Pro Geotech, Inc.

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August 27, 2014

Mr. Naiel Hussein, P.E.
Parsons Brinckerhoff
2545 Farmers Drive, Suite 350
Columbus, Ohio 43235

Reference: **Final Structure Foundation Exploration Report for HAN-75-14.39**
Bridge No. HAN-75-1477 over IR-75 Mainline and Norfolk Southern Railroad
Findlay, Hancock County, Ohio
PID No. 87005
PGI Project No. G13011G

Dear Mr. Hussein:

Enclosed please find our Final Structure Foundation Exploration Report for the above referenced project. Our services included a geotechnical field exploration, laboratory testing, engineering analysis, and related design and construction recommendations. These services have been provided in accordance with our proposal dated January 16, 2013. 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.

S. Sivakumaran

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

Walid Najjar

Walid I. Najjar, P.E.
Senior Geotechnical Engineer

Enclosure
G13011Grpt/HAN-75-1477Bridges/SS/8/27/2014

FINAL
STRUCTION FOUNDATION EXPLORATION REPORT
FOR HAN-75-14.39
BRIDGE NO. HAN-75-1477 OVER IR-75 AND NORFOLK SOUTHERN RAILROAD

HANCOCK COUNTY, OHIO
PGI PROJECT NO. G13011G
PID NO. 87005

PREPARED FOR:

PARSONS BRINCKERHOFF

PREPARED BY:

PRO GEOTECH, INC.

AUGUST 27, 2014

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

APPENDICES

A Boring Location Map
Drilling Logs
Test Boring Profile

B Laboratory Test Results
Compressive Strength of the Rock Core
Rock Core Sample Pictures
Rock Mass Rating Spreadsheets
Bearing Resistance and Settlement Analyses of Footing on Jointed Rock
External Stability Analysis Computer Output for MSE Wall
Global Stability Analyses Computer Output for MSE Wall
Historic Exploration Information
Geotechnical Design Checklist
Laboratory Test Standards
ODOT Soil Classification System

1.0 EXECUTIVE SUMMARY

This report has been prepared for the HAN-75-14.39 project which calls for replacement of the Harrison Street Bridge No. HAN-75-1477 over Interstate Route 75 (IR-75) mainline and Norfolk Southern Railroad as part of redesigning the IR-75/US 68 Interchange in Findlay, Hancock County, Ohio This bridge project begins at Harrison Street station 105+61.16 and ends at Harrison Street station 110+39.66. A total of four (4) project test borings identified as B-167-0-13, B-168-0-13, B-170-0-13, and B-175-0-13 were advanced for bridge and MSE wall foundation design purposes. Test borings B-167-0-13 and B-168-0-13 were advanced in the vicinity of the proposed rear abutment and MSE wall, while test boring B-175-0-13 was advanced in the vicinity of the proposed forward abutment. Test boring B-170-0-13 was advanced in the vicinity of the proposed bridge Pier 2. These structural test borings were advanced to approximate depths ranging from 9.5 to 42.7 feet below the existing ground or Harrison Street pavement surface. Soil and rock information obtained from the historic borings B-004-0-60, B-009-0-60 and B-013-0-60 will also be used to design the piers of the proposed bridge.

Subsurface soil Conditions: The subsurface soils encountered in test borings B-167-0-13, B-170-0-13, and B-175-0-13 consisted of fill materials and in B-168-0-13 consisted of both fill materials and natural soils. The fill material encountered above the natural soils consisted of sandy silt (A-4a), silt (A-4b), silt and clay (A-6a), silty clay (A-6b), clay (A-7-6), and non-plastic sandy silt (A-4a). The approximate thickness of the fill materials ranged from 3.2 feet in test boring B-170-0-13 to 32.3 feet in B-175-0-13 above the natural soils or bedrock. Natural soils encountered below the fill material and above bedrock in test borings B-168-0-13 and B-170-0-13 consisted of silty clay (A-6b). Bedrock was encountered in all test boring locations at approximate depths ranging from 3.2 feet to 32.3 feet below the existing ground or pavement surface. The consistency of the cohesive soils ranged from "medium stiff" to "hard", but was generally “stiff”. The relative density of the non-cohesive soils was “medium dense”.

Bedrock Conditions: The core samples consisted of dolomite of the Tymochtee/Greenfield Group. The dolomite was light gray to gray, moderately to slightly weathered, and strong. Bedding within the dolomite was generally very thin to thin and was highly fractured to moderately fractured. No slickensides were observed and the fractures were typically tight to narrow and slightly rough to very rough. The compressive strength of the project core specimens ranged from 4,596 psi in test boring B-170-0-13 to 20,441 psi in test boring B-175-0-13 which characterizes them as “moderately strong” to "very strong", respectively. Bedrock was also encountered in three historic test borings which were advanced during a subsurface exploration performed in December 1960. These core samples are described as dolomite, gray with massive bedding, hard, broken with trace carbonaceous shale.

The Rock Quality Designation (RQD) for the core samples ranged from 18% to 65% and averaged 45% based on individual runs. The Rock Mass Rating obtained for the bedrock core samples according to LRFD Table 10.4.6.4-1 varied from 42 to 47 and is classified as "Fair Rock".

Bridge Foundation Systems: Soil and rock information obtained from proposed project test borings B-168-0-13, B-170-0-13, and B-175-0-13 and historic project test borings B-004-0-60, B-009-0-60, and B-013-0-60 were used to provide foundation recommendations for this proposed replacement bridge. Since bedrock was encountered at relatively deeper depths below the Harrison Street pavement at the abutment locations, the proposed bridge superstructure loads may be transferred to the underlying bedrock by means of piles at the abutment locations. Since bedrock was encountered at relatively shallower depths below the existing ground surface at the pier locations, the proposed bridge superstructure loads may be transferred to the underlying bedrock by means of spread footings at the pier locations.

The bridge rear abutment above the MSE wall embankment may be supported on end bearing H piles. According to the ODOT *Bridge Design Manual* Section 204.4, the end bearing H-piles should be installed in pre-bored holes with a minimum embedment length of 5 feet into bedrock. End bearing H piles may be used to transfer the proposed bridge superstructure loads to the underlying bedrock at the forward abutment location. The end bearing H piles should be driven to refusal on the underlying dolomite bedrock. The estimated pile parameters for end bearing piles at each boring location are summarized in Table 6.1.1. Based on the factored axial loads acting on the piles, the estimated 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.	Bottom of MSE Wall Elevation	Pile Cut-off Elevation (ft)	Pile Tip Elevation (ft)	Estimated Effective Pile Length (ft)	Pile Type	Pile Size	Maximum Factored Structural Resistance/pile
Rear Abutment							
B-168-0-13	778.5	795.1	772.8	25	H-Pile	10X42	310 kips
Forward Abutment							
B-175-0-13	--	798.7	774.6	25	H-Pile	10X42	310 kips

Spread footings may be used to transfer the loads to the underlying bedrock at the proposed pier locations. Bearing resistance for spread footings on bedrock was evaluated using a semi-empirical method at the test boring B-170-0-13 location. Table 6.1.3 summarizes the factored bearing resistance on bedrock and founding elevation at each test boring location so that PB personnel can evaluate or compare the factored bearing resistance to the factored bearing pressure.

Table 6.1.3 – Estimated Design Parameters at Strength Limit State for Spread Footings

Boring No.	Substructure Location	Top of Bedrock Elevation (feet)	Proposed Bearing Elevation (feet)	Nominal Resistance (ksf)	Factored Resistance (ksf)
B-004-0-60	Pier 1	776.8±	776.5	28.0	13.0
B-170-0-13	Pier 2	775.2±	774.5	28.0	13.0

<u>MSE Wall Foundation Systems</u>: Soil and rock information obtained from structural test borings B-167-0-13 and B-168-0-13 were used to provide foundation recommendations for the proposed MSE Wall at the rear abutment location. As per the boring logs, bedrock was encountered below the bottom of the rear MSE Wall at depths of 1.7 feet in test boring B-167-0-13 and 0.7 feet in test boring B-168-0-13. Since MSE Wall base will bear on bedrock, all existing soils above the bedrock should be removed before the MSE Wall construction. Factored bearing resistance corresponding to bearing elevation at the MSE Wall boring location is summarized in Table 6.2.1. It is estimated that the total and differential settlement of the underlying foundation rock will be within the tolerable total settlement of 12 inches and differential settlement of one percent for MSE Wall. No waiting period is required at the end of MSE Wall Construction.

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

Boring No.	Location	Depth of Bottom of Footing Below Final Grade (feet)	Width of Strip Footing (feet)	Proposed Bearing Elevation (feet)	Factored Bearing Resistance (ksf)
B-167-0-13	Rear MSE Wall	5.7	20.0	778.5	20.0
B-168-0-13	Rear MSE Wall	5.7	20.0	778.5	20.0

External stability of the MSE Wall including sliding on the base, limiting eccentricity, and bearing resistance at the Strength Limit State and settlement analysis at the Service Limit State were performed at the rear and forward abutment location. The External Stability analyses results show that the Capacity Demand Ratio (CDR) value against sliding, CDR value with respect to bearing resistance and eccentricity value are within the acceptable limits for the selected foundation width of the rear MSE Wall. Global stability analyses were performed using the GSTABL7 with STEDwin, version 2.0 program that was developed by Mr. Garry H. Gregory, P.E. to estimate the Factor of Safety for the proposed MSE Walls. Table 6.2.2 summarizes the safety factors for the short term and long term stability of the proposed MSE Walls. Based on this slope stability analysis, the calculated Safety Factors for both short term and long term meet the required Safety Factors specified in the ODOT Embankment Checklist.

Table 6.2.2 –Summary of Critical Factors of Safety for MSE Walls

Boring No	Location	Stability	Method Used	Factor of Safety
B-168-0-13	Rear MSE Wall	Short Term	Circular	2.83
	Rear MSE Wall	Long Term	Circular	2.46

2.0 INTRODUCTION

This report has been prepared for the HAN-75-14.39 project which calls for replacement of the Harrison Street Bridge No. HAN-75-1477 over Interstate Route 75 (IR-75) mainline and Norfolk Southern Railroad as part of redesigning the IR-75/US 68 Interchange in Findlay, Hancock County, Ohio. It represents the intent of Parsons Brinckerhoff (PB) the design engineer, and the Ohio Department of Transportation (ODOT), the owner, to secure subsurface information at the selected locations in accordance with ODOT's *Specifications for Geotechnical Exploration*s, and to obtain recommendations regarding geotechnical factors pertaining to the design and construction of this project.

2.1 Project Description

Present plans call for the design and construction of the proposed Bridge No. HAN-75-1477 which will carry Harrison Street vehicular traffic over IR-75 mainline and Norfolk Southern Railroad. This bridge project begins at Harrison Street station 105+61.16 and ends at Harrison Street station 110+39.66. The design information provided by PB personnel indicates that the proposed replacement bridge will be constructed on the same alignment and location as the existing bridge. The replacement bridge will be a three (3) span structure with a total approximate span length of 478.5 feet. The proposed superstructure will be continuous plate girders with reinforced concrete decking on abutments and piers. The structure will be supported on reinforced concrete stub abutments on capped piles and cap and column piers on spread footing. Mechanically Stabilized Earth (MSE) Wall abutment type will be used at the rear abutment and spill-through abutment type will be used at the forward abutment. This bridge is to be designed utilizing LRFD Bridge Design Specifications. The Site Location Map is shown in Figure 2.1.

This report has been developed based on the field exploration program, laboratory testing, and information secured for site-specific studies. 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.

2.2 Scope of Services

The scope of services for this project was in accordance with Pro Geotech, Inc. (PGI) Proposal No. PG12067 dated January 16, 2013 and governed by ODOT's *Specifications for Geotechnical Explorations* dated January 2007 and updated January 20, 2012 and ODOT's Bridge Design Manual, issued in 2007 and AASHTO LRFD Bridge Design Specifications, 6th 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 the 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 four (4) test borings in the vicinity of proposed Bridge No. HAN-75-1477 over IR 75 mainline and Norfolk Southern Railroad for structural foundation design purposes. These structural test borings for the bridge and MSE Wall were to be advanced to approximate depths ranging from 25.0 feet to 40.0 feet below the existing ground surface or Harrison Street pavement shoulder, and included obtaining 10 to 15 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 bridges and retaining walls including shallow and deep foundations
- Recommendations for MSE walls which will include external stability analysis, settlement, drag-down forces, and lateral earth pressures
- Preparation of ODOT Geotechnical Design Checklists
- Preparation of Geotechnical Structure Foundation Exploration Plans

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, the project site lies on the Huron-Erie Lake Plains Section of the Central Lowland Province. The project site is located within the Findlay Embayment District of the Maumee Lake Plains Region of the Huron-Erie Lake Plains Section. The project site is located at approximate elevations ranging from 778 feet to 808 feet. According to Bulletin 44, *Geology of Water in Ohio* (issued in 1943 and reprinted in 1968), both the Illinoian and Wisconsin Glaciers passed over the area and left a coating of drift materials less than 5 feet in thickness. The main geologic deposit of the project site consists of silty to gravelly Wisconsinan-age lacustrine deposits and wave-planed clay till; ground moraine, flat to gently undulating over Dolomite bedrock of Silurian-age. Based on the *Soil Survey of Hancock County, Ohio* and from the *U.S. Department of Agriculture, Natural Resource Conservation Service* website, the natural site soils in the vicinity of the project area consist primarily of layers of loam, clay loam, fine sandy loam, silty clay loam, and silty clay. These soils are classified as A-4, A-6, and A-7 based on the AASHTO Soil Classification System. However, the project site has incurred cut and fill operations due to construction of existing IR-75. Thus the composition of the surface and subsurface soils has changed from natural in most areas.

Based on information obtained from the Ohio Geological Survey, bedrock in the vicinity of the project site was deposited during the Upper and Lower Silurian Period of the Paleozoic Era and is expected to consist of Tymochtee/Greenfield Group dolomite. Tymochtee Group dolomite is described as shades of gray and brown, very finely crystalline which occurs as thin to massive beds with carbonaceous shale laminae and beds. Greenfield Group dolomite is described as shades of gray and brown; very finely to coarsely crystalline which occurs as massive beds to laminae; argillaceous and locally brecciated in the lower portion. According to ODNR's Ohio Gas and Oil Wells Locator website, many active and abandoned wells are located in the vicinity of the project site. According to ODNR's Ohio Mines Locator website, no abandoned underground or surface mines are present in the immediate vicinity of the project site. Based on the Ohio Division of Geological Survey Interactive Map of Ohio Mineral Industries, an active limestone industrial quarry is located approximately 0.4 miles southwest of the project site. According to ODNR, the project site is located outside of the "Probable Karst Regions" of Ohio and outside of the "Landslide-Prone Areas" of Ohio. According to ODNR website, two (2) earthquakes occurred within Hancock County; one in 1990 with a magnitude of 2.3 Richter Scale and another in 2011 with a magnitude of 2.4 Richter Scale. Their epicenters were located respectively approximately 8.8 miles to the northeast in Big Lick Township and 14.2 miles to the south in Delaware Township.

3.2 Observations

The reconnaissance of the project site was performed by one of PGI's geotechnical engineers in July 2013. The project site is located in a rural area with the closest building located approximately 200 feet from the bridge site. The embankment sections of the existing Harrison Street Bridge approaches generally appeared to be in good condition with no surface erosion observed. No visible signs of embankment slope instability or embankment settlement were observed. The concrete pier columns and caps generally appeared to be in fair to good condition. Few areas of spalling concrete and exposed reinforcement were observed on the pier concrete columns. Light to moderate efflorescence was observed on the exposed abutment concrete surfaces. Longitudinal and traverse cracks, very light in frequency, were observed along the top concrete surface of the deck. Efflorescence, light in frequency was observed along the bottom of the concrete deck. Rust was observed in many places on the steel beams.

4.0 EXPLORATION

4.1 Historic and Project Exploration Program

Historical records of a geotechnical exploration were available from the ODOT Geotechnical Documents Management System ftp site for the existing Harrison Street Bridge No. HAN-75-1477 over IR 75 mainline and Norfolk Southern Railroad. A structure exploration was performed in 1960 under project designation of HAN-25-8.90/HAN-25-14.78 Harrison Ave. over USR 25 & NKP R.R. A total of three (3) historic test borings identified as B-4, B-9, and B-13 (B-004-0-60, B-009-0-60, and B-013-0-60) were advanced in the vicinity of the existing bridge piers. Soil and rock information obtained from the historic borings B-004-0-60 and B-009-0-60 will be used to design the piers of the proposed bridge. All of the relevant historic information discussed above 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 July and August 2013. A total of four (4) project test borings identified as B-167-0-13, B-168-0-13, B-170-0-13, and B-175-0-13 were advanced for bridge and MSE wall foundation design purposes. Test borings B-167-0-13 and B-168-0-13 were advanced in the vicinity of the proposed rear abutment and MSE wall, while test boring B-175-0-13 was advanced in the vicinity of the proposed forward abutment. Test boring B-170-0-13 was advanced in the vicinity of the proposed bridge Pier 2. These structural test borings were advanced to approximate depths ranging from 9.5 to 42.7 feet below the existing ground or Harrison Street pavement surface.

The test borings were marked in the field by PGI based on boring location plans developed by PGI personnel and after obtaining approval from PB 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. Two (2) All Terrain Vehicle (ATV) mounted Diedrich 50 and Diedrich 90 drill rigs and a Truck mounted CME 55 drill rig were 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 NX series core barrel, water method. All test borings were monitored for the presence of groundwater during drilling operations and upon completion. All test borings were backfilled with compacted soil cuttings at the end of drilling operations for safety purposes.

