Pro Geotech, Inc.

CLEVELAND: 3201 E. Royalton Road, Cleveland, Ohio 44147 - Phone 440-717-1415
COLUMBUS: 2572 Oakstone Drive, Unit 4, Columbus, Ohio 43231 - Phone 614 898 9233

January 23, 2015

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-68-1668 over US 68 Ramp A and Norfolk Southern Railroad
Findlay, Hancock County, Ohio
ODOT PID No. 87005 and 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. Sivakum

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

Walid Najjar

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

Enclosure
G13011Grpt/HAN-68-1668Bridges/SS/1/23/2015

FINAL
STRUCTURE FOUNDATION EXPLORATION REPORT
FOR HAN-75-14.39
BRIDGE NO. HAN-68-1668 OVER US 68 RAMP A & NORFOLK SOUTHERN RAILROAD

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

PREPARED FOR:

PARSONS BRINCKERHOFF

PREPARED BY:

PRO GEOTECH, INC.

JANUARY 23, 2015

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

APPENDICES

A Boring Location Map
  Drilling Logs
  Test Boring Profile

B Laboratory Test Results
  Unconfined Compressive Strength of the Rock Core
  Rock Core Samples Pictures
  Rock Mass Rating Spreadsheets
  Bearing Resistance and Settlement Analyses of Footing on Jointed Rock
  Bearing Capacity Analyses Spreadsheets for MSE Wall
  External Stability Analyses Computer Output for MSE Wall
  Global Stability Analyses Computer Output for MSE Wall
  Geotechnical Design Check List
  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 design and construction of the U.S. Route 68 (US 68) Ramp C Bridge No. HAN-68-1668 over US 68 Ramp A and Norfolk Southern Railroad as part of redesigning the IR-75/US 68 Interchange in Findlay, Hancock County, Ohio. A total of six (6) bridge test borings identified as B-126-0-13, B-128-0-13, B-129-0-13, B-130-0-13, B-131-0-13, and B-131-1-13 were advanced for bridge and MSE wall foundations design purposes. Test borings B-130-0-13, B-131-0-13, and B-131-1-13 were advanced in the vicinity of the proposed rear abutment and MSE wall while test borings B-126-0-13 and B-128-0-13 were advanced in the vicinity of the proposed forward abutment and MSE wall. Test boring B-129-0-13 was advanced in the vicinity of the proposed bridge Pier. These structural test borings were advanced to approximate depths ranging from 14.0 to 29.0 feet below the existing ground surface.

Subsurface soil Conditions: The subsurface soils encountered in the test borings consisted primarily of natural soils, however fill material was encountered above natural soils in test borings B-131-0-13 and B-131-1-13 to depths of 8.5 feet and 3.5 feet, respectively. The fill material consisted of sandy silt (A-4a), silt and clay (A-6a), and clay (A-7-6). Natural soils encountered above bedrock in the test borings consisted of both cohesive and non-cohesive soils. Cohesive soils consisted of sandy silt (A-4a), silt and clay (A-6a), and silty clay (A-6b) and non-cohesive soils consisted of non-plastic/granular stone fragments with sand (A-1-b), non-plastic sandy silt (A-4a), and non-plastic silt (A-4b). Bedrock was encountered in all test boring locations at approximate depths ranging from 5.5 feet to 13.5 feet and averaging 7.3 feet below the existing ground surface. The consistency ranged from "medium stiff" to "very stiff", but was generally “stiff”. All of the test borings were terminated after obtaining rock core samples.

Bedrock Conditions: The core samples consisted of dolomite of the Tymochtee/Greenfield Group. The dolomite was light gray to gray, severely to slightly weathered, and strong. Bedding within the dolomite was generally very thin to medium 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 core specimens ranged from 10,625 psi in test boring B-128-0-13 to 25,379 psi in test boring B-131-0-13 which characterizes them as “strong” to “very strong”, respectively. The Rock Quality Designation (RQD) for the core samples ranged from 0% to 63% and averaged 29% based on individual runs and a weighted average of 40%. The Rock Mass Rating obtained for the bedrock core samples according to LRFD Table 10.4.6.4-1 varied from 48 to 60 and is classified as "Fair Rock" to "Good Rock".

Bridge Foundation Systems: Soil and rock information obtained from structural test borings B-126-0-13, B-128-0-13, B-129-0-13, B-130-0-13, B-131-0-13, and B-131-1-13 were used to provide foundation recommendations for the proposed bridge abutments. Since bedrock was encountered at relatively shallow depths below the bottom of the proposed MSE Walls at the proposed abutments and below the existing ground at the proposed pier location, the proposed superstructure loads may be transferred to the underlying bedrock by means of shallow foundations.

Pier: Shallow foundation system consisting of spread footing may be used to transfer the loads to the underlying bedrock at the proposed pier location. Table 6.1.1 summarizes the Factored bearing resistance on bedrock and founding elevation at each test boring location so that PB personnel can verify the bearing pressure at Strength, Extreme Limit, and Service States.

Table 6.1.1 – Estimated Design Parameters at Strength Limit State for Spread Footings

Boring No.	Substructure Location	Top of Bedrock Elevation (feet)	Proposed Bearing Elevation (feet)	Factored Bearing Resistance (ksf)
B-129-0-13	Pier	771.0±	770.0	35.0

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. Since the proposed spread footing 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.

Abutments: According to ODOT *Bridge Design Manual* Section 204.4, MSE Wall supported abutments should be supported on piles regardless of the proximity of bedrock to the MSE Wall foundation. Therefore the proposed superstructure loads at the abutment locations should be transferred to the underlying bedrock by means of 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. These pre-bored holes should be backfilled with Class C concrete up to the top of the leveling pad elevation. The end bearing H-piles should also be installed with a minimum embedment length of 15.0 feet below the bottom of the MSE Wall. The estimated pile parameters for end bearing piles at each boring location are summarized in Table 6.1.2.

Table 6.1.2 - 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
B-131-0-13	779.6	812.4	764.6	50	H-Pile	10X42	310 kips
B-131-0-13	779.6	812.4	764.6	50	H-Pile	12X53	380 kips
B-128-0-13	774.6	802.7	759.6	45	H-Pile	10X42	310 kips
B-128-0-13	774.6	802.7	759.6	45	H-Pile	12X53	380 kips

<u>MSE Wall Foundation Systems</u>: Soil and rock information obtained from test borings; B-130-0-13, B-131-0-13 and B-131-1-13 for the proposed rear MSE Wall and B-126-0-13 and B-128-0-13 for the proposed forward MSE Wall were used to provide foundation recommendations for the proposed MSE Walls. The foundation soils encountered below the bottom of the MSE Walls consisted of both fill and natural soils above bedrock and were generally cohesive in nature. The consistency of these cohesive soils ranged from “stiff to “very stiff” but was generally "stiff”. These cohesive soils encountered in all test boring locations will not support the applied loads from the MSE Walls. Therefore, PGI recommends performing ground improvement on the foundation soils at the rear and forward MSE Walls in the vicinity of these test boring locations. According to recommendations provided by OGE, ground improvements should be performed by removing soils to the bedrock below the bottom of the MSE Walls and replacing it with compacted ODOT Item 304. Table 6.2.1 summarizes the proposed approximate excavation depths below the existing ground and proposed approximately excavation depths below the bottom of the MSE Walls at each test boring location. The ground improvements must be performed in front of the wall and behind the reinforced zone. The removal in front of the wall and behind the reinforced zone should be extended a lateral distance equal to the depth of removal at these two points respectively. Any replacement or backfill material beyond 2 feet behind the reinforcing strips and above the bottom of the leveling pad should consist of Item 203 Embankment, not Item 840 Select Granular Backfill.

Table 6.2.1 – Summary of Excavation Depths for Ground Improvements

Boring No.	MSE Wall Location	Existing Ground Elevation (feet)	Bottom of MSE Wall Elevation (feet)	Existing Bedrock Elevation (feet)	Excavation Depth Below Existing Ground (feet)	Excavation Depth Below MSE Wall (feet)
B-130-0-13	Rear	776.2	779.6	771.7	4.5	7.9
B-131-0-13	Rear	784.3	779.6	770.8	13.5	8.8
B-131-1-13	Rear	779.2	779.6	772.0	7.2	7.6
B-126-0-13	Forward	777.3	774.6	770.8	6.5	3.8
B-128-0-13	Forward	777.7	774.6	772.2	5.5	2.4

Bearing capacity analysis was performed by using effective stress shear strength parameters to estimate the nominal bearing resistance of the strip footings supported on ODOT Item 304 granular soils. Nominal bearing resistance corresponding to bearing elevation at the MSE Wall boring locations is summarized in Table 6.2.2.

Table 6.2.2 – 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-130-0-13	Rear MSE Wall	4.0	31.6	779.6	10.6
B-128-0-13	Forward MSE Wall	3.2	28.2	774.6	9.2

External stability of the MSE Walls including sliding on the base, limiting eccentricity, and bearing resistance at the Strength Limit States and settlement analysis at the Service Limit States were performed at the rear and forward abutment locations. The External Stability analyses results shows that the Capacity Damand 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 and forward MSE Walls.

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 6.2.3 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 6.2.3 –Summary of Critical Factors of Safety for MSE Walls

Boring No	Location	Stability	Method Used	Factor of Safety
B-131-0-13	Rear MSE Wall	Short Term	Circular	2.07
	Rear MSE Wall	Long Term	Circular	1.52
B-128-0-13	Forward MSE Wall	Short Term	Circular	2.20
	Forward MSE Wall	Long Term	Circular	1.77

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2.0 INTRODUCTION

This report has been prepared for the HAN-75-14.39 project which calls for design and construction of the Bridge No. HAN-68-1668 over US 68 Ramp A 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-68-1668 which will carry the US 68 Ramp C vehicular traffic over US 68 Ramp A and Norfolk Southern Railroad. The design information provided by PB personnel indicates that the proposed bridge will be two (2) spans with an approximate total length of 280 feet. The proposed superstructures will be continuous plate girders with reinforced concrete decking on abutments and piers. The sub-structure units will be supported on reinforced concrete integral abutments on capped piles and cap and column piers on spread footings. Retaining walls consisting of Mechanically Stabilized Earth (MSE) Wall System will be used to retain the abutment fill at both rear and forward abutments of this bridge. This bridge is to be designed based on HL-93 loading criteria and the ODOT Bridge Design Manual, issued in 2007 which includes 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.

PROJECT: HAN-75-14.39

BRIDGE NO. HAN-68-1668 OVER US 68 RAMP A & NORFOLK SOUTHERN RAILROAD

SITE LOCATION MAP (FIGURE 2.1)

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 seven (7) test borings in the vicinity of proposed Bridge No. HAN-68-1668 over US 68 Ramp A & NS Railroad and MSE Walls for structural foundation design purposes. These structural test borings for the bridges and MSE Walls were to be advanced to approximate depths ranging from 25.0 feet to 30.0 feet below the existing ground surface and existing Ramp IR 75 SB to US 68 SB pavement shoulder, and included obtaining 5 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 775 feet to 795 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 10 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 the 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 no buildings located within an approximate distance of 1000 feet of the bridge site. The existing Ramps IR-75 SB to US 68 SB and US 68 NB to IR-75 SB run through site. The ramp pavement generally appeared to be in fair condition with light to moderate longitudinal and traverse cracks observed. Tall cattail wetland vegetation in what appear to be wetland areas was observed along the east side of the rear abutment and in areas along the NS Railroad tracks. This site is covered with grass, dense small bushes and few trees and is relatively flat. Standing water was observed along the railroad tracks after several days of rain.

