Pro Geotech, Inc.

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March 11, 2024

Mr. Tom Washko, P.E.
GPD Group
5595 Transportation Blvd. Suite 100
Cleveland, Ohio 44125

Reference: **Final Structure Foundation Exploration Report for Replacement of State Route 88 Bridge over Ohio Turnpike (MP 199.5)**
Portage County, Ohio
OTIC Project No.: 71-22-03
PGI Project No.: G23017G

Dear Mr. Washko:

Enclosed please find our Final Structure Foundation Exploration Report for the above referenced project. Our services included geotechnical field exploration, laboratory testing, engineering analysis, and related design and construction recommendations. These services have been provided in accordance with our proposal dated March 29, 2023. It is important that the items under "Limitations" be precisely followed and complied with.

We appreciate the opportunity of working with you on this project and we invite you to contact us at (440) 717-1415 when we can be of further assistance.

Respectfully,

PRO GEOTECH, INC.

Shan Sivakumaran, P.E.
Project Manager/Geotechnical Engineer

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Senior Geotechnical Engineer

Enclosure
G23017G SR 88 Bridge Report/SS/3/11/2024

FINAL
STRUCTURE FOUNDATION EXPLORATION REPORT
FOR
STATE ROUTE 88 BRIDGE OVER OHIO TURNPIKE (MP 199.5)
REPLACEMENT
PORTAGE COUNTY, OHIO
PGI PROJECT NO. G23017G

PREPARED FOR:

GPD GROUP

PREPARED BY:

PRO GEOTECH, INC.

MARCH 11, 2024

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1.0 EXECUTIVE SUMMARY

This report has been prepared for the proposed replacement of the State Route (SR) 88 Bridge over Ohio Turnpike, Portage County, Ohio. The bridge site is located along SR 88, approximately 3000 feet northeast from SR 303. A total of two (2) test borings identified as B-001-0-23 and B-002-0-23 were advanced at the project site for bridge foundation design purposes. Test borings B-001-0-23 and B-002-0-23 were advanced behind the proposed rear and forward abutments, respectively. These test borings were advanced to approximate depths ranging from 50.5 to 53.7 feet below the existing concrete deck surfaces.

Subsurface Conditions: The subsurface soil conditions were determined from the soil information obtained from test borings B-001-0-23 and B-002-0-23. The subsurface soils encountered in test borings B-001-0-23 and B-002-0-23 consisted of 13.5 feet and 18.5 feet, respectively, of fill/embankment soils over natural soils. The fill/embankment soils were cohesive in nature and consisted of sandy silt (A-4a), silt and clay (A-6a), and silty clay (A-6b). Natural soils encountered below the fill soils consisted primarily of cohesive soils including silt and clay (A-6a), silty clay (A-6b), plastic sandy silt (A-4a), and plastic silt (A-4b). One 4.5-foot-thick layer of non-cohesive soil consisting of coarse and fine sand (A-3a) was encountered in test boring B-001-0-23 between 35.0 and 39.5 feet, above the bedrock. Bedrock was encountered in test borings B-001-0-23 and B-002-0-23 at approximate depths of 39.5 feet and 41.0 feet, respectively below the existing pavement surface. Both test borings were terminated in bedrock after obtaining rock core samples.

Bedrock Condition: The claystone encountered in test borings B-001-0-23 and B-002-0-23 ranged from black to dark gray to light brown and was slightly to moderately weathered, very weak to weak, very thin to thin bedded, and highly to moderately fractured. The rock surface was slightly rough, and the aperture widths were tight. The siltstone encountered in test boring B-001-0-23 ranged from dark gray to light brown and from light brown to gray in test boring B-002-0-23. The siltstone in both test borings was slightly to moderately weathered, slightly to moderately strong, thin to thin bedded, and fractured to moderately fractured. The rock surface was slightly rough, and the aperture widths were tight. Two distinctly different shale deposits were encountered in test boring B-002-0-23; one at the top of Run 1 from 43.7 to 45.0 feet and one at the bottom of Run 2 from 48.0 to 53.7 feet. The shale at the top of Run 1 between 43.7 and 45.0 feet was black with light gray lenses that decreased in thickness and frequency from 43.7 to 45.0 feet. The shale at the bottom of Run 2 between 48.0 and 52.7 feet was gray with

occasional dark inclusions. Both shales were moderately to slightly weathered, very weak, laminated and moderately fractured.

Bridge Foundation Systems: Since bedrock was encountered relatively deeper depths at the rear and forward abutment locations, the proposed superstructure loads may be transferred to the underlying bedrock by means of deep foundations. Deep Foundation consisting of end bearing H-piles may be used to transfer the proposed superstructure loads to the underlying bedrock at the rear and forward abutment locations. MSE Wall will be constructed at the rear and forward abutment locations. The estimated pile parameters for end bearing piles at each boring location are summarized in Table 6.1.1. It is estimated that the maximum total settlement and differential settlement will not exceed one inch and one-half of an inch, respectively.

Table 6.1.1 - Estimated Design Parameters for H-Piles

Boring No.	Pile Cut-off Elevation (ft)	Pile Tip Elevation (ft)	Recommended Pile Order Length (ft)	Pile Type	Pile Size	Maximum Factored Structural Resistance/pile
B-001-0-23	1174.7	1139.7	40	H-Pile	10X42	310 kips
B-002-0-23	1172.7	1134.4	45	H-Pile	10X42	310 kips

MSE Wall Foundation Systems: The foundation soils encountered below the bottom of the MSE Wall consisted of natural soils and were generally cohesive in nature. As per the boring logs, the consistency of these cohesive soils ranged from “hard” to “stiff” but was generally “very stiff” to "hard”. Bearing capacity analyses (AASHTO Article 11.6.3) were performed using total stress soil parameters estimating the nominal bearing resistance of the reinforced MSE soil mass supported on foundation cohesive soils at rear and forward locations. Estimated factored bearing resistance is summarized in Table 6.2.1.

Table 6.2.1 – Estimated Design Parameters at Strength Limit State for MSE Walls

Boring No.	MSE Wall No.	Depth of Bottom of Footing Below Final Grade (feet)	Width of Footing (feet)	Proposed Bearing Elevation (ft.)	Factored Bearing Resistance (ksf)
B-001-0-23	1	3.5	22.0	1157.0	9.0
B-002-0-23	2	3.5	19.0	1158.0	8.0

The External Stability analyses results show that the Capacity Demand Ratio (CDR) value against sliding, CDR value with respect to bearing resistance and eccentricity values are greater than the one for the selected foundation width of the MSE Wall Nos. 1 & 2.

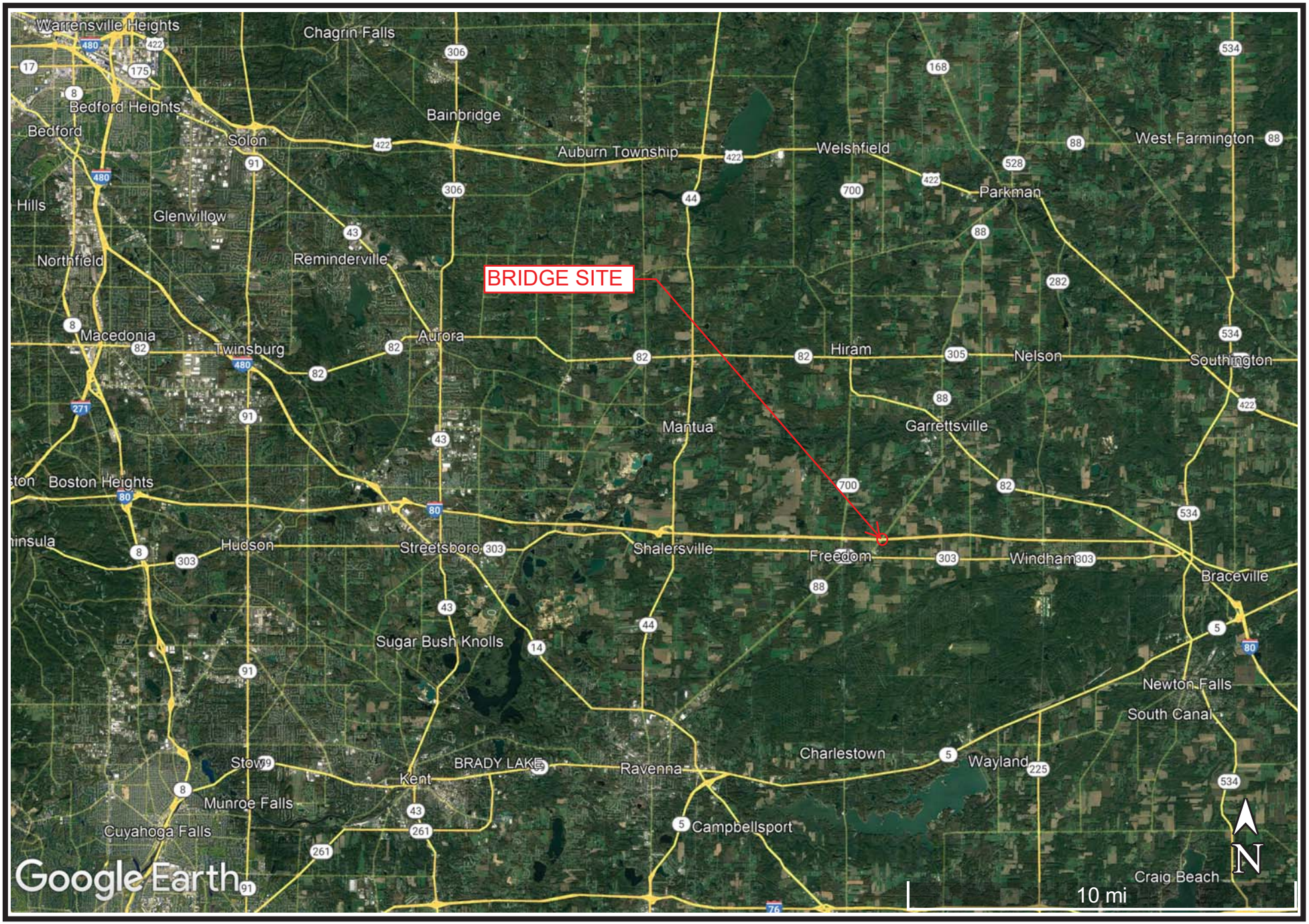
2.0 INTRODUCTION

This report has been prepared for the proposed replacement of the State Route (SR) 88 Bridge over Ohio Turnpike, Portage County, Ohio. The bridge site is located along SR 88, approximately 3000 feet northeast from SR 303. It represents the intent of GPD Group (GPD) the design engineer, and the Ohio Turnpike and Infrastructure Commission (OOTIC), the owner, to secure subsurface information at selected locations in accordance with the ODOT *Specifications for Geotechnical Explorations*, and to obtain recommendations regarding geotechnical factors pertaining to the design and construction of this project.

This report has been developed based on the field exploration program, laboratory testing and information provided by GPD personnel. It must be noted that, as with any exploration program, the site exploration identifies actual subsurface conditions only at those locations where samples were obtained. The data derived through sampling and laboratory testing is reduced by geotechnical engineers and geologists who then render an opinion regarding the overall subsurface conditions and their likely reaction on the site. The actual site conditions may differ from those inferred to exist. Therefore, although a fair amount of subsurface data has been assembled during this exploration, this report may not provide all of the geotechnical data needed for construction of this project. This report was prepared using English units.

2.1 Project Description

Present plans call for the replacement of Bridge which carries SR 88 vehicular traffic over the Ohio Turnpike in Portage County, Ohio. The bridge site is located at the Ohio Turnpike Mile Post of 199.5. Design information provided by GPD personnel indicates that the existing bridge consists of a four-span continuous composite steel beam supported by 2-span continuous steel carrying griders and reinforced concrete substructures supported on piles and drilled shafts. The total span length of the existing bridge is 242.5 feet. The proposed replacement structure will be a two-span structure with an approximate total length of 230.5 feet. The proposed superstructures will be continuous steel plate girders with composite reinforce concrete deck on semi-integral concrete stub abutments supported on piles behind the MSE walls and wall pier supported on existing drilled shafts. Mechanically Stabilized Earth (MSE) Wall System will be installed at the rear and forward abutment locations. The vertical alignment of the bridge will be more or less the same as the existing one. The bridge is to be designed based on HL-93 loading with 60 psf future wearing surface criteria and the ODOT Bridge Design Manual, issued in July 2023 which includes LRFD Bridge Design Specifications. The Site Location Map is shown in Figure 2.1.



SR 88 BRIDGE OVER OHIO TURNPIKE
PORTAGE COUNTY, OHIO
SITE LOCATION MAP (FIG 2.1)

2.2 Scope of Services

The scope of services for this project was in accordance with Pro Geotech, Inc. (PGI) Proposal No. PG23022 dated March 29, 2023 and governed by ODOT's *Specifications for Geotechnical Explorations (SGE)* dated July 2023, ODOT's Geotechnical Design Manual, dated July 20 2023 and ODOT's Bridge Design Manual, issued in 2020 and updated July 2023 which include AASHTO LRFD specifications, current edition, hereafter referred to as ODOT Specifications. Our scope of services consisted of the execution of the following tasks:

Phase I – Planning and Marking Test Borings, which primarily consisted of planning the field portion of our subsurface exploration, performing a site reconnaissance to evaluate the proposed project site from a geotechnical standpoint, reviewing and compiling all existing geology of the project site obtained from ODOT and ODNR sources, marking the test boring locations, obtaining necessary permits, and notifying the Ohio Utility Protection Services (OUPS) about the proposed drilling operations.

Phase II - Test Boring and Sampling Program, which primarily consisted of field verification of the test boring locations with regards to the underground utilities, advancing the test borings at the site, conducting field tests, sampling the subsurface materials, and preparing field drilling logs.

Our scope of services included advancing two (2) test borings at the project site. These test borings were to be advanced in the vicinity of rear and forward bridge abutments for foundation design purposes to approximate depth of 50.0 feet each below the existing pavement surface including obtaining 10 feet of rock core at each boring location. All test borings were advanced in accordance with the ODOT *Specifications for Geotechnical Explorations*. The groundwater conditions were monitored during and upon completion of the drilling operations. PGI provided all of the traffic control needed during the fieldwork.

Phase III - Testing Program, which consisted of performing soil classification and engineering properties tests on selected soil and rock samples and classifying the soils in accordance with the ODOT Soil Classification System.

Phase IV - Geotechnical Exploration Report, which included the following:

- A brief description of the project and our exploration methods
- Typed drilling logs and laboratory test results

- A description of subsurface soil, rock, and groundwater conditions
- Discussions pertaining to earthwork considerations, groundwater management, and construction monitoring
- Foundation recommendations for the bridge including shallow foundations
- Recommendations for MSE walls which will include external stability analysis, settlement, and lateral earth pressures

The scope of services did not include any environmental assessments for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater or air, on, below, or around this site. Any statement in this report or on the boring logs regarding odors, colors or unusual or suspicious items or conditions is strictly for the client's information.

3.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT SITE

3.1 Geology

Based on information obtained from the Physiographic Regions of Ohio map, the bridge site lies at approximate elevations ranging from 1160 feet to 1183 feet within the Killbuck-Glaciated Pittsburgh Plateau of the Glaciated Allegheny Plateaus Section of Ohio. This physiographic region is located within the Appalachian Plateaus Province. The Killbuck-Glaciated Pittsburgh Plateau is characterized by ridges and flat uplands generally above 1200 feet and is covered with thin drift and dissected by steep valleys that alternate between broad drift-filled and narrow rock-walled reaches with moderate relief of 200 feet or less. The geology of the Killbuck-Glaciated Pittsburgh Plateau region generally consists of thin to thick Wisconsinan-age clay to loam till over Mississippian-age and Pennsylvanian-age shales, sandstones, conglomerates and coals. According to Bulletin 44, *Geology of Water in Ohio*, the area was glaciated by the Wisconsin Glacier which deposited relatively thin drift materials ranging from three (3) to 30 feet in thickness and consisting largely of till. Based on the *Quaternary Geology of Ohio,* the main geologic deposit of the project site consists of Ground moraine, flat to gently undulating. Based on the *Soil Survey of Portage County, Ohio* Web Soil Survey Data, the site surface soils in the vicinity of the project area consist primarily of Mahoning silt loam and Udorthents or fill soils, placed during embankment construction for the Ohio Turnpike. Based on the AASHTO Soil Classification System, Mahoning soils generally consist of A-6 (lower plasticity clayey soils).

Based on information obtained from the Ohio Geological Survey, the top bedrock in the vicinity of the project site is anticipated to be present at approximate elevations ranging from 1135 feet to 1143 feet. At these elevations, bedrock is expected to consist of Pennsylvanian-age shale, claystone, siltstone and/or sandstone of the Allegheny and Pottsville Groups Undivided. The shales and siltstones are predominantly gray and black in color. According to Ohio Geological Survey's "Ohio Gas and Oil Wells Locator Interactive Map" no active or abandoned wells located within approximately 1000 feet of the project site. According to Ohio Geological Survey's "Ohio Mine Locator Interactive Map", there are no abandoned underground mines within 10 miles of the project site. According to Ohio Geological Survey's "Active Industrial Mineral Mine Locator Interactive Map", there are no surface mines within or adjacent to the project site. According to Ohio Geological Survey's "Karst Interactive Map", there is no karst region in the vicinity of the project site. According to Ohio Geological Survey's "Ohio Earthquake Epicenters Interactive Map", there have been four (4) earthquake epicenters which have occurred within a 15-mile radius of the project site. The magnitudes of these earthquakes ranged from 1.3 to 3.3 and occurred between 1987 and 2007.

The above soil and bedrock information has been obtained from the Physiographic Regions of Ohio, printed in April 1998, *Bedrock Geologic Map of Ohio* printed in 2006, *Quaternary Geology of Ohio* printed in 1999, *Geology of Water in Ohio (Bulletin 44)* issued in 1943 (reprinted in 1968), the *Soil Survey of Portage County, Ohio* Web Soil Survey published in 1978 by the *U.S. Natural Resources Conservation Service.*

3.2 Observations

The reconnaissance of the project site was performed by one of PGI's geotechnical engineers in September 2023. The project site is located in a rural area with the closest residential house located within an approximate distance of 100 feet from the bridge site. This section of SR 88 consists of one-lane with paved shoulders in each direction. The existing pavement surface consists of asphaltic concrete which appeared to be in fair to poor condition with some longitudinal and transverse cracks. The cracks appeared to be sealed. The existing bridge deck surface consists of concrete which appeared to be in fair condition. Broken concrete areas were observed in some places of the top bridge deck and appeared to be patched. Observed concrete surfaces of abutment and pier walls appeared to be in fair condition. Rust was observed on the steel beams below the bridge deck. Underground water and sewer lines and overhead power lines are present at the project site.