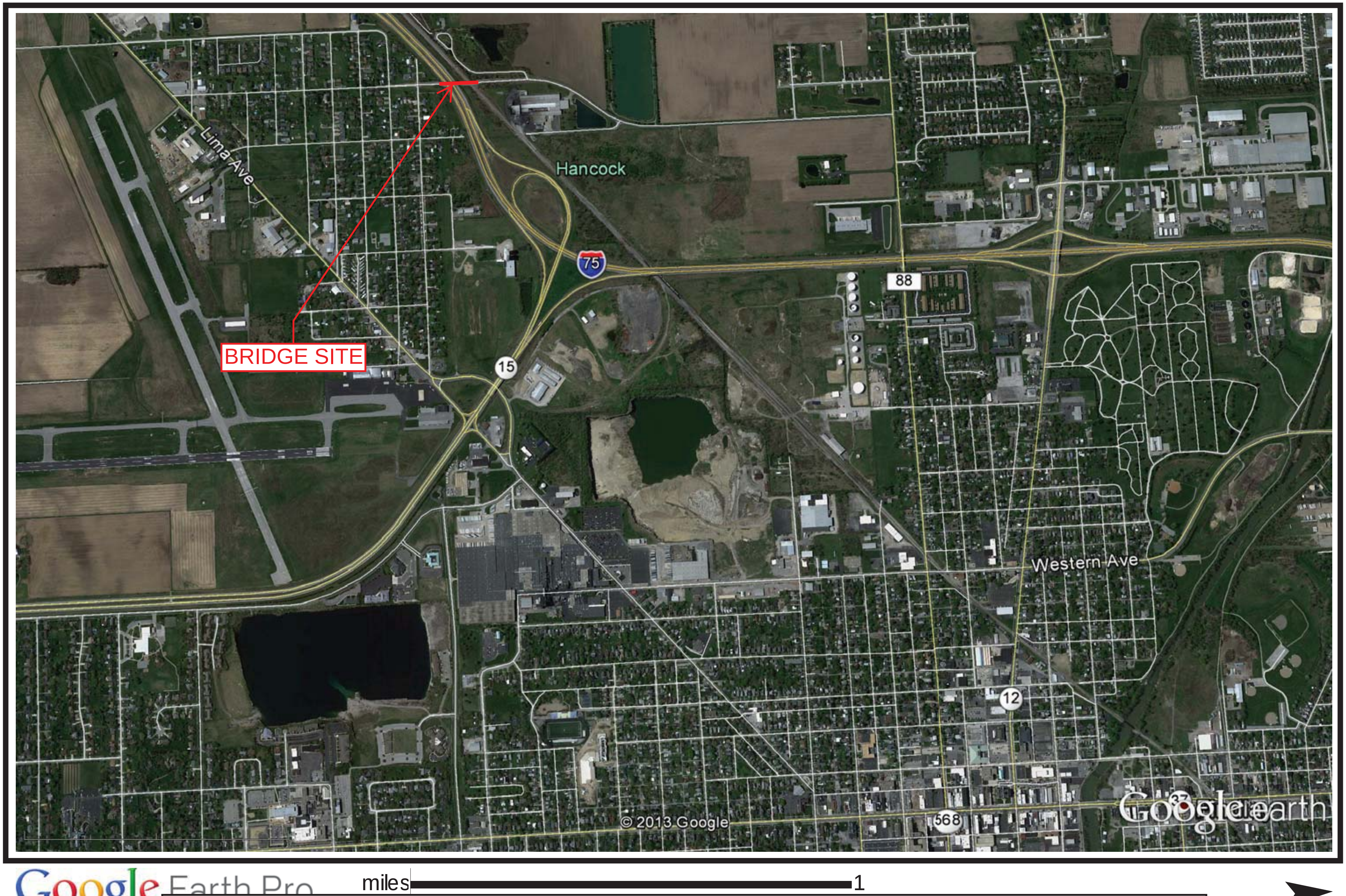
Latitude/longitude and northing/easting coordinates, stations and offsets, and surface elevations at the drilled test boring locations were provided to PGI by PB personnel. The typed drilling logs, Boring Location Map, and soil boring profiles are included in Appendix A. Northing and easting coordinates shown on the Soil Boring Profile sheets are grid. A project adjustment factor (PAF) of 1.00009818 was used to convert the grid coordinates to ground coordinates for this project. 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. The N-values as measured in the field have been corrected to an equivalent rod energy ratio of 60% (N_{60}) in accordance with ODOT's *Specifications for Geotechnical Explorations*. The sample depth shown on the logs and laboratory test results indicate the top of each sampling or testing interval.

4.2 Laboratory Testing Program

All soil and 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 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, Plasticity Index Determination of Soils, and Compressive Strength of Rock Core Samples. 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.



PROJECT: HAN-75-14.39

BRIDGE NO. HAN-75-1477 OVER IR 75 & NORFOLK SOUTHERN RAILROAD

SITE LOCATION MAP (FIGURE 2.1)

5.0 FINDINGS

5.1 Subsurface Soil Conditions

The surficial and subsurface soil conditions in the vicinity of this proposed bridge were determined from the soil information obtained from project test borings B-167-0-13, B-168-0-13, B-170-0-13, and B-175-0-13. Test borings B-167-0-13, B-168-0-13 and B-170-0-13 were advanced through topsoil ranging in thickness from 2 inches to 7 inches and averaging 4.3 inches in thickness. Test boring B-175-0-13 was advanced through the pavement of Harrison Street which consisted of 2 inches of asphalt pavement above 12 inches of concrete pavement above 17 inches of gravel base material. The subsurface soils encountered in test borings B-167-0-13, B-170-0-13, and B-175-0-13 consisted of fill materials and in B-168-0-13 consisted of both fill materials and natural soils. The fill material encountered above the natural soils consisted of sandy silt (A-4a), silt and clay (A-6a), silty clay (A-6b), clay (A-7-6), non-plastic silt (A-4b), and non-plastic sandy silt (A-4a). The approximate thickness of the fill materials ranged from 3.2 feet in test boring B-170-0-13 to 32.3 feet in B-175-0-13 above the natural soils or bedrock. Natural soils encountered below the fill material and above bedrock in test borings B-168-0-13 and B-170-0-13 consisted of silty clay (A-6b). Bedrock was encountered in all test boring locations at approximate depths ranging from 3.2 feet to 32.3 feet below the existing ground or pavement surface.

The laboratory test results indicated that the moisture contents of the tested cohesive soil samples ranged from 9% to 31%. The moisture content of the tested non-cohesive soils was 5%. The consistency of the cohesive soils ranged from "medium stiff" to "hard", but was generally “stiff”. The relative density of the non-cohesive soils was “medium dense”.

One of the four cohesive soil samples tested for Atterberg Limits had a natural moisture content greater than its plastic limit but less than its liquid limit. Normally, soils with moisture contents greater than or equal to their liquid limits are in a liquid state and have no shear strength. Soils with moisture contents greater than or equal to their plastic limits and less than their liquid limits are in a plastic state, and have the potential of volume change under certain loading conditions. All of the test borings were terminated after obtaining rock core samples. 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.

5.2 Bedrock Conditions

Bedrock was encountered in all of the project test borings. Bedrock was split spoon sampled until little or no penetration or recovery was encountered. Bedrock core samples were then obtained using NX diamond impregnated core barrels. 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 dolomite of the Tymochtee/Greenfield Group. The dolomite was light gray to gray, moderately to slightly weathered, and strong. Bedding within the dolomite was generally very thin to thin and was highly fractured to moderately fractured. No slickensides were observed and the fractures were typically tight to narrow and slightly rough to very rough. The compressive strength of the project core specimens ranged from 4,596 psi in test boring B-170-0-13 to 20,441 psi in test boring B-175-0-13 which characterizes them as “moderately strong” to “very strong”, respectively. Bedrock was also encountered in three historic test borings which were advanced during a subsurface exploration performed in December 1960. These core samples are described as dolomite, gray with massive bedding, hard, broken with trace carbonaceous shale.

The Rock Quality Designation (RQD) for the core samples ranged from 18% to 65% and averaged 45% based on individual runs. The results of these measurements are summarized in Table 5.2.1. Table 5.2.2 summarizes the results of compressive strength tests performed at the laboratory on the rock core specimens at various depths. The Rock Mass Rating obtained for the bedrock core samples according to LRFD Table 10.4.6.4-1 varied from 42 to 47 and is classified as “Fair Rock”. The Rock Mass Rating spreadsheets are included 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	Rock Core Run No.	Top of Bedrock Elevations (ft)	Rock Core Run Elevations (ft)	Length of Core Run (ft)	Recovery (%)	RQD (%)
B-167-0-13	NX-1	776.8	775.8	10.0	88	18
B-168-0-13	NX-1	777.8	777.0	3.6	100	55
	NX-2		773.4	6.4	100	65
B-170-0-13	NX-1	775.2	774.1	10.0	98	42
B-175-0-13	NX-1	774.9	774.6	10.0	90	45

Elevations were provided by PB personnel

Table 5.2.2 – Compressive Strength Test Results of Rock Core Specimens

Boring Number	Specimen Depth (ft)	Rock Type	Unit Weight (pcf)	Compressive Strength (psi)
B-167-0-13	7.2	Dolomite	167.24	18,187
B-168-0-13	36.7	Dolomite	177.52	13,284
B-170-0-13	9.2	Dolomite	174.65	4,596
B-175-0-13	37.2	Dolomite	166.80	20,441

5.3 Groundwater Conditions

The groundwater levels were monitored in all of the test boring locations during drilling operations. Groundwater was not encountered during drilling operations in any of the test borings advanced during our field work. Groundwater levels were not recorded upon completion of drilling operations due to water used for rock coring. It should be noted that groundwater elevations are subject to seasonal fluctuations. Groundwater monitoring wells are essential to accurately define the position of the groundwater table; however, installation of monitoring wells was not included in our scope of services. All test borings were backfilled upon completion 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 Harrison Street Bridge No. HAN-75-1477 over IR 75 Mainline and Norfolk Southern Railroad. Site plans provided by PB personnel indicates that the bridge abutments will be supported on piles at the rear and forward abutment locations and Piers 1 and 2 will be supported on spread footing. Mechanically Stabilized Earth (MSE) Wall abutment type will be used at the rear abutment and spill-through abutment type will be used at the forward abutment. Elevation of the bottom of the proposed MSE Walls at the rear abutment location will be 778.5 feet and elevations of the bottom of the spread footing at the proposed Pier 1 and Pier 2 locations will be 776.5 feet and 774.5 feet, respectively. Additional embankment fill, 1.0 foot in thickness at the rear abutment and 4.0 feet in thickness at the forward abutment will be placed on top of existing Harrison Street embankment to the raise the existing grade to the proposed profile grade. The foundation recommendations are provided in accordance with the ODOT *Bridge Design Manual* issued in 2007 using *LRFD Bridge Design Specifications.*

6.1 Bridge Foundation Systems

Soil and rock information obtained from proposed project test borings B-168-0-13, B-170-0-13, and B-175-0-13 and historic project test borings B-004-0-60, B-009-0-60, and B-013-0-60 were used to provide foundation recommendations for this proposed replacement bridge. Project test borings B-168-0-13 and B-175-0-13 were advanced in the vicinity of the proposed rear and forward abutments, respectively. Project test boring B-170-0-13 and historic test borings B-004-0-60 and B-009-0-60 were advanced in the vicinity of proposed pier locations. As outlined in Section 5.1 - "Subsurface Soil Conditions", the top of bedrock was encountered at approximate depths ranging from 27.0 feet to 32.3 feet below the Harrison Street pavement at the abutment locations and at approximate depths ranging from 3.2 feet to 7.5 feet below the existing ground surface at the proposed pier locations. This bedrock consists of dolomite and was encountered to termination depths. Since bedrock was encountered at relatively deeper depths below the Harrison Street pavement at the abutment locations, the proposed bridge superstructure loads may be transferred to the underlying bedrock by means of piles at the abutment locations. Since bedrock was encountered at relatively shallower depths below the existing ground surface at the pier locations, the proposed bridge superstructure loads may be transferred to the underlying bedrock by means of spread footings at the pier locations.

<u>Rear Abutment</u>**:** The bridge rear abutment above the MSE wall embankment may be supported on end bearing H piles. The top of bedrock was encountered at approximate depth of 0.7 feet below the bottom of the rear abutment MSE Wall. Therefore the end bearing H-piles should be installed in pre-bored holes with a minimum embedment length of 5 feet into bedrock according to the ODOT *Bridge Design Manual* Section 204.4. These pre-bored holes should be backfilled with Class C concrete up to the top of the leveling pad elevation. H-pile sizes HP-10X42 may be selected for the rear abutment location. The total factored load on each HP-10X42 pile should not exceed the corresponding maximum structural resistance of 310 kips as per the ODOT *Bridge Design Manual* Section 202.2.3.2.a. Note that the above mentioned structural resistance values can be used only on the axial loaded piles that have a 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 elevation at the rear abutment was extracted from the site plan provided by PB personnel. Pile sections within the MSE Walls should be encased above the existing ground in

corrugated pipe filled with granular material to eliminate any down drag on this portion of the piles and protect against construction operations.

Table 6.1.1 - Estimated Design Parameters for H-Piles

Boring No.	Bottom of MSE Wall Elevation	Pile Cut-off Elevation (ft)	Pile Tip Elevation (ft)	Estimated Effective Pile Length (ft)	Pile Type	Pile Size	Maximum Factored Structural Resistance/pile
Rear Abutment							
B-168-0-13	778.5	795.1	772.8	25	H-Pile	10X42	310 kips
Forward Abutment							
B-175-0-13	--	798.7	774.6	25	H-Pile	10X42	310 kips

<u>Forward Abutment:</u> End bearing H piles may be used to transfer the proposed bridge superstructure loads to the underlying bedrock at the forward abutment location. The end bearing H piles should be driven to refusal on the underlying dolomite 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 forward abutment location. The total factored load on each HP-10X42 pile should not exceed the corresponding maximum structural resistance of 310 kips as per the ODOT *Bridge Design Manual* Section 202.2.3.2.a. 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 elevation at the forward abutment was extracted from the site plan provided by PB personnel. 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. In order to protect the tip of the H piles from damage during pile driving, steel pile points should be installed as per the ODOT *Bridge Design Manual* Section 202.2.3.2.a. At the forward abutment, consolidation of the foundation soils caused by construction of the proposed embankment will be on the order of 0.5 inches. Therefore upward or downward shaft resistance will develop along the pile-soil interface due to relative displacement. Negative shaft resistance may develop along the pile-soil interface above the neutral plane. It is assumed that that the neutral plane will be at the pile tip since pile will bear on bedrock. Unfactored down drag load of 50 kips per pile may be assumed for pile size HP-10X42 at the forward abutment locations. These down drag loads were calculated using Total Stress Method (α Method). The piles should be designed in accordance with section 202.2.3.2.c – "Down Drag Forces on Piles" of the *ODOT Bridge Design Manual* issued in January 2007.

General: 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 July 2007. All H-piles should be installed in accordance with ODOT Item 507 - *Bearing Piles*, of the ODOT *Construction and Material Specifications Manual* dated January 2013. Based on the factored axial loads acting on the piles, the estimated maximum total settlement and differential settlement will not exceed one inch and one half of an inch, respectively. It is recommended that the piles be spaced a minimum of three (3) pile diameters on center. Seepage into the pre-bored holes may occur within the soil overburden during pile installation at the rear abutment. If any water is present in the bottom of the holes, it should be removed before placing concrete. Since piles are extended into bedrock, group effects of the piles can be neglected. The pile supported abutments may experience horizontal movement caused by lateral loads and overturning moments. A lateral load analysis should be performed using LPILE computer software by Ensoft or other comparable pile lateral load analysis software for selected pile size and embedment length to check whether lateral resistance is adequate to support lateral loads and overturning moments. The estimated pile length in Table 6.1.1 should be adjusted based on the outcome of the lateral load analysis. Table 6.1.2 summarizes the weak rock parameters to perform lateral load analyses by PB personnel.

Table 6.1.2 - Estimated Weak Rock Parameters for Lateral Load Analyses

Boring No.	Top Elevation(ft)	Effective Unit Weight (pci)	Youngs's Modulus (psi)	Unconfined Compressive Strength (psi)	RQD (%)	k_rm
B-168-0-13	777.8	0.096	100000	5000	61	0.00005
B-175-0-13	775.0	0.096	100000	5000	45	0.00005

If additional lateral resistance is required, bigger size piles should be considered at the rear abutment location and piles should be installed battered in accordance with Section 303.4.2.4 - "Piles Battered", of the *ODOT Bridge Design Manual* issued in July 2007 at the forward abutment location.

Spread Footings: Spread footings may be used to transfer the loads to the underlying bedrock at the proposed pier locations. Bearing resistance for spread footings on bedrock was evaluated as per AASHTO Article 10.6.3.2.2 (semi-empirical method) at the test boring B-170-0-13 location. The nominal bearing resistance analysis spreadsheet is included in Appendix B. Table 6.1.3 summarizes the factored bearing resistance on bedrock and founding elevation at each test boring location so that PB personnel can evaluate or compare the factored bearing resistance to the factored bearing pressure. A

Resistance Factor (ϕ) of 0.45 was used to compute the Factored Bearing Resistance at the Strength Limit State. A Resistance Factor (ϕ) of 1.0 should be used to compute the Factored Bearing Resistance at the Service Limit State.

Table 6.1.3 – Estimated Design Parameters at Strength Limit State for Spread Footings

Boring No.	Substructure Location	Top of Bedrock Elevation (feet)	Proposed Bearing Elevation (feet)	Nominal Resistance (ksf)	Factored Resistance (ksf)
B-004-0-60	Pier 1	776.8±	776.5	28.0	13.0
B-170-0-13	Pier 2	775.2±	774.5	28.0	13.0

A presumptive nominal bearing resistance of 20 ksf from the LRFD Table C10.6.2.6.1-1 was used for dolomite bedrock to calculate the settlement at the Service Limit State. Settlement of the proposed footings at the pier locations will be due to elastic compression of bedrock. Based on the settlement analysis, it is estimated that the maximum total settlement and differential settlement will not exceed one inch and one-half of an inch, respectively. The settlement calculations are shown on the nominal bearing resistance analysis spreadsheet included in Appendix B. Since the proposed spread footings will be placed on relatively level ground, and shear failure is not anticipated along the foundation bedrock joints, global stability of the footings is not a concern. The proposed pier footings may experience sliding caused by lateral loads. Therefore all pier footings should be keyed into bedrock a minimum of 3 inches in accordance with requirements of Section 204.1, 303.4.1.1, and 606.7 of the *2007 ODOT Bridge Design Manual*. The proposed bottom of pier footings should be placed a minimum of 3.0 feet below the proposed finished ground surface to protect against frost. Please note that the top elevation of the dolomite bedrock may vary with location, and slight adjustments of footing depth may be required in the field. The bedrock footing subgrade should be examined by a competent geotechnical engineer to verify that the maximum factored resistance is being complied with. If any soil or severely weathered bedrock is encountered, it should be removed as directed by an on-site geotechnical engineer and replaced with concrete.