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 IR 75 mainline bridges over Ramps US 68 NB to IR-75 SB and IR 75 SB to US 68 SB which are located approximately 500 feet east of the proposed bridge location. A total of three (3) historic test borings were advanced in the vicinity of the existing bridge. This historic geotechnical exploration performed in December 1984 consists of structure foundation exploration sheets. 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 June, July and August 2013. A total of six (6) bridge test borings identified as B-126-0-13, B-128-0-13, B-129-0-13, B-130-0-13, B-131-0-13, and B-131-1-13 were advanced for bridge and MSE Wall foundations design purposes. Test borings B-130-0-13, B-131-0-13, and B-131-1-13 were advanced in the vicinity of the proposed rear abutment and MSE wall while test borings B-126-0-13 and B-128-0-13 were advanced in the vicinity of the proposed forward abutment and MSE wall. Proposed test boring B-127-0-13 was located in a slight depression in the vicinity of the proposed MSE wall. This test boring could not be advanced due to standing water, more than 1 foot deep that was encountered at the boring location during our fieldwork. Test boring B-129-0-13 was advanced in the vicinity of the proposed bridge pier. These structural test borings were advanced to approximate depths ranging from 14.0 to 29.0 feet below the existing ground surface.

The test borings were marked in the field by PGI based on boring location plans developed by PGI 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 90 and Diedrich 50 drill rigs 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 barrels, water method. All test borings were monitored for the presence of groundwater during and upon completion of drilling operations. All test borings were backfilled with compacted soil cuttings and/or bentonite mix 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 soil tests were performed on selected soil samples for the purpose of soil classification and for analysis of engineering characteristics. These tests consisted of Particle-Size Analysis, Liquid and Plastic Limit, 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.

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 test borings B-126-0-13, B-128-0-13, B-129-0-13, B-130-0-13, B-131-0-13, and B-131-1-13. All test borings with the exception of B-130-0-13 were advanced through topsoil ranging in thickness from 1 inch to 12 inches and averaging 7.6 inches thick. Test boring B-130-0-13 was advance through silt and clay (A-6a), however topsoil was not observed at this boring location. The subsurface soils encountered in the test borings consisted primarily of natural soils, however fill material was encountered above natural soils in test borings B-131-0-13 and B-131-1-13 to depths of 8.5 feet and 3.5 feet, respectively. The fill material consisted of sandy silt (A-4a), silt and clay (A-6a), and clay (A-7-6). Natural soils encountered above bedrock in the test borings consisted of both cohesive and non-cohesive soils. Cohesive soils consisted of sandy silt (A-4a), silt and clay (A-6a), and silty clay (A-6b) and non-cohesive soils consisted of non-plastic/granular stone fragments with sand (A-1-b), non-plastic sandy silt (A-4a), and non-plastic silt (A-4b). Bedrock was encountered in all test boring locations at approximate depths ranging from 5.5 feet to 13.5 feet and averaging 7.3 feet below the existing ground surface.

The laboratory test results indicated that the moisture contents of the tested cohesive soil samples ranged from 7% to 24% and the consistency ranged from "medium stiff" to "very stiff", but was generally “stiff”. The moisture contents of the tested non-cohesive soils ranged from 19% to 29% and the relative density ranged from “dense” to “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 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, severely to slightly weathered, and strong. Bedding within the dolomite was generally very thin to medium 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 core specimens ranged from 10,625 psi in test boring B-128-0-13 to 25,379 psi in test boring B-131-0-13 which characterizes them as "strong" to "very strong", respectively.

The Rock Quality Designation (RQD) for the core samples ranged from 0% to 63% and averaged 29% based on individual runs and a weighted average of 40%. 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 48 to 60 and is classified as "Fair Rock" to "Good 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	Top of Bedrock Elevations (ft)	Rock Core Run No.	Rock Core Run Elevations (ft)	Length of Core Run (ft)	Recovery (%)	RQD (%)
B-126-0-13	770.8	NX-1	769.8	6.5	100	10
		NX-2	763.3	5.0	100	43
B-128-0-13	772.2	NX-1	770.2	2.0	100	30
		NX-2	768.2	4.5	100	37
		NX-3	763.7	10.0	100	60
		NX-4	753.7	5.0	100	6
B-129-0-13	771.0	NX-1	770.5	1.0	96	0
		NX-2	769.5	9.0	69	33
		NX-3	760.5	2.0	100	38
		NX-4	758.5	8.0	83	50
B-130-0-13	771.7	NX-1	770.2	1.2	97	0

Boring Number	Top of Bedrock Elevations (ft)	Rock Core Run No.	Rock Core Run Elevations (ft)	Length of Core Run (ft)	Recovery (%)	RQD (%)
		NX-2	769.0	1.1	98	0
		NX-3	767.9	1.6	100	0
		NX-4	766.3	4.1	100	63
B-131-0-13	770.8	NX-1	770.3	10.0	100	60
B-131-1-13	772.0	NX-1	770.2	10.0	100	30
		NX-2	760.2	5.0	95	63

Elevations were provided by PB personnel

Table 5.2.2 –Compressive Strength Test Results of Rock Core Specimens

Boring No.	Specimen Depth (ft)	Rock Type	Unit Weight (pcf)	Compressive Strength (psi)
B-126-0-13	18.9	Dolomite	168.03	17,355
B-128-0-13	20.2	Dolomite	166.12	10,625
B-129-0-13	23.8	Dolomite	171.17	12,696
B-130-0-13	13.4	Dolomite	167.49	13,284
B-131-0-13	20.7	Dolomite	170.33	25,379
B-131-1-13	19.5	Dolomite	164.60	14,151

5.3 Groundwater Conditions

Groundwater was not encountered during drilling operations prior to coring bedrock in any of the test borings advanced during our field work. Groundwater levels were not recorded upon completion of rock coring 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 U.S. Route 68 (US 68) Ramp C Bridge No. HAN-68-1668 over US 68 Ramp A and Norfolk Southern Railroad. Site plans provided by PB personnel indicates that the bridge abutment above the MSE wall embankment will be supported on piles at the rear and forward abutment locations and will be supported on spread footing at the pier location. Elevations of the bottom of the proposed MSE Walls at the rear and forward abutment locations will be 779.6 and 774.6 feet, respectively and elevation of the bottom of the spread footing at the proposed pier location will be 770.7 feet. The foundation recommendations for bridge and MSE Walls are provided in accordance with the ODOT *Bridge Design Manual* issued in 2007 using AASHTO *LRFD Bridge Design Specifications, 6th Edition.*

6.1 Bridge Foundation Systems

Soil and rock information obtained from structural test borings B-126-0-13, B-128-0-13, B-129-0-13, B-130-0-13, B-131-0-13, and B-131-1-13 was used to provide foundation recommendations for the proposed bridge abutments. Structural test borings B-130-0-13, B-131-0-13 and B-131-1-13 were advanced in the vicinity of the proposed rear abutment while structural test borings B-126-0-13 and B-128-0-13 were advanced in the vicinity of the proposed forward abutment. Structural test boring B-129-0-13 was advanced in the vicinity of proposed pier. As outlined in Section 5.1 - "Subsurface Soil Conditions", the top of bedrock was encountered at a depth of 7.6 feet below the bottom of the rear MSE Wall in test boring B-131-0-13 and 2.4 feet below the bottom of the forward MSE Wall in test boring B-128-0-13. Bedrock was encountered at a depth of 6.5 feet below the existing ground surface in test boring B-129-0-13. Bedrock at these boring locations consists of dolomite and was encountered to termination depth in all four test borings. The Rock Mass Rating obtained for the bedrock core samples according to LRFD Table 10.4.6.4-1 varied from 50 to 62 and is considered as “Fair Rock” to “Good Rock”. Since bedrock was encountered at relatively shallow depths below the bottom of the proposed MSE Walls at the proposed abutments and below the existing ground at the proposed pier locations, the proposed superstructure loads may be transferred to the underlying bedrock by means of shallow foundations.

Pier: Shallow foundation system consisting of spread footing may be used to transfer the loads to the underlying bedrock at the proposed pier location. Bearing resistance for spread footing on bedrock was evaluated as per AASHTO Article 10.6.3.2.2 (semi-empirical method) at the test boring B-129-0-13 location. The nominal bearing resistance analysis spreadsheet is included in Appendix B. Table 6.1.1 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 should be applied 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 6.1.1 – Estimated Design Parameters at Strength Limit State for Spread Footings

Boring No.	Substructure Location	Top of Bedrock Elevation (feet)	Proposed Bearing Elevation (feet)	Factored Bearing Resistance (ksf)
B-129-0-13	Pier	771.0±	770.0	35.0

A presumptive nominal bearing resistance of 30 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 location 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 calculation is shown on the nominal bearing resistance analysis spreadsheet included in Appendix B. Since the proposed spread footing 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 footings supported piers may experience sliding caused by lateral loads. Therefore 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.

Abutments: According to ODOT *Bridge Design Manual* Section 204.4, MSE Wall supported abutments should be supported on piles regardless of the proximity of bedrock to the MSE Wall foundation. Therefore the proposed superstructure loads at the abutment locations should be transferred to the underlying bedrock by means of 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. 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 or HP-12X53 may be selected for the abutment locations depending on the lateral capacity required. The total factored load on each HP-10X42 pile and HP-12X53 pile should not exceed the corresponding maximum structural resistance of 310 kips and 380 kips, respectively 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 6.1.2. The pile cut-off elevations at the abutments were extracted from the final structure site plan provided by PB personnel.

Table 6.1.2 - 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
B-131-0-13	779.6	812.4	764.6	50	H-Pile	10X42	310 kips
B-131-0-13	779.6	812.4	764.6	50	H-Pile	12X53	380 kips
B-128-0-13	774.6	802.7	759.6	45	H-Pile	10X42	310 kips
B-128-0-13	774.6	802.7	759.6	45	H-Pile	12X53	380 kips

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 inch, respectively. It is recommended that the piles be spaced a minimum of three (3) pile diameters on center. Since piles are extended into bedrock, group effects of the piles can be neglected. Pile sections above the bedrock should be encased in corrugated pipe filled with granular material to eliminate any down drag on this portion of the piles and protect against construction operations. 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 6.1.2 should be adjusted based on the outcome of the lateral load analysis. Table 6.1.3 summarizes the weak rock parameters to perform lateral load analyses by PB personnel.

Table 6.1.3 - 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-131-0-13	770.8	0.095	200000	5000	60	0.0005
B-128-0-13	772.2	0.095	200000	5000	40	0.0005

If additional lateral resistance is required, bigger size piles should be considered at the rear and forward abutment locations. 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. 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.