4.0 EXPLORATION

4.1 Historic and Project Exploration Program

Two (2) historical records of geotechnical explorations were available from the OTIC. The first record is for the original design and construction of the bridge which was constructed in 1953 under the project designation of OTC Contract No. C-10. A total of three (3) historic borings identified as B-1, B-4, and B-8 are available for this bridge. N_{60}-values from SPT tests and soil descriptions were not included in these the historic borings. However, the top bedrock elevations were shown in these historic borings B-1 (B-001-0-53), B-4 (B-004-0-53), and B-8 (B-008-0-53). The second record includes a structure foundation exploration performed in 1996 for the third lane widening project between MP 199.05 and 205.46 under the project designation of OTIC Contract 77-96-06. A total of two (2) historic test borings identified as B-1and B-2 are available for this bridge. N_{60}-values from SPT tests and soil descriptions were not included in these historic boring columns. Rock core samples were obtained from these historic borings B-1 (B-001-0-96) and B-2 (B-002-0-96). Recovery, RQD, top of rock elevations, and unconfined compressive strength information are shown on these boring columns. Bedrock information from these historic borings will be used to design the proposed bridge foundations. All the relevant historic information is included in Appendix B.

In order to explore the subsurface conditions at the project site, drilling, sampling, and field-testing operations were performed in September 2023. A total of two (2) test borings identified as B-001-0-23 and B-002-0-23 were advanced at the project site for bridge foundation design purposes. Test borings B-001-0-23 and B-002-0-23 were advanced behind the proposed rear and forward abutments, respectively. These test borings were advanced to approximate depths ranging from 50.5 to 53.7 feet below the existing concrete deck surfaces. These pavement core's locations were shown on the "Boring Location Map" included in Appendix A.

The test boring and pavement core locations were marked in the field by PGI personnel after getting approval from GPD personnel. Site geometry, utility locations, overhead height, and accessibility were also taken into account when locating the test borings**.** At the time of test boring location selection, the vertical soil sampling intervals were determined based on the needs for design and construction of the project. An ATV track mounted drill rig; Acker Recon was used to advance the test borings. All borings were advanced using 3.25-inch inside diameter, continuous flight hollow stem augers (HSA). Representative disturbed samples of the soils were collected at intervals in accordance with the ODOT Specifications. A standard 2.0-inch outside diameter split-barrel sampler was driven into the soil by means of a 140-lb

hammer falling freely through a distance of 30-inches in accordance with the Standard Penetration Test (ASTM D 1586). Where bedrock was encountered, all test borings were advanced and the rock was sampled using type NQ2 series core barrels, water method. All test borings were monitored for the presence of groundwater during drilling operations. All test borings were backfilled with compacted soil cuttings/bentonite mixture at the end of drilling operations for safety purposes.

Latitude/Longitude coordinates, stations and offsets, and surface elevations at the drilled test boring locations were provided to PGI by GPD personnel. The N-values (N_m) as measured in the field have been corrected to equivalent rod energy ratio of 60% (N_{60}) in accordance with ODOT's *Specifications for Geotechnical Explorations*. Drill Rig hammer system was calibrated by energy testing in accordance with ASTM D4633 and drill rod energy ratio, ER was determined. Automatic Hammer were calibrated on 8/23/2023 for Acker Recon (Track) Rig with Drill Rod Energy Ratio of 87.5% for the Drill Rig used at the site. The measured N-values (N_m) were corrected to equivalent rod energy ratio of 60 percent, N_{60}, using the equation: $N_{60} = N_m \times (ER/60)$.

The typed drilling logs are included in Appendix A. These logs show the SPT resistance values (N-values) for each soil sample taken in the test borings and present the classification and description of soils encountered at various depths in the test borings.

4.2 Laboratory Testing Program

All soil/rock samples obtained during the drilling and sampling operations were returned to PGI's geotechnical soils laboratory in Cleveland, Ohio. Upon arrival, the samples were visually examined and classified by a geotechnical engineer and a geologist to verify the classifications made in the field and to note any additional characteristics, which may not have been observed in the field.

Moisture content determination tests were performed on all soil samples as per ODOT specifications. Additional laboratory soil tests were performed on selected soil samples for the purpose of soil classification and for analysis of engineering characteristics. These tests consisted of Particle-Size Analysis, Liquid and Plastic Limit, and Plasticity Index Determination of Soils. . Laboratory rock tests were performed on selected rock core samples and consisted of Point Load Strength of Rock Core specimen. All laboratory tests were performed in accordance with the ASTM or other standards listed in "Laboratory Test Standards" located in Appendix B. The results of the laboratory tests are also included in Appendix B. The soils were classified in accordance with the ODOT Soil Classification System, a description of which is also included in Appendix B.

Upon completion of the laboratory testing, all samples were placed in storage at PGI's Cleveland facility. Unless otherwise requested in writing, the soil and rock samples will be retained through completion and ODOT approval of Stage 2 Plans.

5.0 FINDINGS

5.1 Subsurface Conditions

Existing pavement information provided below is based on the measurements obtained during drilling at test borings B-001-0-23 and B-002-0-23. Concrete was encountered at both test boring locations with a thickness of 9.25 inches at B-001-0-23 and 10.0 inches at B-002-0-23. Road base consisting of slag aggregate was encountered below the concrete at both test boring locations with thicknesses of 26.75 inches at B-001-0-23 and 14.0 inches at B-002-0-23.

The subsurface soil conditions were determined from the soil information obtained from test borings B-001-0-23 and B-002-0-23. The subsurface soils encountered in test borings B-001-0-23 and B-002-0-23 consisted of 13.5 feet and 18.5 feet, respectively, of fill/embankment soils over natural soils. The fill/embankment soils were cohesive in nature and consisted of sandy silt (A-4a), silt and clay (A-6a), and silty clay (A-6b). Natural soils encountered below the fill soils consisted primarily of cohesive soils including silt and clay (A-6a), silty clay (A-6b), plastic sandy silt (A-4a), and plastic silt (A-4b). One 4.5-foot-thick layer of non-cohesive soil consisting of coarse and fine sand (A-3a) was encountered in test boring B-001-0-23 between 35.0 and 39.5 feet, above the bedrock. Bedrock was encountered in test borings B-001-0-23 and B-002-0-23 at approximate depths of 39.5 feet and 41.0 feet, respectively below the existing pavement surface. Both test borings were terminated in bedrock after obtaining rock core samples.

The laboratory test results indicated that the moisture contents of the tested cohesive soil samples ranged from 10% to 21% and the consistency ranged from "medium stiff" to "hard". Two (2) of the eight (8) cohesive soil samples tested for Atterberg Limits had natural moisture contents equal to their plastic limits but less than their liquid limits. Both samples were in test boring B-002-0-23; one sample at a depth of 11.0 feet and the other sample at a depth of 31.0 feet. The other six (6) tested cohesive samples had moisture contents below their plastic limits.

5.2 Bedrock Conditions

Bedrock was cored in the test borings after rock was split spoon sampled until little or no penetration or recovery was encountered. Bedrock core samples were then obtained using an NQ2 diamond impregnated core barrel. The coring operations were performed in accordance with the procedure for Diamond Core Drilling for Site Investigations (ASTM D 2113). The core samples consisted of Pennsylvanian-age claystone, siltstone, shale and sandstone bedrock.

The claystone encountered in test borings B-001-0-23 and B-002-0-23 ranged from black to dark gray to light brown and was slightly to moderately weathered, very weak to weak, very thin to thin bedded, and highly to moderately fractured. The rock surface was slightly rough, and the aperture widths were tight. Note that some small white fossils were encountered in the black claystone from 40.5 to 41.5 feet in test boring B-001-0-23 and this section of the claystone was slightly calcareous and effervesced with dilute hydrochloric acid (HCl). No other portion of the bedrock in either test boring effervesced when tested with HCl. The claystone in test boring B-002-0-23 changed abruptly from black to light brown at 45.3 feet. Note that in test boring B-002-0-23, a two (2) inch thick seam of fine-grained, light brown sandstone was encountered within the claystone at a depth of 46.3 feet.

The siltstone encountered in test boring B-001-0-23 ranged from dark gray to light brown and from light brown to gray in test boring B-002-0-23. The siltstone in both test borings was slightly to moderately weathered, slightly to moderately strong, thin to thin bedded, and fractured to moderately fractured. The rock surface was slightly rough, and the aperture widths were tight.

Two distinctly different shale deposits were encountered in test boring B-002-0-23; one at the top of Run 1 from 43.7 to 45.0 feet and one at the bottom of Run 2 from 48.0 to 53.7 feet. The shale at the top of Run 1 between 43.7 and 45.0 feet was black with light gray lenses that decreased in thickness and frequency from 43.7 to 45.0 feet. The shale at the bottom of Run 2 between 48.0 and 52.7 feet was gray with occasional dark inclusions. Both shales were moderately to slightly weathered, very weak, laminated and moderately fractured. The rock surface in each shale was slightly rough and the aperture widths were tight. The Rock Quality Designation (RQD) for the core samples ranged from 0% to 34%. The results of these measurements are summarized in Table 5.2.1. The point load index strengths of the core specimens range from 230 psi and 433 psi which characterizes them as “very week”. Table 5.2.2 summarizes the results of Point Load Strength Test performed on the rock core specimens at full depth. The unconfined compressive strength of the core specimens obtained from historic borings B-001-0-96 (B-1) and B-002-0-96 (b-2) ranged from 11,220 psi to 11,410 psi which characterizes them as “strong”.

General: For specific conditions at various depths, please refer to the individual test boring logs located in Appendix A of this report. For complete moisture contents and Atterberg limit test results, please refer to the laboratory test results in Appendix B. Refer to the drilling logs in Appendix A and rock core photos in Appendix B for additional bedrock information. Also refer to "Bedrock Descriptions" in Appendix B for general bedrock information.

Table 5.2.1 – Bedrock Information

Boring Number	Top of Bedrock Elevations (ft)	Rock Core Run No.	Rock Core Run Elevations (ft)	Length of Core Run (ft)	Recovery (%)	RQD (%)
B-001-0-23	1143.7	NQ2-1	1142.7	5.0	94	0
		NQ2-2	1137.7	5.0	100	8
B-002-0-23	1140.1	NQ2-1	1137.4	5.0	100	33
		NQ2-2	1132.4	5.0	98	34

Table 5.2.2 – Point Load Strength Test Results of Rock Core Specimens

Boring No.	Specimen Depth (ft)	Specimen Elevation (ft)	Point Load Index (psi)	UCS (psi)
B-001-0-23	40.5 – 50.5	1142.7 – 1132.7	19.18	230
B-002-0-23	43.7 – 53.7	1137.4 – 1127.4	36.05	433

UCS – Unconfined Compressive Strength

5.3 Groundwater Conditions

The groundwater levels were monitored in the test boring locations during drilling operations. Groundwater was encountered at a depth of 28.0 feet during drilling operations in both test borings. Groundwater levels were not recorded upon completion of drilling operations due to use of water for rock coring. It should be noted that groundwater elevations are subject to seasonal fluctuations. Test borings B-001-0-23 and B-002-0-23 were backfilled upon completion of drilling operations for safety purposes.

6.0 ANALYSIS AND RECOMMENDATIONS

Based upon the findings of the field exploration program, laboratory testing, and subsequent engineering analysis, the following sections have been prepared to address the geotechnical aspects related to the design and construction of the SR 88 Bridge over IR-80.

Site plans provided by GPD personnel indicate that the proposed replacement Bridge abutments will be supported on pile foundation at the rear and forward abutment locations while the Pier will be supported on the existing drilled shaft foundation. The proposed abutment type Mechanically Stabilized Earth (MSE) Walls identified as MSE Wall No. 1 and MSE Wall No. 2 will be installed at the rear and forward abutments, respectively. Elevation of the top leveling pad of the MSE Wall No. 1 and MSE Wall No. 2 will be at 1157.50 feet and 1158.50 feet, respectively. The profile grade elevation behind rear and forward abutments of MSE Walls No. 1 and MSE Wall No. 2 will be about 1183.90 feet and 1182.08 feet. The foundation recommendations for the bridge and MSE Wall are provided in accordance with the ODOT *Bridge Design Manual* issued in 2007 using AASHTO *LRFD Bridge Design Specifications, current Edition.*

6.1 Bridge Foundation Systems

Soil and rock information obtained from test borings B-001-0-23 and B-002-0-23 were used to provide foundation recommendations for this proposed replacement bridge. Test boring B-001-0-23 was advanced in the vicinity of the proposed rear abutment while test boring B-002-0-23 was advanced in the vicinity of the forward abutment. As outlined in Section 5.1 - "Subsurface Soil Conditions", the top of bedrock was encountered at an approximate depth of 39.5 feet in test boring B-001-0-23 and 41.0 feet in test boring B-002-0-23. Bedrock at these boring locations consists of claystone, siltstone, shale and sandstone and was encountered to termination depth. Since bedrock was encountered relatively deeper depths at the rear and forward abutment locations, the proposed superstructure loads may be transferred to the underlying bedrock by means of deep foundations.

Deep Foundation consisting of end bearing H-piles may be used to transfer the proposed superstructure loads to the underlying bedrock at the rear and forward abutment locations. MSE Wall will be constructed at the rear and forward abutment locations. According to the ODOT *Bridge Design Manual* Section 605.1-4, prior to driving abutment piles to refusal on bedrock, MSE wall and the bridge approach embankment can be constructed behind the abutment up to the bottom of the footing for a minimum distance of 200 feet behind both abutments. However, the bridge approach embankments exist

on both sides of the abutments. Contractors may select the option of installing the piles before or after constructing MSE walls at the abutment locations. If the contractor select the option of pre-driving abutment piles before constructing MSE walls, the abutment piles should be driven into the soils only as far as necessary so that the pile will remain vertical during MSE wall construction. Pile sleeves should be installed in accordance with the ODOT SS 840.

According to the ODOT *Bridge Design Manual* Section 305.3.5.7, the end bearing H-piles should be installed with a minimum embedment length of 15.0 feet below the bottom of the MSE Wall. Design information provided by GPD personnel indicate that the maximum factored loads along a vertical direction will be 140 kips per highest loaded pile at the rear and forward abutments. The end bearing H-Piles must be driven to refusal on the underlying shale bedrock. Pile refusal can be considered when pile penetration is one inch or less after receiving at least 20 blows from the pile hammer during driving. H-pile sizes HP-10X42 may be selected for the rear and forward abutment locations. The total factored load on each HP-10X42 pile should not exceed the corresponding maximum factored structural resistance of 310 kips as per the ODOT *Bridge Design Manual* Section 305.3.3. Note that the above-mentioned structural resistance values can be used only on the axial loaded piles that have negligible bending moment. The estimated pile parameters for end bearing piles at each boring location are summarized in Table 6.1.1. The pile cut-off elevations at the abutments were obtained from the structure site plan provided by GPD personnel.

Table 6.1.1 - Estimated Design Parameters for H-Piles

Boring No.	Pile Cut-off Elevation (ft)	Pile Tip Elevation (ft)	Recommended Pile Order Length (ft)	Pile Type	Pile Size	Maximum Factored Structural Resistance/pile
Rear Abutment						
B-001-0-23	1174.7	1141.2	40	H-Pile	10X42	310 kips
Forward Abutment (For west side Pile)						
B-002-0-23	1172.7	1135.9	45	H-Pile	10X42	310 kips
Forward Abutment (For East side Pile)						
B-008-0-53	1172.7	1133.3	45	H-Pile	10X42	310 kips

It is estimated that the maximum total settlement and differential settlement of the substructure units will not exceed one inch and one-half of an inch, respectively. Based on the pile tip elevations shown on the Table 6.1.1, H-piles will be installed below the bottom of MSE Wall Nos. 1 and 2 with the embedment length of 17.3 feet and 23.4 feet, respectively which is more than required minimum

embedment length of 15.0 feet. All H-piles should be installed in accordance with ODOT Item 507 - *Bearing Piles*, of the ODOT *Construction and Material Specifications Manual* dated July 2023. For detailed pile foundation design refer to Section 303.4.2 - "Pile Foundations" and other related sections of the *ODOT Bridge Design Manual* issued in 2020. If it is assured that the piles are driven to refusal on bedrock, then neither a static load test nor a dynamic pile bearing capacity test will be necessary. It is recommended that the piles be spaced a minimum of three (3) pile diameters on center. If additional lateral resistance is required, larger size piles should be considered at the abutment locations. Since piles are to be driven to bear on bedrock, steel points are required to protect the tip of the H pile as per the ODOT *Bridge Design Manual* Section 305.3.5.6. Damage could be caused to existing residential buildings within approximately 200 feet of the proposed driving locations due to induced vibrations during pile driving operations. Since the closest residential house is located within an approximate distance of 100 feet from the bridge site, vibrations impact assessment must be performed in accordance with BDM C305.3.6.

Since approach embankments exist on both sides of the bridge, no embankment fill placement will be required along the abutment approaches. MSE Walls will be constructed at the proposed abutment locations after removing section of the existing embankment and replacing with MSE Wall reinforced fill. The foundation soils below the MSE Wall were already consolidated under the weight of existing embankment soils. Therefore, consolidation settlement due to the volume change in the foundation soils are not anticipated due to construction of MSE Wall fill at each abutment location and waiting period is not required before construction of MSE Wall at both abutments.

6.2 MSE Wall Foundation Systems

Strait Line MSE Wall System will be constructed at the rear and forward abutment locations in order to retain soils behind the abutments. The site plan provided by GPD personnel indicates that the top leveling pad elevations for the proposed MSE Wall No. 1 will be at 1157.50 feet and proposed MSE Wall No. 2 will be at 1158.50 feet. The profile grade elevation behind the MSE Wall Nos. 1 and 2 will be about 1183.90 feet and 1182.08 feet. The maximum height of MSE Wall No. 1 from top of the leveling pad to SR 88 roadway profile grade behind the rear abutment will be about 26.40 feet. The foundation width of the MSE Wall No.1 will be 21.12 feet based upon a minimum strap length equal to 80% of the maximum wall height as per Reinforced Earth Co. recommendations and it is initially assumed 22.0 feet for the MSE Wall external stability calculations. The length of the MSE Wall No.1 will be 125.17 feet. Soil and rock information obtained from project test boring B-001-0-23 was used to

provide foundation recommendations for the proposed MSE Wall No. 1. The maximum height of MSE Wall No.2 from top of the leveling pad to SR 88 roadway profile grade behind the forward abutment will be about 23.58 feet. The foundation width of the MSE Wall No.2 will be 18.86 feet based upon a minimum strap length equal to 80% of the maximum wall height as per Reinforced Earth Co. recommendations and it is initially assumed 19.0 feet for the MSE Wall external stability calculations. The length of this MSE Wall will be 125.17 feet. Soil and rock information obtained from project test boring B-002-0-23 was used to provide foundation recommendations for the proposed MSE Wall No. 2.