6.2 MSE Wall Foundation Systems

Design information provided by PB personnel indicates that the maximum height of the MSE Wall will be 27.6 feet at the rear abutment location. MSE Wall base will directly bear on bedrock. The foundation width of the MSE Wall will be 19.3 feet based upon a minimum strap length equal to 70% of the wall height. However, it is assumed 20 feet for the MSE Wall external stability calculations. It is assumed that maximum applied bearing pressures at the Service Limit State will be 4000 psf for rear MSE Wall. Soil and rock information obtained from structural test borings B-167-0-13 and B-168-0-13 were used to provide foundation recommendations for the proposed MSE Wall at the rear abutment location. According to site plans provided by PB personnel, elevation of the bottom of the proposed MSE Wall at the rear abutment location will be 778.5 feet. As per the boring logs, bedrock was encountered below the bottom of the rear MSE Wall at depths of 1.7 feet in test boring B-167-0-13 and 0.7 feet in test boring B-168-0-13. Since MSE Wall base will bear on bedrock, all existing soils above the bedrock should be removed before the MSE Wall construction.

Bearing capacity analysis (as per AASHTO Article 10.6.3.2.2) was performed based on the rock parameters (per Carter and Kulhawy (1988)) to estimate the nominal bearing resistance of the strip footings supported on bedrock. Results of the bearing capacity analysis are attached in Appendix B. Estimated factored bearing resistance at each boring location 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. It is estimated that the total and differential settlement of the underlying foundation rock will be within the tolerable total settlement of 12 inches and differential settlement of one percent for MSE Wall. No waiting period is required at the end of MSE Wall Construction.

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

Boring No.	Location	Depth of Bottom of Footing Below Final Grade (feet)	Width of Strip Footing (feet)	Proposed Bearing Elevation (feet)	Factored Bearing Resistance (ksf)
B-167-0-13	Rear MSE Wall	5.7	20.0	778.5	20.0
B-168-0-13	Rear MSE Wall	5.7	20.0	778.5	20.0

External stability of the MSE Wall including sliding on the base, limiting eccentricity, and bearing resistance at the Strength Limit State and settlement analysis at the Service Limit State were performed at the rear abutment location. These external stability analyses were performed utilizing the MSEW Version

3.0, developed by Dov Leshchinsky, Ph.D., ADAMA Engineering. For the external stability analysis, 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 as per BDM Section 204.6.2.1.F. The uniform surcharge load due to traffic was assumed to be 250 psf as per AASHTO LRFD Table 3.11.6.4-1 for the MSE Wall height of 27.6 feet. Abutment configuration at the rear location was obtained from the site plans for the external stability analysis. Computer output of the MSE Wall external stability analyses is 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 computer output. The External Stability analyses results show that the Capacity Demand Ratio (CDR) value against sliding, CDR value with respect to bearing resistance and eccentricity value are within the acceptable limits for the selected foundation width of the rear MSE Wall.

Global stability analyses were performed using the GSTABL7 with STEDwin, version 2.0 program that was developed by Mr. Garry H. Gregory, P.E. to estimate the Factor of Safety for the proposed MSE Walls. The foundation soil profiles below the proposed MSE Walls were estimated from information obtained from the test borings. The phreatic surface was assumed as top of bedrock. For slope stability analysis, shear strength soil parameters used in this analysis were obtained from the laboratory tests performed on the undisturbed soil samples obtained from the ramp test borings and from our experience with similar types of soils. Trial failure surfaces were generated using the method of slices for short term and long-term stability. The Modified Bishop Method of slices was used to generate circular trial failure surfaces. Table 6.2.2 summarizes the safety factors for the short term and long term stability of the proposed MSE Wall. Based on this slope stability analysis, the calculated Safety Factors for both short term and long term meet the required Safety Factors specified in the ODOT Embankment Checklist. Slope analyses critical failure circles are included in Appendix B.

Table 6.2.2 –Summary of Critical Factors of Safety for MSE Walls

Boring No	Location	Stability	Method Used	Factor of Safety
B-168-0-13	Rear MSE Wall	Short Term	Circular	2.83
	Rear MSE Wall	Long Term	Circular	2.46

The foundation rock subgrade should be examined by competent geotechnical personnel before placing granular materials. If any soils 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. The MSE Wall design should be in accordance with the ODOT Bridge Design Manual issued in January 2007, Section 204.6.2.1. The backfill material in the reinforced zone and retained soil zone should be as per Section 204.6.2.1 F specifications.

6.3 Lateral Earth Pressures and Abutment Drainage

In order to resist the horizontal loads from the rear abutment and MSE Wall, a minimum of one row of soil reinforcements should be attached to the back row of piles. 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 Approach Slab Design Parameters

During construction of the project, the proposed approach slabs will be constructed on the existing embankment subgrade fill soils. The soil information obtained from the upper sample of test borings B-168-0-13 and B-175-0-13 which were advanced at the rear and forward abutments, respectively were used to obtain the pavement design parameters. The subgrade analysis was performed in accordance with Geotechnical Bulletin-GB1 from ODOT released on January 19, 2012. Based on the analysis, the following conclusions are presented. An average Group Index of 6.5 and a subsequent CBR value of 7 was obtained from the ODOT *GB1 Subgrade Analysis.* From the Group Index, the Soil Support Value, Resilient Modulus of Subgrade Soils, and Modulus of Subgrade Reaction were estimated based on the ODOT Subgrade Resilient Modulus Estimation Method, illustrated in 203-3, "Pavement Design & Rehabilitation Manual". This estimation was prepared based on the soil encountered within a depth of approximately 3.5 feet below the proposed subgrade. The pavement design parameter information is summarized in Table 6.4.1.

Table 6.4.1 – Summary of Approach Slab Design Parameters

Parameter	Value
Group Index	6.5
CBR	7
Soil Support Value (SSV)	4.9
Resilient Modulus (psi)	8,400
Modulus of Subgrade Reaction (K, pci)	165

6.5 Groundwater Management

Groundwater was not encountered in any of the test borings during drilling operations. If water infiltration is anticipated, it can be controlled through the use of sump pumps. It must be noted that the groundwater levels during construction may vary due to seasonal fluctuations, and groundwater may occur where not encountered previously.

6.6 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 2013 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). All topsoil should be removed before the start of construction. A drainage ditch which appears to be wetland will cross the proposed rear MSE Wall on the west side of the proposed structure. If proposed cut slopes for the structure foundation are to be exposed for an extended period of time, they must be constructed using a two (2) horizontal to one (1) vertical slope for excavation in cohesive soils. Soil and rock excavations are expected during construction of the project. It is expected that some harder, less weathered bedrock will be present in the pre-bored holes. Therefore special drilling equipment should be required.

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.

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.

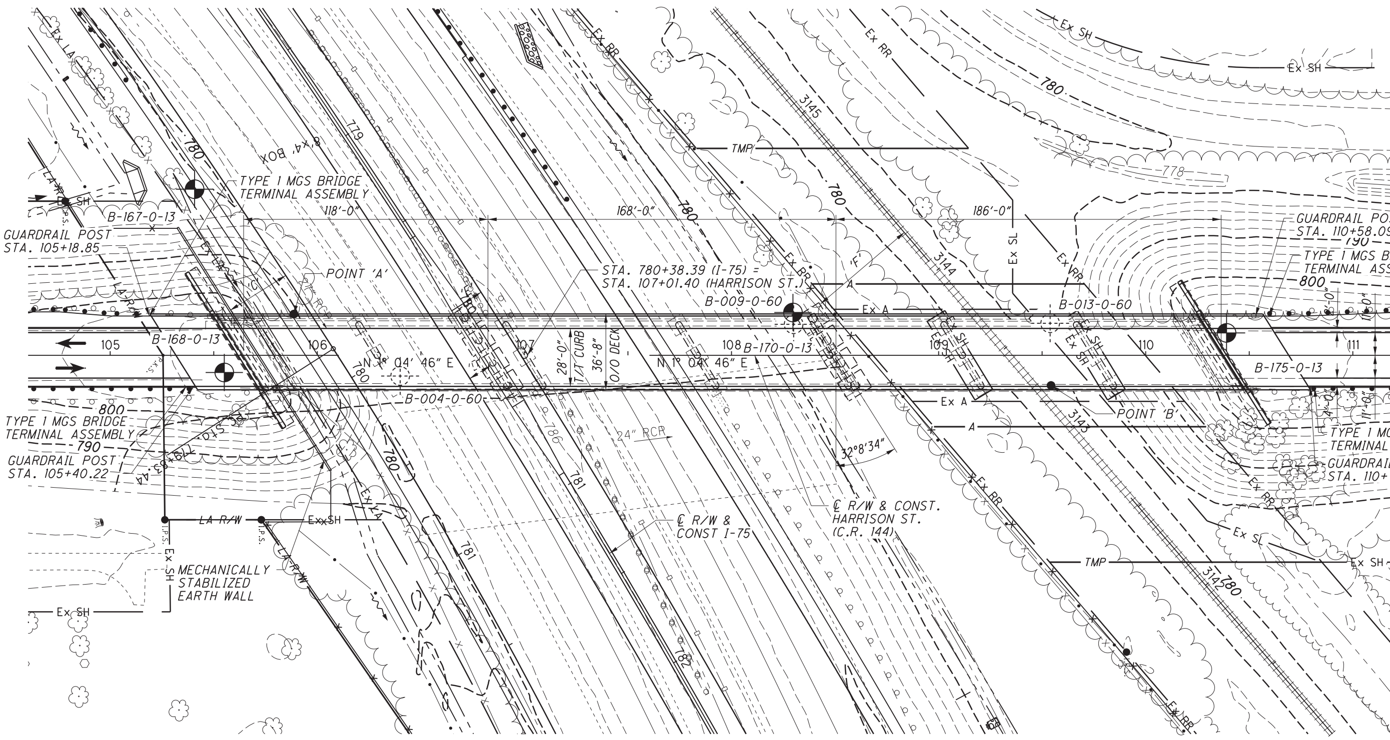
7.3 All earthwork and foundation construction must be performed under the supervision of a Professional Engineer in accordance with ODOT Construction Specifications.

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



HORIZONTAL SCALE IN FEET

DRAWN SS

CHECKED SS

BORING LOCATION PLAN

BRIDGE NO. HAN-75-1477 OVER I-75 & NORFOLK SOUTHERN RAILROAD

HAN-75-14.39

PID No. 87005

PROJECT: HAN-75-14.39	DRILLING FIRM / OPERATOR: B-M / JOSH DEAN	DRILL RIG: DIEDRICH D-90 ATV	STATION / OFFSET: 778+81.0, 74.7' RT	EXPLORATION ID B-167-0-13
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: PGI / W. NAJJAR	HAMMER: DIEDRICH AUTOMATIC	ALIGNMENT: IR-75 BASELINE	
PID: 87005 BR ID: HAN-75-1477	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 9/18/12	ELEVATION: 780.3 (MSL) EOB: 9.5 ft.	PAGE 1 OF 1
START: 8/6/13 END: 8/6/13	SAMPLING METHOD: SPT/NX	ENERGY RATIO (%): 80.2	COORD: 41.022508180, 83.681785180	

MATERIAL DESCRIPTION AND NOTES	ELEV. 780.3	DEPTHS	SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
TOPSOIL (7" THICK)	779.7																	
VERY STIFF, BROWN AND DARK BROWN, **CLAY**, LITTLE SAND, TRACE STONE FRAGS & ROOTS, FILL, MOIST	776.8	1–3	4 6 11	23	100	SS-1	4.5+	-	-	-	-	-	-	-	-	22	A-7-6 (V)	
LIGHT GRAY DOLOMITE BEDROCK @4.5'; AUGER REFUSAL AND STARTED CORING BEDROCK	775.8	TR 4	50/2"	-	100	SS-2	-	-	-	-	-	-	-	-	-	3	Rock (V)	
DOLOMITE, LIGHT GRAY, MODERATELY TO SLIGHTLY WEATHERED, VERY STRONG, VERY THIN TO THIN BEDDED, HIGHLY FRACTURED TO MODERATELY FRACTURED, FEW ANGULAR FRACTURES, TIGHT APERTURE WIDTH, ROUGH TO SLIGHTLY ROUGH. @7.2'; COMPRESSIVE STRENGTH = 18,187 psi	770.8	5–9 EOB	18		88	NX-1											CORE	

NOTES: GROUNDWATER WAS NOT ENCOUNTERED DURING DRILLING. NO READING WAS TAKEN UPON COMPLETION DUE TO WATER USED DURING ROCK CORING OPERATIONS.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: HOLE WAS BACKFILLED WITH SOIL CUTTINGS

PROJECT: HAN-75-14.39	DRILLING FIRM / OPERATOR: DLZ / JOHN	DRILL RIG: CME 55 TRUCK	STATION / OFFSET: 105+55.1, 8.3' RT	EXPLORATION ID B-168-0-13
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: PGI / W. NAJJAR	HAMMER: CME AUTOMATIC	ALIGNMENT: HARRISON ROAD BL	
PID: 87005 BR ID: HAN-75-1477	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 6/13/13	ELEVATION: 804.8 (MSL) EOB: 37.8 ft.	PAGE 1 OF 2
START: 7/23/13 END: 7/23/13	SAMPLING METHOD: SPT/NX	ENERGY RATIO (%): 70.2	COORD: 41.022545080, 83.681464130	

MATERIAL DESCRIPTION AND NOTES	ELEV. 804.8
TOPSOIL (2" THICK)	804.6
MEDIUM STIFF, DARK BROWN, **SANDY SILT**, SOME CLAY, TRACE STONE FRAGMENTS, FILL, DAMP	801.3
MEDIUM STIFF, BROWN, **SILT AND CLAY**, SOME SAND, TRACE STONE FRAGMENTS, FILL, MOIST	799.8
GRAY, NON-PLASTIC SILT LAYER	798.3
STIFF TO VERY STIFF, BROWN, **SILT AND CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, FILL, DAMP @8.5'; BROWN AND GRAY @16.0'; VERY STIFF	786.3
VERY STIFF, BROWN AND DARK BROWN, **SILTY CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, FILL, DAMP @23.5'; BROWN	780.1
STIFF, BLACK, **CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, FILL, MOIST	778.8
VERY STIFF, BROWN AND GRAY, **SILTY CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, DAMP	777.8 (TR)
POSSIBLE DOLOMITE BEDROCK @27.8'; AUGER REFUSAL AND STARTED CORING BEDROCK	777.0

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
3 2 2	5	44	SS-1	4.5+	10	6	14	35	35	26	16	10	15	A-4a (7)
2 3 4	8	78	SS-2	4.5+	8	6	17	40	29	29	15	14	18	A-6a (8)
3 4 6	12	72	SS-3	1.50	-	-	-	-	-	-	-	-	14	A-6a (V)
8 4 6	12	61	SS-4	4.5+	-	-	-	-	-	-	-	-	13	A-6a (V)
4 4 7	13	61	SS-5	4.5+	-	-	-	-	-	-	-	-	15	A-6a (V)
3 5 5	12	72	SS-6	3.50	-	-	-	-	-	-	-	-	15	A-6a (V)
6 8 10	21	78	SS-7	3.50	-	-	-	-	-	-	-	-	15	A-6a (V)
3 7 10	20	100	SS-8	4.5+	-	-	-	-	-	-	-	-	18	A-6b (V)
4 6 9	18	83	SS-9A&B	4.00	-	-	-	-	-	-	-	-	13	A-6b (V)
				3.00	-	-	-	-	-	-	-	-	29	A-7-6 (V)
3 8 9	20	78	SS-10	3.50	-	-	-	-	-	-	-	-	17	A-6b (V)
55		100	NX-1											CORE

PID: 87005	BR ID: HAN-75-1477	PROJECT: HAN-75-14.39	STATION / OFFSET: 105+55.1, 8.3' RT	START: 7/23/13	END: 7/23/13	PG 2 OF 2	B-168-0-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 774.8	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
DOLOMITE, GRAY, MODERATELY TO SLIGHTLY WEATHERED, STRONG, VERY THIN TO THIN BEDDED, FRACTURED TO MODERATELY FRACTURED, FEW ANGULAR FRACTURES, TIGHT TO NARROW APERTURE WIDTH, ROUGH TO SLIGHTLY ROUGH. *(continued)*	773.4	31																
DOLOMITE, GRAY, MODERATELY TO SLIGHTLY WEATHERED, STRONG, VERY THIN TO THIN BEDDED, HIGHLY FRACTURED TO MODERATELY FRACTURED, FEW ANGULAR FRACTURES, TIGHT TO NARROW APERTURE WIDTH, ROUGH TO SLIGHTLY ROUGH. @36.7'; COMPRESSIVE STRENGTH = 13,284 psi	767.0	32–37 EOB	65		100	NX-2											CORE	

NOTES: GROUNDWATER WAS NOT ENCOUNTERED DURING DRILLING. NO READING WAS TAKEN UPON COMPLETION DUE TO WATER USED DURING ROCK CORING OPERATIONS.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: HOLE WAS BACKFILLED WITH SOIL CUTTINGS

PROJECT: HAN-75-14.39 TYPE: BRIDGE REPLACEMENT PID: 87005 BR ID: HAN-75-1477 START: 6/26/13 END: 6/26/13	DRILLING FIRM / OPERATOR: OTB / JOHN SAMPLING FIRM / LOGGER: PGI / F.BUSHER DRILLING METHOD: 3.25" HSA SAMPLING METHOD: SPT/NX	DRILL RIG: DIEDRICH D-50 ATV HAMMER: DIEDRICH AUTOMATIC CALIBRATION DATE: 12/10/11 ENERGY RATIO (%): 81.7	STATION / OFFSET: 108+29.7, 20.7' LT ALIGNMENT: HARRISON ROAD BL ELEVATION: 778.4 (MSL) EOB: 14.3 ft. COORD: 41.023299070, 83.681563970	EXPLORATION ID B-170-0-13 PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 778.4	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
TOPSOIL (4" THICK)	778.1																	
MEDIUM STIFF, BLACK, **CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS & ROOTS, FILL, MOIST	775.2	1–3	3 2 3	7	50	SS-1	3.00	-	-	-	-	-	-	-	-	31	A-7-6 (V)	
POSSIBLE DOLOMITE BEDROCK @3.5'; NO SPLIT SPOON RECOVERY NOTE: AUGERED TO 4.3' AND STARTED CORING BEDROCK	774.1	TR — 3–4	50/3"	-	0	SS-2	-	-	-	-	-	-	-	-	-	-		
DOLOMITE, LIGHT GRAY, MODERATELY TO SLIGHTLY WEATHERED, MODERATELY STRONG, VERY THIN TO THIN BEDDED, FRACTURED TO MODERATELY FRACTURED, FEW ANGULAR FRACTURES, TIGHT APERTURE WIDTH, SLIGHTLY ROUGH. @9.2'; COMPRESSIVE STRENGTH = 4,596 psi	764.1	4–14, EOB	42		98	NX-1											CORE	

