	11.5	55.0

Factored Horizontal Loads and Moments

Limit State	P(EH) (K/ft)	P(LSh) (K/ft)	P(ALh) (K/ft)	P(WA) (K/ft)	Ph(TOT) (K/ft)	Mh (K-ft/ft)
Strength 1-a	1.6	0.0	0.0	2.5	4.1	14.1
Strength 1-b	2.7	0.0	0.0	2.5	5.2	18.4
Service 1	1.8	0.0	0.0	2.5	4.3	15.8

Eccentricity

Limit State	Pv(TOT) (K/ft)	Mv (K-ft/ft)	Ph(TOT) (K/ft)	Mh (K-ft/ft)	X(0) (ft)	e (B) (ft)	q (applied) (ksf)
Strength 1-a	10.3	49.5	4.1	14.1	3.44	1.07	1.5
Strength 1-b	14.6	70.3	5.2	18.4	3.55	0.96	2.1
Service 1	11.5	55.0	4.3	15.8	3.41	1.10	1.7

where:
X(0) = [Mv - Mh] / Pv(TOT)
e(B) = B/2 - X(0)
q(applied) = P(EV) / 2X(0)

Eccentricity Check:		
Maximum Design Eccentricity (e) =	1.10 (feet)	
Maximum Allowable Eccentricity =	2.26 (feet)	OK

Performed by: James Samples 1/28/2025
Checked by: Eric Kistner 1/29/2025

Sliding

Foundation Soil Type =	Granular	
Strength of Foundation Soil (Su) =	N/A	ksf
Resistance Factor for Sliding (Φs) =	1	(see Table 10.5.5.2.2-1)
Friction Angle of Foundation Soil (φf) =	34.0	deg.

Sliding Check:
Sliding Force = 4.1 (ksf)
Factored Sliding Resistance (Q_R) = 6.9 (ksf) **OK**

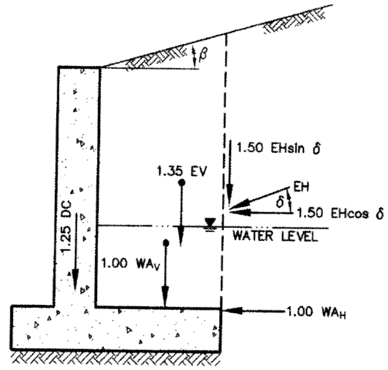


Figure C11.5.5-1 Typical Application of Load Factors for Bearing Resistance.

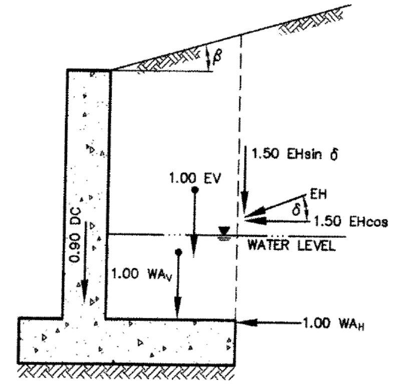


Figure C11.5.5-2 Typical Application of Load Factors for Sliding and Eccentricity.

Bearing Capacity

Foundation Soil Type =	Granular	
Resistance Factor for Bearing (Φb)=	0.55	(see Table 10.5.5.2.2-1)
Effective Footing Width (B') =	6.8	feet

	Cohesive Foundation Soil		Granular Foundation Soil		
Cohesion (~Su) of foundation soil (c) =	1.5	ksf	n/a	ksf	
Friction Angle of Foundation Soil (φf) =	0.0	degrees	34.0	degrees	
Bearing capacity factor (Nc) =	5.14		n/a		
Bearing capacity factor (Ngamma) =	0.0		41.0		(see Table 10.6.3.1.2c-2 on page 10-39)
Bearing capacity factor (Nq) =	0.0		29.0		(see Table 10.6.3.1.2c-2 on page 10-39)
Nominal Bearing Resistance (q ult)=	7.7	ksf	13.7	ksf	
Factored Unit Bearing Resistance (q R) =	4.2	ksf	7.6	ksf	