The site plan provided by GPD personnel indicates that the bearing elevation of the proposed MSE Wall No. 1 at 1157.00 feet and proposed MSE Wall No. 2 at 1158.00 feet. The foundation soils encountered below the bottom of the MSE Wall consisted of natural soils and were generally cohesive in nature. As per the boring logs, the consistency of these cohesive soils ranged from “hard” to “stiff” but was generally “very stiff” to "hard”. Therefore, shallow foundations may be designed to transfer the applied design loads from the MSE Walls to the underlying natural soils at the test boring abutment locations. Bearing capacity analyses (AASHTO Article 11.6.3) were performed using total stress soil parameters estimating the nominal bearing resistance of the reinforced MSE soil mass supported on foundation cohesive soils at rear and forward locations. Groundwater level was estimated at elevation 1155.2 feet at MSE Walls No. 1 and at elevation 1153.1 feet at MSE Wall No. 2 based on the groundwater level reading obtained from test borings. Result of the MSE Wall Nos. 1 and 2 bearing resistance capacity analysis is included in Appendix B. Estimated factored bearing resistance is summarized in Table 6.2.1. A resistance factor (ϕ) of 0.65 (per Table AASHTO LRFD Table 11.5.6-1) was applied to compute the factored bearing resistance at Strength Limit State. The foundation soils below the MSE Wall were already consolidated under the weight of the existing embankment soils. Therefore, it is estimated that the maximum total settlement and differential settlement will not exceed one inch and one-half of an inch, respectively.

Table 6.2.1 – Estimated Design Parameters at Strength Limit State for MSE Walls

Boring No.	MSE Wall No.	Depth of Bottom of Footing Below Final Grade (feet)	Width of Footing (feet)	Proposed Bearing Elevation (feet)	Factored Bearing Resistance (ksf)
B-001-0-23	1	3.5	22.0	1157.0	9.0
B-002-0-23	2	3.5	19.0	1158.0	8.0

According to Supplemental Specification SS840, the existing soils supporting the proposed MSE walls should be excavated to a minimum 12 inches and replaced with compacted ODOT Item 203 Granular Material, Type C. The excavated MSE Wall subgrade should be examined by competent geotechnical personnel. If any fill materials/highly compressible materials or areas of low bearing capacity with excessive moisture (soft pockets) are encountered, they should be removed as directed by on site geotechnical personnel and replaced with ODOT Item 203 Granular Material, Type C. External stability of the MSE Wall including sliding on the base, limiting eccentricity, and bearing resistance at the Strength and Service Limit States was performed using hand calculations for the MSE Wall Nos. 1 and 2. For this external stability analysis, the MSE Wall was considered as a rigid body. Shear strength parameters of the reinforced soil; bulk unit weight = 120 pcf and phi angle = 34° and shear strength parameters of the retaining soil; bulk unit weight = 120 pcf and phi angle = 30° were assumed. The uniform surcharge load due to traffic was assumed to be 250 psf as per AASHTO LRFD Table 3.11.6.4-2 for the MSE Wall height of more than equal to 20.0 feet. Abutment configuration at the rear and forward abutment locations were obtained from the site plans for the external stability analysis. Hand calculation sheets of MSE Walls No. 1 and 2 external stability analyses are included in Appendix B. Load and resistance factors used with respect to the various potential failure modes and limit states of the MSE Wall are shown in the Hand calculation sheets. The External Stability analyses results show that the Capacity Demand Ratio (CDR) value against sliding, CDR value with respect to bearing resistance and eccentricity values are greater than the one for the selected foundation width of the MSE Wall Nos. 1 & 2.

Since the MSE Wall bottom at both abutment locations will be placed below the grade of IR 80, on relatively level area, and installed piles within the MSE Walls will be driven to bedrock, global stability of the MSA Wall No1 and 2 are not a concern. The MSE Wall design should be constructed in accordance with the ODOT Supplemental Speciation SS840.

6.3 Lateral Earth Pressures and Abutment Drainage

In order to resist the horizontal bridge and backwall forces, a minimum of one row of soil reinforcements should be attached to the backside of the forward abutment footing. The MSE Wall system supplier must be responsible for internal stability design, including checking both pullout and rupture of the reinforcements and abutment drainage. Freely draining material must be placed behind the bridge abutments in accordance with ODOT Item 518 - “Drainage of Structures”. The porous backfill should be placed a minimum of two (2) feet in thickness normal to the abutment walls. It is suggested

that filter fabric, ODOT Item 712.09, Type A, be placed between Item 518 porous backfill material and Item 203 embankment material. This will ensure that fine particles from within the embankment do not migrate into the voids of the porous backfill.

6.4 Groundwater Management

Groundwater was encountered at a depth of 28.0 feet during drilling operations in both test borings B-001-0-23 and B-002-0-23. Groundwater will be anticipated for excavations extending below the groundwater level encountered during the drilling operations. Water seepage is anticipated in the proposed excavations. Low to moderate to high volume pumping or dewatering may be required. Please note that the groundwater levels may vary due to seasonal fluctuations and groundwater may appear during excavation where it was not previously encountered.

6.5 Earthwork and Construction Monitoring

All excavation and backfilling operations should be conducted in accordance with ODOT's *Construction and Materials Specifications*, Item 503 - "Excavation for Structures" issued in January 2023 and under the supervision of competent geotechnical personnel. All excavations should comply with all current and applicable local, state, and federal safety codes, regulations and practices, including the Occupational Safety and Health Administration (OSHA). The proposed cut slopes for the structure foundation excavation must be constructed using a two (2) horizontal to one (1) vertical slope in cohesive soils. Soil and rock excavations are expected during construction of the project.

All fill material must be approved by a qualified geotechnical engineer prior to placement. The fill materials should be placed in lifts of eight (8) inches in thickness (loose measure) and be compacted to an unyielding condition in accordance with ODOT 203.07 “Compaction and Moisture Requirements” specifications. The top 12 inches of the fill in pavement subgrade areas should be placed in lifts of eight (8) inches in thickness (loose measure) and be compacted to an unyielding condition in accordance with ODOT 204.03 “Compaction of the Subgrade” specifications. All in-place density tests should be performed as per Supplement 1015 “Compaction Testing of Unbound Materials” during earthwork construction. The tests should be performed by a qualified soil technician in accordance with the appropriate ASTM procedures.

7.0 LIMITATIONS

This report is subject to the following conditions and limitations:

7.1 The subsurface conditions described are based on an examination of the soil and rock samples at the sampling intervals. Varying soil deposits, including fill material, may exist between the sampling intervals and between the test boring locations. Variation in subsurface conditions from those indicated in this report may become apparent during the earthwork and/or installation of the foundations. Such variations may require changes and/or modifications in our recommendations. Such changes may cause time delays and/or additional costs. Owners must be made aware of these limitations and must incorporate them in the design budget and scheduling of the project.

7.2 The design of the proposed project does not vary from the technical information provided and specified in this report. All changes in the design must be reviewed by our geotechnical engineers. PGI cannot assume any responsibility for interpretations made by others of the subsurface conditions and their behavior based on this report.

7.3 All earthwork and foundation construction must be performed under the supervision of a Professional Engineer in accordance with ODOT Construction Specifications.

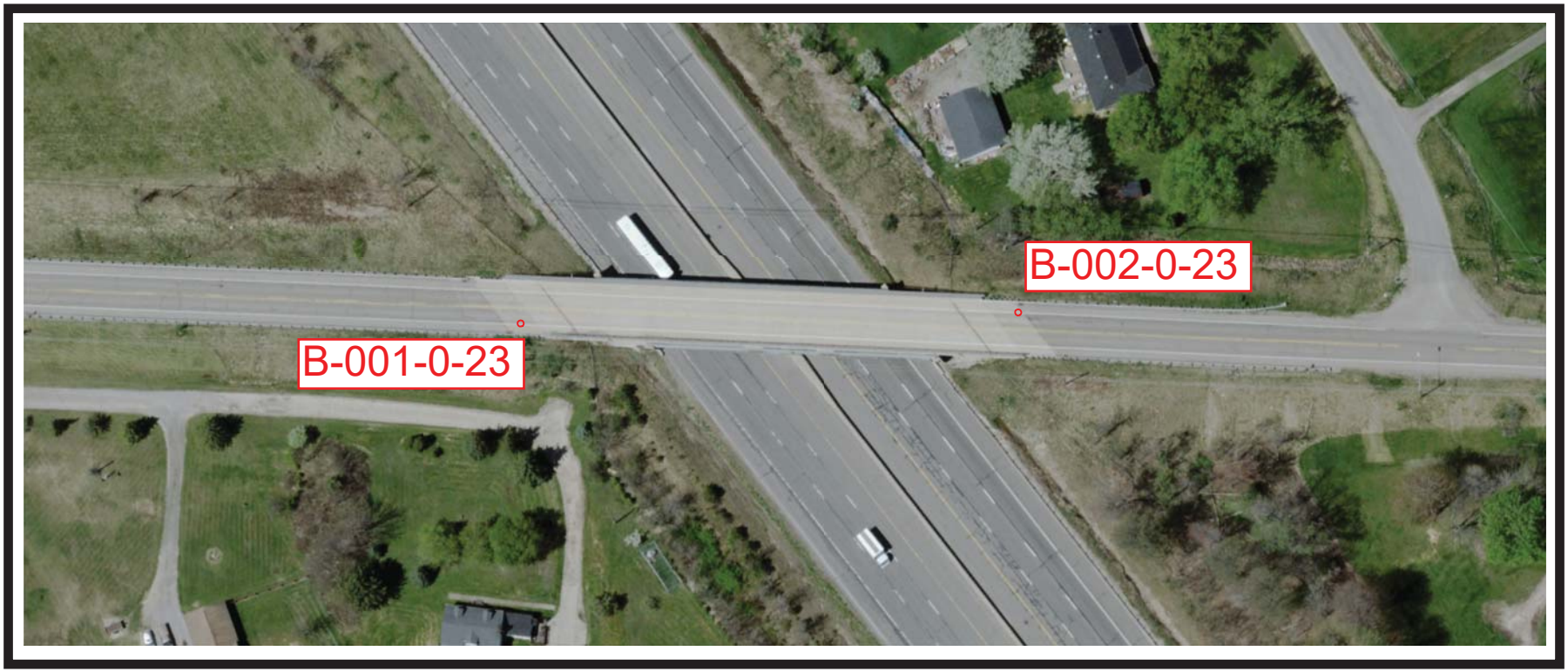
7.4 The subsurface exploration for this project is strictly from a geotechnical standpoint. An environmental site assessment was not included in the scope of these geotechnical services.

7.5 All sheeting, shoring, and bracing of trenches, pits and excavations should be made the responsibility of the contractor and should comply with all current and applicable local, state and federal safety codes, regulations and practices, including the Occupational Safety and Health Administration (OSHA).

APPENDICES

APPENDIX A

SR 88 BRIDGE OVER OHIO TURNPIKE
BORING LOCATION MAP



N

PROJECT: SR 88 BRIDGE OVER IR-80	DRILLING FIRM / OPERATOR: GEI / TROY	DRILL RIG: ACKER RECON TRACK	STATION / OFFSET: 8+53, 8' RT.
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: GEI / TMR	HAMMER: ACKER AUTOMATIC	ALIGNMENT: SR 88 CENTERLINE
PID: STR ID:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 8/21/23	ELEVATION: 1183.2 (MSL) EOB: 50.5 ft.
START: 9/20/23 END: 9/20/23	SAMPLING METHOD: SPT / NQ2	ENERGY RATIO (%): 87.5	LAT / LONG: 41.242078, -81.128900

EXPLORATION ID B-001-0-23

PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 1183.2	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
CONCRETE PAVEMENT (9.25 INCHES IN THICKNESS)	1182.4																	
GRAY SLAG ROADBASE (26.75 INCHES IN THICKNESS)	1180.2	1																
HARD, BROWN, **SANDY SILT**, SOME CLAY, LITTLE STONE FRAGMENTS, FILL, DAMP	1178.2	2	21 13 4	25	100	SS-1	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	
STIFF, **SILTY CLAY**, SOME SAND, TRACE STONE FRAGMENTS, FILL, DAMP	1169.7	3	3 4 4	12	100	SS-2	4.5+	-	-	-	-	-	-	-	-	11	A-4a (V)	
HARD, BROWN, **SILT AND CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, DAMP	1159.7	4																
HARD TO STIFF, BROWN TO GRAY, **SANDY SILT**, SOME TO LITTLE CLAY, LITTLE STONE FRAGMENTS	1148.2	5																
@26.0'; VERY STIFF		6	2 2 3	7	78	SS-3	1.25	-	-	-	-	-	-	-	-	16	A-6b (V)	
@28.5'; VERY STIFF, GRAY		7																
@31.0'; STIFF, GRAY		8	1 2 3	7	78	SS-4	2.00	-	-	-	-	-	-	-	-	18	A-6b (V)	
@33.5'; VERY STIFF, GRAY, LITTLE CLAY		9																
DENSE, GRAY, **COARSE AND FINE SAND**, LITTLE FINES, TRACE TO WITH ROCK FRAGMENTS, WET	1143.7	10	1 2 3	7	67	SS-5	2.00	7	9	16	27	41	38	20	18	19	A-6b (10)	
BLACK WEATHERED CLAYSTONE. AUGER REFUSAL AT 40.5 FEET.	1142.7	11																
CLAYSTONE, BLACK TO DARK GRAY, SLIGHTLY WEATHERED, VERY WEAK TO WEAK, VERY THIN TO THIN BEDDED, HIGHLY FRACTURED TO FRACTURED, SLIGHTLY ROUGH, TIGHT APERTURE WIDTH. TRACE WHITE FOSSILS FROM 40.5 TO 41.5 FEET SLIGHTLY CALCAREOUS FROM 40.5 TO 42.3 FEET. EFFERVESCES WITH DILUTE HYDROCHLORIC ACID.	1133.4	12																
@40.5' - 50.5'; POINT LOAD INDEX STRENGTH = 230 PSI		13	3 4 9	19	72	SS-6	4.5+	-	-	-	-	-	-	-	-	16	A-6a (V)	
SILTSTONE, DARK GRAY TO LIGHT BROWN, SLIGHTLY WEATHERED, WEAK, THIN TO MEDIUM BEDDED, FRACTURED TO MODERATELY FRACTURED, SLIGHTLY ROUGH, TIGHT APERTURE WIDTH. OCCASIONAL GRAY AND BROWN INCLUSIONS THROUGHOUT SILTSTONE	1132.7	14																
		15																
		16	5 8 9	25	100	SS-7	4.5+	-	-	-	-	-	-	-	-	15	A-6a (V)	
		17																
		18																
		19	9 14 18	47	100	SS-8	4.5+	-	-	-	-	-	-	-	-	16	A-6a (V)	
		20																
		21																
		22	6 9 13	32	100	SS-9	4.5+	4	4	8	32	52	35	20	15	16	A-6a (10)	
		23																
		24	7 9 11	29	100	SS-10	4.5+	-	-	-	-	-	-	-	-	12	A-4a (V)	
		25																
		26	3 7 12	28	28	SS-11	3.00	-	-	-	-	-	-	-	-	12	A-4a (V)	
		27																
		28 (W)																
		29	3 4 8	18	67	SS-12	2.0-4.5+	-	-	-	-	-	-	-	-	12	A-4a (V)	
		30																
		31	0 0 4	6	100	SS-13	1.25	15	9	23	32	21	21	15	6	13	A-4a (4)	
		32																
		33																
		34	7 8 11	28	100	SS-14	4.00	13	13	26	34	14	18	14	4	12	A-4a (3)	
		35																
		36	13 16 13	42	100	SS-15	-	-	-	-	-	-	-	-	-	14	A-3a (V)	
		37																
		38																
		39 (TR)	7 8 30	55	100	SS-16	-	-	-	-	-	-	-	-	-	9	A-3a (V)	
		40																
		41																
		42																
		43	0		94	NQ2-1											CORE	
		44																
		45																
		46																
		47																
		48	8		100	NQ2-2											CORE	
		49																
		50 (EOB)																

NOTES: GROUNDWATER WAS ENCOUNTERED AT 28.0' BELOW GROUND SURFACE DURING DRILLING AND NO WATER READING WAS TAKEN UPON COMPLETION BECAUSE WATER WAS USED FOR ROCK CORING OPER

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PAVEMENT WAS REPLACED WITH 1.0 BAG ASPHALT PATCH; BACKFILLED WITH 0.5 BAG SOIL CUTTINGS/BENTONITE PELLETS

STANDARD ODOT SOIL BORING LOG (11 X 17) - OH DOT.GDT - 11/2/23 12:43 - \\GEOTECHSERVER\SHARED FOLDERS\COMPANY\PUBLIC\PROJECT FILES\23 PROJECTS\G23017G-GPD-OTC(71-22-03)\LAB DATA SHEETS\G23017GGINT.GPJ