NOTES: GROUNDWATER WAS NOT ENCOUNTERED DURING DRILLING. NO READING WAS TAKEN UPON COMPLETION DUE TO WATER USED DURING ROCK CORING OPERATIONS.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: HOLE WAS BACKFILLED WITH SOIL CUTTINGS

PROJECT: HAN-75-14.39	DRILLING FIRM / OPERATOR: DLZ / JOHN	DRILL RIG: CME 55 TRUCK	STATION / OFFSET: 110+39.0, 10.8' LT	EXPLORATION ID B-175-0-13
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: PGI / W. NAJJAR	HAMMER: CME AUTOMATIC	ALIGNMENT: HARRISON ROAD BL	
PID: 87005 BR ID: HAN-75-1477	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 6/13/13	ELEVATION: 807.3 (MSL) EOB: 42.7 ft.	PAGE 1 OF 2
START: 7/24/13 END: 7/24/13	SAMPLING METHOD: SPT/NX	ENERGY RATIO (%): 70.2	COORD: 41.023873480, 83.681524220	

MATERIAL DESCRIPTION AND NOTES	ELEV. 807.3
ASPHALT PAVEMENT (2" THICK)	807.1
CONCRETE PAVEMENT (12" THICK)	806.1
GRAVEL BASE (17" THICK)	804.9
MEDIUM STIFF TO STIFF, BROWN TO GRAY, **SANDY SILT**, SOME CLAY, LITTLE TO TRACE STONE FRAGMENTS, FILL, DAMP @3.5'; STIFF, TRACE STONE FRAGMENTS @8.5'; STIFF, TRACE STONE FRAGMENTS @11.0'; STIFF, GRAY, TRACE STONE FRAGMENTS	793.8
VERY STIFF, BROWN, **SILT AND CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, FILL, MOIST	791.3
MEDIUM DENSE, GRAY, **NON-PLASTIC SANDY SILT**, LITTLE STONE FRAGMENTS, FILL, DAMP	787.8
VERY STIFF, BROWN, **SILT AND CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, FILL, DAMP	777.8

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
3 4 3	8	28	SS-1	4.5+	11	12	19	24	34	21	16	5	11	A-4a (5)
2 4 5	11	61	SS-2	4.5+	7	9	21	39	24	20	15	5	12	A-4a (6)
6 6 5	13	67	SS-3	4.5+	-	-	-	-	-	-	-	-	9	A-4a (V)
3 6 6	14	56	SS-4	3.75	-	-	-	-	-	-	-	-	13	A-4a (V)
6 7 6	15	78	SS-5	4.5+	-	-	-	-	-	-	-	-	12	A-4a (V)
2 4 15	22	83	SS-6	2.50	-	-	-	-	-	-	-	-	19	A-6a (V)
17 12 13	29	89	SS-7	3.00	-	-	-	-	-	-	-	-	5	A-4a (V)
7 5 4	11	50	SS-8	4.00	-	-	-	-	-	-	-	-	5	A-4a (V)
6 8 12	23	100	SS-9	4.5+	-	-	-	-	-	-	-	-	14	A-6a (V)
7 11 17	33	67	SS-10A&B	4.5+	-	-	-	-	-	-	-	-	14	A-6a (V)
				4.5+	-	-	-	-	-	-	-	-	25	A-7-6 (V)

DEPTHS: 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29

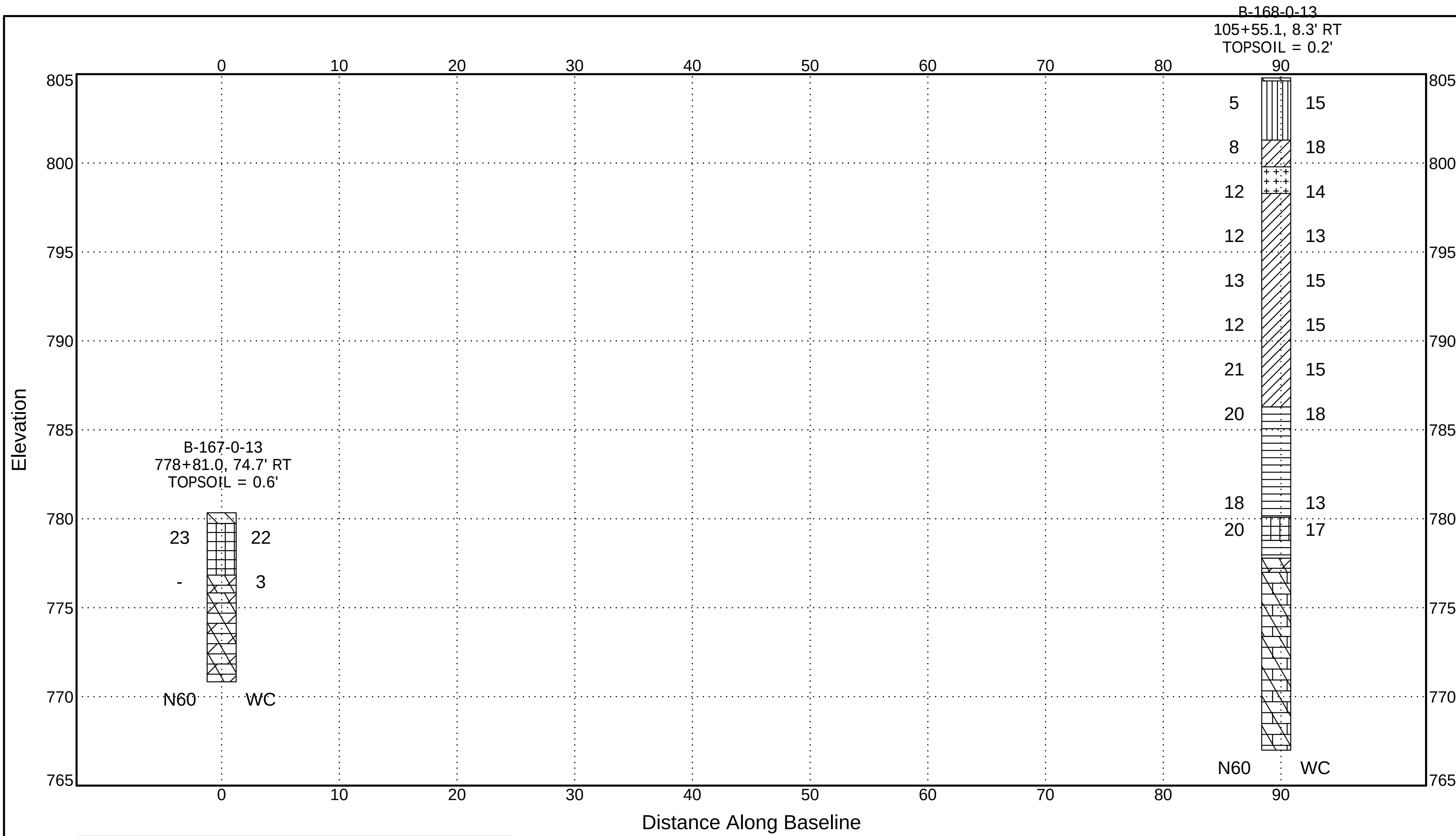
BACK FILL

PID: 87005	BR ID: HAN-75-1477	PROJECT: HAN-75-14.39	STATION / OFFSET: 110+39.0, 10.8' LT	START: 7/24/13	END: 7/24/13	PG 2 OF 2	B-175-0-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 777.3	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
HARD, BLACK, **CLAY**, SOME SAND, TRACE STONE FRAGMENTS, FILL, MOIST *(continued)*	775.0	31, 32																
POSSIBLE DOLOMITE BEDROCK @32.7'; AUGER REFUSAL AND STARTED CORING BEDROCK	774.6	TR																
DOLOMITE, LIGHT GRAY, MODERATELY TO SLIGHTLY WEATHERED, VERY STRONG, VERY THIN TO THIN BEDDED, FRACTURED TO MODERATELY FRACTURED, FEW ANGULAR FRACTURES, TIGHT TO NARROW APERTURE WIDTH, SLIGHTLY ROUGH. @37.2'; COMPRESSIVE STRENGTH = 20,441 psi	764.6	33–42, EOB	45		90	NX-1											CORE	

NOTES: GROUNDWATER WAS NOT ENCOUNTERED DURING DRILLING. NO READING WAS TAKEN UPON COMPLETION DUE TO WATER USED DURING ROCK CORING OPERATIONS.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: HOLE WAS BACKFILLED WITH SOIL CUTTINGS



Borehole	North	East	Elev.	Depth
B-167-0-13	496173	1642428	780.3	9.5
B-168-0-13	496186	1642517	804.8	37.8

DISTANCES:
Beginning 0
Ending 90
VIEWING ANGLES (degrees):
Horizontal 0.0
Vertical 0.0

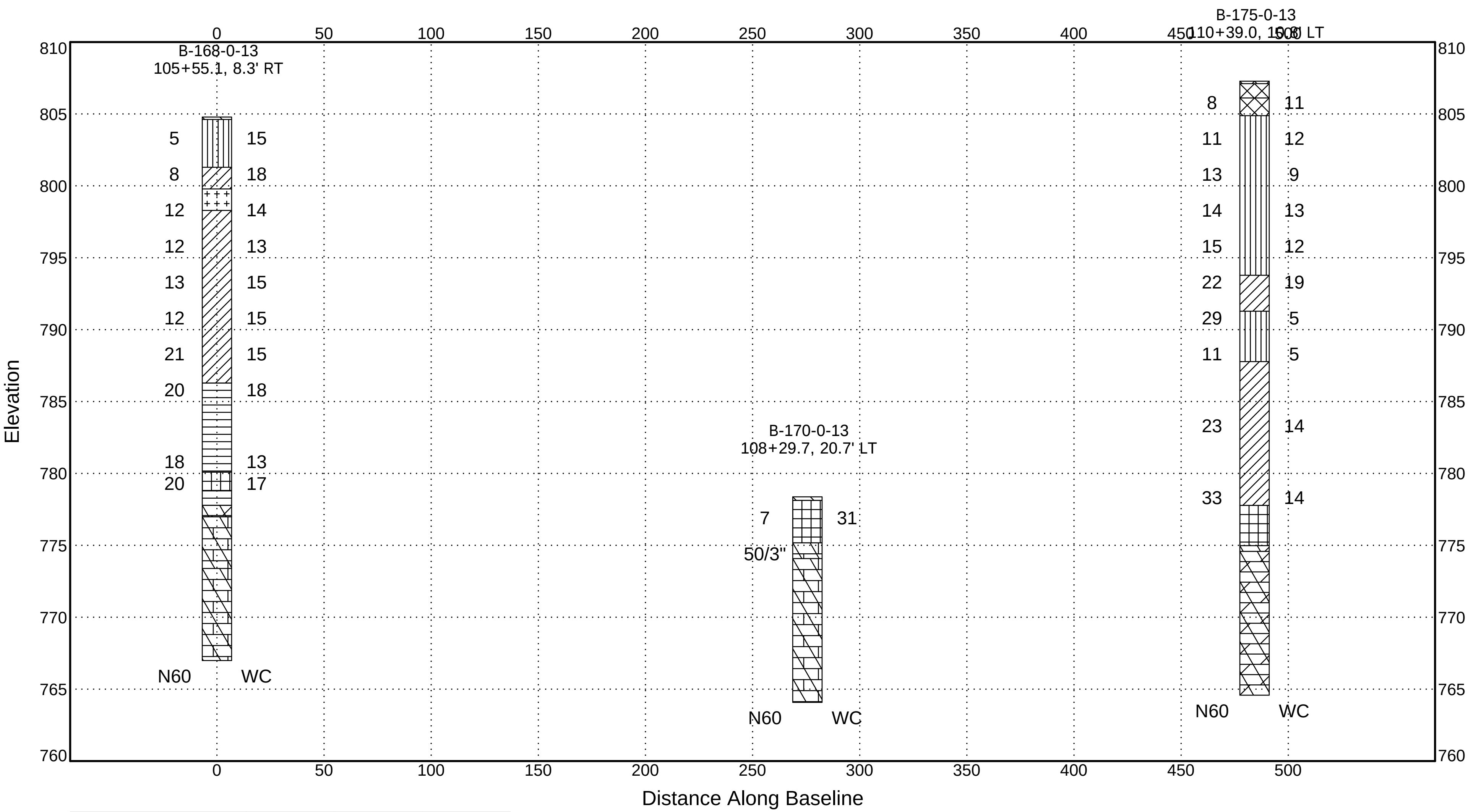
Position	North	East
Left, Front	496173	1642428
Right, Front	496186	1642518
Left, Back	496173	1642428
Right, Back	496186	1642518

SOIL BORINGS PROFILE
BRIDGE NO. HAN-75-1477

HAN-75-14.39

FINDLAY, HANCOCK COUNTY, OHIO

PROJECT #	DATE	PLATE
87005	Apr 14	1



Borehole	North	East	Elev.	Depth
B-168-0-13	496186	1642517	804.8	37.8
B-170-0-13	496461	1642493	778.4	14.3
B-175-0-13	496670	1642507	807.3	42.7

DISTANCES:

Beginning 0

Ending 500

VIEWING ANGLES (degrees):

Horizontal 0.0

Vertical 0.0

Position	North	East
Left, Front	496185	1642512
Right, Front	496685	1642499
Left, Back	496185	1642512
Right, Back	496685	1642499

SOIL BORINGS PROFILE
Bridge No. HAN-75-1477

HAN-75-14.39

FINDLAY, HANCOCK COUNTY, OHIO

PROJECT #	DATE	PLATE
87005	Mar 14	2

PROFILE ODOT-PRIMENG.GDT-3/23/14 21:35-\\CLEDC01\PUBLIC\PROJECT FILES\13 PROJECTS\G13011G HAN-75\LAB DATA SHEETS\BRIDGES\1477 HARRISON ST.GPJ

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-167-0-13	SS-1	1.0	22											BROWN AND DARK BROWN CLAY, LITTLE SAND, TRACE STONE FRAGS & ROOTS	A-7-6 (V)
B-167-0-13	SS-2	3.5	3											GRAY DOLOMITE BEDROCK	Rock (V)
B-168-0-13	SS-1	1.0	15	26	16	10		10	6	14	35	70	35	DARK BROWN SANDY SILT, SOME CLAY, TRACE STONE FRAGMENTS (FILL)	A-4a (7)
B-168-0-13	SS-2	3.5	18	29	15	14		8	6	17	39	69	29	BROWN SILT AND CLAY, SOME SAND, TRACE STONE FRAGMENTS (FILL)	A-6a (8)
B-168-0-13	SS-3	6.0	14											BROWN SILT AND CLAY, LITTLE SAND WITH GRAY, NON-PLASTIC SILT LAYER (FILL)	A-6a (V)
B-168-0-13	SS-4	8.5	13											BROWN AND GRAY SILT AND CLAY, LITTLE SAND, TRACE STONE FRAGMENTS (FILL)	A-6a (V)
B-168-0-13	SS-5	11.0	15											BROWN SILT AND CLAY, LITTLE SAND, TRACE STONE FRAGMENTS (FILL)	A-6a (V)
B-168-0-13	SS-6	13.5	15											BROWN SILT AND CLAY, LITTLE SAND, TRACE STONE FRAGMENTS (FILL)	A-6a (V)
B-168-0-13	SS-7	16.0	15											BROWN SILT AND CLAY, LITTLE SAND, TRACE STONE FRAGMENTS (FILL)	A-6a (V)
B-168-0-13	SS-8	18.5	18											BROWN AND DARK BROWN SILTY CLAY, LITTLE SAND, TRACE STONE FRAGS (FILL)	A-6b (V)
B-168-0-13	SS-9A	23.5	13											BROWN SILTY CLAY, LITTLE SAND, TRACE STONE FRAGMENTS (FILL)	A-6b (V)
B-168-0-13	SS-9B	24.7	29											BLACK CLAY, LITTLE SAND, TRACE STONE FRAGMENTS (FILL)	A-7-6 (V)
B-168-0-13	SS-10	25.0	17											BROWN AND GRAY SILTY CLAY, LITTLE SAND, TRACE STONE FRAGMENTS	A-6b (V)
B-170-0-13	SS-1	1.0	31											BLACK CLAY, LITTLE SAND, TRACE STONE FRAGMENTS & ROOTS (FILL)	A-7-6 (V)
B-170-0-13	SS-2	3.5												NO RECOVERY	
B-175-0-13	SS-1	1.0	11	21	16	5		11	12	19	24	58	34	BROWN SANDY SILT, SOME CLAY, LITTLE STONE FRAGMENTS (FILL)	A-4a (5)
B-175-0-13	SS-2	3.5	12	20	15	5		7	9	21	40	63	24	BROWN SANDY SILT, SOME CLAY, TRACE STONE FRAGMENTS (FILL)	A-4a (6)
B-175-0-13	SS-3	6.0	9											BROWN SANDY SILT, SOME CLAY, LITTLE STONE FRAGMENTS (FILL)	A-4a (V)
B-175-0-13	SS-4	8.5	13											BROWN SANDY SILT, SOME CLAY, TRACE STONE FRAGMENTS (FILL)	A-4a (V)
B-175-0-13	SS-5	11.0	12											GRAY SANDY SILT, SOME CLAY, TRACE STONE FRAGMENTS (FILL)	A-4a (V)
B-175-0-13	SS-6	13.5	19											BROWN SILT AND CLAY, LITTLE SAND, TRACE STONE FRAGMENTS (FILL)	A-6a (V)
B-175-0-13	SS-7	16.0	5											GRAY, NON-PLASTIC SANDY SILT, LITTLE STONE FRAGMENTS (FILL)	A-4a (V)
B-175-0-13	SS-8	18.5	5											GRAY, NON-PLASTIC SANDY SILT, LITTLE STONE FRAGS (FILL) W/SILT AND CLAY LAYER	A-4a (V)
B-175-0-13	SS-9	23.5	14											BROWN SILT AND CLAY, LITTLE SAND, TRACE STONE FRAGMENTS (FILL)	A-6a (V)
B-175-0-13	SS-10A	28.5	14											BROWN SILT AND CLAY, LITTLE SAND, TRACE STONE FRAGMENTS	A-6a (V)
B-175-0-13	SS-10B	29.5	25											BLACK CLAY, SOME SAND, TRACE STONE FRAGMENTS (FILL)	A-7-6 (V)

Pro Geotech, Inc.