Group	Pv(TOT) (K/ft)	Mv (K-ft/ft)	Ph(TOT) (K/ft)	Mh (K-ft/ft)	X(0) (ft)	e (B)' (ft)	q (applied) (ksf)
Strength 1-a	10.3	49.5	4.1	14.1	3.4	1.1	1.5
Strength 1-b	14.6	70.3	5.2	18.4	3.6	0.9	2.0
Service 1	11.5	55.0	4.3	15.8	3.4	1.1	1.7

where:
X(0) = [Mv(TOT) - Mh(TOT)] / V(TOT)
e(B) = B/2 - X(0)
q(applied) = V(TOT) / 2X(0)

Bearing Check:
Required Nominal Bearing Resistance = 2.0 (ksf)
Factored Bearing Resistance (q R) = 7.6 (ksf) **OK**

Bearing Capacity Factors

phi angle	N(gamma)	Nq
28.0	17	15
30.0	22	18
32.0	30	23
34.0	41	29
36.0	58	38
38.0	78	49
40.0	110	64
42.0	155	85
44.0	225	115
46.0	330	160

** Factors from Table 10.6.3.1.2c-2 (page 10-39) in the LRFD Bridge Design Specifications (2004)

Notes:
Modify cells in yellow only! Use necessary tables when appropriate.
Horizontal earth pressure is calculated using Coulomb's Active Earth Pressure Theory.
Embedment depth in granular bearing capacity calculations is assumed to be 3 feet.
The live load surcharge should only be used in this sheet when the backfill is horizontal.
Use cohesive/granular dropdown menus to select the appropriate equations for analyses.

Performed by: James Samples 1/28/2025
Checked by: Eric Kistner 1/29/2025

Retaining Wall Analysis Using AASHTO LRFD Bridge Design Specifications

Stantec Consulting Services Inc.

EXTERNAL STABILITY ANALYSIS
Stantec Project: **173401733**
Analysis Location: **WAR-350-8.30 Gabion Wall (4 baskets, bottom of leveling pad)**

Height of Wall above Footing (H1) =	12.5	feet
Thickness of Footing (T) =	0.0	feet
Height of Wall (H) =	12.5	feet
Unit Weight of Retained Soil Mass (γs) =	60.6	pcf
Unit Weight of Wall Mass (γm) =	100.0	pcf
Unit Weight of Water (γw) =	62.4	pcf
Live Load Surcharge (LS) =	0.0	psf
Friction Angle of Retained Soil (ϕf)=	30.0	degrees
Coefficient of Lateral Earth Pressure (Ka) =	0.45	
Length of Heel (L1) =	0.0	feet
Length of Toe (L2) =	0.0	feet
Width of Wall (W1) =	9.0	feet
Width of Footing (B) =	9.0	feet
Water Table Height Above Bottom of Footing (Wt) =	9.5	feet
Interface Friction Angle (δ) =	20.0	degrees
Angle of Wall Backface (θ) =	96.0	degrees
Horizontal Abutment Live Load, P(ALh) =	0.0	K/ft
Distance of P(ALh) Above Top of Wall (d)=	0.0	feet
Slope of Backfill (X:1) =	2.0	
Angle of Backfill (β) =	26.6	degrees
Backfill Height (H')=	12.5	feet

Buoyant weight used due to water table at top of wall
(see Table 3.5.1-1 on page 3-14)

AASHTO Recommended Load Factors for Retaining Wall Analyses
(see Table 3.4.1-1 & -2 on page 3-12)

	Limit State	EV	EH	LS	DC	WA
(Eccentricity, Sliding)	Strength 1-a	1.00	0.90	1.75	0.90	1.00
(Bearing Capacity)	Strength 1-b	1.35	1.50	1.75	1.25	1.00
	Service 1	1.00	1.00	1.00	1.00	1.00

Permanent Loads:
DC = Dead Load of Structural Components
EH = Horizontal Earth Pressure
ES = Earth Surcharge Load
EV = Vertical Earth Pressure