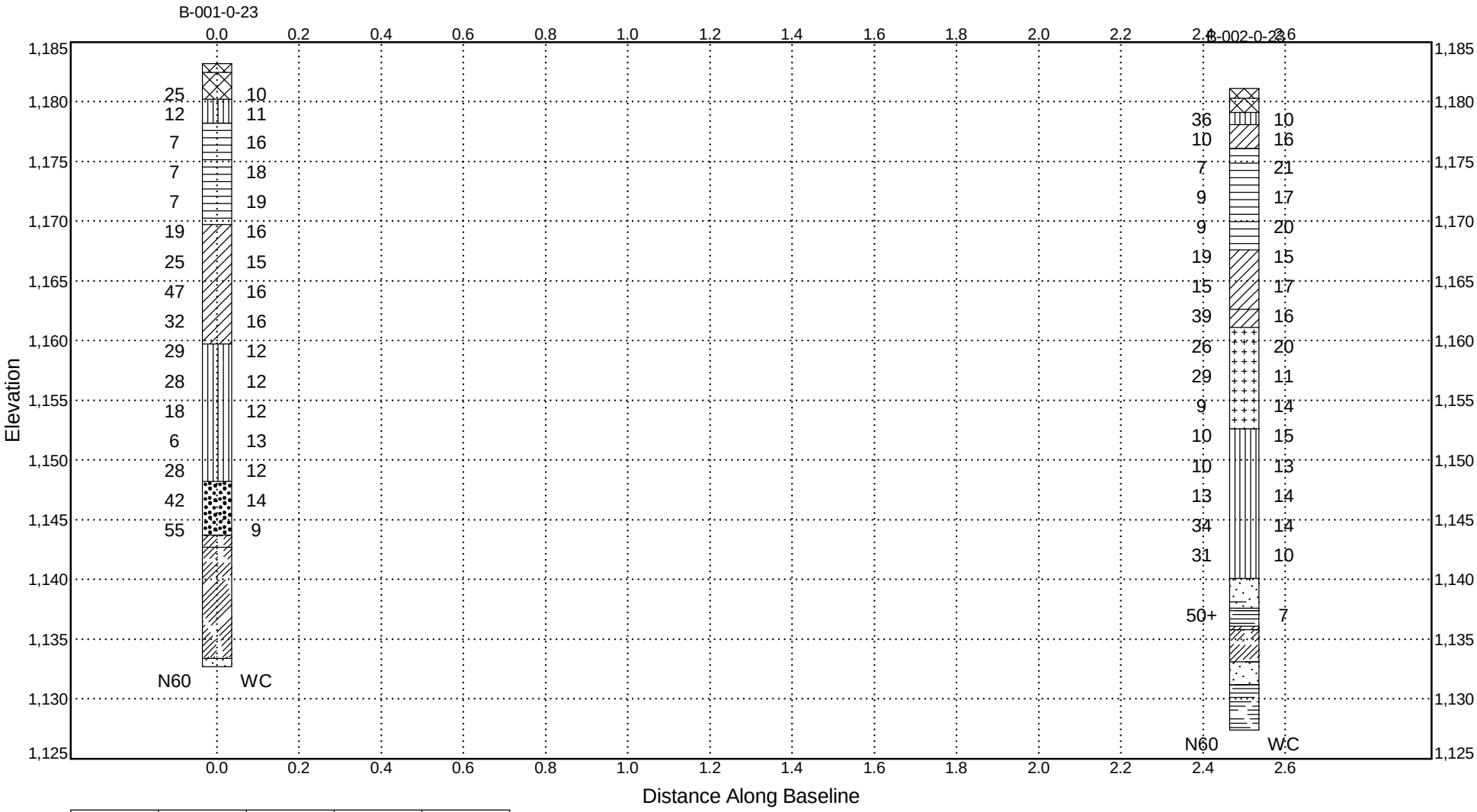
PROJECT: SR 88 BRIDGE OVER IR-80	DRILLING FIRM / OPERATOR: GEI / TROY	DRILL RIG: ACKER RECON TRACK	STATION / OFFSET: 11+50, 11' LT.	EXPLORATION ID B-002-0-23
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: GEI / TMR	HAMMER: ACKER AUTOMATIC	ALIGNMENT: SR 88 CENTERLINE	
PID: STR ID:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 8/21/23	ELEVATION: 1181.1 (MSL) EOB: 53.7 ft.	PAGE 1 OF 1
START: 9/19/23 END: 9/19/23	SAMPLING METHOD: SPT / NQ2	ENERGY RATIO (%): 87.5	LAT / LONG: 41.242764, -81.128314	

MATERIAL DESCRIPTION AND NOTES	ELEV. 1181.1	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
CONCRETE PAVEMENT (10 INCHES IN THICKNESS)	1180.3																	
SLAG AGGREGATE BASE (14 INCHES IN THICKNESS)	1179.1	1																
HARD, BROWN, **SANDY SILT**, SOME CLAY, LITTLE STONE FRAGMENTS, FILL, DAMP	1178.1	2 3	7 9 16	36	100	SS-1	4.5+	-	-	-	-	-	-	-	-	10	A-4a (V)	
VERY STIFF, BROWN, **SILT AND CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, FILL, DAMP	1176.1	4 5	4 3 4	10	61	SS-2	3.25	-	-	-	-	-	-	-	-	16	A-6a (V)	
STIFF TO VERY STIFF, GRAY TO BROWN, **SILTY CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, FILL, MOIST TO DAMP		6 7	1 2 3	7	94	SS-3	1.25	-	-	-	-	-	-	-	-	21	A-6b (V)	
@8.5'; VERY STIFF, DAMP		8 9 10	2 2 4	9	78	SS-4	3.25	-	-	-	-	-	-	-	-	17	A-6b (V)	
@11.0'; STIFF, MOIST	1167.6	11 12 13	2 2 4	9	89	SS-5	1.50	5	5	11	31	48	36	20	16	20	A-6b (10)	
VERY STIFF TO HARD, BROWN, **SILT AND CLAY**, LITTLE TO TRACE SAND, TRACE STONE FRAGMENTS, FILL, DAMP	1162.6	14 15 16 17 18	3 6 7	19	100	SS-6	3.50	-	-	-	-	-	-	-	-	15	A-6a (V)	
			2 3 7	15	94	SS-7	2.00	-	-	-	-	-	-	-	-	17	A-6a (V)	
HARD, BROWN, **SILT AND CLAY**, TRACE SAND, TRACE STONE FRAGMENTS, DAMP	1161.1	19 20	8 10 17	39	100	SS-8	4.5+	-	-	-	-	-	-	-	-	16	A-6a (V)	
HARD TO VERY STIFF, BROWN, **PLASTIC SILT**, SOME CLAY, TRACE SAND, TRACE STONE FRAGMENTS, DAMP		21 22 23	6 8 10	26	100	SS-9	4.50	2	3	4	65	26	27	21	6	20	A-4b (8)	
		24 25	6 9 11	29	61	SS-10	4.5+	-	-	-	-	-	-	-	-	11	A-4b (V)	
@26.0'; VERY STIFF	1152.6	26 27 28 W	2 3 3	9	100	SS-11	3.50	-	-	-	-	-	-	-	-	14	A-4b (V)	
SOFT TO VERY STIFF, GRAY, **SANDY SILT**, LITTLE CLAY, TRACE TO SOME STONE FRAGMENTS, MOIST TO DAMP		29 30	4 3 4	10	67	SS-12	0.50	-	-	-	-	-	-	-	-	15	A-4a (V)	
@31.0'; STIFF		31 32	1 3 4	10	56	SS-13	1.75	10	9	25	39	17	18	13	5	13	A-4a (4)	
@33.5'; STIFF		33 34 35	4 4 5	13	100	SS-14	2.00	-	-	-	-	-	-	-	-	14	A-4a (V)	
		36 37 38	5 10 13	34	56	SS-15	-	-	-	-	-	-	-	-	-	14	A-4a (V)	
@38.5'; VERY STIFF, SOME STONE FRAGMENTS, DAMP		39	8 10 11	31	100	SS-16A	2.50	29	11	18	27	15	22	15	7	10	A-4a (1)	
@39.5'; VERY STIFF, SOME STONE FRAGMENTS, DAMP	1140.1	40 41 TR				SS-16B	3.25	-	-	-	-	-	-	-	-	10	A-4a (V)	
GRAY WEATHERED SILTSTONE	1137.6	42 43																
SHALE, BLACK AND LIGHT GRAY, MODERATELY TO SLIGHTLY WEATHERED, VERY WEAK, LAMINATED, HIGHLY FRACTURED TO FRACTURED, SLIGHTLY ROUGH, TIGHT APERTURE WIDTH. LIGHT GRAY LENSES DECREASE IN THICKNESS AND FREQUENCY FROM 43.7 TO 45 FEET..	1137.4	44	50/0"	–	100	SS-18	-	-	-	-	-	-	-	-	-	7	Rock (V)	
	1136.1 1135.8	45 46 47	33		100	NQ2-1											CORE	
CLAYSTONE, BLACK, SLIGHTLY WEATHERED, VERY WEAK, VERY THIN BEDDED, HIGHLY FRACTURED, SLIGHTLY ROUGH, TIGHT APERTURE WIDTH..	1133.1	48																
CLAYSTONE, LIGHT BROWN, MODERATELY TO SLIGHTLY WEATHERED, VERY WEAK, THIN BEDDED, HIGHLY TO MODERATELY FRACTURED, SLIGHTLY ROUGH, TIGHT APERTURE WIDTH. NOTE 2.0 INCH LIGHT BROWN, FINE GRAINED SANDSTONE SEAM AT 46.3 FEET..	1131.2	49 50 51	34		98	NQ2-2											CORE	
@43.7' - 53.7'; POINT LOAD INDEX STRENGTH = 433 PSI																		
SILTSTONE, LIGHT BROWN TO GRAY, SLIGHTLY TO MODERATELY WEATHERED, VERY WEAK, THIN BEDDED, HIGHLY TO MODERATELY FRACTURED, SLIGHTLY ROUGH, TIGHT APERTURE WIDTH..	1127.4	52 53 EOB																
SHALE, GRAY, SLIGHTLY WEATHERED, VERY WEAK, LAMINATED, HIGHLY TO MODERATELY FRACTURED, SLIGHTLY ROUGH, TIGHT APERTURE WIDTH..																		

NOTES: GROUNDWATER WAS ENCOUNTERED AT 28.0' BELOW GROUND SURFACE DURING DRILLING AND NO WATER READING WAS TAKEN UPON COMPLETION BECAUSE WATER WAS USED FOR ROCK CORING OPER

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PAVEMENT WAS REPLACED WITH 1.0 BAG ASPHALT PATCH; BACKFILLED WITH 0.5 BAG SOIL CUTTINGS/BENTONITE PELLETS

STANDARD ODOT SOIL BORING LOG (11 X 17) - OH DOT.GDT - 11/2/23 12:43 - \\GEOTECHSERVER\SHARED FOLDERS\COMPANY\PUBLIC\PROJECT FILES\23 PROJECTS\G23017G-GPD-OTC(71-22-03)\LAB DATA SHEETS\G23017GGINT.GPJ



Borehole	North	East	Elev.	Depth
B-001-0-23			1183.2	50.5
B-002-0-23			1181.1	53.7

DISTANCES:
Beginning 0
Ending 3
VIEWING ANGLES (degrees):
Horizontal 0.0
Vertical 0.0

Position		
Left, Front		
Right, Front		
Left, Back		
Right, Back		

SOIL/ROCK BORINGS PROFILE

SR 88 BRIDGE OVER OHIO TURNPIKE

PORTAGE COUNTY, OHIO

PROJECT #	DATE	PLATE
G23017G	Nov 23	1

PROFILE ODOT - PRIMENG.GDT - 11/3/23 16:13 - \\GEOTECHSERVER\SHARED FOLDERS\COMPANY\PUBLIC\PROJECT FILES\23 PROJECTS\G23017G-GPD-OTC(71-22-03)\LAB DATA SHEETS\G23017GGINT.GI

APPENDIX B

Boring Number	Sample Number	Depth (ft)	Water Content %	Liquid Limit %	Plastic Limit %	Plast. Index	Specific Gravity	Agg. %	Coarse Sand %	Fine Sand %	Silt %	Silt&Clay Comb. %	Clay %	Soil Description	Class. Symbol
B-001-0-23	SS-1	2.0	10											GRAY SLAG AGGREGATE BASE (FILL)	A-1-a (V)
B-001-0-23	SS-2	3.5	11											BROWN SANDY SILT, SOME CLAY, LITTLE STONE FRAGMENTS (FILL)	A-4a (V)
B-001-0-23	SS-3	6.0	16											BROWN SILTY CLAY, SOME SAND, TRACE STONE FRAGMENTS (FILL)	A-6b (V)
B-001-0-23	SS-4	8.5	18											BROWN SILTY CLAY, SOME SAND, TRACE STONE FRAGMENTS (FILL)	A-6b (V)
B-001-0-23	SS-5	11.0	19	38	20	18		6	9	16	27	68	41	BROWN SILTY CLAY, SOME SAND, TRACE STONE FRAGMENTS (FILL)	A-6b (10)
B-001-0-23	SS-6	13.5	16											BROWN SILT AND CLAY, LITTLE SAND, TRACE STONE FRAGMENTS	A-6a (V)
B-001-0-23	SS-7	16.0	15											BROWN SILT AND CLAY, LITTLE SAND, TRACE STONE FRAGMENTS	A-6a (V)
B-001-0-23	SS-8	18.5	16											BROWN SILT AND CLAY, LITTLE SAND, TRACE STONE FRAGMENTS	A-6a (V)
B-001-0-23	SS-9	21.0	16	35	20	15		4	4	8	32	84	52	BROWN SILT AND CLAY, LITTLE SAND, TRACE STONE FRAGMENTS	A-6a (10)
B-001-0-23	SS-10	23.5	12											BROWN SANDY SILT, SOME CLAY, LITTLE STONE FRAGMENTS	A-4a (V)
B-001-0-23	SS-11	26.0	12											BROWN SANDY SILT, SOME CLAY, LITTLE STONE FRAGMENTS	A-4a (V)
B-001-0-23	SS-12	28.5	12											GRAY SANDY SILT, SOME CLAY, LITTLE STONE FRAGMENTS	A-4a (V)
B-001-0-23	SS-13	31.0	13	21	15	6		14	9	23	32	53	21	GRAY SANDY SILT, SOME CLAY, LITTLE STONE FRAGMENTS	A-4a (4)
B-001-0-23	SS-14	33.5	12	18	14	4		13	13	26	34	48	14	GRAY, SANDY SILT, LITTLE CLAY, LITTLE STONE FRAGMENTS	A-4a (3)
B-001-0-23	SS-15	36.0	14											GRAY COARSE AND FINE SAND, LITTLE FINES, TRACE STONE FRAGMENTS	A-3a (V)
B-001-0-23	SS-16	38.5	9											GRAY COARSE AND FINE SAND, LITTLE FINES, W/ CLAYSTONE FRAGS	A-3a (V)
B-002-0-23	SS-1	2.0	10											BROWN SANDY SILT, SOME CLAY, LITTLE STONE FRAGMENTS (FILL)	A-4a (V)
B-002-0-23	SS-2	3.5	16											BROWN SILT AND CLAY, LITTLE SAND, TRACE STONE FRAGMENTS (FILL)	A-6a (V)
B-002-0-23	SS-3	6.0	21											GRAY SILTY CLAY, LITTLE SAND, TRACE STONE FRAGMENTS (FILL)	A-6b (V)
B-002-0-23	SS-4	8.5	17											BROWN SILTY CLAY, LITTLE SAND, TRACE STONE FRAGMENTS (FILL)	A-6b (V)
B-002-0-23	SS-5	11.0	20	36	20	16		4	5	11	31	79	48	BROWN SILTY CLAY, LITTLE SAND, TRACE STONE FRAGMENTS (FILL)	A-6b (10)
B-002-0-23	SS-6	13.5	15											BROWN SILT AND CLAY, LITTLE SAND, TRACE STONE FRAGMENTS (FILL)	A-6a (V)
B-002-0-23	SS-7	16.0	17											BROWN SILT AND CLAY, LITTLE SAND, TRACE STONE FRAGMENTS (FILL)	A-6a (V)
B-002-0-23	SS-8	18.5	16											BROWN SILT AND CLAY, TRACE SAND, TRACE STONE FRAGMENTS	A-6a (V)
B-002-0-23	SS-9	21.0	20	27	21	6		1	3	4	66	91	26	BROWN SILT, SOME CLAY, TRACE SAND, TRACE STONE FRAGMENTS	A-4b (8)
B-002-0-23	SS-10	23.5	11											BROWN SILT, SOME CLAY, TRACE SAND, TRACE STONE FRAGMENTS	A-4b (V)
B-002-0-23	SS-11	26.0	14											BROWN SILT, SOME CLAY, TRACE SAND, TRACE STONE FRAGMENTS	A-4b (V)
B-002-0-23	SS-12	28.5	15											GRAY SANDY SILT, LITTLE CLAY, TRACE STONE FRAGMENTS	A-4a (V)
B-002-0-23	SS-13	31.0	13	18	13	5		10	9	25	39	56	17	GRAY SANDY SILT, LITTLE CLAY, TRACE STONE FRAGMENTS	A-4a (4)

Pro Geotech, Inc.

TR.-TRACE, BR.-BROWN, LI.-LITTLE, S/F-STONE FRAGMENTS, SO.-SOME, RB-ROADBASE, NP-NON-PLASTIC, POSS-POSSIBLE

Summary of Laboratory Results

Client: GPD GROUP
Project: SR 88 BRIDGE OVER IR-80
Location: PORTAGE COUNTY, OHIO
Pro. Number: G23017G

PRO US_LAB_ODOT_SUMMARY_ODOT - OH DOT.GDT - 11/2/23 12:48 - \\GEOTECHSERVER\SHARED FOLDERS\COMPANY\PUBLIC\PROJECT FILES\23 PROJECTS\G23017G-GPD-OTC(71-22-03)\LAB D

Boring Number	Sample Number	Depth (ft)	Water Content %	Liquid Limit %	Plastic Limit %	Plast. Index	Specific Gravity	Agg. %	Coarse Sand %	Fine Sand %	Silt %	Silt&Clay Comb. %	Clay %	Soil Description	Class. Symbol
B-002-0-23	SS-14	33.5	14											GRAY SANDY SILT, LITTLE CLAY, TRACE STONE FRAGMENTS	A-4a (V)
B-002-0-23	SS-15	36.0	14											GRAY SANDY SILT, LITTLE CLAY, TRACE STONE FRAGMENTS	A-4a (V)
B-002-0-23	SS-16A	38.5	10	22	15	7		30	11	18	27	42	15	GRAY SANDY SILT, SOME STONE FRAGMENTS, LITTLE CLAY	A-4a (1)
B-002-0-23	SS-16B	39.5	10											GRAY SANDY SILT, SOME STONE FRAGMENTS, LITTLE CLAY	A-4a (V)
B-002-0-23	SS-17	43.5	7											GRAY SILTSTONE FRAGMENTS	Rock (V)

Pro Geotech, Inc.

TR.-TRACE, BR.-BROWN, LI.-LITTLE, S/F-STONE FRAGMENTS, SO.-SOME, RB-ROADBASE, NP-NON-PLASTIC, POSS-POSSIBLE

Summary of Laboratory Results

Client: GPD GROUP
Project: SR 88 BRIDGE OVER IR-80
Location: PORTAGE COUNTY, OHIO
Pro. Number: G23017G

PRO US_LAB_ODOT_SUMMARY_ODOT - OH DOT.GDT - 11/2/23 12:48 - \\GEOTECHSERVER\SHARED FOLDERS\COMPANY\PUBLIC\PROJECT FILES\23 PROJECTS\G23017G-GPD-OTC(71-22-03)\LAB D

Point Load Test (ASTM D 5731)

Project: SR 88 Bridge **Boring No.:** B-001-0-23 **Date:** 10/31/2023

Project No.: G23017G **Depth Range:** 40.5'-50.5' **Technician:** NA

Rock Description: PENNSYLVANIAN-AGE CLAYSTONE, SILTSTONE, SHALE, AND SANDSTONE BEDROCK

Type of Test (Axial/Block/Diametral): Axial

No.	Type	W (mm)	D (mm)	L (psi)	P (lb)	D_e^2 (mm^2)	I_s (psi)	F	$I_{s(50)}$ (psi)
1	Axial ⊥	50.0	25.0	420.0	623.70	1593	252.62	0.904	228.26
2	Axial ⊥	50.3	41.0	200.0	297.00	2626	72.97	1.011	73.78
3	Axial ⊥	50.2	26.0	100.0	148.50	1660	57.71	0.912	52.63
4	Axial ⊥	50.3	19.0	5.0	7.43	1216	3.94	0.850	3.35
5	Axial ⊥	50.7	26.0	5.0	7.43	1678	2.85	0.914	2.61
6	Axial ⊥	50.6	19.0	5.0	7.43	1224	3.91	0.852	3.33
7	Axial ⊥	50.6	26.0	5.0	7.43	1675	2.86	0.914	2.61
8	Axial ⊥	50.5	35.0	5.0	7.425	2250	2.13	0.977	2.08
9	Axial ⊥	50.4	28.0	5.0	7.43	1797	2.67	0.928	2.48
10	Axial ⊥	50.2	36.0	5.0	7.43	2301	2.08	0.982	2.04
11	Axial ⊥	50.2	28.0	60.0	89.10	1790	32.12	0.928	29.79

Note: Bedrock was in dry Condition.