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

Summary of Laboratory Results

Client: PARSONS BRINKERHOFF
Project: HAN-75-14.39 - Bridge No. HAN-75-1477
Location: FINDLAY, HANCOCK COUNTY, OHIO
PID Number: 87005

Pro Geotech, Inc.

Compressive Strength of Rock
ASTM D 7012

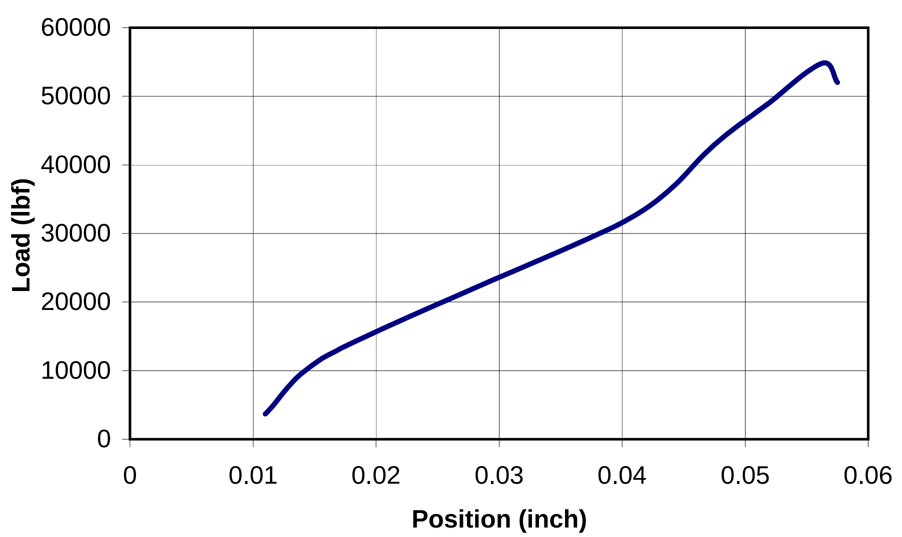
PROJECT	HAN-75-14.39	PGI PROJECT NO.	G13011G	DATE	9/17/2013

BORING NUMBER	B-167-0-13	TOP DEPTH (FT)	7.2	BOTTOM DEPTH (FT)	7.5
SAMPLE NUMBER	NX-1	DISTRICT	1	PID NO.	87005
COUNTY	HANCOCK	ROUTE	75	SECTION	1477
STATION	778+81	OFFSET	75.0'	OFFSET DIRECTION	RT

FORMATION	TYMOCHTEE / GREENFIELD GROUP
DESCRIPTION	Dolomite, light gray, moderately to slightly weathered, very strong.

MEASUREMENT	LENGTH (INCH)	DIAMETER (INCH)			
1	4.173	1.953		LENGTH/DIAMETER	2.13
2	4.187	1.962		CORRECTION FACTOR	1.00
3	4.169	1.959		AREA (SQ. INCH)	3.011
				MASS (GRAMS)	552.04
AVERAGE	4.176	1.958		UNIT WEIGHT (LBS/FT^3)	167.24

MAXIMUM LOAD (LBS)	54761
COMPRESSIVE STRENGTH (PSI)	18187
TIME OF TEST (MINUTES)	4:14
LOADING DIRECTION	PERPENDICULAR TO BEDDING
TECHNICIAN	FBUSHER



BEFORE TESTING

AFTER FAILURE

Pro Geotech, Inc.

Compressive Strength of Rock
ASTM D 7012

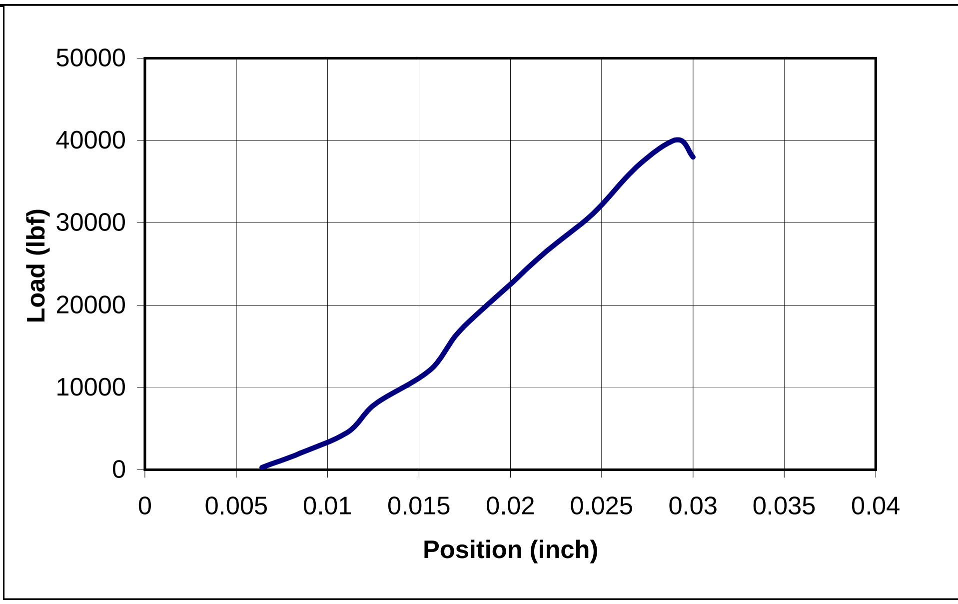
PROJECT	HAN-75-14.39	PGI PROJECT NO.	G13011G	DATE	9/16/2013

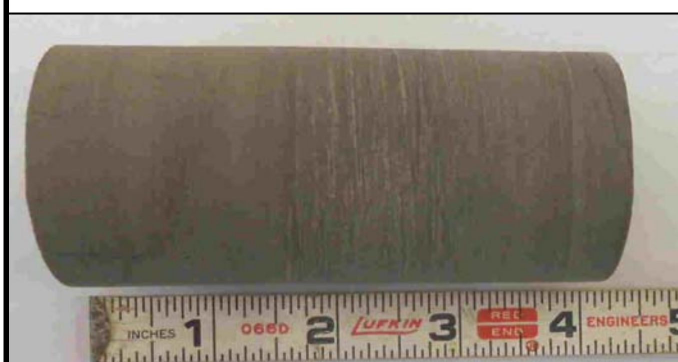
BORING NUMBER	B-168-0-13	TOP DEPTH (FT)	30.9	BOTTOM DEPTH (FT)	31.2
SAMPLE NUMBER	NX-1 & 2	DISTRICT	1	PID NO.	87005
COUNTY	HANCOCK	ROUTE	75	SECTION	1477
STATION	105+55	OFFSET	8.0'	OFFSET DIRECTION	RT

FORMATION	TYMOCHTEE / GREENFIELD GROUP
DESCRIPTION	Dolomite, gray, moderately weathered, strong.

MEASUREMENT	LENGTH (INCH)	DIAMETER (INCH)		LENGTH/DIAMETER	2.05
1	4.023	1.964		CORRECTION FACTOR	1.00
2	4.016	1.958		AREA (SQ. INCH)	3.018
3	4.014	1.959		MASS (GRAMS)	565.07
AVERAGE	4.018	1.960		UNIT WEIGHT (LBS/FT^3)	177.52

MAXIMUM LOAD (LBS)	40093
COMPRESSIVE STRENGTH (PSI)	13284
TIME OF TEST (MINUTES)	3.32
LOADING DIRECTION	PERPENDICULAR TO BEDDING
TECHNICIAN	FBUSHER





BEFORE TESTING

AFTER FAILURE

Pro Geotech, Inc.

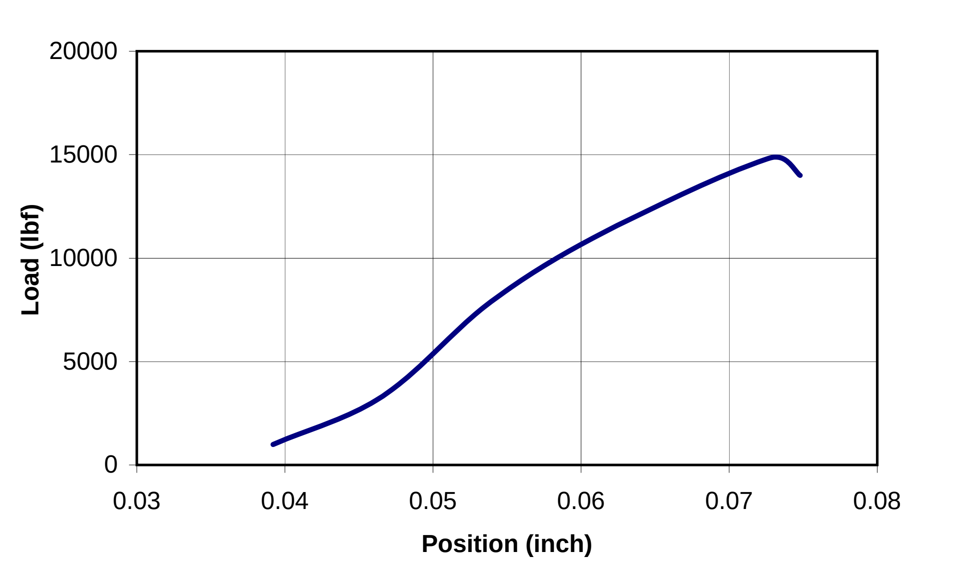
Compressive Strength of Rock
ASTM D 7012

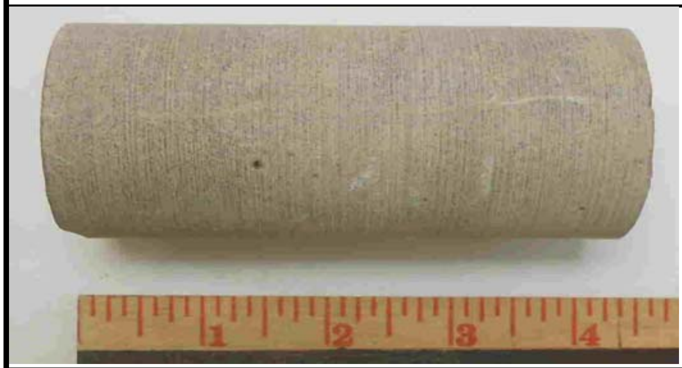
PROJECT	HAN-75-14.39 STRUCTURE	PGI PROJECT NO.	G13011G	DATE	9/6/2013
BORING NUMBER	B-170-0-13	TOP DEPTH (FT)	9.2	BOTTOM DEPTH (FT)	9.5
SAMPLE NUMBER	NX-1	DISTRICT	1	PID NO.	87005
COUNTY	HANCOCK	ROUTE	75	SECTION	1477
STATION	108+30	OFFSET	21.0'	OFFSET DIRECTION	LT

FORMATION	TYMOCHTEE / GREENFIELD GROUP
DESCRIPTION	Dolomite, gray, moderately weathered, strong.

MEASUREMENT	LENGTH (INCH)	DIAMETER (INCH)		
1	3.923	2.024	LENGTH/DIAMETER	1.95
2	4.016	2.026	CORRECTION FACTOR	1.00
3	3.916	2.026	AREA (SQ. INCH)	3.222
			MASS (GRAMS)	583.66
AVERAGE	3.952	2.025	UNIT WEIGHT (LBS/FT^3)	174.65

MAXIMUM LOAD (LBS)	14852
COMPRESSIVE STRENGTH (PSI)	4596
TIME OF TEST (MINUTES)	2:10
LOADING DIRECTION	PERPENDICULAR TO BEDDING
TECHNICIAN	FBUSHER





BEFORE TESTING

AFTER FAILURE

Pro Geotech, Inc.

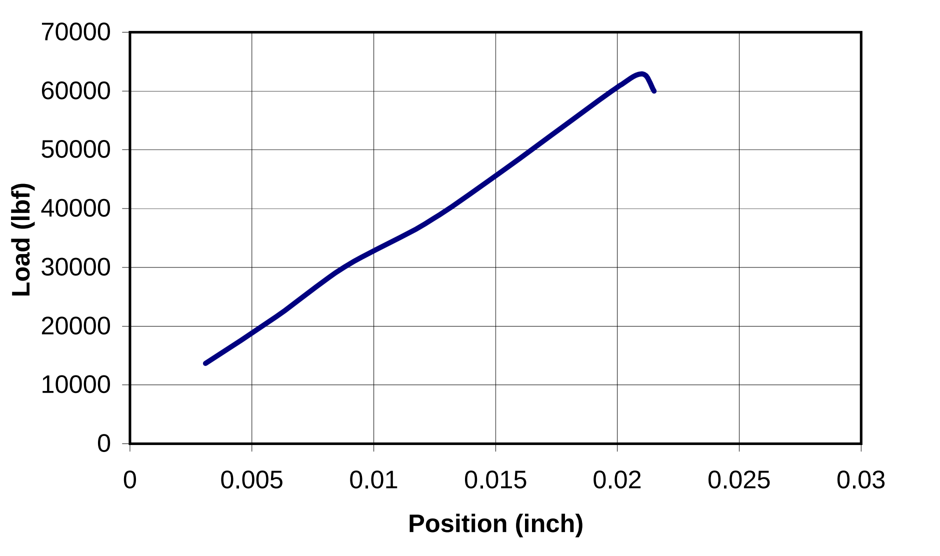
Compressive Strength of Rock
ASTM D 7012

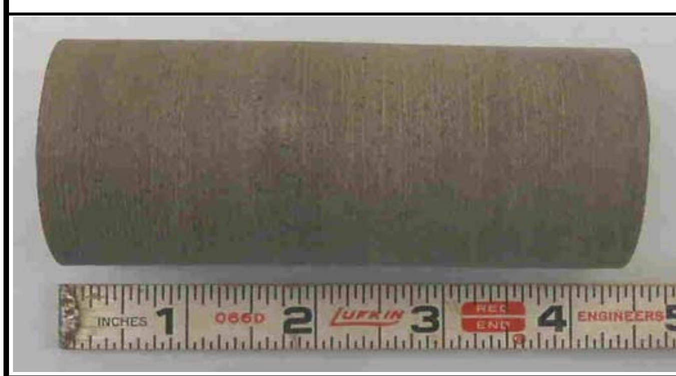
PROJECT	HAN-75-14.39	PGI PROJECT NO.	G13011G	DATE	9/6/2013
	STRUCTURE				
BORING NUMBER	B-175-0-13	TOP DEPTH (FT)	16.2	BOTTOM DEPTH (FT)	16.5
SAMPLE NUMBER	NX-1	DISTRICT	1	PID NO.	87005
COUNTY	HANCOCK	ROUTE	75	SECTION	1477
STATION	110+39	OFFSET	11.0'	OFFSET DIRECTION	LT

FORMATION	TYMOCHTEE / GREENFIELD GROUP
DESCRIPTION	Dolomite, light gray, moderately to slightly weathered, strong.

MEASUREMENT	LENGTH (INCH)	DIAMETER (INCH)		
1	4.097	1.991	LENGTH/DIAMETER	2.06
2	4.106	1.975	CORRECTION FACTOR	1.00
3	4.041	1.974	AREA (SQ. INCH)	3.079
			MASS (GRAMS)	550.22
AVERAGE	4.081	1.980	UNIT WEIGHT (LBS/FT^3)	166.80

MAXIMUM LOAD (LBS)	62938
COMPRESSIVE STRENGTH (PSI)	20441
TIME OF TEST (MINUTES)	3:20
LOADING DIRECTION	PERPENDICULAR TO BEDDING
TECHNICIAN	FBUSHER





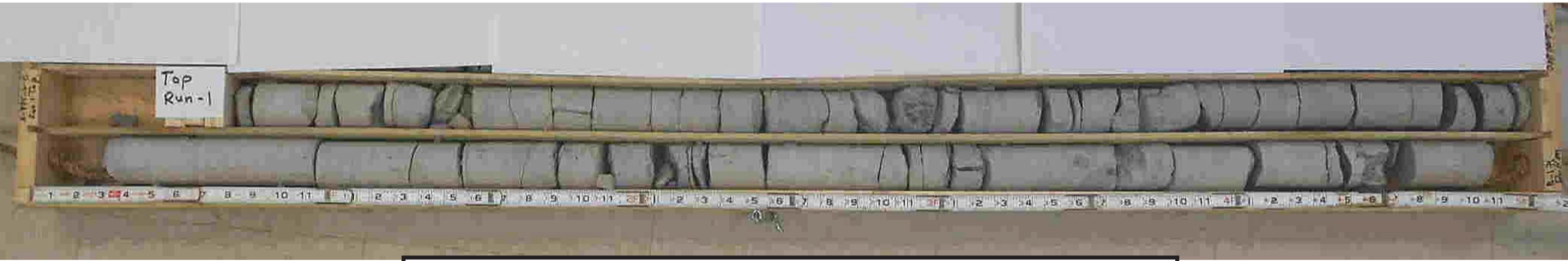
BEFORE TESTING

AFTER FAILURE

COMPANY: PGI DRILLED BY: B-M
PROJECT: HAN-75-14.39
BRIDGE NO.: HAN-75-1477 Harrison Street over IR-75
BORING: B-167-0-13 BOX 1/1
DATE of CORING: 8/6/13
RUN-1: 4.5' - 9.5'
REC: 88% RQD: 18%

COMPANY: PGI DRILLED BY: DLZ
PROJECT: HAN-75-14.39
BRIDGE NO.: HAN-75-1477 Harrison Street over IR-75
BORING: B-168-0-13 BOX 1/1
DATE of CORING: 7/23/13
RUN-1: 27.8' - 31.4' REC: 100% RQD: 55%
RUN-2: 31.4' - 37.8' REC: 100% RQD: 65%

COMPANY: PGI DRILLED BY: OTB
PROJECT: HAN-75-14.39
BRIDGE NO.: HAN-75-1477 Harrison Street over IR-75
BORING: B-170-0-13 BOX 1/1
DATE of CORING: 6/26/13
RUN-1: 4.3' - 14.3'
REC: 98% RQD: 42%



COMPANY: PGI DRILLED BY: DLZ
PROJECT: HAN-75-14.39
BRIDGE NO.: HAN-75-1477 Harrison Street over IR-75
BORING: B-175-0-13 BOX 1/1
DATE of CORING: 7/24/13
RUN-1: 37.2' - 47.2'
REC: 90% RQD: 45%