Temporary Loads:
LL = Vehicular Live Load
LS = Live Load Surcharge
WA = Water Load

Vertical Loads and Moments (Unfactored)

Load	VL (K/ft)	Moment Arm X(0) About Toe (ft)	Mv (K-ft/ft)
P(EV)	0.0	9.02	0.0
P(EHv)	0.7	9.02	6.3
P(LSv)	0.0	9.02	0.0
P(DC)	11.3	4.51	51.0
P(WA)	0.0	9.02	0.0

Horizontal Loads and Moments (Unfactored)

Load	HL (K/ft)	Moment Arm X(0) About Toe (ft)	Mh (K-ft/ft)
P(EHh)	2.0	4.17	8.3
P(LSh)	0.0	6.25	0.0
P(ALh)	0.0	12.50	0.0
P(WA)	2.8	3.17	8.9

Factored Vertical Loads and Moments

Limit State	P(EV) (K/ft)	P(EH) (K/ft)	P(LSv) (K/ft)	P(DC) (K/ft)	P(WA) (K/ft)	Pv(TOT) (K/ft)	Mv (K-ft/ft)
Strength 1-a	0.0	0.6	0.0	10.2	0.0	10.8	51.6
Strength 1-b	0.0	1.1	0.0	14.1	0.0	15.2	73.2
Service 1	0.0	0.7	0.0	11.3	0.0	12.0	57.3

Factored Horizontal Loads and Moments

Limit State	P(EH) (K/ft)	P(LSh) (K/ft)	P(ALh) (K/ft)	P(WA) (K/ft)	Ph(TOT) (K/ft)	Mh (K-ft/ft)
Strength 1-a	1.8	0.0	0.0	2.8	4.6	16.4
Strength 1-b	3.0	0.0	0.0	2.8	5.8	21.4
Service 1	2.0	0.0	0.0	2.8	4.8	18.2

Eccentricity

Limit State	Pv(TOT) (K/ft)	Mv (K-ft/ft)	Ph(TOT) (K/ft)	Mh (K-ft/ft)	X(0) (ft)	e (B) (ft)	q (applied) (ksf)
Strength 1-a	10.8	51.6	4.6	16.4	3.26	1.25	1.7
Strength 1-b	15.2	73.2	5.8	21.4	3.41	1.10	2.2
Service 1	12.0	57.3	4.8	18.2	3.26	1.25	1.8

where:
X(0) = [Mv - Mh] / Pv(TOT)
e(B) = B/2 - X(0)
q(applied) = P(EV) / 2X(0)

Eccentricity Check:		
Maximum Design Eccentricity (e) =	1.25 (feet)	
Maximum Allowable Eccentricity =	2.26 (feet)	OK

Performed by: James Samples 1/28/2025
Checked by: Eric Kistner 1/29/2025

Sliding

Foundation Soil Type =	Granular	
Strength of Foundation Soil (Su) =	N/A	ksf
Resistance Factor for Sliding (Φs) =	0.9	(see Table 10.5.5.2.2-1)
Friction Angle of Foundation Soil (ϕf) =	34.0	deg.

Sliding Check:		
Sliding Force =	4.6 (ksf)	
Factored Sliding Resistance (Q_R) =	6.6 (ksf)	OK

Bearing Capacity

Foundation Soil Type =	Granular	
Resistance Factor for Bearing (Φb)=	0.45	(see Table 10.5.5.2.2-1)
Effective Footing Width (B') =	6.5	feet

	Cohesive Foundation Soil		Granular Foundation Soil	
Cohesion (~Su) of foundation soil (c) =	1.5	ksf	n/a	ksf
Friction Angle of Foundation Soil (ϕf) =	0.0	degrees	34.0	degrees
Bearing capacity factor (Nc) =	5.14		n/a	
Bearing capacity factor (Ngamma) =	0.0		41.0	(see Table 10.6.3.1.2c-2 on page 10-39)
Bearing capacity factor (Nq) =	0.0		29.0	(see Table 10.6.3.1.2c-2 on page 10-39)
Nominal Bearing Resistance (q ult)=	7.7	ksf	13.4	ksf
Factored Unit Bearing Resistance (q R) =	3.5	ksf	6.0	ksf