Mean Corrected Point Load Index $I_{s(50)}$ ⊥ (psi)	19.18

UCS = $I_{s(50)}$ x 12 (psi)	230

Piston Area = 1.485 sq. Inches

L = Applied Pressure
P = Failure Load
⊥ = Load Applied Perpendicular to Bedding
W = Core Sample Diameter
D = Height of Sample
UCS = Unconfined Compressive Strength

Point Load Test (ASTM D 5731)

Project: SR 88 Bridge **Boring No.:** B-002-0-23 **Date:** 10/30/2023

Project No.: G23017G **Depth Range:** 43.7'-53.7' **Technician:** NA

Rock Description: PENNSYLVANIAN-AGE CLAYSTONE, SILTSTONE, SHALE, AND SANDSTONE BEDROCK

Type of Test (Axial/Block/Diametral): Axial

No.	Type	W (mm)	D (mm)	L (psi)	P (lb)	D_e^2 (mm^2)	I_s (psi)	F	$I_{s(50)}$ (psi)
1	Axial ⊥	50.1	23.5	30	44.55	1498	19.18	0.891	17.09
2	Axial ⊥	50.3	40.2	100	148.50	2572	37.25	1.006	37.49
3	Axial ⊥	50.6	22.6	5	7.43	1455	3.29	0.885	2.92
4	Axial ⊥	50.7	33.5	100	148.50	2163	44.30	0.968	42.88
5	Axial ⊥	50.7	22.0	5	7.43	1420	3.37	0.881	2.97
6	Axial ⊥	50.0	20.0	5	7.43	1273	3.76	0.859	3.23
7	Axial ⊥	50.3	31.0	5	7.43	1985	2.41	0.949	2.29
8	Axial ⊥	50.1	19.0	140	207.90	1212	110.67	0.850	94.03
9	Axial ⊥	50.0	37.0	200	297.00	2355	81.35	0.987	80.26
10	Axial ⊥	50.0	21.0	400	594.00	1337	286.65	0.869	248.99
11	Axial ⊥	50.0	21.0	70	103.95	1337	50.16	0.869	43.57

Note: Bedrock was in dry Condition.

Mean Corrected Point Load Index $I_{s(50)}$ ⊥ (psi)	36.05

UCS = $I_{s(50)}$ x 12 (psi)	433

Piston Area = 1.485 sq. Inches

L = Applied Pressure
P = Failure Load
⊥ = Load Applied Perpendicular to Bedding
W = Core Sample Diameter
D = Height of Sample
UCS = Unconfined Compressive Strength

COMPANY: PGI DRILLED BY: GEI
PROJECT: OTIC 71-22-23
BRIDGE NO.: SR 88 BRIDGE
BORING: B-001-0-23 BOX 1/1
DATE of CORING: 9/20/23
RUN-1/NQ2-1: 40.5' - 45.5' REC: 94% RQD: 0
RUN-2/NQ2-2: 45.5' - 50.5' REC: 100% RQD: 8%

COMPANY: PGI DRILLED BY: GEI
PROJECT: OTIC 71-22-23
BRIDGE NO.: SR 88 BRIDGE
BORING: B-002-0-23 BOX 1/1
DATE of CORING: 9/19/23
RUN-1/NQ2-1: 43.7' - 48.7' REC: 100% RQD: 33%
RUN-2/NQ2-2: 48.7' - 53.7' REC: 98% RQD: 34%

BEARING CAPACITY ANALYSIS

AASHTO Article 10.6.3.1.2 and Munfakh (2001)

Project	**OTIC SR 88 Bridge Rear Abutment MSE Wall (No. 1)**
Project#	**G23017G**
Bore#	**B-001-0-23**
Method	**AASHTO Eqn 10.6.3.1.2a**
Foundation Dimension	
Width of Footing (B_f) (feet)	22.0
Length of Footing (L_f) (feet)	125.0
Length (L_f)/Width (B_f) (>5 is continuous footing)	5.7
Type of Footing	Strip
Footing Bearing Elevation (feet)	1057.0
Depth of Footing (D_f) Feet below Proposed Grade	3.5
Depth of groundwater Table (D_w) below Footing (ft)	3.9
Height of Slope (Hs) (feet)	Flat Ground
Soil Parameters	
Undrained Shear Strength/Cohesion (psf)	2700
Angle of internal friction (Phi) Degrees	0
Unit Weight of soil above base of footing (pcf)	120
Unit Weight of soil below base of footing (pcf)	122
Bearing Capacity Factors	
N_c	5.14
N_q	1.00
$N\gamma$	0.00
Shape Correction Factors	
s_c	1.020
s_q	1.000
s_γ	1.000
Load Inclination Factors	
ic	1.0
iq	1.0
i_γ	1.0
Correction for Water Table	
D_f+1.5B_f	36.5
C_{wq}	0.5
$C_{w\gamma}$	0.5
Embedment Depth Correction Factor	
Df/Bf	0.2
d_q	1.0
Bearing Capacity Terms	
Cohesion Term	14156
Surcharge Term	210
Unit Weight Term	0
Nominal Bearing Resistance (psf)	14366
Resistance Factor for bearing (per AASHTO Table 11.5.7-1)	0.65
Factored Bearing Resistance (psf)	9338

AASHTO Eqn 10.6.3.1.2a

qn = c*Nc*Sc*ic + (Gamma)*Df*Nq*sq*dq*iq*Cwq+0.5*(Gamma)*Bf*Nγ*sγ*iγ*Cwγ

BEARING CAPACITY ANALYSIS

AASHTO Article 10.6.3.1.2 and Munfakh (2001)

Project	**OTIC SR 88 Bridge Forward Abutment MSE Wall (No. 2)**
Project#	**G23017G**
Bore#	**B-002-0-23**
Method	**AASHTO Eqn 10.6.3.1.2a**
Foundation Dimension	
Width of Footing (B_f) (feet)	19.0
Length of Footing (L_f) (feet)	125.0
Length (L_f)/Width (B_f) (>5 is continuous footing)	6.6
Type of Footing	Strip
Footing Bearing Elevation (feet)	1058.0
Depth of Footing (D_f) Feet below Proposed Grade	3.5
Depth of groundwater Table (D_w) below Footing (ft)	4.9
Height of Slope (Hs) (feet)	Flat Ground
Soil Parameters	
Undrained Shear Strength/Cohesion (psf)	2500
Angle of internal friction (Phi) Degrees	0
Unit Weight of soil above base of footing (pcf)	120
Unit Weight of soil below base of footing (pcf)	120
Bearing Capacity Factors	
N_c	5.14
N_q	1.00
$N\gamma$	0.00
Shape Correction Factors	
s_c	1.030
s_q	1.000
s_γ	1.000
Load Inclination Factors	
ic	1.0
iq	1.0
i_γ	1.0
Correction for Water Table	
D_f+1.5B_f	32.0
C_{wq}	0.5
$C_{w\gamma}$	0.5
Embedment Depth Correction Factor	
Df/Bf	0.2
d_q	1.0
Bearing Capacity Terms	
Cohesion Term	13236
Surcharge Term	210
Unit Weight Term	0
Nominal Bearing Resistance (psf)	13446
Resistance Factor for bearing (per AASHTO Table 11.5.7-1)	0.65
Factored Bearing Resistance (psf)	8740

AASHTO Eqn 10.6.3.1.2a

qn = c*Nc*Sc*ic + (Gamma)*Df*Nq*sq*dq*iq*Cwq+0.5*(Gamma)*Bf*Nγ*sγ*iγ*Cwγ

Mechanically Stabilized Earth Wall (MSE Wall No. 1)
External Stability Calculations
State Route 88 Bridge over Ohio Turnpike (IR-80)

STEP 1. ESTABLISH PROJECT REQUIREMENTS

MSE wall height, H = 26.40 ft.
Length of wall = 125.17 ft.
Design Life = 100 years
Precast panel units: 5 ft wide x 5 ft tall x 0.5 ft thick
Type of reinforcement: Grade 65 (Fy = 65 ksi), steel bar mat with W15 and W11 wires.
Assume wires to be galvanized with zinc coating of 3.386 mils.
No seismic considerations

STEP 2. EVALUATE PROJECT PARAMETERS

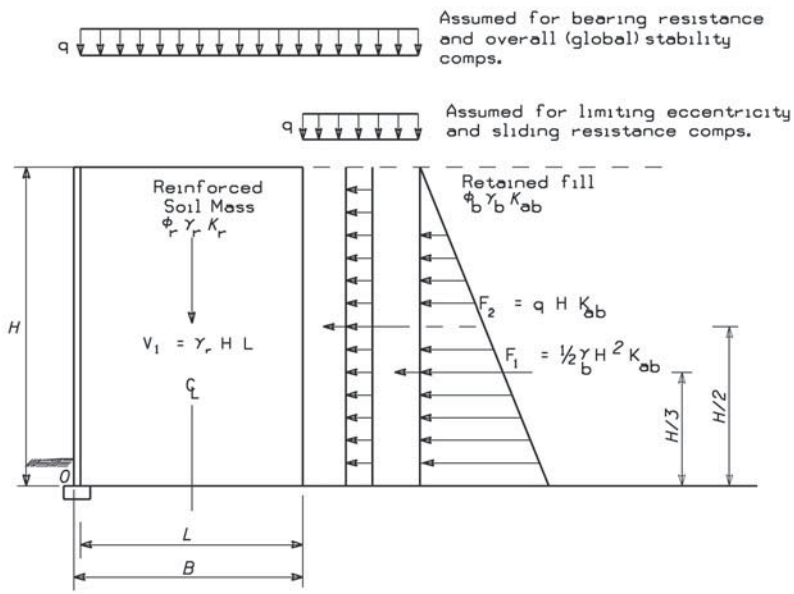
Reinforced backfill, $\Phi'_r = 34°$, $\gamma_r = 120$ pcf
Retained backfill, $\Phi'_f = 30°$, $\gamma_f = 120$ pcf
Foundation soil, Cohesive soil (A-4a) with $\Phi'_{fd} = 00°$, $\gamma_{fd} = 122$ pcf
Replace 6" foundation soil with Granular Type C to protect, Granular Material, $\Phi'_{fd} = 32°$, $\gamma_{fd} = 120$ pcf
Factored Bearing resistance of foundation soil at the Strength Limit State was calculated and is included in the Appendix B of the Geotechnical Exploration Report.
Factored Bearing resistance of foundation soil at the Service Limit State was calculated and Settlement analysis spreadsheet was included in the Appendix B of the Geotechnical Exploration Report.
Live load surcharge q = 250 psf

STEP 3. ESTIMATE DEPTH OF EMBEDMENT AND LENGTH OF REINFORCEMENT

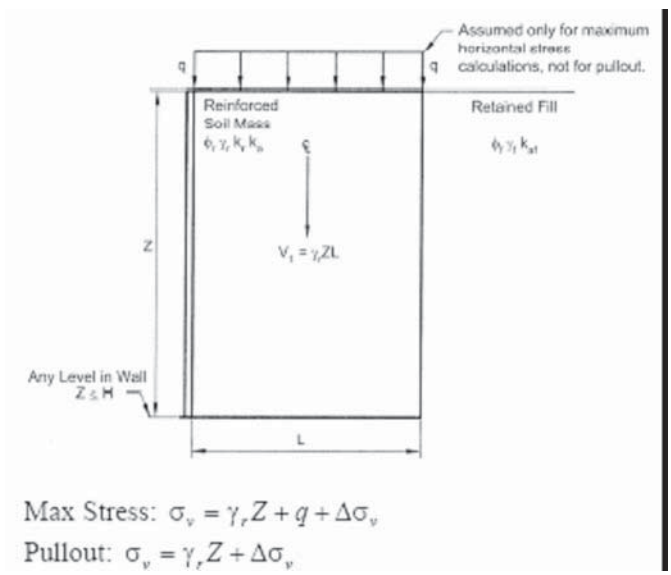
The minimum embedment depth in front of wall is equal to 3.5 feet based on the depth required for frost protection. Due to the level backfill, the minimum initial length of reinforcement is assumed to be 0.8H = 21.12 ft. and assumed L=22.0 ft. This length will be verified as part of the design process. The length of the reinforcement is assumed to be constant throughout the height to limit differential settlements across the reinforced zone because differential settlements could overstress the reinforcements.

STEP 4. ESTIMATE UNFACTORED LOADS

Attached Figure 4.2 shows the unfactored loads and equations for calculating unfactored loads and moment arms about Point A.
The moments are a product of the respective forces and moment arms. Each force is assigned a designation representing the applicable LRFD load type.
To compute the numerical values of various forces and moments, the parameters provided in Step 2 are used. Using the values of the various friction angles, the coefficients of lateral earth pressure are as follows:
Compute Active Earth Pressure Compute the coefficient of active earth pressure (ka) using Coulomb Theory.



(a)



(b)

Figure 4-2. Legend for computation of forces and moments (not-to-scale).

LRFD [Eq 3.11.5.3-1] with the wall backfill material interface friction angle δ, set equal to β (i.e. δ=β) LRFD [11.10.5.2]. The retained backfill soil will be used (i.e., $k_a=k_{af}$).

$\Phi'_f = 30$ deg, $\beta = 0$, $\theta = 90$ deg, $\delta = \beta$

$\Gamma = ((1 + ((\sin(\Phi'_f + \delta) \sin(\Phi'_f - \beta))/ (\sin(\theta - \delta) \sin(\theta + \beta)))^{1/2})^2$

$k_{af} = (\sin(\theta + \Phi'_f)^2 / (\Gamma \sin\theta^2 \sin(\theta - \delta))) = k_a = 0.333$

Table 4.1 - Equations of computing unfactored vertical forces and moments

Vertical Force Force/Length Unit	**LRFD Load Type**	**Moment arm Force/Length Unit @ Point A**
Vertical Force $V_1 = (\gamma_r)(H)(L)$	EV	L/2
Vertical Force $V_S = (q)L$	LS	L/2
Note: Live load surcharge q = 250 psf		

Table 4.2 - Equations of computing unfactored horizontal forces and moments

Horizontal Force Force/Length Unit	**LRFD Load Type**	**Moment arm Force/Length Unit @ Point A**
Horizontal Force $F_1 = ½(K_{af})(\gamma_f)H^2$	EH	H/3
Horizontal Force $F_2 = (K_{af})(q)](H)$	LS	H/2

The unfactored forces and moments in Tables 4.3 and 4.4 form the basis of all computations in this problem.

Table 4.3 - Unfactored vertical forces and moments

Force	**Value k/ft.**	**Moment Arm @ Point A, ft.**	**Moment**	**Moment at Point A, k-ft./ft.**
$V_1 =$	69.70	11.0	$MV_1=$	766.70
$V_s =$	5.50	11.0	$MV_s =$	60.50

Table 4.4 - Unfactored horizontal forces and moments

Force	**Value k/ft.**	**Moment Arm @ Point A, ft.**	**Moment**	**Moment at Point A, k-ft./ft.**
$F_1 =$	13.93	8.80	$MF_1 =$	122.58
$F_2 =$	2.20	13.2	$MF_2 =$	29.04

The unfactored forces and moments should be multiplied by the appropriate load factors based on the LRFD load types to perform the analysis for various load combinations such as Strength I, Service I, etc. The load factors for various load types relevant to this problem are discussed in Step 5.

STEP 5. SUMMARIZE APPLICABLE LOAD AND RESISTANCE FACTORS

Table 5.1 summarizes the load factors for the various LRFD load type shown in second column of Tables 4.1 and 4.2.

Table 5.1 - Summary of applicable load factors

Load Combination	Load Factors (According to BDM S11.10.2.2 In Table 5.1)		
	EV	**EH**	**LS**
Strength I (maximum)	1.35	1.50	1.75
Strength I (minimum)	1.00	1.50	1.75
Service I	1.00	1.00	1.00

Throughout the computations in this problem, the forces and moments in Tables 4.3 and 4.4 should be multiplied by appropriate load factors in Table 5.1. For example, if computations are being done for Strength I (maximum) load combination, the forces and moments corresponding to load V_1 should be multiplied by 1.35 which is associated with load type EV assigned to load V_1. For computation of factored resistances during evaluation of strength limits states, appropriate resistance factors have to be used. Table 5.2 summarizes the applicable resistance factors. For service limit state, all resistance factors are equal to 1.0.

Table 5.2-Summary of applicable resistance factors for evaluation of resistances

Item	Resistance Factors	AASHTO (2014)
Sliding of MSE wall on foundation soil	$\Phi_s = 1.00$	Table 11.5.7-1
Bearing resistance	$\Phi_b = 0.65$	Table 11.5.7-1

STEP 6. EVALUATE EXTERNAL STABILITY OF MSE WALL

The external stability of MSE wall is a function of the various forces and moments shown in Figure 4.2. In the LRFD context the forces and moments need to be categorized into various load types. The primary load types for this problem are soil loads (EV, EH) and live load (LS).

6.1 Sliding Resistance at Base of MSE Wall

The purpose of these computations is to evaluate the sliding resistance at the base of the MSE wall. Since the computations are related to sliding resistance, the beneficial contribution of live load surcharge to resisting forces and moments is neglected. The computations for sliding resistance at the base of the MSE wall are illustrated in Table 6.1. Note that sliding resistance is a strength limit state check and therefore service limit state calculations are not performed. Since the friction angle of foundation soil, Φ'_{fd} , is less than the friction angle for reinforced soil, Φ'_r, the sliding check will be performed using Φ'_{fd}. The critical values based on max/min result in the extreme force effect and govern the sliding mode of failure.