6.2 MSE Wall Foundation Systems

Based on the site plan provided by PB personnel, the maximum height of the MSE Walls will be 45.1 feet and 40.3 feet at the rear and forward abutment locations, respectively. The foundation width of the MSE Walls at the rear and forward abutment locations will be 31.6 feet and 28.2 feet based upon a minimum strap length equal to 70% of the wall height. It is assumed that maximum applied bearing pressures at the Service Limit State will be 8000 psf and 7300 psf, respectively at the rear and forward MSE Walls. Soil and rock information obtained from test borings; B-130-0-13, B-131-0-13 and B-131-1-13 for the proposed rear MSE Wall and B-126-0-13 and B-128-0-13 for the proposed forward MSE Wall was used to provide foundation recommendations for the proposed MSE Walls. According to site plans provided by PB personnel, elevations of the bottom of the proposed MSE Walls at the rear and forward abutment locations will be 779.6 and 774.6 feet, respectively. As per the boring logs, bedrock was encountered at depths ranging from 7.6 feet to 8.8 feet below the bottom of the rear MSE Wall while bedrock was encountered at depths ranging from 2.4 feet to 3.8 feet below the bottom of the forward MSE Wall.

The foundation soils encountered below the bottom of the MSE Walls consisted of both fill and natural soils above bedrock and were generally cohesive in nature. The consistency of these cohesive soils ranged from “stiff to “very stiff” but was generally "stiff”. These cohesive soils encountered in all test boring locations will not support the applied loads from the MSE Walls. Therefore, PGI recommends performing ground improvement on the foundation soils at the rear and forward MSE Walls in the vicinity of these test boring locations. Ground improvements should be performed by removing the soils below the bottom of the MSE Walls and replacing it with compacted ODOT Item 203 Granular Material, Type C, in accordance with Supplemental Specification 840. However, according to recommendations provided by OGE, ground improvements should be performed by removing soils to the bedrock below the bottom of the MSE Walls and replacing it with compacted ODOT Item 304. Table 6.2.1 summarizes the proposed approximate excavation depths below the existing ground and approximate excavation depths below the bottom of the MSE Walls at each test boring location. The ground improvements must be performed in front of the wall and behind the reinforced zone. The removal in front of the wall and behind the reinforced zone should be extended a lateral distance equal to the depth of removal at these two points respectively. Any replacement or backfill material beyond 2 feet behind the reinforcing strips and above the bottom of the leveling pad should consist of Item 203 Embankment, not Item 840 Select Granular Backfill. The excavated foundation soil subgrade should be examined by competent geotechnical personnel. If any areas of low bearing capacity with excessive moisture (soft pockets) soils are encountered, they should be removed as directed by on site geotechnical personnel and replaced with ODOT Item 304.

Table 6.2.1 – Summary of Excavation Depths for Ground Improvements

Boring No.	MSE Wall Location	Existing Ground Elevation (feet)	Bottom of MSE Wall Elevation (feet)	Existing Bedrock Elevation (feet)	Excavation Depth Below Existing Ground (feet)	Excavation Depth Below MSE Wall (feet)
B-130-0-13	Rear	776.2	779.6	771.7	4.5	7.9
B-131-0-13	Rear	784.3	779.6	770.8	13.5	8.8
B-131-1-13	Rear	779.2	779.6	772.0	7.2	7.6
B-126-0-13	Forward	777.3	774.6	770.8	6.5	3.8
B-128-0-13	Forward	777.7	774.6	772.2	5.5	2.4

Bearing capacity analysis was performed by using effective stress shear strength parameters to estimate the nominal bearing resistance of the strip footings supported on 304 granular soils. Groundwater level was assumed to be at the base of the MSE Wall at the rear and forward abutment locations. Results of the bearing capacity analysis are attached in Appendix B. Factored bearing resistance corresponding to bearing elevation at the MSE Wall boring locations is summarized in Table 6.2.2. 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 6.2.2 – 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-130-0-13	Rear MSE Wall	4.0	31.6	779.6	10.6
B-128-0-13	Forward MSE Wall	3.2	28.2	774.6	9.2

External stability of the MSE Walls including sliding on the base, limiting eccentricity, and bearing resistance at the Strength Limit States and settlement analysis at the Service Limit States were performed at the rear and forward abutment locations. These external stability analyses were performed utilizing the MSEW Version 3.0, developed by Dov Leshchinsky, Ph.D., ADAMA Engineering. Global stability analyses of MSE Walls were also performed at the rear and forward abutment locations. 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. The uniform surcharge load due to traffic was assumed to be 250 psf. Abutment configuration at the rear and forward locations was obtained from the site plans for the global stability analysis. Computer output of the MSE Walls external stability analyses are included in Appendix B. Load and resistance factors used with respect to the various potential failure modes and limit states of the MSE Wall are shown in the computer output. The External Stability analyses results shows 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 and forward MSE Walls.

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 6.2.3 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. Slope analyses critical failure circles are included in Appendix B.

Table 6.2.3 –Summary of Critical Factors of Safety for MSE Walls

Boring No	Location	Stability	Method Used	Factor of Safety
B-131-0-13	Rear MSE Wall	Short Term	Circular	2.07
	Rear MSE Wall	Long Term	Circular	1.52
B-128-0-13	Forward MSE Wall	Short Term	Circular	2.20
	Forward MSE Wall	Long Term	Circular	1.77

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 abutment and MSE Walls, 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 proposed embankment subgrade fill soils. Therefore, the soil parameters derived from the actual fill soils should be used for pavement design. Representative samples of proposed borrow materials should be tested and CBR values should be derived prior to construction.

6.5 Groundwater Management

The groundwater level 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. 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. The tests should be performed by a qualified soil technician under the supervision of PGI or other geotechnical-engineering firm and in accordance with the appropriate ASTM procedures.

7.0 LIMITATIONS

This report is subject to the following conditions and limitations:

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

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

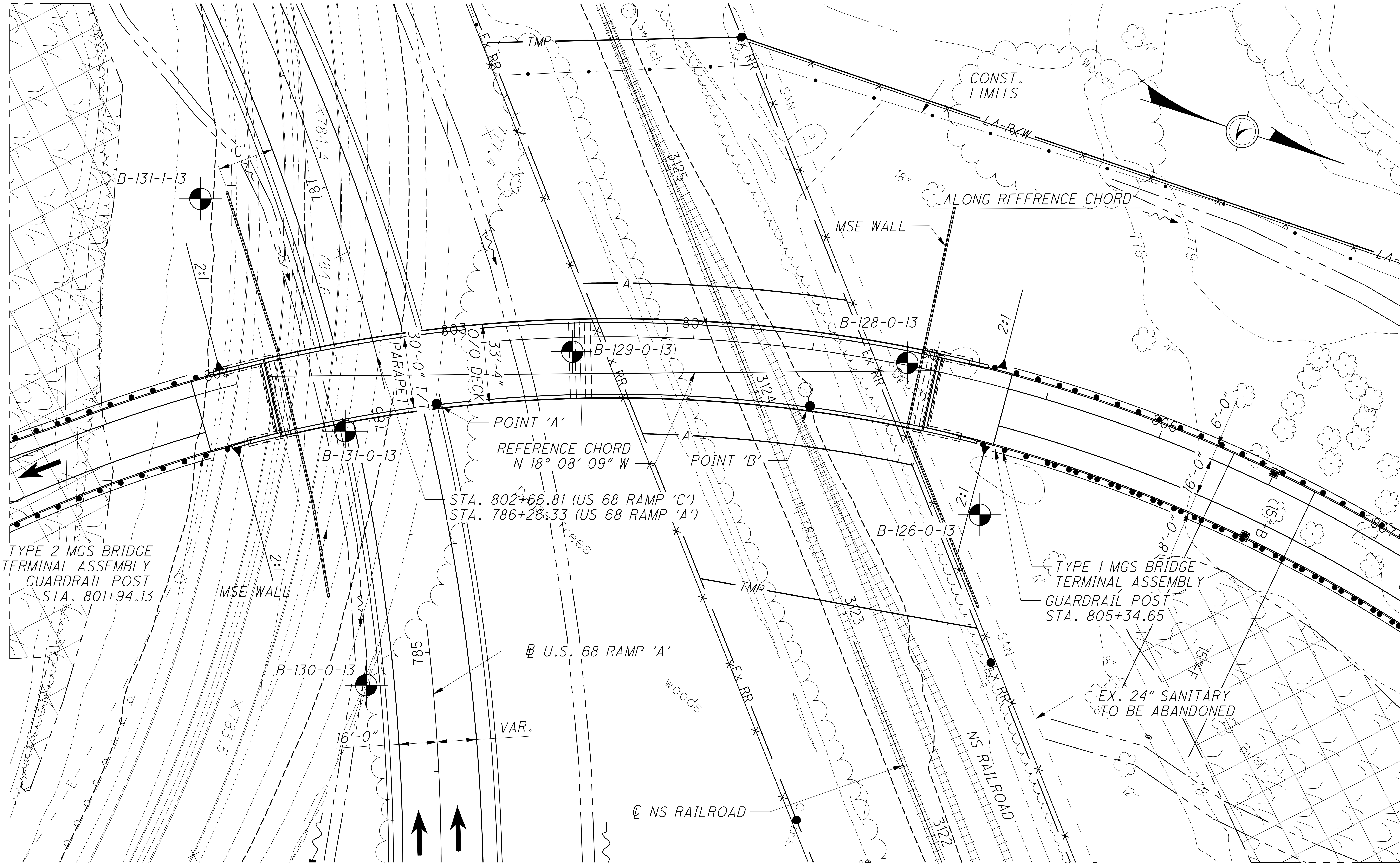
7.3 All earthwork and foundation construction must be performed under the supervision of a Professional Engineer in accordance with ODOT Construction Specifications.