ROCK MASS RATING From Table 10.4.6.4-1	
Project: HAN-75-14.39	**Project No.:** G13011G
Structure: Bridge No. HAN-75-1477 over IR-75 and NS Railroad	
Boring No.: B-167-0-13	**Substructure Unit:** MSE Wall
Strength of Intact Rock Material	
Uniaxial Compressive Strength	2619 ksf
Relative Rating	8
Drill Core Quality RQD	
RQD	18%
Relative Rating	2
Joint Conditions	
Spacing of Joints	2" to 1'
Relative Rating	8
Conditions of Joints	Slightly Rough Surfaces, Separation < 0.05", and Hard Joint Wall
Relative Rating	17
Groundwater Conditions	
Relative Rating	7
Strike & Dip Orientation of Joint	
Relative Rating	0
Total Mass Rating	42
Class No	III
Description	Fair Rock
Boring No.: B-168-0-13	**Substructure Unit:** Rear Abutment
Strength of Intact Rock Material	
Uniaxial Compressive Strength	1912 ksf
Relative Rating	6
Drill Core Quality RQD	
RQD	61%
Relative Rating	10
Joint Conditions	
Spacing of Joints	2" to 1'
Relative Rating	6
Conditions of Joints	Slightly Rough Surfaces, Separation < 0.05", and Hard Joint Wall
Relative Rating	18
Groundwater Conditions	
Relative Rating	7
Strike & Dip Orientation of Joint	
Relative Rating	0
Total Mass Rating	47
Class No	III
Description	Fair Rock

ROCK MASS RATING From Table 10.4.6.4-1	
Project: HAN-75-14.39	**Project No.:** G13011G
Structure: Bridge No. HAN-75-1477 over IR-75 and NS Railroad	
Boring No.: B-170-0-13	**Substructure Unit:** Pier 2
Strength of Intact Rock Material	
Uniaxial Compressive Strength	662 ksf
Relative Rating	3
Drill Core Quality RQD	
RQD	42%
Relative Rating	6
Joint Conditions	
Spacing of Joints	2" to 1'
Relative Rating	8
Conditions of Joints	Slightly Rough Surfaces, Separation < 0.05", and Hard Joint Wall
Relative Rating	18
Groundwater Conditions	
Relative Rating	7
Strike & Dip Orientation of Joint	
Relative Rating	0
Total Mass Rating	42
Class No	III
Description	Fair Rock
Boring No.: B-175-0-13	**Substructure Unit:** Forward Abutment
Strength of Intact Rock Material	
Uniaxial Compressive Strength	2943 ksf
Relative Rating	9
Drill Core Quality RQD	
RQD	45%
Relative Rating	7
Joint Conditions	
Spacing of Joints	2" to 1'
Relative Rating	7
Conditions of Joints	Slightly Rough Surfaces, Separation < 0.05", and Hard Joint Wall
Relative Rating	17
Groundwater Conditions	
Relative Rating	7
Strike & Dip Orientation of Joint	
Relative Rating	0
Total Mass Rating	47
Class No	III
Description	

Bearing Resistance and Settlement Analyses of Footing on Jointed Rock

Project: HAN-75-14.39-HAN-75-1477 **Project No.:** G13011G
Boring No.: B-170-0-13 **Substructure Unit:** Pier 2

Rock Parameters	
Rock Mass Rating (RMR) (From AASHTO LRFD Table 10.4.6.4.1)	42
Class No. (From AASHTO LRFD Table 10.4.6.4.3)	III
Quality Description (From AASHTO LRFD Table 10.4.6.4.3)	Fair Rock
Uniaxial Compressive Strength of Rock (q_u, ksf) (From Laboratory Test (ASTM D 7012))	662
Presumptive Bearing Resistance for Spread Footing at Service Limit State (ksf) (From AASHTO LRFD Table C10.6.2.6.1-1)	20
Nominal Resistance of Concrete (ksf) = $0.3*f'_c$	173
Fractured Rock Mass Parameters "s" and "m" (From AASHTO LRFD Table 10.4.6.4.4)	m= 0.118 s= 0.000082
Poisson's Ratio of Intact Rock (From AASHTO LRFD Table C10.4.6.5-2)	0.29
Average Elastic Modulus for Intact Rock, E_i (ksi) (From Load vs Displacement from Lab Test, ASTM D 7012)	452
Elastic Modulus of Rock Mass, E_m (ksi) (From AASHTO LRFD Eq 10.4.6.5-1)	915
Reduction Factor (E_m/E_i) (From AASHTO LRFD Table 10.4.6.5-1)	0.12
Elastic Modulus of Rock Mass (E_m) (ksi) (From AASHTO LRFD Eq 10.4.6.5-2)	54
Assumed Em (ksi)	100
Nominal Bearing Resistance (Carter and Kulhawy (1988))	qult=$(\sqrt{s}+(m\sqrt{s}+s)^{0.5})$qu (**At the Strength Limit State**)
Effective Length of Footing, L (feet)	14
Effective Width of Footing, B (feet)	14
L/B	1.0
Type of Footing	Square
Depth of Footing Below Ground, D (feet)	1.6
Unit Weight of Soil above base of footing, y_q (pcf)	125
Unit Weight of Rock below base of footing, y_y (pcf)	165
Nominal Bearing Resistance (ksf) (Per AASHTO LRFD Article 10.6.3.2.2) (Carter and Kulhawy (1988))	28
Resistance Factor (From LRFD Table 10.5.5.2.2-1)	0.45
Factored Resistance (ksf)	13
Settlement Analysis (From LRFD Eq 10.6.2.4.4-3)=	$q_o(1-v^2)((B*I_p/(144*E_m))$ (**At the Service Limit State**)
Rigidity Factors, B_z for L/B (For Rigid Footing) (From AASHTO LRFD Table 10.6.2.4.2-1)	1.08
Influence Coefficient, I_p = $L/B)^{1/2}/B_z$ (From AASHTO LRFD Eq 10.6.2.4.4-4)	0.926
Nominal Bearing Rsistance (ksf)	20
Elastic Settlement p (inches)	0.198

Bearing Resistance and Settlement Analyses of Footing on Jointed Rock	
Project: HAN-75-14.39-HAN-75-1477	**Project No.:** G13011G
Boring No.: B-168-0-13	**Substructure Unit:** Rear MSE Wall
Rock Parameters	
Rock Mass Rating (RMR) (From AASHTO LRFD Table 10.4.6.4.1)	47
Class No. (From AASHTO LRFD Table 10.4.6.4.3)	III
Quality Description (From AASHTO LRFD Table 10.4.6.4.3)	Fair Rock
Uniaxial Compressive Strength of Rock (q_u, ksf) (From Laboratory Test (ASTM D 7012))	1912
Presumptive Bearing Resistance for Spread Footing at Service Limit State (ksf) (From AASHTO LRFD Table C10.6.2.6.1-1)	20
Nominal Resistance of Concrete (ksf) = 0.3*f'_c	173
Fractured Rock Mass Parameters "s" and "m" (From AASHTO LRFD Table 10.4.6.4.4)	m= 0.128 s= 0.00009
Poisson's Ratio of Intact Rock (From AASHTO LRFD Table C10.4.6.5-2)	0.29
Average Elastic Modulus for Intact Rock, E_i (ksi) (From Load vs Displacement from Lab Test, ASTM D 7012)	2910
Elastic Modulus of Rock Mass, E_m (ksi) (From AASHTO LRFD Eq 10.4.6.5-1)	1220
Reduction Factor (E_m/E_i) (From AASHTO LRFD Table 10.4.6.5-1)	0.15
Elastic Modulus of Rock Mass (E_m) (ksi) (From AASHTO LRFD Eq 10.4.6.5-2)	437
Assumed Em (ksi)	250
Nominal Bearing Resistance (Carter and Kulhawy (1988))	qult=($\sqrt{s}$+(m$\sqrt{s}$+s)$^{0.5}$)qu (**At the Strength Limit State**)
Effective Length of Footing, L (feet)	155
Effective Width of Footing, B (feet)	17.5
L/B	8.9
Type of Footing	Continuous
Depth of Footing Below Ground, D (feet)	5.7
Unit Weight of Soil above base of footing, y_q (pcf)	125
Unit Weight of Rock below base of footing, y_y (pcf)	165
Nominal Bearing Resistance (ksf) (Per AASHTO LRFD Article 10.6.3.2.2) (Carter and Kulhawy (1988))	87
Resistance Factor (From LRFD Table 10.5.5.2.2-1)	0.65
Factored Resistance (ksf)	57
Settlement Analysis (From LRFD Eq 10.6.2.4.4-3)= $q_o(1-v^2)((B*I_p/(144*E_m))$ (**At the Service Limit State**)	
Rigidity Factors, B_z for L/B (For Rigid Footing) (From AASHTO LRFD Table 10.6.2.4.2-1)	1.35
Influence Coefficient, I_p = L/B)$^{1/2}$/B_z (From AASHTO LRFD Eq 10.6.2.4.4-4)	2.205
Nominal Bearing Rsistance (ksf)	4
Elastic Settlement p (inches)	0.047

AASHTO 2007-2010 (LRFD)
HAN-75-14.39-Bridge No. HAN-75-1477

MSEW(3.0): Update # 14.93

PROJECT IDENTIFICATION

Title:	HAN-75-14.39-Bridge No. HAN-75-1477
Project Number:	87005
Client:	PB
Designer:	SS
Station Number:	105+71

Description:

External Stability Analysis of the Rear MSE Wall

Company's information:

Name: Pro Geotech, Inc
Street:

,
Telephone #:
Fax #:
E-Mail:

Original file path and name: M:\Project Files\13 Projects\G13011G HAN-75\Analysis Fi.....
.....77 Rear MSE Wall.BEN

Original date and time of creating this file: Tues April 8, 2014

PROGRAM MODE: ANALYSIS
of a BRIDGE ABUTMENT
using METAL STRIPS as reinforcing material.

SOIL DATA

Soil above reinforcement has the following properties:

Unit weight, γ	125.0 lb/ft ³
Design value of internal angle of friction, ϕ	30.0 °

REINFORCED SOIL

Unit weight, γ	120.0 lb/ft ³
Design value of internal angle of friction, ϕ	34.0 °

RETAINED SOIL

Unit weight, γ	120.0 lb/ft ³
Design value of internal angle of friction, ϕ	30.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{equiv.}$	125.0 lb/ft ³
Equivalent internal angle of friction, $\phi_{equiv.}$	30.0 °
Equivalent cohesion, $c_{equiv.}$	0.0 lb/ft ²

Water table is at wall base elevation

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Ka (external stability) = 0.3333 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 30.14 N_γ= 22.40

SEISMICITY

Not Applicable

INPUT DATA: Geometry and Surcharge loads (of a BRIDGE ABUTMENT)

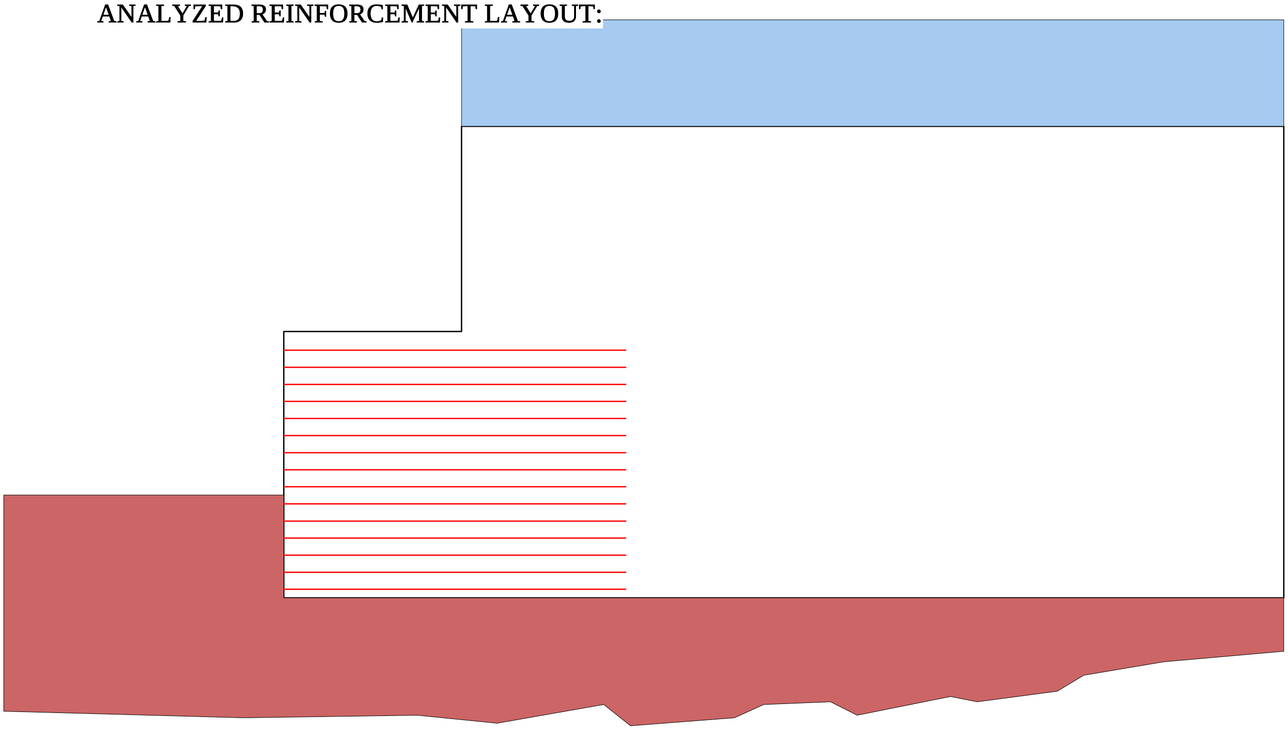
Design height, Hd	15.60	[ft]	{ Embedded depth is E = 6.00 ft, and height above top of finished bottom grade is H = 9.60 ft }
Batter, ω	0.0	[deg]	
Backslope, β	0.0	[deg]	
Backslope rise	0.0	[ft]	Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

U N I F O R M S U R C H A R G E

Uniformly distributed dead load is 0.0 [lb/ft ²], and live load is 375.0 [lb/ft ²]

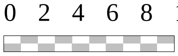
ABUTMENT GEOMETRY (On pile foundation.)
Abutment's width, bf = 8.00 at distance from back of wall, cf = 2.40 [ft].
Footing's dimension: height, h' = 12.00, width, b = 2.10, and thickness, t = 3.30 [ft].
Dimensions of bridge bearing plate: height, fh = 0.50, width, fw = 1.80 [ft].

ANALYZED REINFORCEMENT LAYOUT:



SCALE:

0 2 4 6 8 10 [ft]



AASHTO 2007-2010 (LRFD) Input Data

INTERNAL STABILITY

Load factor for vertical earth pressure, EV, from Table 3.4.1-2:	$\gamma_{p\text{-}EV}$	1.35	
Load factor for earthquake loads, EQ, from Table 3.4.1-1:	$\gamma_{p\text{-}EQ}$	1.00	
Load factor for live load surchrge, LS, from Figure C11.5.5-3(b): (Same as in External Stability).	$\gamma_{p\text{-}LS}$	1.75	
Load factor for dead load surchrge, ES: (Same as in External Stability).	$\gamma_{p\text{-}ES}$	1.50	
Resistance factor for reinforcement tension from Table 11.5.6-1:	ϕ	Static	Combined static/seismic
Metal Strips:		0.75	1.00
Resistance factor for reinforcement tension in connectors from Table 11.5.6-1:	ϕ	Static	Combined static/seismic
Metal Strips:		0.75	1.00
Resistance factor for reinforcement pullout from Table 11.5.6-1:	ϕ	0.90	1.20

EXTERNAL STABILITY

Load factor for vertical earth pressure, EV, from Table 3.4.1-2 and Figure C11.5.5-2:

		Static		Combined Static/Seismic
Sliding and Eccentricity	$\gamma_{p\text{-}EV}$	1.00	$\gamma_{p\text{-}EQ}$	1.00
Bearing Capacity	$\gamma_{p\text{-}EV}$	1.35	$\gamma_{p\text{-}EQ}$	1.35

Load factor of active lateral earth pressure, EH, from Table 3.4.1-2 and Figure C11.5.5-2:	$\gamma_{p\text{-}EH}$	1.50
Load factor of active lateral earth pressure during earthquake (does not multiply P_{AE} and P_{IR}):	$(\gamma_{p\text{-}EH})_{EQ}$	1.50
Load factor for earthquake loads, EQ, from Table 3.4.1-1 (multiplies P_{AE} and P_{IR}):	$\gamma_{p\text{-}EQ}$	1.00

Resistance factor for shear resistance along common interfaces from Table 11.5.6-1:

		Static	Combined Static/Seismic
Reinforced Soil and Foundation	ϕ_τ	1.00	1.00
Reinforced Soil and Reinforcement	ϕ_τ	1.00	1.00

Resistance factor for bearing capacity of shallow foundation from Table 11.5.6-1:

	Static	Combined Static/Seismic
ϕ_b	0.65	0.65

ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, CDR = 1.34, factored bearing load = 5093 lb/ft².