Table 6.1 - Computations for evaluation of sliding resistance of MSE wall

Item	Unit	Str. I (max)	Str. I (min)	Ser. I
Lateral load on the MSE wall, $H_m = F_1+F_2$	k/ft.	24.75	24.75	NA

Item	Unit	Str. I (max)	Str. I (min)	Ser. I
Vertical load at base of MSE wall without LS surcharge = V_1	k/ft.	94.10	69.70	NA
Nominal sliding resistance at base of MSE wall, V_{Nm} = tan $\Phi'_{fd})(V_1)$	k/ft.	58.80	43.55	NA
Sliding resistance at base of MSE wall, $V_{Fm}= \Phi_s * V_{Nm}$	k/ft.	58.80	43.55	NA
Is $V_{Fm} > H_m$?	-	Yes	Yes	NA
Capacity:Demand Ratio (CDR) = $V_{Fm:}H_m$	Dim	2.376	1.760	NA
CRITICAL VALUES BASED ON MAX/MIN				
Minimum V_{Fm} (V_{Fmmin})	k/ft.	43.55		
Maximum H_m (H_{mmax})	k/ft.	24.75		
Is $VF_{mmin} > H_{mmax}$?	-	Yes		
Capacity:Demand Ratio (CDR) = $V_{Fmmin:}H_{mmax}$	Dim	1.760		

6.2 Limiting Eccentricity at Base of MSE Wall

The purpose of these computations is to evaluate the limiting eccentricity at the base of the MSE wall. Since the computations are related to limiting eccentricity, the beneficial contribution of live load surcharge to resisting forces and moments is neglected. The computations for limiting eccentricity at the base of the MSE wall are illustrated in Table 6.2. Limiting eccentricity is a strength limit state check and therefore service limit state calculations are not performed. The critical values based on max/min result in the extreme force effect and govern the limiting eccentricity mode of failure.

Table 6.2 - Computations for evaluation of limiting eccentricity for MSE wall

Item	Unit	Str. I (max)	Str. I (min)	Ser. I
Total vertical load at base of MSE wall without LS, $V_A = V_1$	k/ft.	94.10	69.70	N/A
Resisting moments about Point A without LS surcharge= $M_{RA} = MV_1$	k-ft./ft.	1035.05	766.70	N/A
Overturning moments about Point A = M_{OA} = MF_1+MF_2	k-ft./ft.	234.71	234.71	N/A
Net moment about Point A = $M_A = M_{RA} – M_{OA}$	k-ft./ft.	800.64	531.99	N/A
Location of the resultant force on base of MSE wall from Point A, a = M_A/V_A	ft.	8.51	7.63	N/A
Eccentricity at base of MSE wall, $e_L = L/2 – a$	ft.	2.49	3.37	N/A
Limiting eccentricity, e = L/3 for strength limit state	ft.	7.33	7.33	N/A
Is the resultant within limiting value of e?	-	Yes	Yes	N/A
Calculated e_L/L	-	0.113	0.153	N/A
CRITICAL VALUES BASED ON MAX/MIN				

Item	Unit	Str. I (max)	Str. I (min)	Ser. I
Max. Overturning moments about Point A, M_{OA-C}	k-ft./ft.	234.71		
Mini. Resisting moments about Point A, M_{RA-C}	k-ft./ft.	766.70		
Net moment about Point A, $M_{A-C} = M_{RA-C} - M_{OA-C}$	k-ft./ft.	531.99		
Vertical force, V_{A-C}	k/ft.	69.70		
Location of resultant from Point A, $a_{nl} = M_{A-C}/V_{A-C}$	ft.	7.63		
Eccentricity from center of wall base, e_L=0.5*L - a_{nl}	ft.	3.37		
Limiting eccentricity, e = L/3	ft.	7.33		
Is the limiting eccentricity criteria satisfied?	-	Yes		
Effective width of base of MSE wall, B' = L-2e_L	Ft.	15.26		
Calculated e_L/L	-	0.153		

6.3 Bearing Resistance at base of MSE Wall

For bearing resistance computations, the effect of live load surcharge is included since it creates larger bearing stresses. The bearing stress at the base of the MSE wall can be computed as follows:

$\sigma_v = \Sigma v/(L - 2e_L)$

where $\Sigma V = R = V_1 + V_S$ is the resultant of vertical forces and the load eccentricity e_L is calculated by principles of statics using appropriate loads and moments with the applicable load factors.
In LRFD, σ_v is compared with the factored bearing resistance when computed for strength limit state and used for settlement analysis when computed for service limit state. The various computations for evaluation of bearing resistance are presented in Table 6.3. The Strength I (max) load combination results in the extreme force effect in terms of maximum bearing stress and therefore governs the bearing resistance mode of failure. The Service I load combination is evaluated to compute the bearing stress for settlement analysis.

Table 6.3 - Computations for evaluation of bearing resistance for MSE wall

Item	Unit	Str. I (max)	Str. I (min)	Ser. I
Vertical load at base of MSE wall including LS on top, $\Sigma V = R = V_1 + V_S$	k/ft.	103.72	79.33	75.2
Resisting moments @ Point A on the MSE wall, $M_{RA} = MV_1+MV_S$	k-ft./ft.	1140.92	872.58	827.20
Overturning moments @ Point A on the MSE wall, $M_{OA} = MF_1+MF_2$	k-ft./ft.	234.69	234.69	151.62
Net moment at Point A, $M_A = M_{RA} - M_{OA}$	k-ft./ft.	906.23	637.89	675.58

Item	Unit	Str. I (max)	Str. I (min)	Ser. I
Location of Resultant from Point A, $a = M_A/\Sigma V$	Ft.	8.74	8.04	8.98
Eccentricity from center of wall base, $e_L = 0.5*L - a$	Ft.	2.26	2.96	2.02
Limiting eccentricity, e = L/4 for strength limit states and e= L/6 for service limit state	Ft.	5.50	5.50	3.67
Is the resultant within limiting value of e_L?	-	Yes	Yes	Yes
Effective width of base of MSE wall, $B' = L-2e_L$	Ft.	17.48	16.08	17.96
Bearing stress due to MSE wall $=\Sigma V/(L-2e_L) = \sigma v$	Ksf	5.93	4.93	4.19
Bearing resistance, ($q_{nf\text{-}str}$ for strength) or ($q_{nf\text{-}ser}$ for service) (given)	Ksf	9.0	9.0	9.0
Is bearing stress less than the bearing resistance?	-	Yes	Yes	Yes
Capacity:Demand Ratio (CDR) = q_{nf}:σv	Dim	1.52	1.83	2.15
CRITICAL VALUES BASED ON MAX/MIN				
Resisting moments about Point A, $M_{RA\text{-}C}$	k-ft./ft.	872.58		
Overturning moments about Point A, $M_{OA\text{-}C}$	k-ft./ft.	234.69		
Net moment about Point A, $M_{A\text{-}C} = M_{RA\text{-}C} - M_{OA\text{-}C}$	k-ft./ft.	637.89		
Vertical force, ΣV_C	k/ft.	79.33		
Location of resultant from Point A, $a = M_{A\text{-}C}/\Sigma V_C$	Ft.	8.04		
Eccentricity from center of wall base, $e_L = 0.5*L - a$	Ft.	2.96		
Limiting eccentricity, e = L/4	Ft.	5.50		
Is the limiting eccentricity criteria satisfied?	-	Yes		
Effective width of base of MSE wall, $B' = L-2e_L$	Ft.	16.08		
Bearing stress, $\Sigma VC / (L-2e_L) = \sigma v\text{-}c$	Ksf	4.93		
Factored Bearing resistance, qnf-str (given)	Ksf	9.00		
Is bearing stress < bearing resistance?	Dim	Yes		
Capacity:Demand Ratio (CDR) = qnf-str:σv-c	Dim	1.83		

6.4 Settlement Analysis

The foundation soils below the MSE Wall were already consolidated under the weight of existing embankment soils and no settlement analyses was performed at Service Limit State.

References: FHWA NHI-10-025 Example E4 – Level backfill with LS

Mechanically Stabilized Earth Wall (MSE Wall No. 2)
External Stability Calculations
State Route 88 Bridge over Ohio Turnpike (IR-80)

STEP 1. ESTABLISH PROJECT REQUIREMENTS

MSE wall height, H = 23.58 ft.
Length of wall = 125.17 ft.
Design Life = 100 years
Precast panel units: 5 ft wide x 5 ft tall x 0.5 ft thick
Type of reinforcement: Grade 65 (Fy = 65 ksi), steel bar mat with W15 and W11 wires.
Assume wires to be galvanized with zinc coating of 3.386 mils.
No seismic considerations

STEP 2. EVALUATE PROJECT PARAMETERS

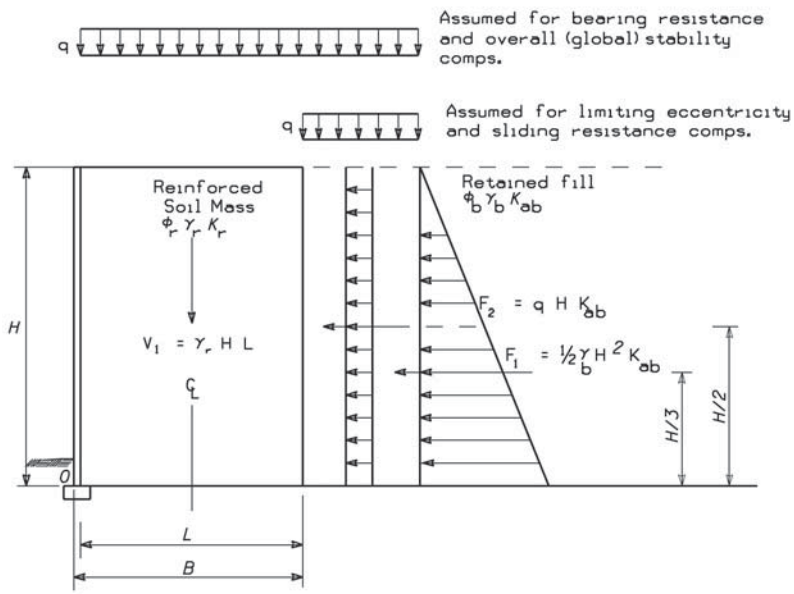
Reinforced backfill, $\Phi'_r = 34°$, $\gamma_r = 120$ pcf
Retained backfill, $\Phi'_f = 30°$, $\gamma_f = 120$ pcf
Foundation soil, Cohesive soil (A-4b) with $\Phi'_{fd} = 00°$, $\gamma_{fd} = 120$ pcf
Replace 12" foundation soil with Granular Type C to protect, Granular Material, $\Phi'_{fd} = 32°$, $\gamma_{fd} = 120$ pcf
Factored Bearing resistance of foundation soil at the Strength Limit State was calculated and is included in the Appendix B of the Geotechnical Exploration Report.
Factored Bearing resistance of foundation soil at the Service Limit State was calculated and Settlement analysis spreadsheet was included in the Appendix B of the Geotechnical Exploration Report.
Live load surcharge q = 250 psf

STEP 3. ESTIMATE DEPTH OF EMBEDMENT AND LENGTH OF REINFORCEMENT

The minimum embedment depth in front of wall is equal to 3.5 feet based on the depth required for frost protection. Due to the level backfill, the minimum initial length of reinforcement is assumed to be 0.8H = 18.86 ft. and assumed L=19.0 ft. This length will be verified as part of the design process. The length of the reinforcement is assumed to be constant throughout the height to limit differential settlements across the reinforced zone because differential settlements could overstress the reinforcements.

STEP 4. ESTIMATE UNFACTORED LOADS

Attached Figure 4.2 shows the unfactored loads and equations for calculating unfactored loads and moment arms about Point A.
The moments are a product of the respective forces and moment arms. Each force is assigned a designation representing the applicable LRFD load type.
To compute the numerical values of various forces and moments, the parameters provided in Step 2 are used. Using the values of the various friction angles, the coefficients of lateral earth pressure are as follows:
Compute Active Earth Pressure Compute the coefficient of active earth pressure (ka) using Coulomb Theory.



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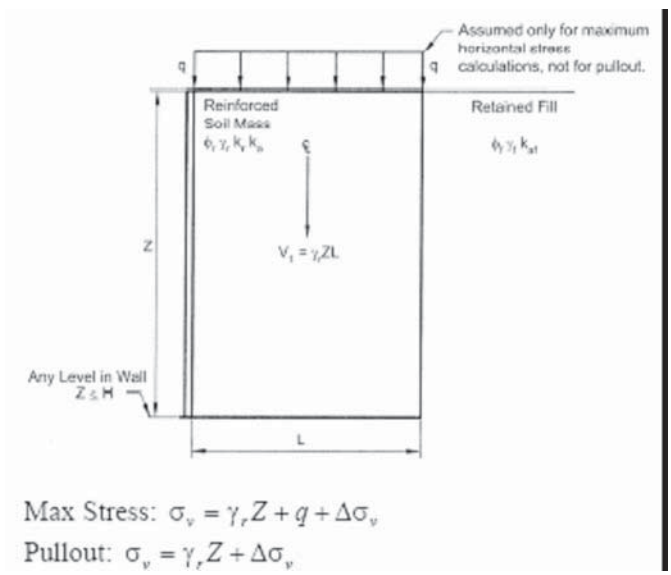


(b)

Figure 4-2. Legend for computation of forces and moments (not-to-scale).

LRFD [Eq 3.11.5.3-1] with the wall backfill material interface friction angle δ, set equal to β (i.e. δ=β) LRFD [11.10.5.2]. The retained backfill soil will be used (i.e., $k_a=k_{af}$).

$\Phi'_f = 30$ deg, $\beta = 0$, $\theta = 90$ deg, $\delta = \beta$

$\Gamma = ((1 + ((\sin(\Phi'_f + \delta) \sin(\Phi'_f - \beta))/ (\sin(\theta - \delta) \sin(\theta + \beta)))^{1/2})^2$

$k_{af} = (\sin(\theta + \Phi'_f)^2 / (\Gamma \sin\theta^2 \sin(\theta - \delta))) = k_a = 0.333$

Table 4.1 - Equations of computing unfactored vertical forces and moments

Vertical Force Force/Length Unit	**LRFD Load Type**	**Moment arm Force/Length Unit @ Point A**
Vertical Force $V_1 = (\gamma_r)(H)(L)$	EV	L/2
Vertical Force $V_S = (q)L$	LS	L/2
Note: Live load surcharge q = 250 psf		

Table 4.2 - Equations of computing unfactored horizontal forces and moments

Horizontal Force Force/Length Unit	**LRFD Load Type**	**Moment arm Force/Length Unit @ Point A**
Horizontal Force $F_1 = ½(K_{af})(\gamma_f)H^2$	EH	H/3
Horizontal Force $F_2 = (K_{af})(q)](H)$	LS	H/2

The unfactored forces and moments in Tables 4.3 and 4.4 form the basis of all computations in this problem.

Table 4.3 - Unfactored vertical forces and moments

Force	**Value k/ft.**	**Moment Arm @ Point A, ft.**	**Moment**	**Moment at Point A, k-ft./ft.**
V_1 =	53.76	9.5	MV_1=	510.72
V_s =	4.75	9.5	MV_s =	45.03

Table 4.4 - Unfactored horizontal forces and moments

Force	**Value k/ft.**	**Moment Arm @ Point A, ft.**	**Moment**	**Moment at Point A, k-ft./ft.**
F_1 =	11.11	7.86	MF_1 =	87.32
F_2 =	1.96	11.79	MF_2 =	23.11

The unfactored forces and moments should be multiplied by the appropriate load factors based on the LRFD load types to perform the analysis for various load combinations such as Strength I, Service I, etc. The load factors for various load types relevant to this problem are discussed in Step 5.

STEP 5. SUMMARIZE APPLICABLE LOAD AND RESISTANCE FACTORS

Table 5.1 summarizes the load factors for the various LRFD load type shown in second column of Tables 4.1 and 4.2.

Table 5.1 - Summary of applicable load factors

Load Combination	Load Factors (According to BDM S11.10.2.2 In Table 5.1)		
	EV	EH	LS
Strength I (maximum)	1.35	1.50	1.75
Strength I (minimum)	1.00	1.50	1.75
Service I	1.00	1.00	1.00

Throughout the computations in this problem, the forces and moments in Tables 4.3 and 4.4 should be multiplied by appropriate load factors in Table 5.1. For example, if computations are being done for Strength I (maximum) load combination, the forces and moments corresponding to load V_1 should be multiplied by 1.35 which is associated with load type EV assigned to load V_1. For computation of factored resistances during evaluation of strength limits states, appropriate resistance factors have to be used. Table 5.2 summarizes the applicable resistance factors. For service limit state, all resistance factors are equal to 1.0.

Table 5.2-Summary of applicable resistance factors for evaluation of resistances

Item	Resistance Factors	AASHTO (2014)
Sliding of MSE wall on foundation soil	$\Phi_s = 1.00$	Table 11.5.7-1
Bearing resistance	$\Phi_b = 0.65$	Table 11.5.7-1

STEP 6. EVALUATE EXTERNAL STABILITY OF MSE WALL

The external stability of MSE wall is a function of the various forces and moments shown in Figure 4.2. In the LRFD context the forces and moments need to be categorized into various load types. The primary load types for this problem are soil loads (EV, EH) and live load (LS).

6.1 Sliding Resistance at Base of MSE Wall

The purpose of these computations is to evaluate the sliding resistance at the base of the MSE wall. Since the computations are related to sliding resistance, the beneficial contribution of live load surcharge to resisting forces and moments is neglected. The computations for sliding resistance at the base of the MSE wall are illustrated in Table 6.1. Note that sliding resistance is a strength limit state check and therefore service limit state calculations are not performed. Since the friction angle of foundation soil, Φ'_{fd} , is less than the friction angle for reinforced soil, Φ'_r, the sliding check will be performed using Φ'_{fd}. The critical values based on max/min result in the extreme force effect and govern the sliding mode of failure.