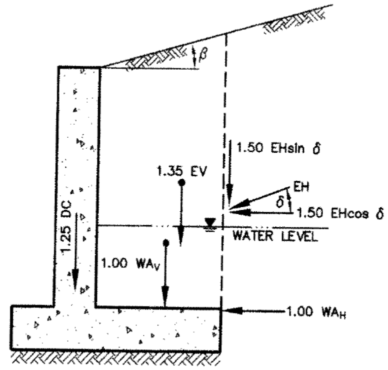


Figure C11.5.5-1 Typical Application of Load Factors for Bearing Resistance.

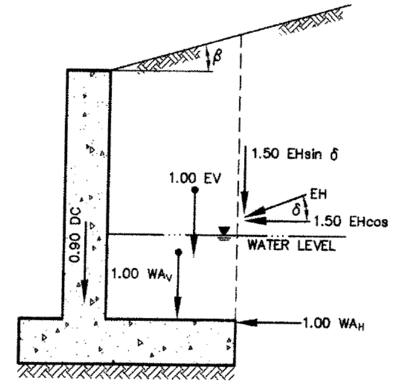


Figure C11.5.5-2 Typical Application of Load Factors for Sliding and Eccentricity.

Group	Pv(TOT) (K/ft)	Mv (K-ft/ft)	Ph(TOT) (K/ft)	Mh (K-ft/ft)	X(0) (ft)	e (B)' (ft)	q (applied) (ksf)
Strength 1-a	10.8	51.6	4.6	16.4	3.3	1.2	1.6
Strength 1-b	15.2	73.2	5.8	21.4	3.4	1.1	2.2
Service 1	12.0	57.3	4.8	18.2	3.3	1.2	1.8

where:
X(0) = [Mv(TOT) - Mh(TOT)] / V(TOT)
e(B) = B/2 - X(0)
q(applied) = V(TOT) / 2X(0)

Bearing Check:		
Required Nominal Bearing Resistance =	2.2 (ksf)	
Factored Bearing Resistance (q R) =	6.0 (ksf)	OK

Bearing Capacity Factors

phi angle	N(gamma)	Nq
28.0	17	15
30.0	22	18
32.0	30	23
34.0	41	29
36.0	58	38
38.0	78	49
40.0	110	64
42.0	155	85
44.0	225	115
46.0	330	160