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

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

APPENDICES

APPENDIX A



PLAN

Boring Location Plan
Bridge No. HAN-68-1668 over US 68 Ramp A and Norfolk Southern Railroad

HAN-75-14.39
PID No. 87005

PROJECT: HAN-75-14.39 TYPE: NEW BRIDGE PID: 87005 STR ID: HAN-68-1668 START: 7/19/13 END: 7/22/13	DRILLING FIRM / OPERATOR: B-M / JOSH DEAN SAMPLING FIRM / LOGGER: PGI / F.BUSHER DRILLING METHOD: 3.25" HSA SAMPLING METHOD: SPT/NX	DRILL RIG: DIEDRICH D-90 ATV HAMMER: DIEDRICH AUTOMATIC CALIBRATION DATE: 9/18/12 ENERGY RATIO (%): 80.2	STATION / OFFSET: 805+35.88, 54.18 RT ALIGNMENT: MSE WALL ELEVATION: 777.3 (MSL) EOB: 19.0 ft. COORD: 41.027319290, 83.676028700	EXPLORATION ID B-126-0-13 PAGE 1 OF 1

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
TOPSOIL	776.7																	
MEDIUM STIFF, MOTTLED BROWN AND GRAY, **CLAY**, SOME SAND, MOIST	773.8	1–3	2 3 4	9	67	SS-1	1.75	-	-	-	-	-	-	-	-	24	A-7-6 (V)	
LOOSE, BROWN, **NON-PLASTIC SILT**, LITTLE SAND, MOIST	771.3	4–6	3 3 4	9	67	SS-2	-	-	-	-	-	-	-	-	-	29	A-4b (V)	
MEDIUM DENSE, GRAY, **STONE FRAGMENTS WITH SAND AND SILT**, DAMP	770.8	TR	14 50/2"	-	50	SS-3	-	45	13	9	21	12	18	14	4	7	A-2-4 (0)	
GRAY DOLOMITE BEDROCK NOTE: AUGERED TO 7.5' AND BEGAN CORING BEDROCK	769.8	7																
DOLOMITE, LIGHT GRAY, HIGHLY TO MODERATELY WEATHERED, STRONG, VERY THIN TO THIN BEDDED, HIGHLY TO MODERATELY FRACTURED, TIGHT APERTURE WIDTH, SLIGHTLY ROUGH, FEW ANGULAR FRACTURES.	763.3	8–14	10		100	NX-1											CORE	
DOLOMITE, LIGHT GRAY, SLIGHTLY WEATHERED, STRONG, VERY THIN TO THIN BEDDED, HIGHLY TO MODERATELY FRACTURED, TIGHT APERTURE WIDTH, SLIGHTLY ROUGH, FEW ANGULAR FRACTURES. @17.9'; COMPRESSIVE STRENGTH = 17355 psi	758.3	15–19 EOB	43		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: BACKFILLED WITH SOIL CUTTINGS

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 6/16/14 13:34 - M:\PROJECT FILES\13 PROJECTS\G13011G HAN-75\LAB DATA SHEETS\BRIDGES\1668 RR FLYOVER.GPJ

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 6/16/14 13:34 - M:\PROJECT FILES\13 PROJECTS\G13011G HAN-75\LAB DATA SHEETS\BRIDGES\1668 RR FLYOVER.GPJ

PROJECT: HAN-75-14.39
TYPE: NEW BRIDGE
PID: 87005 STR ID: HAN-68-1668
START: 7/22/13 END: 7/22/13

DRILLING FIRM / OPERATOR: B-M / JOSH DEAN
SAMPLING FIRM / LOGGER: PGI / F.BUSHER
DRILLING METHOD: 3.25" HSA
SAMPLING METHOD: SPT/NX

DRILL RIG: DIEDRICH D-90 ATV
HAMMER: DIEDRICH AUTOMATIC
CALIBRATION DATE: 9/18/12
ENERGY RATIO (%): 80.2

STATION / OFFSET: 804+89.8, 1.16 LT
ALIGNMENT: ABUTMENT
ELEVATION: 777.7 (MSL) EOB: 29.0 ft.
COORD: 41.027184670, 83.676213590

EXPLORATION ID B-128-0-13
PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 777.7	DEPTHS	SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
TOPSOIL	777.0																	
STIFF, BROWN, **SILTY CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, MOIST @2.5'; PUSHED SHELBY TUBE	775.2	1–2	3 3 4	9	56	SS-1	2.50	-	-	-	-	-	-	-	-	22	A-6b (V)	
STIFF, BROWN AND GRAY, **SILT**, SOME CLAY, LITTLE SAND, MOIST	773.2	3–4			100	ST-2	4.00	-	-	-	-	-	-	-	-	22	A-4b (V)	
VERY STIFF, BROWN, **SANDY SILT**, SOME CLAY, SOME STONES FRAGMENTS, DAMP	772.2	5 (TR)	6 15 56	95	67	SS-3	2.25	21	7	12	34	26	25	17	8	14	A-4a (5)	
GRAY DOLOMITE BEDROCK NOTE: AUGERED TO 7.5' AND BEGAN CORING BEDROCK	770.2	6–7																
DOLOMITE, LIGHT GRAY, HIGHLY TO MODERATELY WEATHERED, STRONG, VERY THIN TO THIN BEDDED, HIGHLY FRACTURED TO FRACTURED, TIGHT APERTURE WIDTH, SLIGHTLY ROUGH, FEW VERTICAL FRACTURES.	768.2	8–9	30		100	NX-1											CORE	
DOLOMITE, LIGHT GRAY, MODERATELY TO SLIGHTLY WEATHERED, STRONG, VERY THIN TO THIN BEDDED, HIGHLY TO MODERATELY FRACTURED, TIGHT APERTURE WIDTH, SLIGHTLY ROUGH.	763.7	10–13	37		100	NX-2											CORE	
DOLOMITE, LIGHT GRAY, MODERATELY TO SLIGHTLY WEATHERED, STRONG, VERY THIN TO THIN BEDDED, HIGHLY FRACTURED TO MODERATELY FRACTURED, TIGHT APERTURE WIDTH, SLIGHTLY ROUGH, FEW ANGULAR FRACTURES. @20.2'; COMPRESSIVE STRENGTH = 10625 psi	753.7	14–23	60		100	NX-3											CORE	
DOLOMITE, LIGHT GRAY, MODERATELY TO SLIGHTLY WEATHERED, STRONG, VERY THIN TO THIN BEDDED, HIGHLY FRACTURED TO MODERATELY FRACTURED, TIGHT APERTURE WIDTH, SLIGHTLY ROUGH, FEW VERTICAL FRACTURES.	748.7	24–29 (EOB)	6		100	NX-4											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: BACKFILLED WITH SOIL CUTTINGS

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 6/16/14 13:34 - M:\PROJECT FILES\13 PROJECTS\G13011G HAN-75\LAB DATA SHEETS\BRIDGES\1668 RR FLYOVER.GPJ

PROJECT: HAN-75-14.39
TYPE: NEW BRIDGE
PID: 87005 STR ID: HAN-68-1668
START: 7/17/13 END: 7/17/13

DRILLING FIRM / OPERATOR: B-M / JOSH DEAN
SAMPLING FIRM / LOGGER: PGI / F.BUSHER
DRILLING METHOD: 3.25" HSA
SAMPLING METHOD: SPT/NX

DRILL RIG: DIEDRICH D-90 ATV
HAMMER: DIEDRICH AUTOMATIC
CALIBRATION DATE: 9/18/12
ENERGY RATIO (%): 80.2

STATION / OFFSET: 803+49.16, 5.62 RT
ALIGNMENT: PIER
ELEVATION: 777.5 (MSL) EOB: 27.0 ft.
COORD: 41.026816080, 83.676070100

EXPLORATION ID B-129-0-13

PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 777.5	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
TOPSOIL	776.7																	
MEDIUM STIFF, BROWN, MOTTLED GRAY, **SILTY CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, MOIST	774.0	1–3	2 2 3	7	67	SS-1	2.75	-	-	-	-	-	-	-	-	20	A-6b (V)	
STIFF, BROWN AND GRAY TO GRAY, **SILT AND CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, MOIST		4–5	4 4 4	11	100	SS-2	4.00	-	-	-	-	-	-	-	-	21	A-6a (V)	
@6.0'; GRAY	771.0	6	4 100/2"	-	63	SS-3	2.50	-	-	-	-	-	-	-	-	20	A-6a (V)	
GRAY DOLOMITE BEDROCK	770.5	TR 7																
DOLOMITE, LIGHT GRAY, MODERATELY TO SLIGHTLY WEATHERED, STRONG, VERY THIN TO THIN BEDDED, HIGHLY FRACTURED TO FRACTURED, TIGHT APERTURE WIDTH, SLIGHTLY ROUGH, FEW VERTICAL FRACTURES.	769.5	7–8	0		96	NX-1											CORE	
DOLOMITE, LIGHT GRAY, SLIGHTLY TO MODERATELY WEATHERED, STRONG, VERY THIN TO THIN BEDDED, HIGHLY TO MODERATELY FRACTURED, TIGHT APERTURE WIDTH, SLIGHTLY ROUGH, FEW ANGULAR AND VERTICAL FRACTURES.	760.5	8–17	33		69	NX-2											CORE	
DOLOMITE, LIGHT GRAY, SLIGHTLY WEATHERED, STRONG, VERY THIN TO THIN BEDDED, HIGHLY FRACTURED TO FRACTURED, TIGHT APERTURE WIDTH, SLIGHTLY ROUGH.	758.5	17–19	38		100	NX-3											CORE	
DOLOMITE, LIGHT GRAY, SLIGHTLY WEATHERED, STRONG, VERY THIN TO THIN BEDDED, HIGHLY TO MODERATELY FRACTURED, TIGHT APERTURE WIDTH, SLIGHTLY ROUGH, FEW ANGULAR FRACTURES. @23.8'; COMPRESSIVE STRENGTH = 12696 psi	750.5	19–27 EOB	50		83	NX-4											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

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 6/16/14 13:34 - M:\PROJECT FILES\13 PROJECTS\G13011G HAN-75\LAB DATA SHEETS\BRIDGES\1668 RR FLYOVER.GPJ

PROJECT: HAN-75-14.39	DRILLING FIRM / OPERATOR: B-M / JOSH DEAN	DRILL RIG: DIEDRICH D-90 ATV	STATION / OFFSET: 802+33.69, 135.11 RT
TYPE: NEW BRIDGE	SAMPLING FIRM / LOGGER: PGI / F.BUSHER	HAMMER: DIEDRICH AUTOMATIC	ALIGNMENT: MSE WALL
PID: 87005 STR ID: HAN-68-1668	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 9/18/12	ELEVATION: 776.2 (MSL) EOB: 14.0 ft.
START: 7/18/13 END: 7/18/13	SAMPLING METHOD: SPT/NX	ENERGY RATIO (%): 80.2	COORD: 41.026712010, 83.675488970

EXPLORATION ID B-130-0-13

PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 776.2	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
STIFF, BROWN, **SILT AND CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, DAMP	772.7		3 / 3 / 4	9	100	SS-1	2.25	-	-	-	-	-	-	-	-	16	A-6a (V)	
MEDIUM DENSE, BROWN, **NON-PLASTIC SILT**, TRACE SAND, MOIST	771.7	TR	4 / 6 / 48	72	78	SS-2	-	-	-	-	-	-	-	-	-	19	A-4b (V)	
GRAY DOLOMITE BEDROCK NOTE: AUGERED TO 6.0' AND BEGAN CORING BEDROCK	770.2																	
DOLOMITE, LIGHT GRAY, MODERATELY TO SLIGHTLY WEATHERED, STRONG, VERY THIN TO THIN BEDDED, HIGHLY FRACTURED TO FRACTURED, TIGHT APERTURE WIDTH, SLIGHTLY ROUGH, FEW VERTICAL FRACTURES.	769.0		0		97	NX-1											CORE	
	767.9		0		98	NX-2											CORE	
DOLOMITE, LIGHT GRAY, MODERATELY TO SLIGHTLY WEATHERED, STRONG, VERY THIN TO THIN BEDDED, HIGHLY FRACTURED TO FRACTURED, TIGHT APERTURE WIDTH, SLIGHTLY ROUGH, FEW ANGULAR FRACTURES.	766.3		0		100	NX-3											CORE	
DOLOMITE, LIGHT GRAY, SLIGHTLY WEATHERED, STRONG, VERY THIN TO THIN BEDDED, HIGHLY FRACTURED TO FRACTURED, TIGHT APERTURE WIDTH, SLIGHTLY ROUGH, FEW ANGULAR FRACTURES.			63		100	NX-4											CORE	
DOLOMITE, LIGHT GRAY, SLIGHTLY WEATHERED, STRONG, VERY THIN TO THIN BEDDED, FRACTURED TO MODERATELY FRACTURED, TIGHT APERTURE WIDTH, SLIGHTLY ROUGH, FEW ANGULAR FRACTURES. @13.4'; COMPRESSIVE STRENGTH = 13284 psi	762.2	EOB 14																