Table 6.1 - Computations for evaluation of sliding resistance of MSE wall

Item	Unit	Str. I (max)	Str. I (min)	Ser. I
Lateral load on the MSE wall, $H_m = F_1+F_2$	k/ft.	20.10	20.10	NA

Item	Unit	Str. I (max)	Str. I (min)	Ser. I
Vertical load at base of MSE wall without LS surcharge = V_1	k/ft.	72.58	53.76	NA
Nominal sliding resistance at base of MSE wall, V_{Nm} = tan $\Phi'_{fd})(V_1)$	k/ft.	45.35	33.59	NA
Sliding resistance at base of MSE wall, $V_{Fm}= \Phi_s *V_{Nm}$	k/ft.	45.35	33.59	NA
Is $V_{Fm} > H_m$?	-	Yes	Yes	NA
Capacity:Demand Ratio (CDR) = V_{Fm}:H_m	Dim	2.256	1.671	NA
CRITICAL VALUES BASED ON MAX/MIN				
Minimum V_{Fm} (V_{Fmmin})	k/ft.	33.59		
Maximum H_m (H_{mmax})	k/ft.	20.10		
Is $VF_{mmin} > H_{mmax}$?	-	Yes		
Capacity:Demand Ratio (CDR) = V_{Fmmin}:H_{mmax}	Dim	1.671		

6.2 Limiting Eccentricity at Base of MSE Wall

The purpose of these computations is to evaluate the limiting eccentricity at the base of the MSE wall. Since the computations are related to limiting eccentricity, the beneficial contribution of live load surcharge to resisting forces and moments is neglected. The computations for limiting eccentricity at the base of the MSE wall are illustrated in Table 6.2. Limiting eccentricity is a strength limit state check and therefore service limit state calculations are not performed. The critical values based on max/min result in the extreme force effect and govern the limiting eccentricity mode of failure.

Table 6.2 - Computations for evaluation of limiting eccentricity for MSE wall

Item	Unit	Str. I (max)	Str. I (min)	Ser. I
Total vertical load at base of MSE wall without LS, $V_A = V_1$	k/ft.	72.58	53.76	N/A
Resisting moments about Point A without LS surcharge= $M_{RA} = MV_1$	k-ft./ft.	689.47	510.72	N/A
Overturning moments about Point A = M_{OA} = MF_1+MF_2	k-ft./ft.	171.44	171.44	N/A
Net moment about Point A = $M_A = M_{RA} – M_{OA}$	k-ft./ft.	518.03	339.28	N/A
Location of the resultant force on base of MSE wall from Point A, a = M_A/V_A	ft.	7.14	6.31	N/A
Eccentricity at base of MSE wall, $e_L = L/2 – a$	ft.	2.36	3.19	N/A
Limiting eccentricity, e = L/3 for strength limit state	ft.	6.33	6.33	N/A
Is the resultant within limiting value of e?	-	Yes	Yes	N/A
Calculated e_L/L	-	0.124	0.168	N/A
CRITICAL VALUES BASED ON MAX/MIN				

Item	Unit	Str. I (max)	Str. I (min)	Ser. I
Max. Overturning moments about Point A, M_{OA-C}	k-ft./ft.	171.44		
Mini. Resisting moments about Point A, M_{RA-C}	k-ft./ft.	510.72		
Net moment about Point A, $M_{A-C} = M_{RA-C} - M_{OA-C}$	k-ft./ft.	339.28		
Vertical force, V_{A-C}	k/ft.	53.76		
Location of resultant from Point A, $a_{nl} = M_{A-C}/V_{A-C}$	ft.	6.31		
Eccentricity from center of wall base, e_L=0.5*L - a_{nl}	ft.	3.19		
Limiting eccentricity, e = L/3	ft.	6.33		
Is the limiting eccentricity criteria satisfied?	-	Yes		
Effective width of base of MSE wall, B' = L-2e_L	Ft.	12.62		
Calculated e_L/L	-	0.168		

6.3 Bearing Resistance at base of MSE Wall

For bearing resistance computations, the effect of live load surcharge is included since it creates larger bearing stresses. The bearing stress at the base of the MSE wall can be computed as follows:

$\sigma_v = \Sigma v/(L - 2e_L)$

where $\Sigma V = R = V_1 + V_S$ is the resultant of vertical forces and the load eccentricity e_L is calculated by principles of statics using appropriate loads and moments with the applicable load factors.
In LRFD, σ_v is compared with the factored bearing resistance when computed for strength limit state and used for settlement analysis when computed for service limit state. The various computations for evaluation of bearing resistance are presented in Table 6.3. The Strength I (max) load combination results in the extreme force effect in terms of maximum bearing stress and therefore governs the bearing resistance mode of failure. The Service I load combination is evaluated to compute the bearing stress for settlement analysis.

Table 6.3 - Computations for evaluation of bearing resistance for MSE wall

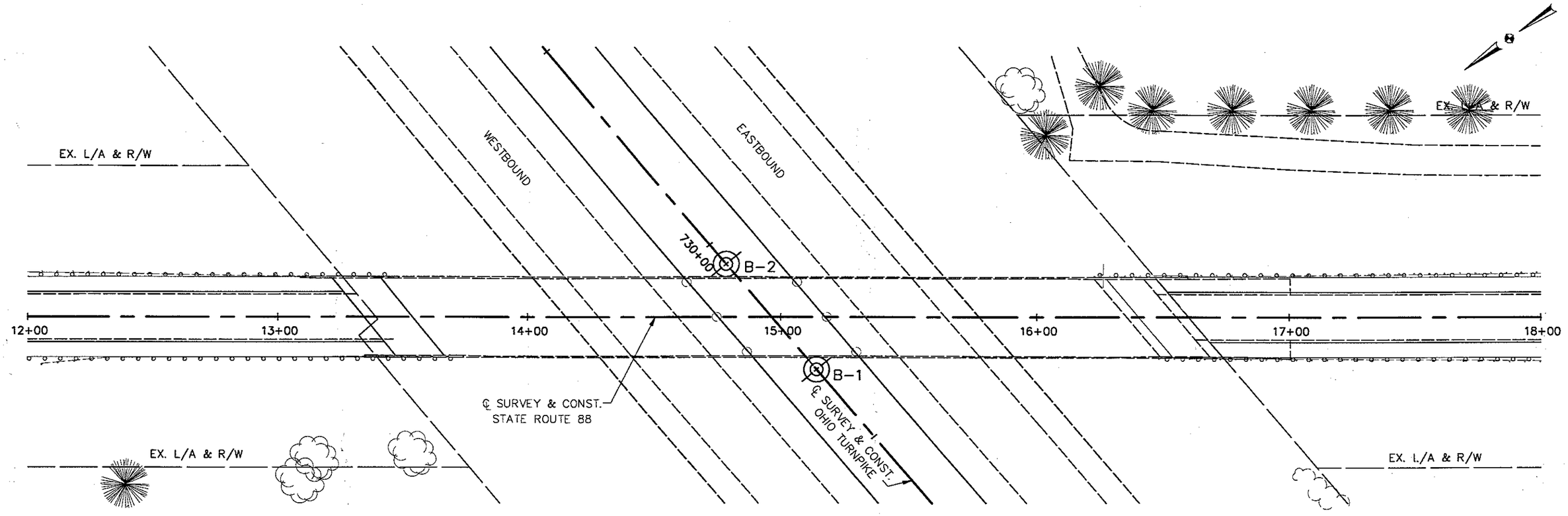
Item	Unit	Str. I (max)	Str. I (min)	Ser. I
Vertical load at base of MSE wall including LS on top, $\Sigma V = R = V_1 + V_S$	k/ft.	80.89	62.07	58.51
Resisting moments @ Point A on the MSE wall, $M_{RA} = MV_1+MV_S$	k-ft./ft.	768.45	589.70	555.85
Overturning moments @ Point A on the MSE wall, $M_{OA} = MF_1+MF_2$	k-ft./ft.	171.44	171.44	110.44
Net moment at Point A, $M_A = M_{RA} - M_{OA}$	k-ft./ft.	597.01	418.26	445.41

Item	Unit	Str. I (max)	Str. I (min)	Ser. I
Location of Resultant from Point A, a = $M_A/\Sigma V$	Ft.	7.38	6.74	7.61
Eccentricity from center of wall base, e_L = 0.5*L – a	Ft.	2.12	2.76	1.89
Limiting eccentricity, e = L/4 for strength limit states and e= L/6 for service limit state	Ft.	4.75	4.75	3.17
Is the resultant within limiting value of e_L?	-	Yes	Yes	Yes
Effective width of base of MSE wall, B' = L-2e_L	Ft.	14.76	13.48	15.22
Bearing stress due to MSE wall =$\Sigma V/(L-2e_L) = \sigma v$	Ksf	5.48	4.60	3.84
Bearing resistance, ($q_{nf\text{-}str}$ for strength) or ($q_{nf\text{-}ser}$ for service) (given)	Ksf	8.00	8.00	8.00
Is bearing stress less than the bearing resistance?	-	Yes	Yes	Yes
Capacity:Demand Ratio (CDR) = q_{nf}:σv	Dim	1.46	1.74	2.08
CRITICAL VALUES BASED ON MAX/MIN				
Resisting moments about Point A, $M_{RA\text{-}C}$	k-ft./ft.	589.70		
Overturning moments about Point A, $M_{OA\text{-}C}$	k-ft./ft.	171.44		
Net moment about Point A, $M_{A\text{-}C} = M_{RA\text{-}C} - M_{OA\text{-}C}$	k-ft./ft.	418.26		
Vertical force, ΣV_C	k/ft.	62.07		
Location of resultant from Point A, a = $M_{A\text{-}C}/\Sigma V_C$	Ft.	6.74		
Eccentricity from center of wall base, e_L = 0.5*L – a	Ft.	2.76		
Limiting eccentricity, e = L/4	Ft.	4.75		
Is the limiting eccentricity criteria satisfied?	-	Yes		
Effective width of base of MSE wall, B' = L-2e_L	Ft.	13.48		
Bearing stress, ΣVC / (L-2e_L) = σv-c	Ksf	4.60		
Factored Bearing resistance, qnf-str (given)	Ksf	8.0		
Is bearing stress < bearing resistance?	Dim	Yes		
Capacity:Demand Ratio (CDR) = qnf-str:σv-c	Dim	1.74		

6.4 Settlement Analysis

The foundation soils below the MSE Wall were already consolidated under the weight of existing embankment soils and no settlement analyses was performed at Service Limit State.

References: FHWA NHI-10-025 Example E4 – Level backfill with LS



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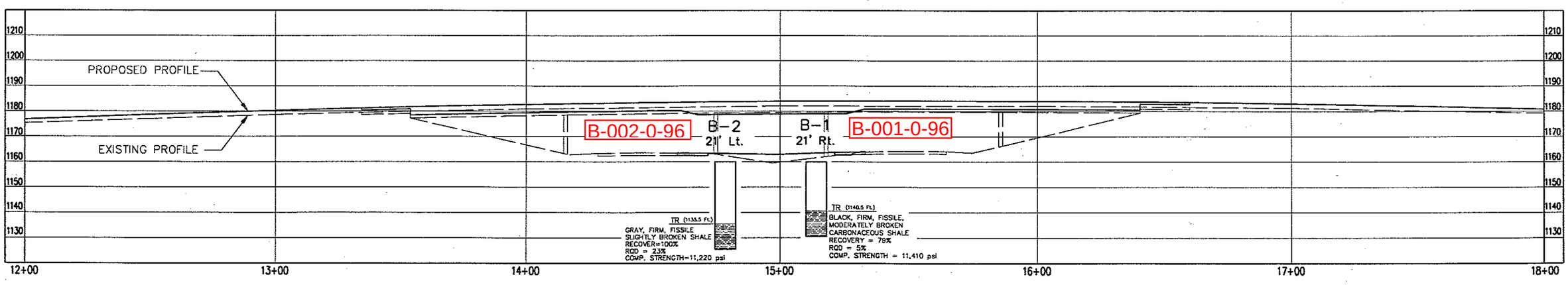


ELEVATION

NO.	REVISIONS	BY	DATE

OHIO TURNPIKE COMMISSION

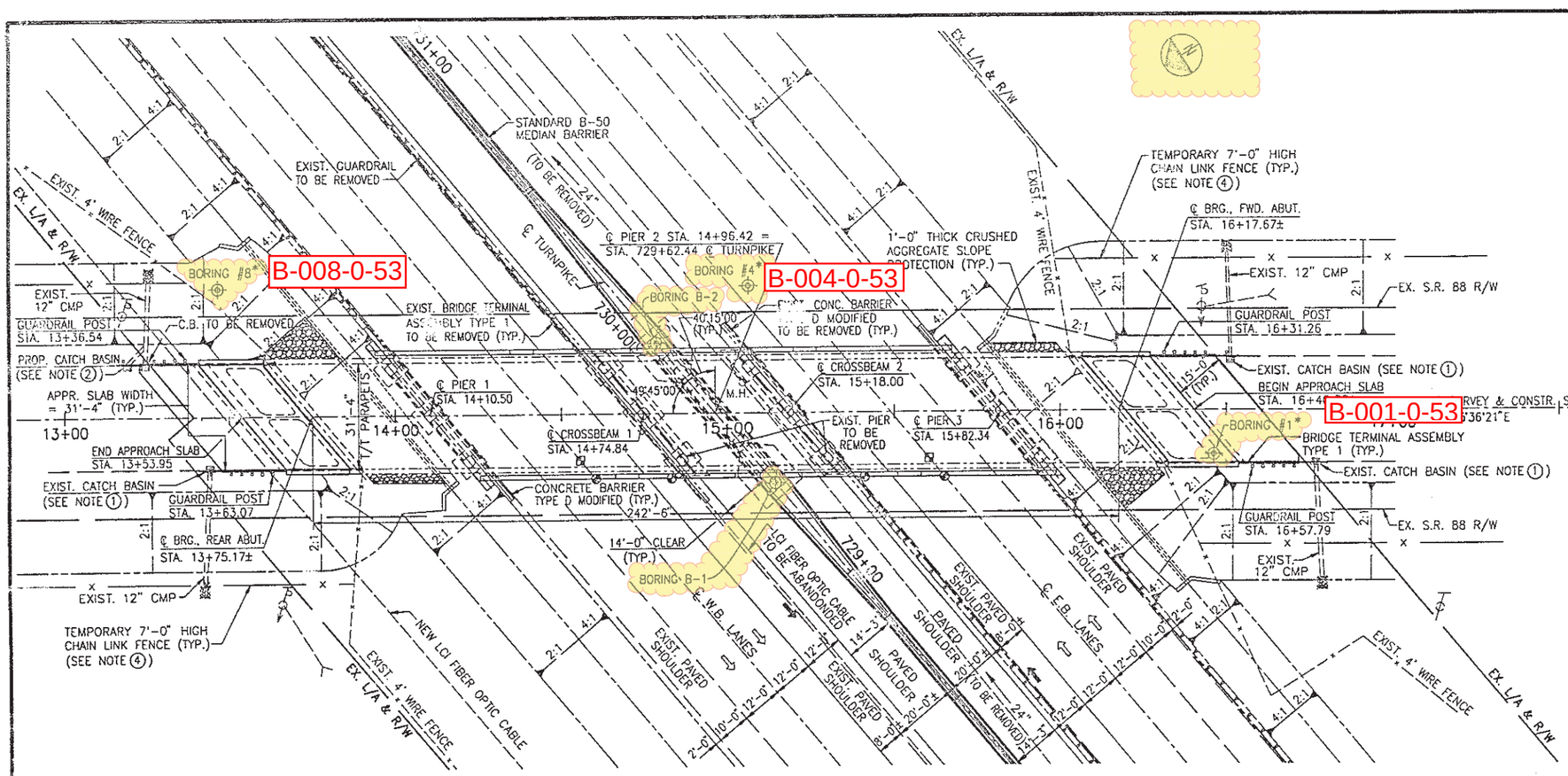
STRUCTURE FOUNDATION INVESTIGATION
S.R. 88 OVER THE OHIO TURNPIKE

RESOURCE INTERNATIONAL INC.
281 ENTERPRISE DR.
WESTERVILLE, OHIO 43081
(614) 885-1959

DESIGNED:	CHECKED: GPH	DATE: 2/96
DRAWN: JRA	IN CHARGE: GPH	SCALE: 1"=20'

1 / 1 SECTION E-2 SHEET OF

RESOURCE INTERNATIONAL INC.
PROJECT NO. W-5100
FILENAME: 5100S1-1.DWG
DATE: 22_FEB_1996

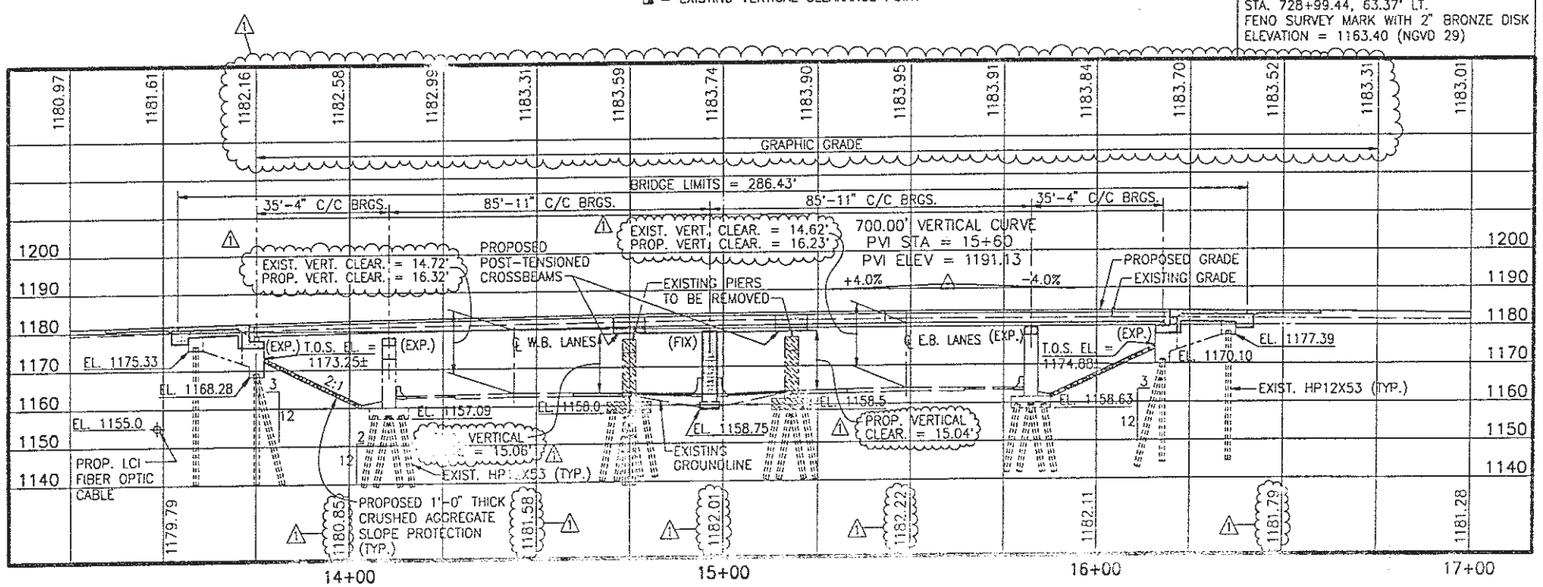


PLAN

⊕ = PROPOSED VERTICAL CLEARANCE POINT
⊠ = EXISTING VERTICAL CLEARANCE POINT

BENCH MARK

STA. 728+99.44, 63.37' LT.
FENO SURVEY MARK WITH 2" BRONZE DISK
ELEVATION = 1163.40 (NGVD 29)



℄ PROFILE

* DATA FOR EXISTING BORINGS 1, 4, AND 8 OBTAINED FROM ORIGINAL DRAWING SHEET 34 OF 36, CONTRACT NO. C-10 DATED 2-17-53.