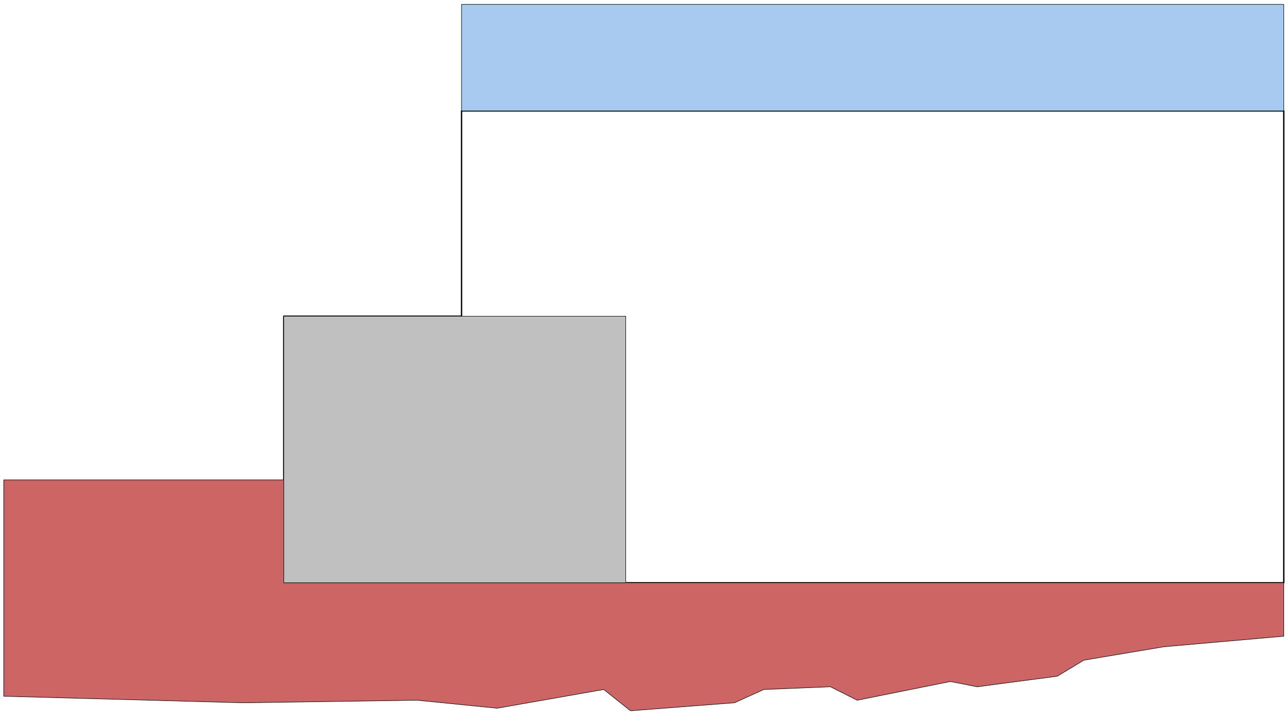
Foundation Interface: Direct sliding, CDR = 1.030, Eccentricity, e/L = 0.2324, CDR-overturning = 1.88

METAL STRIP				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	CDR [pullout resistance]	CDR [connection break]	CDR [metal strip strength]	Metal strip strength CDR	Pullout resistance CDR	Direct sliding CDR	Eccentricity e/L	Product name
1	0.50	20.00	1	N/A	3.43	3.43	3.432	4.126	1.214	0.2232	---
2	1.50	20.00	1	N/A	3.46	3.46	3.459	4.116	1.236	0.2047	---
3	2.50	20.00	1	N/A	3.48	3.48	3.483	4.060	1.258	0.1862	---
4	3.50	20.00	1	N/A	3.50	3.50	3.496	3.972	1.281	0.1677	---
5	4.50	20.00	1	N/A	3.51	3.51	3.509	3.891	1.303	0.1491	---
6	5.50	20.00	1	N/A	3.53	3.53	3.530	3.780	1.325	0.1302	---
7	6.50	20.00	1	N/A	3.56	3.56	3.559	3.663	1.347	0.1109	---
8	7.50	20.00	1	N/A	3.60	3.60	3.597	3.542	1.367	0.0911	---
9	8.50	20.00	1	N/A	3.64	3.64	3.643	3.451	1.386	0.0705	---
10	9.50	20.00	1	N/A	3.71	3.71	3.708	3.340	1.402	0.0489	---
11	10.50	20.00	1	N/A	3.79	3.79	3.794	3.237	1.416	0.0258	---
12	11.50	20.00	1	N/A	3.89	3.89	3.888	3.134	1.424	0.0007	---
13	12.50	20.00	1	N/A	3.99	3.99	3.987	3.068	1.427	-0.0271	---
14	13.50	20.00	1	N/A	4.09	4.09	4.089	2.967	1.421	-0.0590	---
15	14.50	20.00	1	N/A	2.65	2.65	2.653	1.814	1.403	-0.0967	---

BEARING CAPACITY for GIVEN LAYOUT

	STATIC	SEISMIC	UNITS
(Water table is at wall base elevation)			
Factored bearing resistance. q-n	6821	N/A	[lb/ft ²]
Factored bearing load, σ_V	5093.5	N/A	[lb/ft ²]
Eccentricity, e	2.51	N/A	[ft]
Eccentricity, e/L	0.126	N/A	
CDR calculated	1.34	N/A	
Base length	20.00	N/A	[ft]

Unfactored applied bearing pressure = (Unfactored R) / [L - 2 * (Unfactored e)] =
Unfactored R = 55439.97 [lb/ft], L = 20.00, Unfactored e = 1.74 [ft], and Sigma = 3356.37 [lb/ft ²]



SCALE:

0 2 4 6 8 10 [ft]

DIRECT SLIDING for GIVEN LAYOUT (for METAL STRIPS reinforcements)

Along reinforced and foundation soils interface: CDR-static = 1.030

#	Metal strip Elevation [ft]	Metal strip Length [ft]	CDR Static	CDR Seismic	Metal strip Type #	Product name
1	0.50	20.00	1.214	N/A	1	---
2	1.50	20.00	1.236	N/A	1	---
3	2.50	20.00	1.258	N/A	1	---
4	3.50	20.00	1.281	N/A	1	---
5	4.50	20.00	1.303	N/A	1	---
6	5.50	20.00	1.325	N/A	1	---
7	6.50	20.00	1.347	N/A	1	---
8	7.50	20.00	1.367	N/A	1	---
9	8.50	20.00	1.386	N/A	1	---
10	9.50	20.00	1.402	N/A	1	---
11	10.50	20.00	1.416	N/A	1	---
12	11.50	20.00	1.424	N/A	1	---
13	12.50	20.00	1.427	N/A	1	---
14	13.50	20.00	1.421	N/A	1	---
15	14.50	20.00	1.403	N/A	1	---

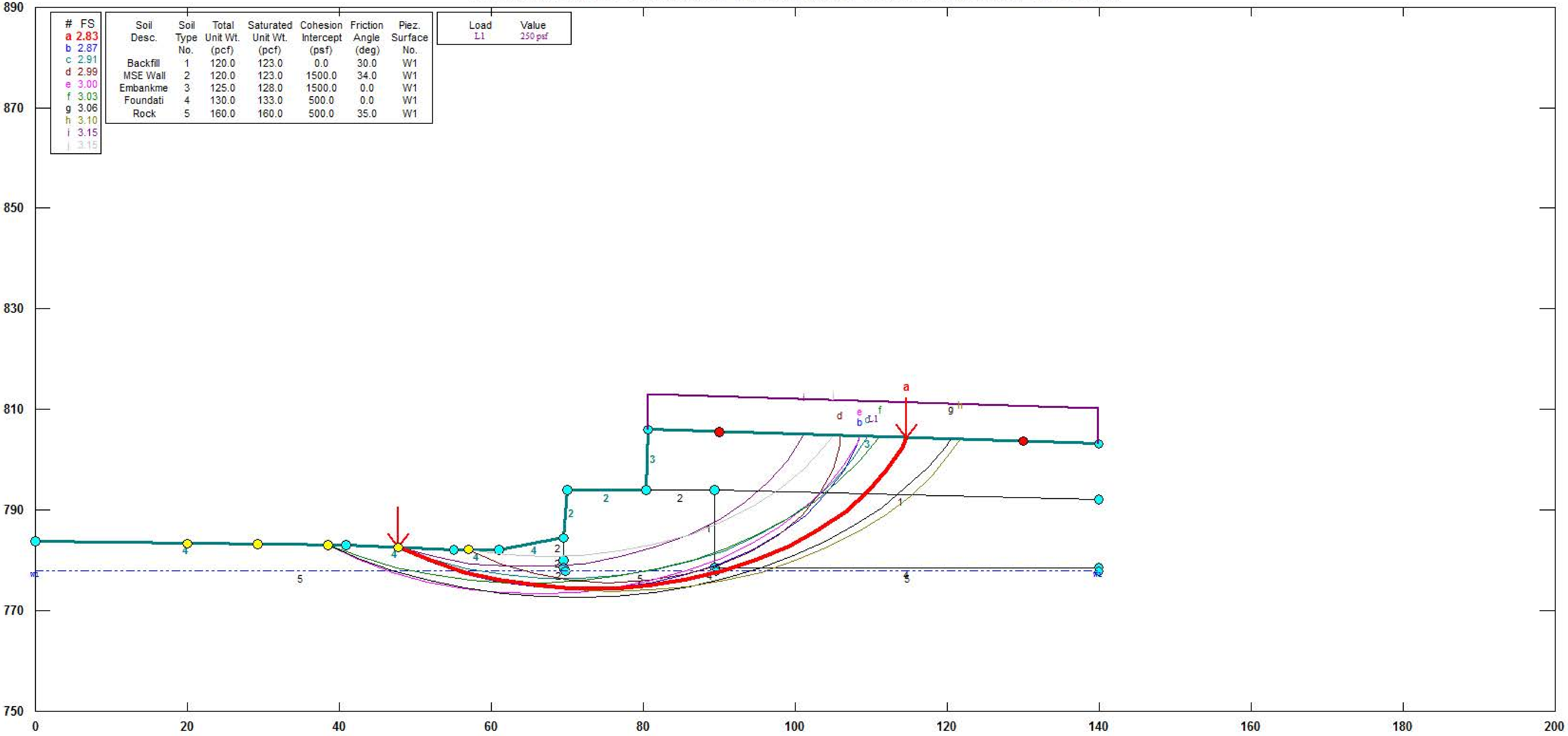
ECCENTRICITY for GIVEN LAYOUT (for Simplified Method)

At interface with foundation: e/L static = 0.2324; Overturning: CDR-static = 1.88

#	Metal strip Elevation [ft]	Metal strip Length [ft]	e / L Static	e / L Seismic	Metal strip Type #	Product name
1	0.50	20.00	0.2232	N/A	1	---
2	1.50	20.00	0.2047	N/A	1	---
3	2.50	20.00	0.1862	N/A	1	---
4	3.50	20.00	0.1677	N/A	1	---
5	4.50	20.00	0.1491	N/A	1	---
6	5.50	20.00	0.1302	N/A	1	---
7	6.50	20.00	0.1109	N/A	1	---
8	7.50	20.00	0.0911	N/A	1	---
9	8.50	20.00	0.0705	N/A	1	---
10	9.50	20.00	0.0489	N/A	1	---
11	10.50	20.00	0.0258	N/A	1	---
12	11.50	20.00	0.0007	N/A	1	---
13	12.50	20.00	-0.0271	N/A	1	---
14	13.50	20.00	-0.0590	N/A	1	---
15	14.50	20.00	-0.0967	N/A	1	---

HAN-75-14.39- Bridge No. HAN-75-1477- Global Stability of MSE Wall-Short Term

M:\PROJEC~2\13PROJ~1\G13011~1\ANALYS~1\BRIDGE~1\HARRIS~1\B168-0ST.PL2 Run By: John Smith, XYZ Company 8/27/2014 3:01PM



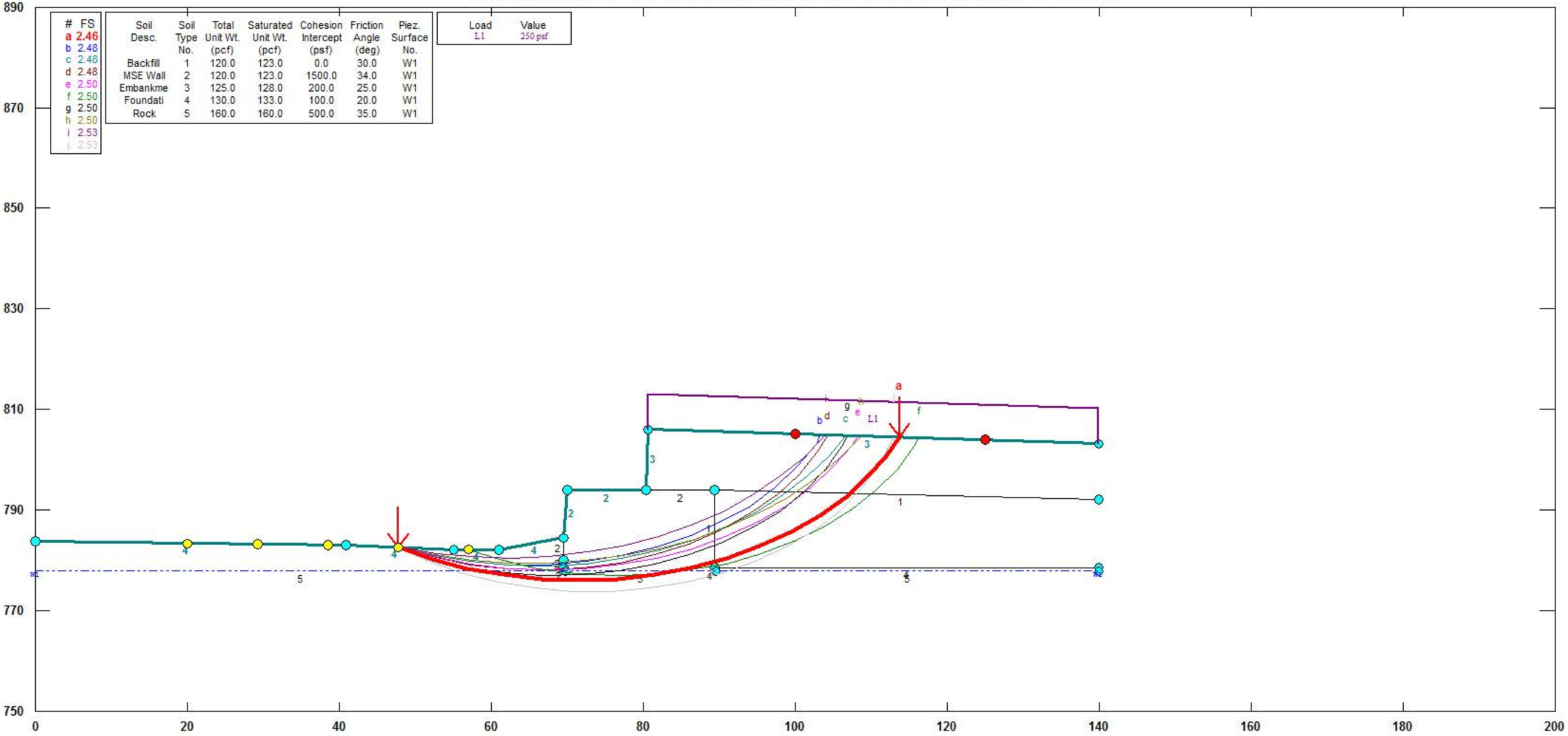
GSTABL7 v.2 FSmin=2.83

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

HAN-75-14.39- Bridge No. HAN-75-1477- Global Stability of MSE Wall-Long Term

M:\PROJEC~2\13PROJ~1\G13011~1\ANALYS~1\BRIDGE~1\HARRIS~1\B1680RLT.PL2 Run By: John Smith, XYZ Company 8/27/2014 3:07PM



GSTABL7 v.2 FSmin=2.46
Safety Factors Are Calculated By The Modified Bishop Method

LEGEND

Gravel

Sandy gravel

Silty gravel

Clayey gravel

Silty sandy gravel

Clayey sandy gravel

Gravelly sand

Silty gravelly sand

Clayey gravelly sand

Sand

Silty sand

Clayey sand

Gravelly silt

Sandy gravelly silt

Gravelly sandy silt

Sandy silt

Silt

Clayey silt

Gravelly clay

Sandy gravelly clay

Gravelly sandy clay

Sandy clay

Silty clay

Silt and clay

Clay

Dolomite

T.H.B.R. Top of Hard Broken Rock

Auger Boring - Plan View

Drive Sample - Core Boring - Plan View

Drive Rod Penetration Resistance Soundings - Plan View

A Indicates Auger Boring

B Indicates Drive Sample - Core Boring

Horizontal bar indicates the depth the sample was taken

X/Y Figures to the left of boring log in profile view
X = Plastic Limit
Y = Moisture Content

P/Q Figures to the right of boring log in profile view indicate the number of blows for "Standard Penetration Test" P = First 6 inches
Q = Second 6 inches