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: NEW BRIDGE
PID: 87005 STR ID: HAN-68-1668
START: 8/16/13 END: 8/16/13

DRILLING FIRM / OPERATOR: B-M / JOSH DEAN
SAMPLING FIRM / LOGGER: PGI / W. NAJJAR
DRILLING METHOD: 3.25" HSA
SAMPLING METHOD: SPT/NX

DRILL RIG: DIEDRICH D-90 ATV
HAMMER: DIEDRICH AUTOMATIC
CALIBRATION DATE: 9/18/12
ENERGY RATIO (%): 80.2

STATION / OFFSET: 802+47.81, 28.54 RT
ALIGNMENT: ABUTMENT
ELEVATION: 784.3 (MSL) EOB: 24.0 ft.
COORD: 41.026596180, 83.675846370

EXPLORATION ID B-131-0-13

MATERIAL DESCRIPTION AND NOTES	ELEV. 784.3	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
TOPSOIL (1" THICK)	784.2																	
VERY STIFF, BROWN, **SILT AND CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, FILL, DAMP		1–2	3-6-6	16	89	SS-1	4.5+	-	-	-	-	-	-	-	-	13	A-6a (V)	
		4–5	2-8-8	21	83	SS-2	4.5+	-	-	-	-	-	-	-	-	12	A-6a (V)	
	778.3																	
VERY STIFF, BROWN, **SANDY SILT**, SOME CLAY, TRACE STONE FRAGMENTS, FILL, DAMP		6–7	7-8-9	23	44	SS-3	4.5+	-	-	-	-	-	-	-	-	11	A-4a (V)	
	775.8																	
STIFF, BROWN AND GRAY, **SILTY CLAY**, LITTLE SAND, MOIST		9–10	4-5-6	15	100	SS-4	3.00	0	2	17	39	42	37	20	17	21	A-6b (11)	
	773.3																	
STIFF, BROWN AND GRAY, **SILT AND CLAY**, LITTLE SAND, LITTLE STONE FRAGMENTS, DAMP		11–12	4-4-5	12	83	SS-5	3.25	-	-	-	-	-	-	-	-	16	A-6a (V)	
@13.5'; NO SPLIT SPOON RECOVERY	770.8	TR	50/5"	-	0	SS-6	-	-	-	-	-	-	-	-	-	-		
POSSIBLE DOLOMITE BEDROCK	770.3																	
DOLOMITE, GRAY, SLIGHTLY TO MODERATELY WEATHERED, STRONG, THIN BEDDED, HIGHLY TO MODERATELY FRACTURED, TIGHT APERTURE WIDTH, SLIGHTLY ROUGH, FEW ANGULAR AND VERTICAL FRACTURES.		14–24	60		100	NX-1											CORE	
@20.7'; COMPRESSIVE STRENGTH = 25379 psi																		
	760.3	EOB 24																

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

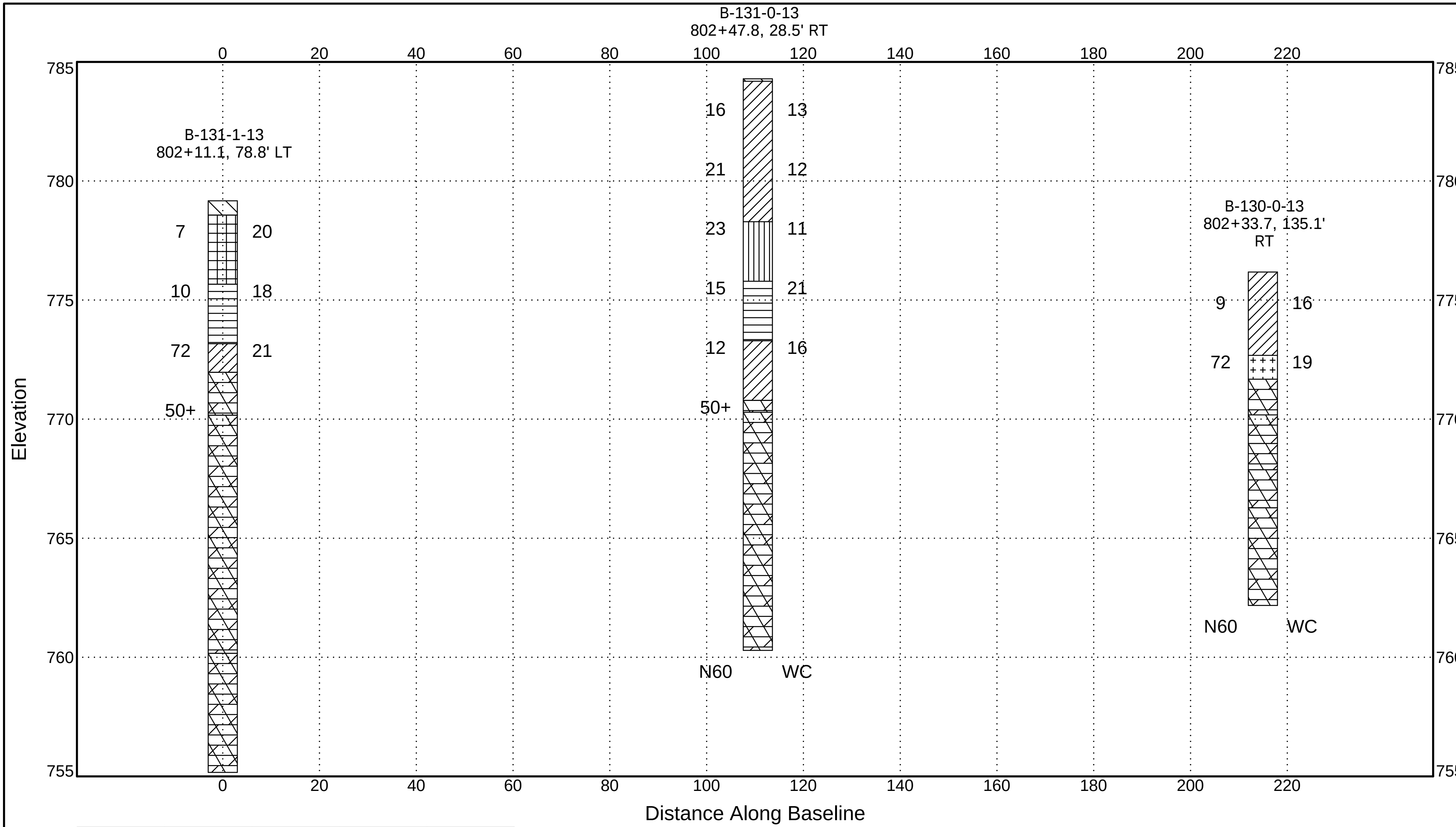
STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 6/16/14 13:34 - M:\PROJECT FILES\13 PROJECTS\G13011G HAN-75\LAB DATA SHEETS\BRIDGES\1668 RR FLYOVER.GPJ

PROJECT: HAN-75-14.39	DRILLING FIRM / OPERATOR: OTB / JOHN	DRILL RIG: DIEDRICH D-50 ATV	STATION / OFFSET: 802+11.12, 78.84 LT	EXPLORATION ID B-131-1-13
TYPE: NEW BRIDGE	SAMPLING FIRM / LOGGER: PGI / F.BUSHER	HAMMER: DIEDRICH AUTOMATIC	ALIGNMENT: MSE WALL	
PID: 87005 STR ID: HAN-68-1668	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 12/10/11	ELEVATION: 779.2 (MSL) EOB: 24.0 ft.	PAGE 1 OF 1
START: 6/25/13 END: 6/25/13	SAMPLING METHOD: SPT/NX	ENERGY RATIO (%): 81.7	COORD: 41.026355560, 83.676109910	

MATERIAL DESCRIPTION AND NOTES	ELEV. 779.2	DEPTHS	SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
TOPSOIL	778.6	0.6																
MEDIUM STIFF, DARK GRAY, **CLAY**, TRACE SAND WITH ASPHALT PIECES, FILL, DAMP	775.7	1–3.5	2 3 2	7	39	SS-1	3.50	-	-	-	-	-	-	-	-	20	A-7-6 (V)	
STIFF, MOTTLED BROWN AND GRAY, **SILTY CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, MOIST	773.2	3.5–6	2 3 4	10	44	SS-2	3.00	-	-	-	-	-	-	-	-	18	A-6b (V)	
VERY STIFF, BROWN AND GRAY, **SILT AND CLAY**, TRACE SAND, TRACE STONE FRAGMENTS, MOIST	772.0	6–7.2 (TR)	7 30 23	72	56	SS-3	2.75	1	1	4	54	40	33	22	11	21	A-6a (8)	
GRAY DOLOMITE BEDROCK @8.5': NO SPLIT SPOON RECOVERY NOTE: AUGERED TO 9.0' AND BEGAN CORING BEDROCK	770.2	7.2–9					-	-	-	-	-	-	-	-	-	-	Rock (V)	
		8.5	50/3"	-	0	SS-4	-	-	-	-	-	-	-	-	-	-		
DOLOMITE, LIGHT GRAY, SLIGHTLY TO MODERATELY WEATHERED, STRONG, VERY THIN TO THIN BEDDED, HIGHLY TO MODERATELY FRACTURED, TIGHT APERTURE WIDTH, SLIGHTLY ROUGH, FEW ANGULAR AND VERTICAL FRACTURES.	760.2	9–19	30		100	NX-1											CORE	
DOLOMITE, LIGHT GRAY, SLIGHTLY WEATHERED, STRONG, VERY THIN TO THIN BEDDED, HIGHLY TO MODERATELY FRACTURED, TIGHT APERTURE WIDTH, SLIGHTLY ROUGH. @19.5'; COMPRESSIVE STRENGTH = 14151 psi	755.2	19–24 (EOB)	63		95	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



Borehole	North	East	Elev.	Depth
B-130-0-13	497681	1644186	776.2	14.0
B-131-0-13	497640	1644087	784.3	24.0
B-131-1-13	497554	1644013	779.2	24.0

DISTANCES:
Beginning 0
Ending 220
VIEWING ANGLES (degrees):
Horizontal 0.0
Vertical 0.0