BORING NO.	APPROXIMATE TOP OF ROCK ELEVATION
1 *	1143
4 *	1128
8 *	1135
B-1	1140
B-2	1135

NOTES:

1. EXISTING CATCH BASINS TO BE RECONSTRUCTED TO GRADE. REFER TO SHEETS 164 & 165 OF 219 FOR DETAILS.
2. PROPOSED CATCH BASIN TO REPLACE EXISTING AND TIE IN TO EXISTING 12" CMP. REFER TO SHEETS 164 & 165 OF 219 FOR DETAILS.
3. ALL UTILITY POLES ARE TO REMAIN. (DO NOT DISTURB.)
4. FOR TEMPORARY FENCE DETAILS REFER TO SHEETS 164 & 165 OF 219.
5. EARTHWORK LIMITS SHOWN ARE APPROXIMATE. ACTUAL SLOPES SHALL CONFORM TO PLAN CROSS SECTIONS.

EXISTING STRUCTURE

TYPE: 5-SPAN CONTINUOUS COMPOSITE STEEL BEAMS W/REINFORCED CONCRETE PIERS AND ABUTMENTS
SPANS: 35'-4", 64'-4", 43'-2", 64'-4", 35'-4" C/C BRGS.
ROADWAY: 31'-4" TOE/TOE BARRIERS
SKEW: 40°15'00" (R.F.)
DESIGN LOADING: HS20-44 (CASE II) AND ALTERNATE MILITARY LOADING W/O FUTURE WEARING SURFACE
APPROACH SLABS: O.T.C. STANDARD (15'-0" LONG, 10" THICK)
CROWN: 3/16"/FT.
ALIGNMENT: TANGENT
WEARING SURFACE: MONOLITHIC CONCRETE
STRUCTURE FILE NUMBER: 6729789

REHABILITATED STRUCTURE

TYPE: 5-SPAN CONTINUOUS COMPOSITE STEEL BEAMS SUPPORTED BY 2-SPAN CONTINUOUS STEEL CARRYING GIRDERS AND REINFORCED CONCRETE SUBSTRUCTURES
SPANS: 85'-11", 85'-11" C/C BRGS. (CARRYING GIRDERS) 35'-4", 64'-4", 43'-2", 64'-4", 35'-4" C/C BRGS. (BEAMS)
ROADWAY: 31'-4" TOE/TOE BARRIERS
SKEW: 40°15'00" (R.F.)
DESIGN LOADING: HS20-44 (CASE II) AND ALTERNATE MILITARY LOADING
APPROACH SLABS: O.T.C. STANDARD (15'-0" LONG, 10" THICK)
CROWN: 3/16"/FT.
ALIGNMENT: TANGENT
WEARING SURFACE: MONOLITHIC CONCRETE
AVERAGE DAILY TRAFFIC: 4,150 (1996) 5,580 (2016)
AVERAGE DAILY TRUCK TRAFFIC: 208 (1996) 446 (2016)
COORDINATES: LAT.: N41°14'33" LONG.: W81°07'42"

NO.	REVISIONS	BY	DATE
1	REVISED PROFILE GRADE ELEVS. & VERTICAL CLEARANCES AND ADDED GRAPHIC GRADE LIMITS.	ms	3-96

OHIO TURNPIKE COMMISSION

SITE PLAN
STATE ROUTE 88 OVER
OHIO TURNPIKE (MP 199.5)

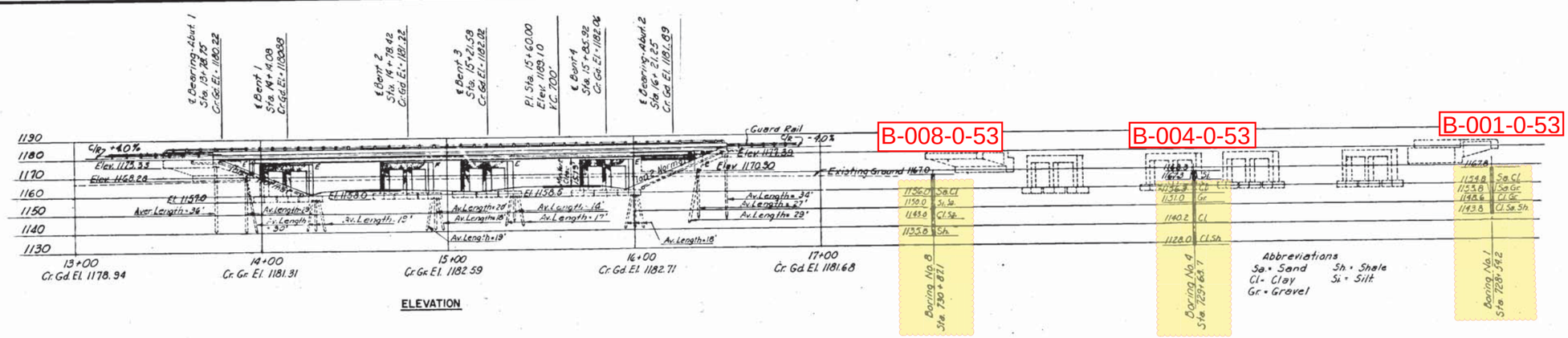
ms consultants, inc.
YOUNGSTOWN, OHIO

DESIGNED:	G.K.	CHECKED:	W.H.	DATE:	10-24-95
DRAWN:	R.A.M.	IN CHARGE:	D.F.S.	SCALE:	N.T.S.

STR #1 ROM C:\D6153208\SITEPLN4 1=20 13:14 03/07/96

71 M-3

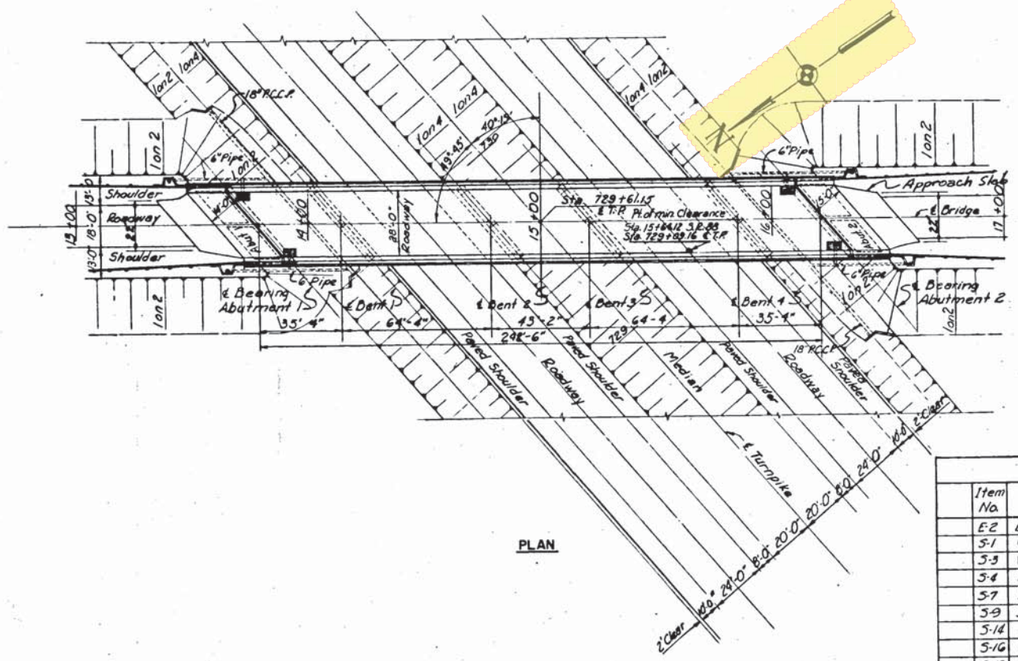
QUANTITIES					
No.	Item	Location & Remarks	Unit	Estimated Quantity	Final Quantity



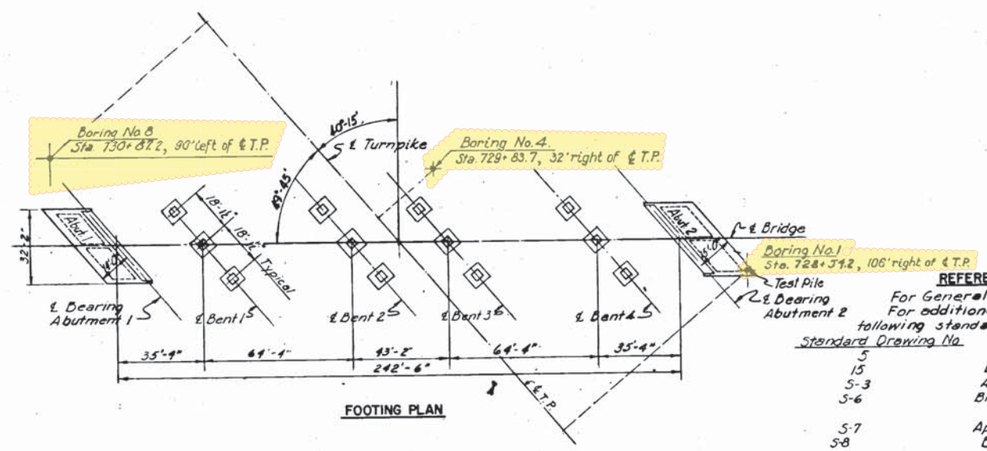
ELEVATION

BORING DATA

(See Supplemental Boring Data.)



PLAN



FOOTING PLAN

REFERENCES

For General Notes see Sheet 1
For additional details refer to the following standard drawings.

Standard Drawing No.	Title
5	Handrail Details
15	Drainage Details
S-3	Abutments - Roadway = 28'-0"
S-6	Bridge Floor - Roadway = 28'-0" and 24'-0"
S-7	Approach Slabs.
S-8	Bents

CF = 30

QUANTITIES

Item No.	Item	Unit	Abutment 1 Est Q	Abutment 1 Final Q	Abutment 2 Est Q	Abutment 2 Final Q	Bent 1 Est Q	Bent 1 Final Q	Bent 2 Est Q	Bent 2 Final Q	Bent 3 Est Q	Bent 3 Final Q	Bent 4 Est Q	Bent 4 Final Q	Superstructure Est Q	Superstructure Final Q	Totals Est Q	Totals Final Q
E-2	Excav. for Structures (Dry)	Cu.Yds.	100	106.0	100	105.6	72	43.2	59	40.8	47	33.6	57	34.9			435	366.1
S-1	Conc. for Structures, Class C	Cu.Yds.	107	106.10	110	108.47	48	47.04	51	50.38	48	46.72	48	46.78	236	242.11	648	647.60
S-3	Waterproofing, Type C	Sq.Yds.													875	839.0	875	839.0
S-4	Reinforcing Steel	Lbs.	12,520	11,895.84	12,870	12,400.89	6,125	6,071.37	6,115	6,177.68	6,125	6,065.25	6,125	6,065.25	53,900	53,666.19	103,780	107,143.02
S-7	Structural Steel	Lbs.													195,620	198,388	195,620	198,388
S-9	Struct. Exp. &/or Contr. Joints	Lbs.													9,100	8,904	9,100	8,904
S-14	Metal Handrail, Type B	Lin.Ft.													562	562.08	562	562.08
S-16	First Test Pile	Each			0	1	1	0									1	1
S-18	Furnishing Bearing Piles, Steel 12BP53	Lin.Ft.	318	350	338	340	220	269	250	283	233	264	233	264			1,592	1,750
S-18	Driving Bearing Piles, Steel 12BP53	Lin.Ft.	318	329.15	338	277.55	220	243.30	250	264.70	233	213.81	233	235.58			1,592	1,564.09
S-10	Splice for Bearing Piles, Steel 12BP53	Each	10	0	10	0	13	0	16	0	13	0	13	0			73	0
T-35	Asph. Conc. Surf. Course, Type C	Cu.Yds.													30	29.10	30	29.10
S-29	Scuppers, Type B	Each													4	4	4	4
S-29	Drainage of Struct, 6" W.I. Pipe	Lin.Ft.													120	112.96	120	112.96
S-29	Drainage of Struct, 6" A.Pipe	Lin.Ft.													160	202.20	160	202.20

No.	Revision	By	Date
2	As-Built Plans	H.R.L.	
1	Minor checking, revisions	R.S.H.	7-21-59

OHIO TURNPIKE COMMISSION

OHIO TURNPIKE PROJECT NO. 1

STRUCTURE NO. 29
STATE ROUTE 88
GENERAL PLAN AND ELEVATION

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONTRACTING ENGINEER
DESIGN SECTION D-3

DESIGNED: W.R.S.-E.L. CHECKED: H.L.M. DATE: 2-17-58
DRAWN: D.M.E. IN CHARGE: L.D.S.W. SCALE: 1" = 30'

CONTRACT NO. C-10 SHEET 34 OF 36

LABORATORY TEST STANDARDS

STANDARD	REFERENCE NUMBER
I. Soil/Rock Testing	
Description and Identification of Soils (Visual-Manual Procedures)	ASTM D 2488
Classification of Soils for Engineering Purposes (USCS*).*	ASTM D 2487
Laboratory Determination of Water (Moisture) Content of Soil and Rock	ASTM D 2216
Classification for Sizes of Aggregate for Road and Bridge Construction	ASTM D 488
Liquid Limit, Plastic Limit, and Plasticity Index of Soils	ASTM D 4318
Shrinkage Factors of Soils by Mercury Method	ASTM D 427
Moisture, Ash, and Organic Matter of Peat and Other Organic Soils	ASTM D 2974
Specific gravity of Soils	ASTM D 854
Direct Shear Test of Soils under Consolidated Drained Conditions	ASTM D 3080
Particle-Size Analysis of Soils	ASTM D 422
Unconfined Compressive Strength of Cohesive Soils	ASTM D 2166
Compressive Strength of Intact Rock Core Specimens	ASTM D 7012
Slake Durability Index of Shale/Similar Weak Rock Test	ASTM D 4644
Point Load Test of Rock Core Specimens	ISRM* / ASTM D5731
CBR (California Bearing Ration) of Laboratory-Compacted Soils	ASTM D 1883
Laboratory Compaction Characteristics of Soil using Standard Effort	ASTM D 698
Laboratory Compaction Characteristics of Soil using Modified Effort	ASTM D 1557
One-Dimensional Consolidation Properties of Soils	ASTM D 2435
One-Dimensional Swell or Settlement Potential of Cohesive Soils	ASTM D 4546
Ph of Soil	ASTM D 4972

*ISRM – International Society for Rock Mechanics

II. Concrete Testing

Compressive Strength for Cylindrical Concrete Specimens	ASTM C-39
Acid-Soluble Chloride in Mortar and Concrete	ASTM C 1152

CLASSIFICATION OF SOILS

Ohio Department of Transportation

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

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

MATERIAL CLASSIFIED BY VISUAL INSPECTION

- Sod and Topsoil
- Pavement or Base
- Uncontrolled Fill (Describe)
- Bouldery Zone
- Peat, S-Sedimentary W-Woody F-Fibrous L-Loamy & etc

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

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

1) STRENGTH OF SOIL:

Non-Cohesive (granular) Soils - Compactness	
Description	**Blows Per Ft.**
Very Loose	≤ 4
Loose	5 – 10
Medium Dense	11 – 30
Dense	31 – 50
Very Dense	> 50

2) COLOR :

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

3) PRIMARY COMPONENT

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

Cohesive (fine grained) Soils - Consistency

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

4) COMPONENT MODIFIERS:

Description	Percentage By Weight
Trace	0% - 10%
Little	10% - 20%
Some	20% - 35%
"And"	35% -50%

5) Soil Organic Content

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

6) Relative Visual Moisture

Description	Criteria	
	Cohesive Soil	**Non-cohesive Soils**
Dry	Powdery; Cannot be rolled; Water content well below the plastic limit	No moisture present
Damp	Leaves very little moisture when pressed between fingers; Crumbles at or before rolled to $^1/_8$"; Water content below plastic limit	Internal moisture, but no to little surface moisture
Moist	Leaves small amounts of moisture when pressed between fingers; Rolled to $^1/_8$" or smaller before crumbling; Water content above plastic limit to -3% of the liquid limit	Free water on surface, moist (shiny) appearance
Wet	Very mushy; Rolled multiple times to $^1/_8$" or smaller before crumbles; Near or above the liquid limit	Voids filled with free water, can be poured from split spoon.

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

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

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

3) **WEATHERING**

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

4) **RELATIVE STRENGTH**

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

5) **TEXTURE**

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

6) **BEDDING**

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

7) DESCRIPTORS

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

8) DISCONTINUITIES

a) Discontinuity Types

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

b) Degree of Fracturing

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

c) Aperture Width

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

d) Surface Roughness

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

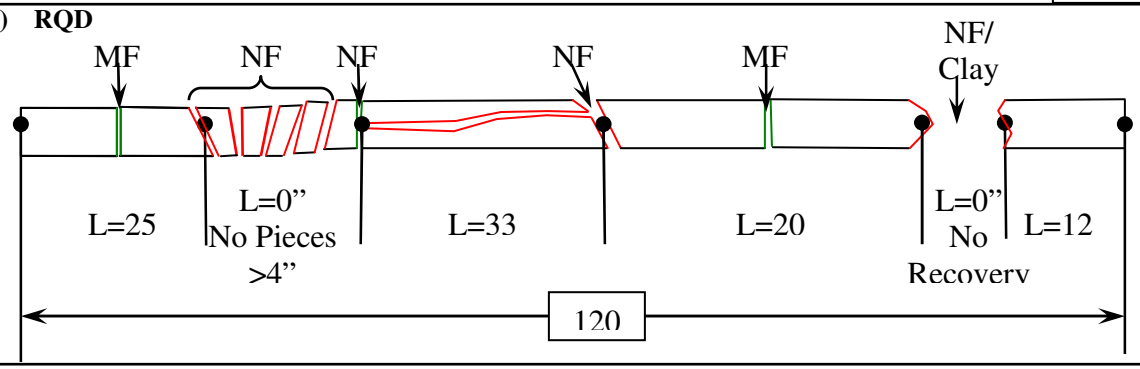
10) LOSS

$$Run\ Loss = \left(\frac{L_R - R_R}{L_R}\right) * 100 \quad Unit\ Loss = \left(\frac{L_U - R_U}{L_U}\right) * 100$$

L_R=Run Length R_R=Run Recovery

L_U=Rock Unit Length R_U=Rock Unit Recovery

9) RQD



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

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