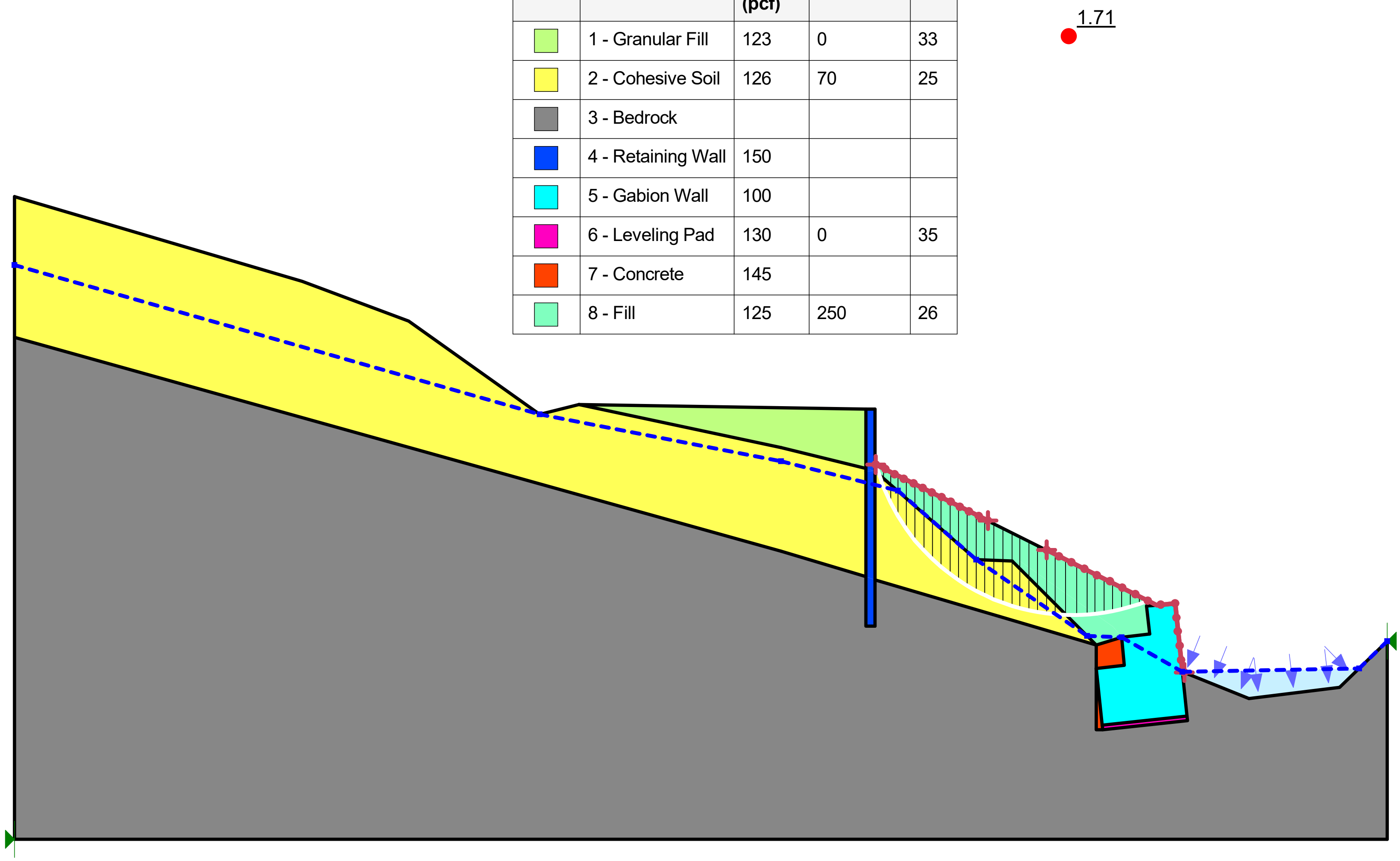
** Factors from Table 10.6.3.1.2c-2 (page 10-39) in the LRFD Bridge Design Specifications (2004)

Notes:
Modify cells in yellow only! Use necessary tables when appropriate.
Horizontal earth pressure is calculated using Coulomb's Active Earth Pressure Theory.
Embedment depth in granular bearing capacity calculations is assumed to be 3 feet.
The live load surcharge should only be used in this sheet when the backfill is horizontal.
Use cohesive/granular dropdown menus to select the appropriate equations for analyses.