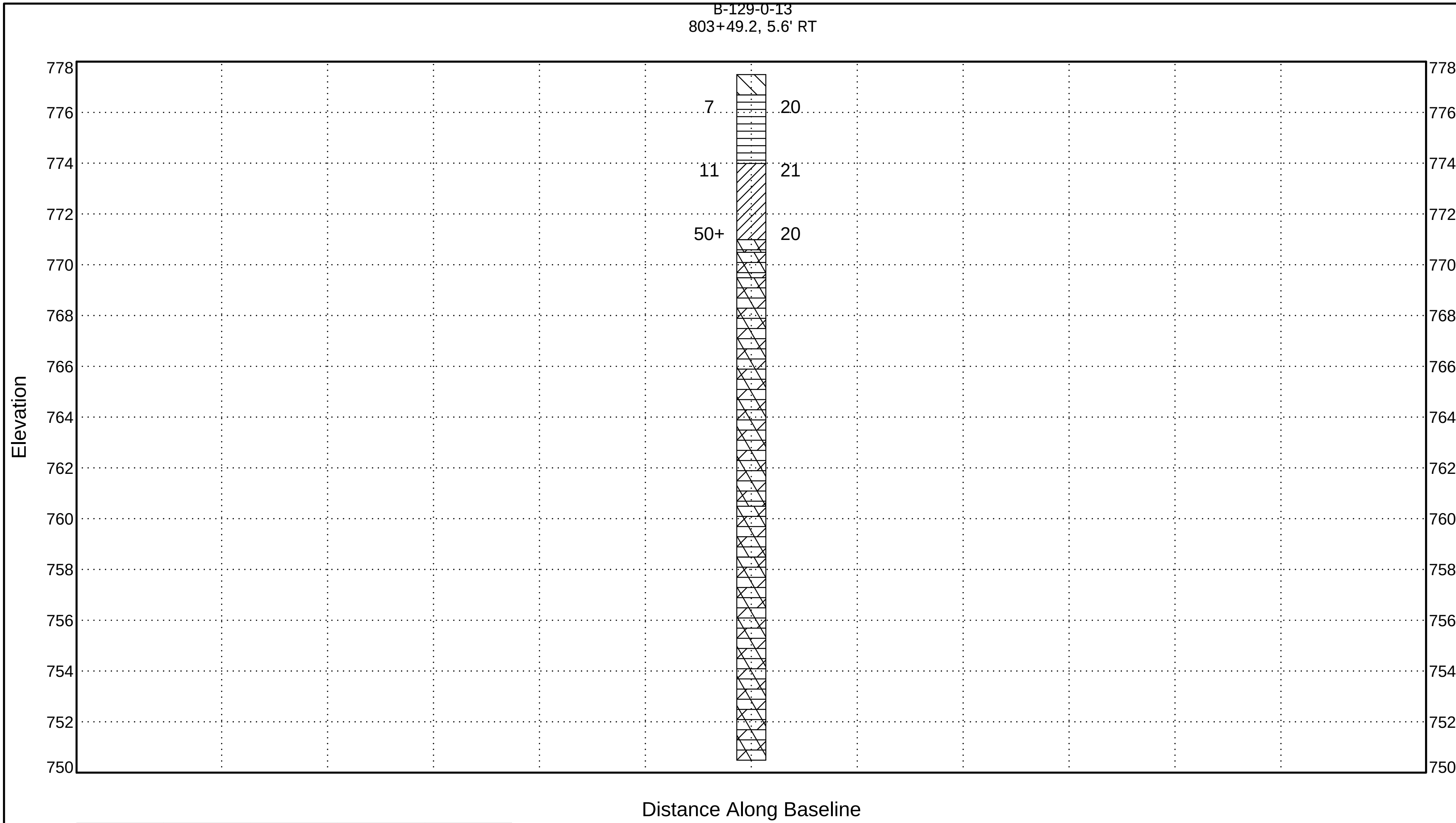
Position	North	East
Left, Front	497562	1644007
Right, Front	497690	1644186
Left, Back	497562	1644007
Right, Back	497690	1644186

SOIL BORINGS PROFILE
BRIDGE REAR ABUTMENT AND MSE WALL

HAN-75-14.39-BRIDGE NO. HAN-68-1668

FINDLAY, HANCOCK COUNTY, OHIO

PROJECT #	DATE	PLATE
87005	Mar 14	1



Borehole	North	East	Elev.	Depth
B-129-0-13	497721	1644026	777.5	27.0

DISTANCES:
Beginning 0
Ending 2
VIEWING ANGLES (degrees):
Horizontal 0.0
Vertical 0.0

Position	North	East
Left, Front		
Right, Front		
Left, Back		
Right, Back		

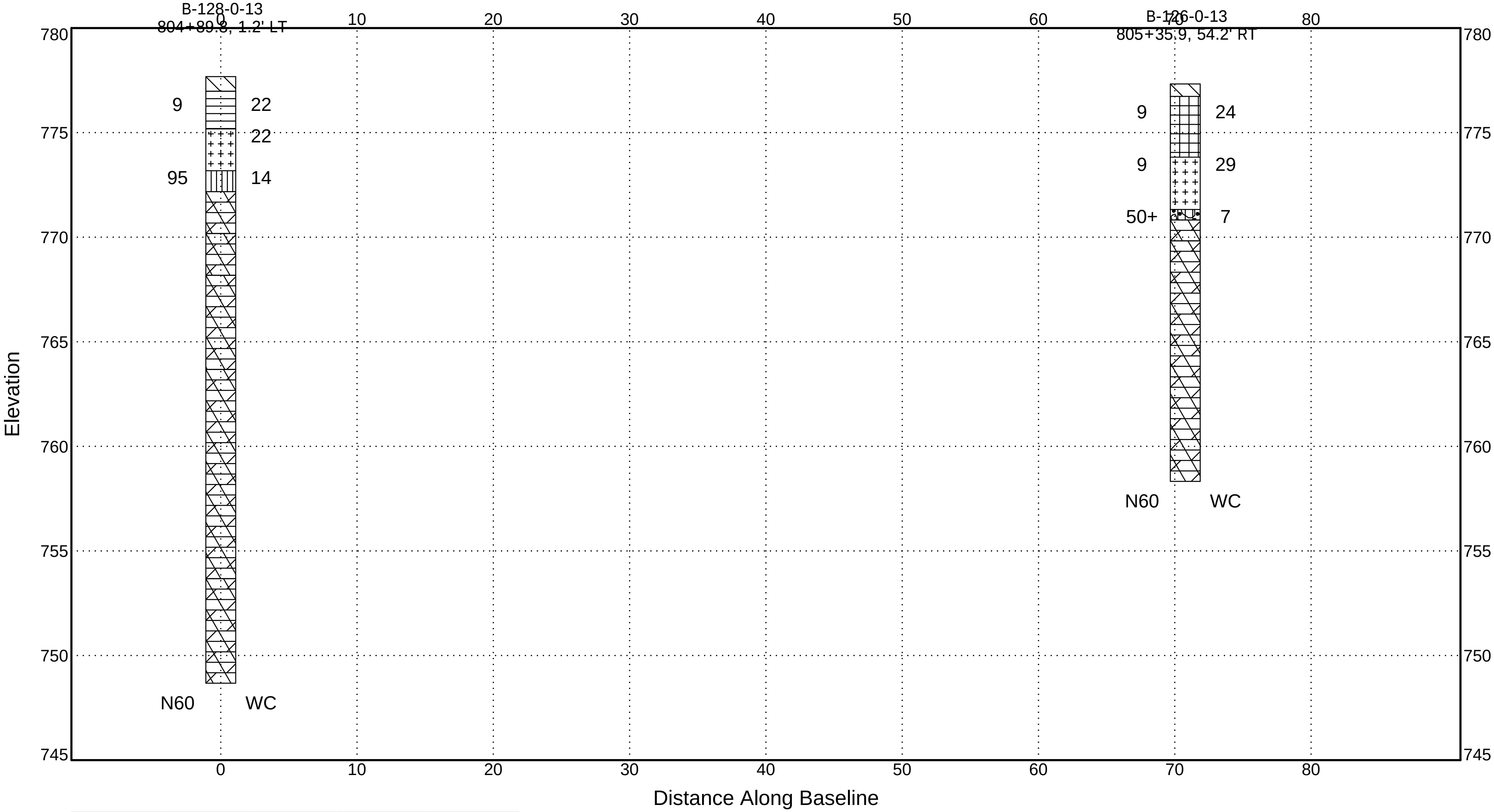
SOIL BORINGS PROFILE
BRIDGE PIER

HAN-75-14.39-BRIDGE NO. HAN-68-1668

FINDLAY, HANCOCK COUNTY, OHIO

PROJECT #	DATE	PLATE
87005	Mar 14	1

PROFILE ODOT-PRIMENG.GDT-3/15/14 06:58-\\CLEDC01\PUBLIC\PROJECT FILES\13 PROJECTS\G13011G HAN-75\LAB DATA SHEETS\BRIDGES\1668 RR FLYOVER.GPJ



Borehole	North	East	Elev.	Depth
B-126-0-13	497905	1644040	777.3	19.0
B-128-0-13	497856	1643989	777.7	29.0

DISTANCES:
Beginning 0
Ending 80
VIEWING ANGLES (degrees):
Horizontal 0.0
Vertical 0.0

Position	North	East
Left, Front	497856	1643989
Right, Front	497911	1644047
Left, Back	497856	1643989
Right, Back	497911	1644047

SOIL BORINGS PROFILE
FORWARD ABUTMENT AND MSE WALL

HAN-75-14.39-Bridge No. HAN-68-1668

FINDLAY, HANCOCK COUNTY, OHIO

PROJECT #	DATE	PLATE
87005	Mar 14	1

PROFILE ODOT-PRIMENG.GDT-3/15/14 06:50-\\CLEDC01\PUBLIC\PROJECT FILES\13 PROJECTS\G13011G HAN-75\LAB DATA SHEETS\BRIDGES\1668 RR FLYOVER.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-126-0-13	SS-1	1.0	24											BROWN, MOTTLED GRAY, CLAY, SOME SAND	A-7-6 (V)
B-126-0-13	SS-2	3.5	29											BROWN, NON-PLASTIC SILT, LITTLE SAND	A-4b (V)
B-126-0-13	SS-3	6.0	7	18	14	4		45	13	9	21	33	12	GRAY STONE FRAGMENTS WITH SAND AND SILT	A-2-4 (0)
B-128-0-13	SS-1	1.0	22											BROWN SILTY CLAY, LITTLE SAND, TRACE STONE FRAGMENTS W/TOPSOIL	A-6b (V)
B-128-0-13	ST-2	2.5	22											BROWN AND GRAY SILT, SOME CLAY, LITTLE SAND	A-4b (V)
B-128-0-13	SS-3	4.5	14	25	17	8		21	7	12	35	60	26	BROWN SANDY SILT, SOME CLAY, SOME STONE FRAGMENTS	A-4a (5)
B-129-0-13	SS-1	1.0	20											MOTTLED BROWN AND GRAY SILTY CLAY, LITTLE SAND, TRACE STONE FRAGMENTS	A-6b (V)
B-129-0-13	SS-2	3.5	21											BROWN AND GRAY SILT AND CLAY, LITTLE SAND, TRACE STONE FRAGMENTS	A-6a (V)
B-129-0-13	SS-3	6.0	20											GRAY SILT AND CLAY, LITTLE SAND, TRACE STONE FRAGMENTS	A-6a (V)
B-130-0-13	SS-1	1.0	16											BROWN SILT AND CLAY, LITTLE SAND, TRACE STONE FRAGMENTS	A-6a (V)
B-130-0-13	SS-2	3.5	19											BROWN, NON-PLASTIC SILT, TRACE SAND	A-4b (V)
B-131-0-13	SS-1	1.0	13											BROWN SILT AND CLAY, LITTLE SAND, TRACE STONE FRAGMENTS (FILL)	A-6a (V)
B-131-0-13	SS-2	3.5	12											BROWN SILT AND CLAY, LITTLE SAND, TRACE STONE FRAGMENTS (FILL)	A-6a (V)
B-131-0-13	SS-3	6.0	11											BROWN SANDY SILT, SOME CLAY, TRACE STONE FRAGMENTS (FILL)	A-4a (V)
B-131-0-13	SS-4	8.5	21	37	20	17		0	2	17	39	81	42	BROWN AND GRAY SILTY CLAY, LITTLE SAND	A-6b (11)
B-131-0-13	SS-5	11.0	16											BROWN AND GRAY SILT AND CLAY, LITTLE SAND, LITTLE STONE FRAGMENTS	A-6a (V)
B-131-0-13	SS-6	13.5												NO RECOVERY	
B-131-1-13	SS-1	1.0	20											DARK GRAY CLAY, TRACE SAND WITH ASPHALT PIECES (FILL)	A-7-6 (V)
B-131-1-13	SS-2	3.5	18											BROWN, MOTTLED GRAY SILTY CLAY, LITTLE SAND, TRACE STONE FRAGMENTS	A-6b (V)
B-131-1-13	SS-3A	6.0	21	33	22	11		1	1	4	54	94	40	BROWN AND GRAY SILT AND CLAY, TRACE SAND, TRACE STONE FRAGMENTS	A-6a (8)
B-131-1-13	SS-4	8.5													

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-68-1668
Location: FINDLAY, HANCOCK COUNTY, OHIO
PID Number: 87005

Pro Geotech, Inc.