Drive Rod Penetration Resistance Soundings - Depth Penetrated

Casing

Resistance 'R' < 10,000 lbs

Resistance 'R' > 10,000 lbs

1/16 Indicates final measurement of penetration in inches

Sod & Topsoil - Visual Classification

Berm material - Visual Classification

Indicates water elevation

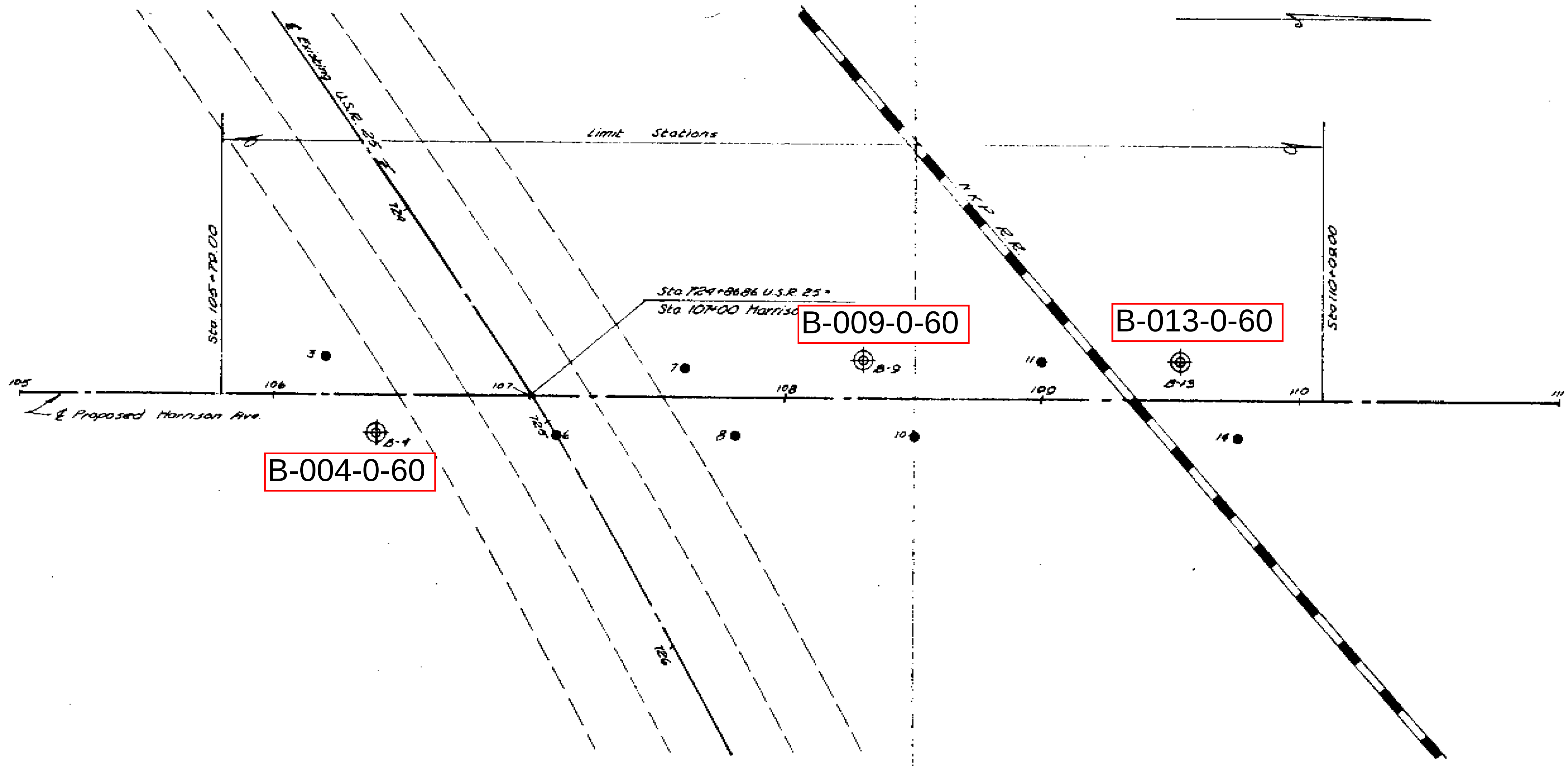
Footing

Footing on pile

Capped pile

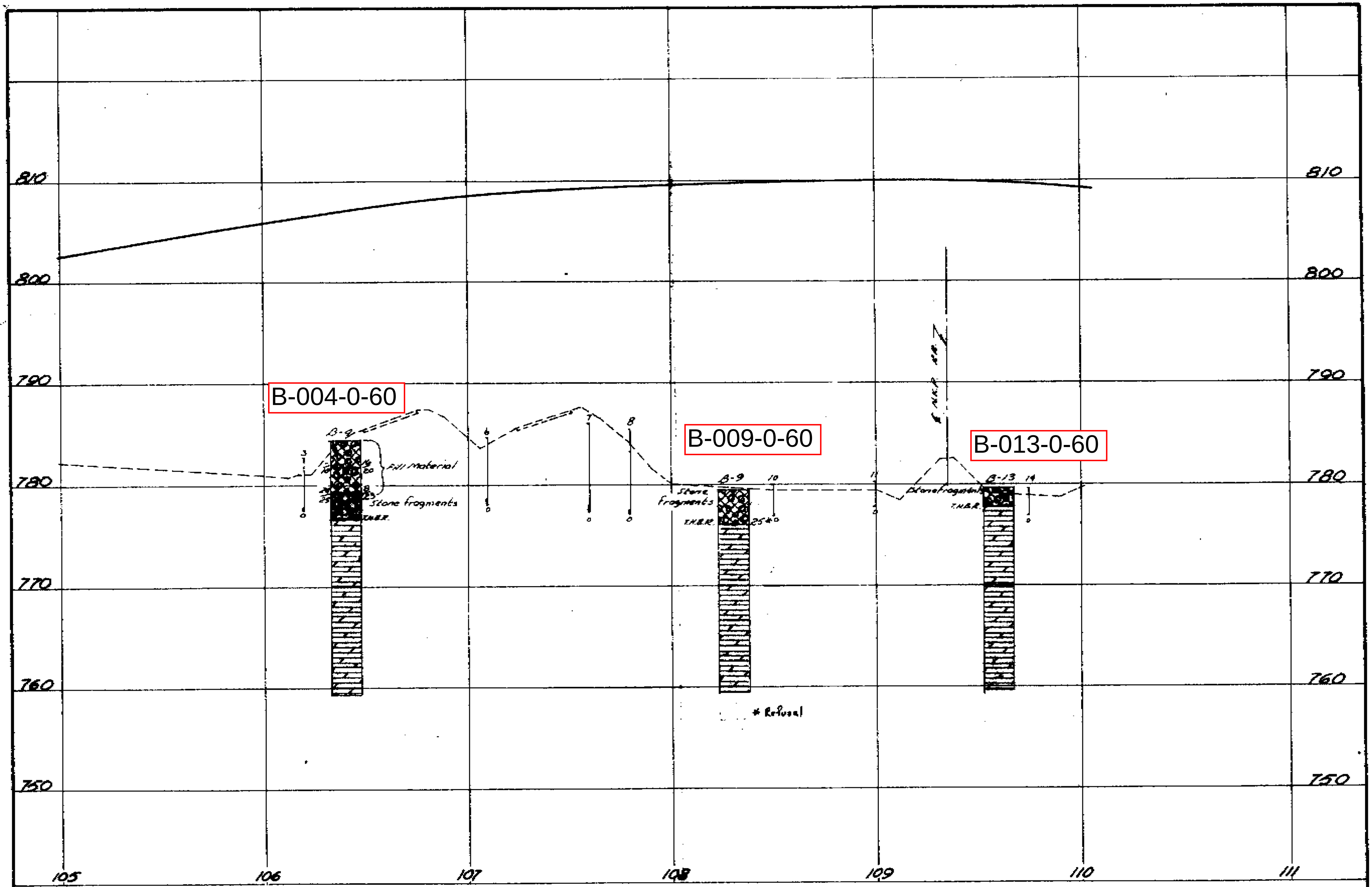
HAN-25-1478
HARRISON AVE.
OVER
U.S.R. 25 & N.K.P. R. R.

Drafting By G. V. Z.



BORING PLAN
SCALE: 1"=50'

HAN-25-1478
HARRISON AVE.
OVER
U.S.R. 25 & N.K.P. R.R.



SUBSURFACE PROFILE

HOR. SCALE - 1" = 50'
VERT. SCALE - 1" = 10'

HAN-25-1478
HARRISON AVE.
OVER
U.S.R. 25 & N.K.P. R.R.

State of Ohio
Department of Highways
Testing Laboratory

LOG OF BORING Historic Boring B-004-0-60

Date Started 12-14-60 Sampler Type SS Dia. 1 3/8" Water Elev. ____
Date Completed 12-14-60 Casing Length NONE Dia. ____

Project Identification HANCOCK
HAN-25-8.90
HAN-25-1478 HARRISON AVE.
OVER USR 25 & NKP R.R.

Boring No. B-4 Station & Offset 106+40, 15'RT(FIRST PIER) Surface Elev. 784.3

Elev.	Depth	Std. Pen. (N)	Rec. ft.	Loss ft.	Description	Field No.	Lab. Nos. So.	% Agg.	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	SHTL Class
784.3	0															
781.8	2	16/20			Brown Clayey Gravel - Fill Material	1	90944	V	I	S	U	A	L		10	
779.3	4	8/23			Dark-Brown Clay & Stone Fragments	2	90945	37	3	11	23	26	36	12	25	
776.8	8				TOP OF HARD BROKEN ROCK											
	10		2.4	0.1												
	12–14		5.0	0.0	Dolomite, gray, massive-bedded, hard, broken, with trace of carbonaceous shale. Core loss: 8%											
	18		4.6	0.4												
759.3	24		4.9	0.1	BOTTOM OF BORING											

Particle Sizes: Agg= >2.00mm, Coarse Sand=2.00-0.42 mm, Fine Sand=0.42-0.074mm, Silt=0.074-0.005mm, Clay= <0.005mm

State of Ohio
Department of Highways
Testing Laboratory

LOG OF BORING

Historic Boring B-009-0-60

Date Started 12-13-60 Sampler Type SS Dia 1 3/8" Water Elev.
Date Completed 12-14-60 Casing: Length Dia

Project Identification HANCOCK
HAN-25-8.90
HAN-25-1478 HARRISON AVE.
OVER USR 25 & NKP R.R.

Boring No. B-9 Station & Offset 108+30, 15' LT (FOURTH PIER) Surface Elev. 779.4

Elev.	Depth	Std. Pen. (N)	Rec. ft.	Loss ft.	Description	Field No	Lab. Nos. So.	% Agg.	% C.S.	% F.S.	% Silt	% Clay	L.L.	PI	W.C.	SHTL Class
779.4	0															
775.9	4	25*			Clay and Stone Fragments	1	-	V	I	S	U	A	L			
			1.2	0.8	TOP OF HARD BROKEN ROCK											
			0.9	1.6												
			2.3	0.2												
			5.0	0.0	Dolomite, gray, massive-bedded, hard, broken, with trace of carbonaceous shale at 19.6'. Core loss: 11%											
			5.0	0.0												
759.4	20				BOTTOM OF BORING											

*REFUSAL

Particle Sizes: Agg = >2.00mm, Coarse Sand = 2.00-0.42mm, Fine Sand = 0.42-0.074mm, Silt = 0.074-0.005mm, Clay = <0.005mm

SHEET 6 of 14 sheets

State of Ohio
Department of Highways
Testing Laboratory

LOG OF BORING Historic Boring B-013-0-60

Water Elev. Immediate
After Hours.

Project Identification HANCOCK
HAN-25-8.00
HAN-25-1478 HARRISON AVE.
OVER USR 25 & NKP R.R.

Date Started 12-15-60 Date Completed 12-15-60

Boring No. B-13 Station & Offset 109+54, 15'LT(SIXTH PIER) Surface Elev. 779.8

Elev.	Depth	Rec. ft	Loss ft	Description
779.8	0			Clay and Stone Fragments
777.8	2			TOP OF HARD, BROKEN ROCK
	4	1.2	1.8	
	6			
	8	4.0	0.0	Dolomite, gray, massive-bedded, hard, broken, with trace of carbonaceous shale. Core loss: 14%
	10			
	12			
	14	4.9	0.1	
	16			
	18	4.6	0.4	
759.8	20			BOTTOM OF BORING
	22			
	24			
	26			
	28			
	30			
	32			
	34			
	36			

SHEET 7 of 14 sheets

Form FE-21

VI.D. Geotechnical Reports

C-R-S: HAN-75-14.39-Bridge No. HAN-75-1477	PID:87005	Reviewer:SS	Date:8/27/2014

General		
Y N $\boxed{X}$ 1	Has the first complete version of a geotechnical report being submitted been labeled as 'Draft'?	
$\boxed{Y}$ N X 2	Subsequent to ODOT's review and approval, has the complete version of the revised geotechnical report being submitted been labeled 'Final'?	
$\boxed{Y}$ N X 3	Have all geotechnical reports being submitted been titled correctly as prescribed in Section 705.1 of the SGE?	

Report Body		
$\boxed{Y}$ N X 4	Do all geotechnical reports being submitted contain an Executive Summary as described in Section 705.2 of the SGE?	
$\boxed{Y}$ N X 5	Do all geotechnical reports being submitted contain an Introduction as described in Section 705.3 of the SGE?	
$\boxed{Y}$ N X 6	Do all geotechnical reports being submitted contain a section titled "Geology and Observations of the Project," as described in Section 705.4 of the SGE?	
$\boxed{Y}$ N X 7	Do all geotechnical reports being submitted contain a section titled "Exploration," as described in Section 705.5 of the SGE?	
$\boxed{Y}$ N X 8	Do all geotechnical reports being submitted contain a section titled "Findings," as described in Section 705.6 of the SGE?	
$\boxed{Y}$ N X 9	Do all geotechnical reports being submitted contain a section titled "Analyses and Recommendations," as described in Section 705.7 of the SGE?	

VI.D. Geotechnical Reports

Appendices	
$\boxed{Y}$ N X 10 Do all geotechnical reports being submitted contain all applicable Appendices as described in Section 705.8 of the SGE?	
$\boxed{Y}$ N X 11 Do the Appendices present a site Boring Plan showing all boring locations as described in Section 705.8.1 of the SGE?	
$\boxed{Y}$ N X 12 Do the Appendices include boring logs as described in Section 705.8.2 of the SGE?	
$\boxed{Y}$ N X 13 Do the Appendices present reports of undisturbed test data as described in Section 705.8.3 of the SGE?	
$\boxed{Y}$ N X 14 Do the Appendices present calculations in a logical format to support recommendations as described in Section 705.8.4 of the SGE?	

Notes:

IV.A Foundations/Structures - Non-bridge Applications

C-R-S: HAN-75-14.39-Bridge No. HAN-75-1477	PID:87005	Reviewer:SS	Date:8/27/2014

If you do not have such a foundation or structure on the project, you do not have to fill out this checklist.

Soil and Bedrock Strength Data		
Y $\boxed{\text{N}}$ X 1	Has the shear strength of the foundation soils been determined? Check method used: □ laboratory shear tests □ estimation from SPT or field tests	Bridge Foundations bear on bedrock
Y N X 2	Have sufficient soil shear strength, consolidation, and other parameters been determined so that the required allowable loads for the foundation/structure can be designed?	
$\boxed{\text{Y}}$ N X 3	Has the shear strength of the foundation bedrock been determined? Check method used: □ laboratory shear tests $\boxed{□}$ other List Other items:	Compression Strength Test

Notes:

Stage 1:

IV.A Foundations/Structures - Non-bridge Applications

Spread Footings				
$\boxed{Y}$ N	4		Are there spread footings on the project? If no, go to Question **11**	
$\boxed{Y}$ N X	5		Has the recommended bottom of footing elevation and reason for this recommendation been provided?	
Y N $\boxed{X}$		a	Has the recommended bottom of footing elevation taken scour from streams or other water flow into account?	
	6		Were representative sections analyzed for the entire length of the structure for the following:	
$\boxed{Y}$ N X		a	bearing capacity?	
Y $\boxed{N}$ X		b	sliding?	To be analyzed by PB
Y $\boxed{N}$ X		c	overturning?	To be analyzed by PB
$\boxed{Y}$ N X		d	settlement?	
Y $\boxed{N}$ X	7		Has the need for a shear key been evaluated?	To be evaluated by PB
Y $\boxed{N}$ X		a	If needed, have the details been included in the plans?	To be included by PB
Y N $\boxed{X}$	8		If special conditions exist (e.g. geometry, sloping rock, varying soil conditions), was the bottom of footing “stepped” to accommodate them?	
$\boxed{Y}$ N X	9		Has the recommended allowable soil or rock bearing pressure been provided?	
Y N $\boxed{X}$	10		If weak soil is present at the proposed foundation level, has the removal / treatment of this soil been developed and included in the plans?	
Y N $\boxed{X}$		a	Have the procedure and quantities related to this removal / treatment been included in the plans?	

Notes:

Stage 1:

IV.A Foundations/Structures - Non-bridge Applications

Pile Structures			
$\boxed{Y}$ N	11	Are there piles on the project? If no, go to Question **17**	
$\boxed{Y}$ N	12	Has an appropriate pile type been selected? Check the type selected: ☐ H-pile (driven) ☐ H-pile (drilled) ☐ Cast In-place Concrete ☐ other List Other items:	
$\boxed{Y}$ N X	13	Have the estimated pile length or tip elevation and section (diameter) been specified? Check method used: ☐ SPILE, DRIVEN, or equivalent software ☐ hand calculations	
	14	If required for design, have sufficient soil parameters been provided and calculations performed to evaluate the:	
Y $\boxed{N}$ X		a Lateral load capacity and maximum deflection of the piles?	Lateral Load Analysis will be performed by PB
$\boxed{Y}$ N X		b Vertical load capacity and maximum settlement of the piles?	
$\boxed{Y}$ N X		c Negative skin friction on piles driven through new embankment or soft foundation layers?	
Y N $\boxed{X}$		d Potential for and impact of lateral squeeze from soft foundation soils?	
$\boxed{Y}$ N X	15	If piles are to be driven to bedrock, have "pile points" been recommended to assure secure contact with the rock surface, as per BDM 202.2.3.2.a?	
Y N $\boxed{X}$	16	If subsurface obstacles exist, has preboring been recommended to avoid these obstructions?	

Notes:

Stage 1:

IV.A Foundations/Structures - Non-bridge Applications

Drilled Shafts				
Y $\boxed{N}$	17	Are there drilled shafts on the project?	
		If no, go to the next checklist.	
Y N X	18	Have the drilled shaft diameter and embedment length been specified?	
Y N X	19	Have the recommended drilled shaft diameter and embedment been developed based on side friction and end bearing for vertical loading situations?	
	20	For shafts undergoing lateral loading, have the following been determined:	
Y N X		a. maximum lateral shear	
Y N X		b. maximum bending moment	
Y N X		c. maximum deflection	
Y N X		d. reinforcement design	
Y N X	21	Generally, bedrock sockets are 6" smaller in diameter than the soil embedment section of the drilled shaft. Has this factor been accounted for in the drilled shaft design?	
Y N X	22	If a bedrock socket is required below soil embedment, have separate quantities been estimated based on shaft diameters and materials to be excavated?	
Y N X	23	Has the site been assessed for groundwater influence?	
Y N X		a If yes, if artesian flow is a potential concern, does the design address control of groundwater flow during construction?	
Y N X	24	If special construction features (e.g., slurry, casing, load tests) are required, have all the proper items been included in the plans?	

Notes:

Stage 1

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

STANDARD	REFERENCE NUMBER
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 x 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

Cohesive (fine grained) Soils - Consistency

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

2) Color :

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

3) Primary Component

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

4) Component Modifiers:

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

5) Soil Organic Content

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

6) Relative Visual Moisture

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

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.

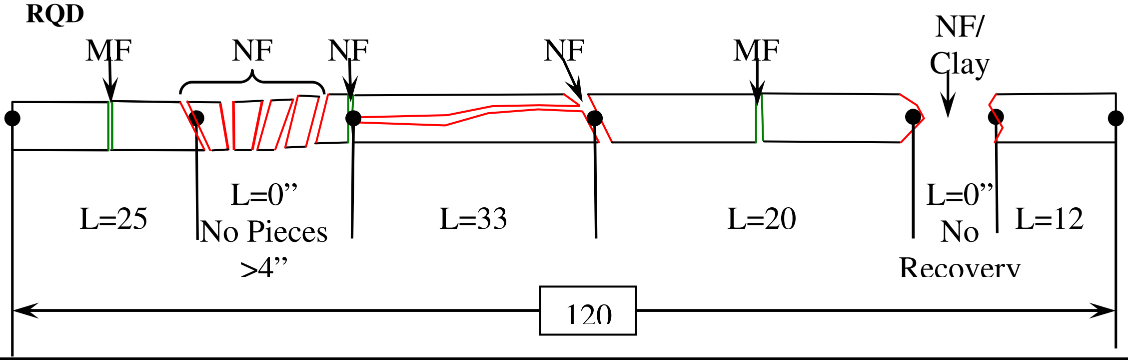
10) LOSS

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

L_R=Run Length R_R=Run Recovery

L_U=Rock Unit Length R_U=Rock Unit Recovery

9) RQD



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

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