Performed by: James Samples 1/28/2025
Checked by: Eric Kistner 1/29/2025

APPENDIX E
GLOBAL STABILITY ANALYSES

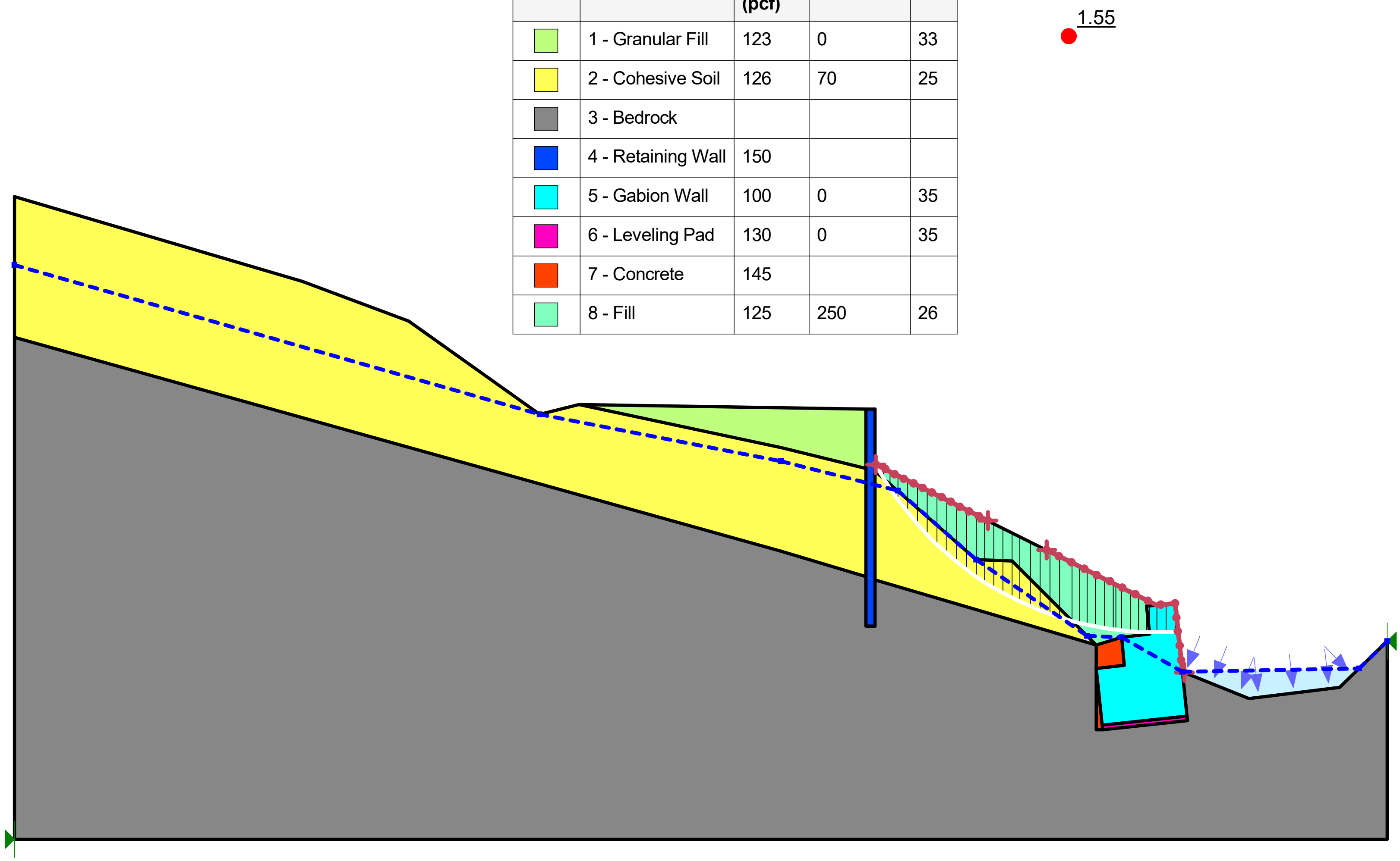
Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	1 - Granular Fill	123	0	33
	2 - Cohesive Soil	126	70	25
	3 - Bedrock			
	4 - Retaining Wall	150		
	5 - Gabion Wall	100		
	6 - Leveling Pad	130	0	35
	7 - Concrete	145		
	8 - Fill	125	250	26



Slope Stability	
WAR-350_gabion.gsz	
01/31/2025	1:210

Performed by: James Samples 1/31/2025
Reviewed by: Robert Lopina 1/31/2025

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	1 - Granular Fill	123	0	33
	2 - Cohesive Soil	126	70	25
	3 - Bedrock			
	4 - Retaining Wall	150		
	5 - Gabion Wall	100	0	35
	6 - Leveling Pad	130	0	35
	7 - Concrete	145		
	8 - Fill	125	250	26



Slope Stability

WAR-350_gabion.gsz

01/31/2025 1:210

Performed by: James Samples 1/31/2025
Reviewed by: Robert Lopina 1/31/2025