Compressive Strength of Rock
ASTM D 7012

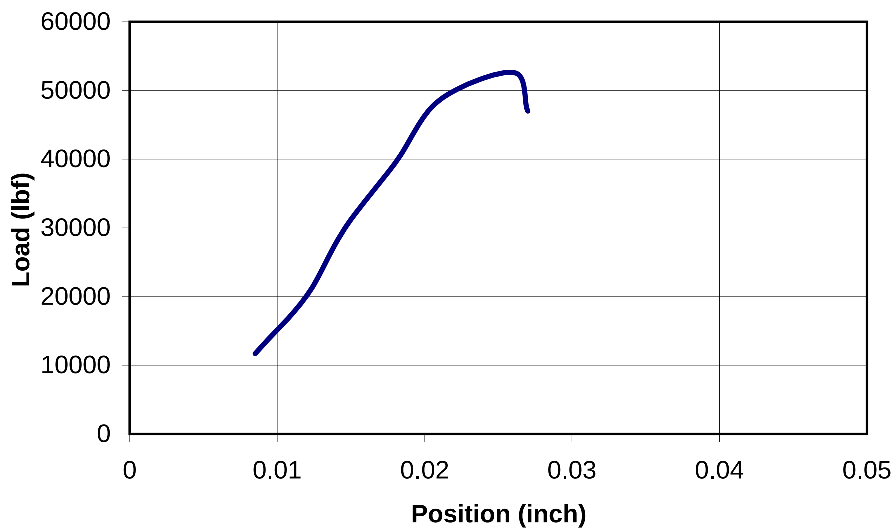
PROJECT	HAN-75-14.39	PGI PROJECT NO.	G13011G	DATE	9/16/2013

BORING NUMBER	B-126-0-13	TOP DEPTH (FT)	17.9	BOTTOM DEPTH (FT)	18.2
SAMPLE NUMBER	NX-2	DISTRICT	1	PID NO.	87005
COUNTY	HANCOCK	ROUTE	US 68	SECTION	1668
STATION	805+35.9	OFFSET	54.2	OFFSET DIRECTION	RT

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

MEASUREMENT	LENGTH (INCH)	DIAMETER (INCH)		
1	4.005	1.960	LENGTH/DIAMETER	2.04
2	4.011	1.964	CORRECTION FACTOR	1.00
3	4.009	1.971	AREA (SQ. INCH)	3.033
			MASS (GRAMS)	536.15
AVERAGE	4.008	1.965	UNIT WEIGHT (LBS/FT^3)	168.03

MAXIMUM LOAD (LBS)	52632
COMPRESSIVE STRENGTH (PSI)	17355
TIME OF TEST (MINUTES)	2:40
LOADING DIRECTION	PERPENDICULAR TO BEDDING
TECHNICIAN	F. BUSHER



BEFORE TESTING

AFTER FAILURE

Pro Geotech, Inc.

Compressive Strength of Rock
ASTM D 7012

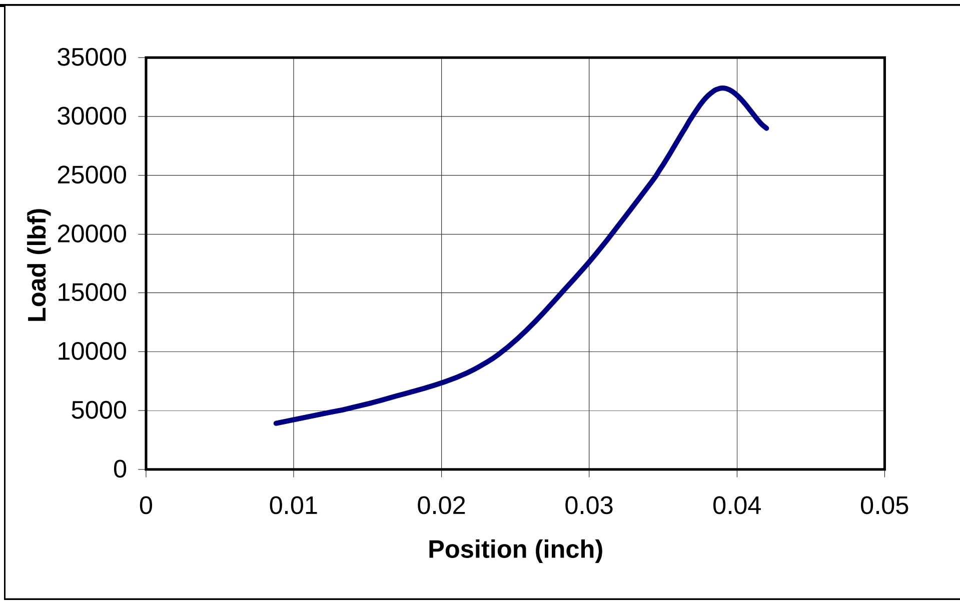
PROJECT	HAN-75-14.39	PGI PROJECT NO.	G13011G	DATE	9/16/2013

BORING NUMBER	B-128-0-13	TOP DEPTH (FT)	20.2	BOTTOM DEPTH (FT)	20.5
SAMPLE NUMBER	NX-1	DISTRICT	1	PID NO.	87005
COUNTY	HANCOCK	ROUTE	US 68	SECTION	1668
STATION	804+89.8	OFFSET	1.16	OFFSET DIRECTION	LT

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

MEASUREMENT	LENGTH (INCH)	DIAMETER (INCH)		LENGTH/DIAMETER	2.04
1	4.011	1.965		CORRECTION FACTOR	1.00
2	4.017	1.958		AREA (SQ. INCH)	3.038
3	4.019	1.977		MASS (GRAMS)	531.94
AVERAGE	4.016	1.967		UNIT WEIGHT (LBS/FT^3)	166.12

MAXIMUM LOAD (LBS)	32276
COMPRESSIVE STRENGTH (PSI)	10625
TIME OF TEST (MINUTES)	3.32
LOADING DIRECTION	PERPENDICULAR TO BEDDING
TECHNICIAN	F. BUSHER



BEFORE TESTING

AFTER FAILURE

Pro Geotech, Inc.

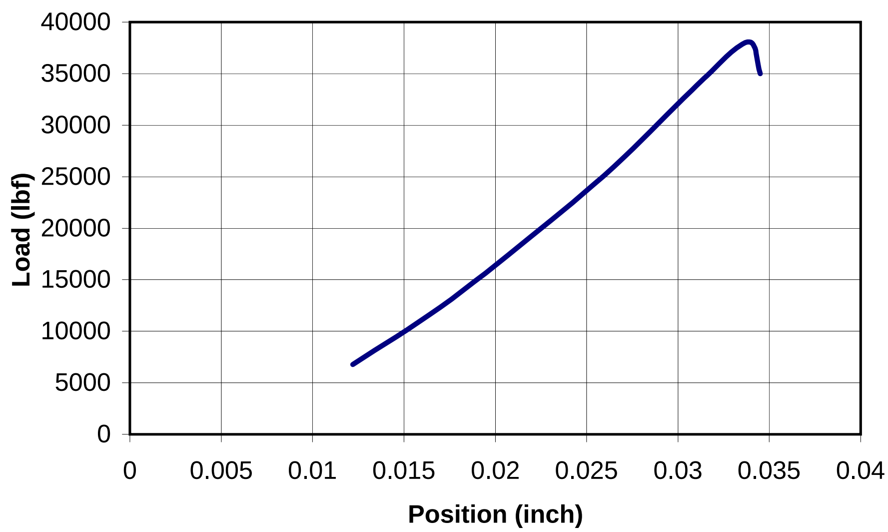
Compressive Strength of Rock
ASTM D 7012

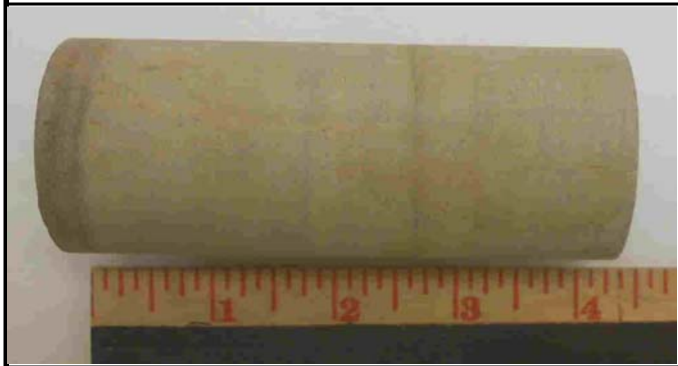
PROJECT	HAN-75-14.39	PGI PROJECT NO.	G13011G	DATE	9/6/2013
	STRUCTURE				
BORING NUMBER	B-129-0-13	TOP DEPTH (FT)	23.8	BOTTOM DEPTH (FT)	24.1
SAMPLE NUMBER	NX-3	DISTRICT	1	PID NO.	87005
COUNTY	HANCOCK	ROUTE	US 68	SECTION	1668
STATION	803+49.16	OFFSET	5.62	OFFSET DIRECTION	RT

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

MEASUREMENT	LENGTH (INCH)	DIAMETER (INCH)		
1	4.053	1.955	LENGTH/DIAMETER	2.07
2	4.055	1.950	CORRECTION FACTOR	1.00
3	4.055	1.957	AREA (SQ. INCH)	2.999
			MASS (GRAMS)	546.28
AVERAGE	4.054	1.954	UNIT WEIGHT (LBS/FT^3)	171.17

MAXIMUM LOAD (LBS)	38071
COMPRESSIVE STRENGTH (PSI)	12696
TIME OF TEST (MINUTES)	3.32
LOADING DIRECTION	PERPENDICULAR TO BEDDING
TECHNICIAN	F. BUSHER





BEFORE TESTING

AFTER FAILURE

Pro Geotech, Inc.

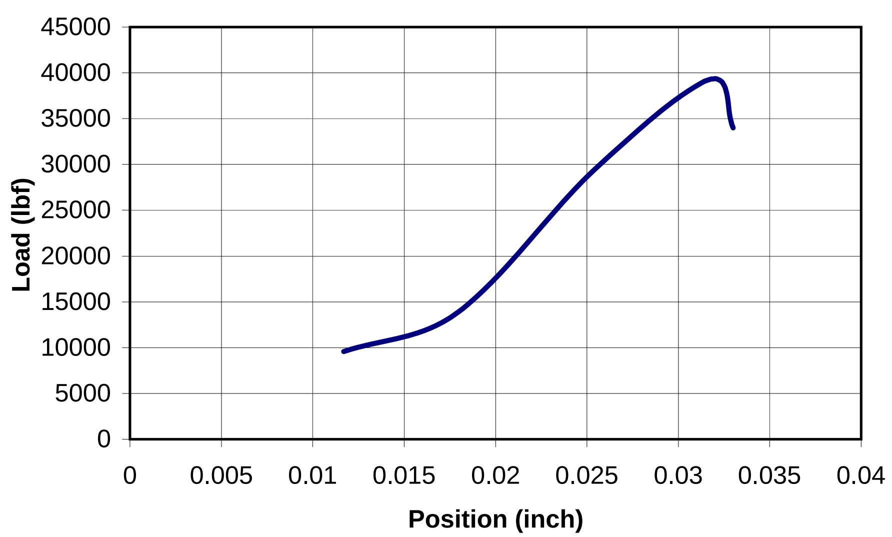
Compressive Strength of Rock
ASTM D 7012

PROJECT	HAN-75-14.39	PGI PROJECT NO.	G13011G	DATE	9/30/2013
	STRUCTURE	US 68 Ramp D Bridge No. HAN-68-1668 over Lima Ave Ramp A			
BORING NUMBER	B-130-0-13	TOP DEPTH (FT)	13.4	BOTTOM DEPTH (FT)	13.7
SAMPLE NUMBER	NX-1	DISTRICT	1	PID NO.	87005
COUNTY	HANCOCK	ROUTE	US 68	SECTION	1668
STATION	802+33.7	OFFSET	135.1	OFFSET DIRECTION	RT

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

MEASUREMENT	LENGTH (INCH)	DIAMETER (INCH)		LENGTH/DIAMETER	2.26
1	4.362	1.942		CORRECTION FACTOR	1.00
2	4.358	1.939		AREA (SQ. INCH)	2.957
3	4.460	1.940		MASS (GRAMS)	571.16
AVERAGE	4.393	1.940		UNIT WEIGHT (LBS/FT^3)	167.49

MAXIMUM LOAD (LBS)	39281
COMPRESSIVE STRENGTH (PSI)	13284
TIME OF TEST (MINUTES)	3:10
LOADING DIRECTION	PERPENDICULAR TO BEDDING
TECHNICIAN	F. BUSHER



BEFORE TESTING

AFTER FAILURE

Pro Geotech, Inc.

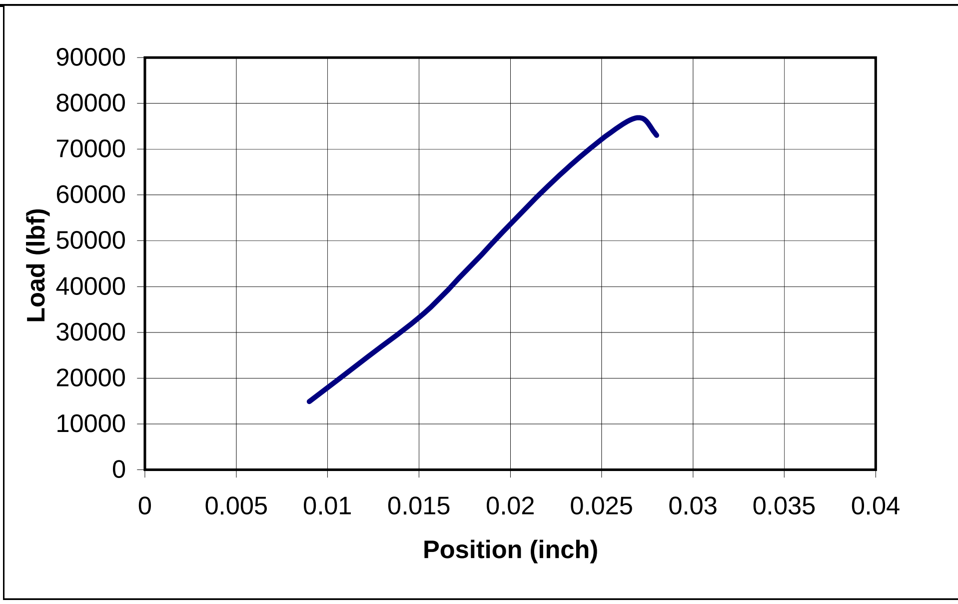
Compressive Strength of Rock
ASTM D 7012

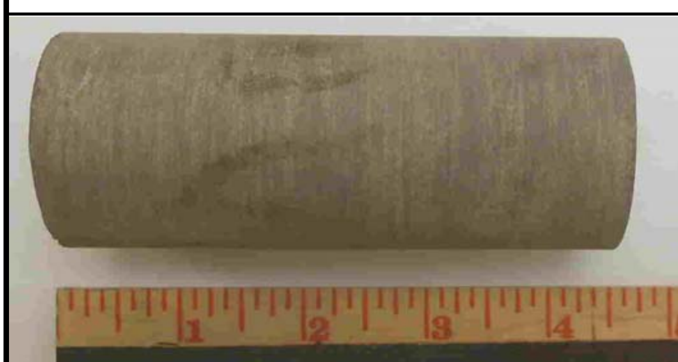
PROJECT	HAN-75-14.39	PGI PROJECT NO.	G13011G	DATE	9/30/2013
	STRUCTURE	US 68 Ramp D Bridge No. HAN-68-1668 over Lima Ave Ramp A			
BORING NUMBER	B-131-0-13	TOP DEPTH (FT)	20.7	BOTTOM DEPTH (FT)	21.0
SAMPLE NUMBER	NX-1	DISTRICT	1	PID NO.	87005
COUNTY	HANCOCK	ROUTE	US 68	SECTION	1668
STATION	802+47.81	OFFSET	28.54	OFFSET DIRECTION	RT

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

MEASUREMENT	LENGTH (INCH)	DIAMETER (INCH)		
1	4.029	1.959	LENGTH/DIAMETER	2.05
2	4.010	1.964	CORRECTION FACTOR	1.00
3	4.016	1.967	AREA (SQ. INCH)	3.027
			MASS (GRAMS)	543.92
AVERAGE	4.018	1.963	UNIT WEIGHT (LBS/FT^3)	170.33

MAXIMUM LOAD (LBS)	76835
COMPRESSIVE STRENGTH (PSI)	25379
TIME OF TEST (MINUTES)	3:10
LOADING DIRECTION	PERPENDICULAR TO BEDDING
TECHNICIAN	F. BUSHER





BEFORE TESTING

AFTER FAILURE

Pro Geotech, Inc.

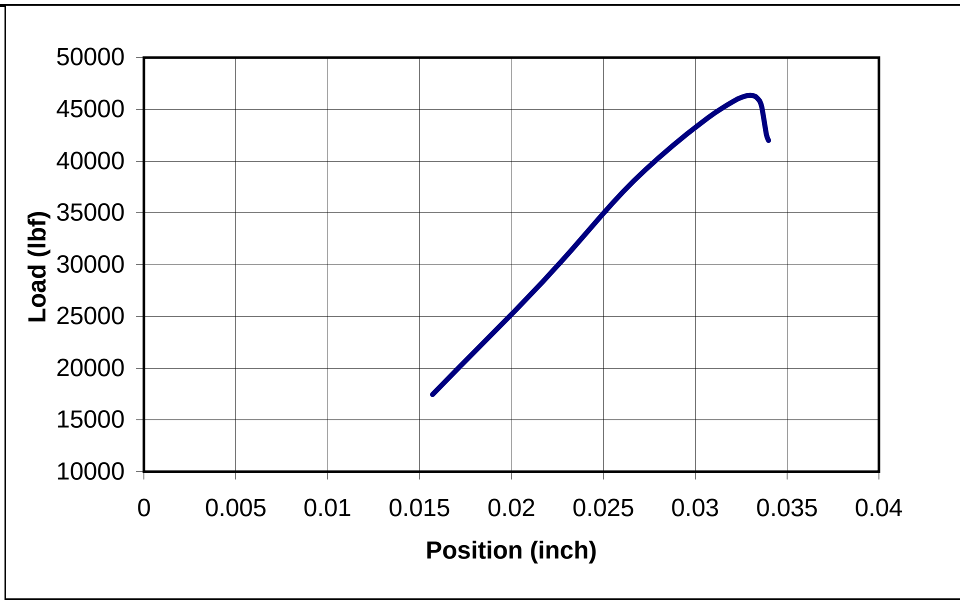
Compressive Strength of Rock
ASTM D 7012

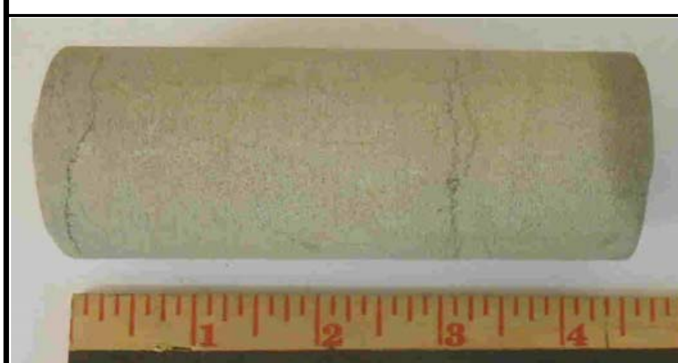
PROJECT	HAN-75-14.39	PGI PROJECT NO.	G13011G	DATE	9/27/2013
	STRUCTURE	US 68 Ramp D Bridge No. HAN-68-1668 over Lima Ave Ramp A			
BORING NUMBER	B-131-1-13	TOP DEPTH (FT)	19.5	BOTTOM DEPTH (FT)	19.8
SAMPLE NUMBER	NX-2	DISTRICT	1	PID NO.	87005
COUNTY	HANCOCK	ROUTE	US 68	SECTION	1668
STATION	802+11.12	OFFSET	78.8'	OFFSET DIRECTION	LT

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

MEASUREMENT	LENGTH (INCH)	DIAMETER (INCH)		LENGTH/DIAMETER	2.03
1	4.150	2.039		CORRECTION FACTOR	1.00
2	4.139	2.045		AREA (SQ. INCH)	3.273
3	4.140	2.040		MASS (GRAMS)	585.86
AVERAGE	4.143	2.041		UNIT WEIGHT (LBS/FT^3)	164.60

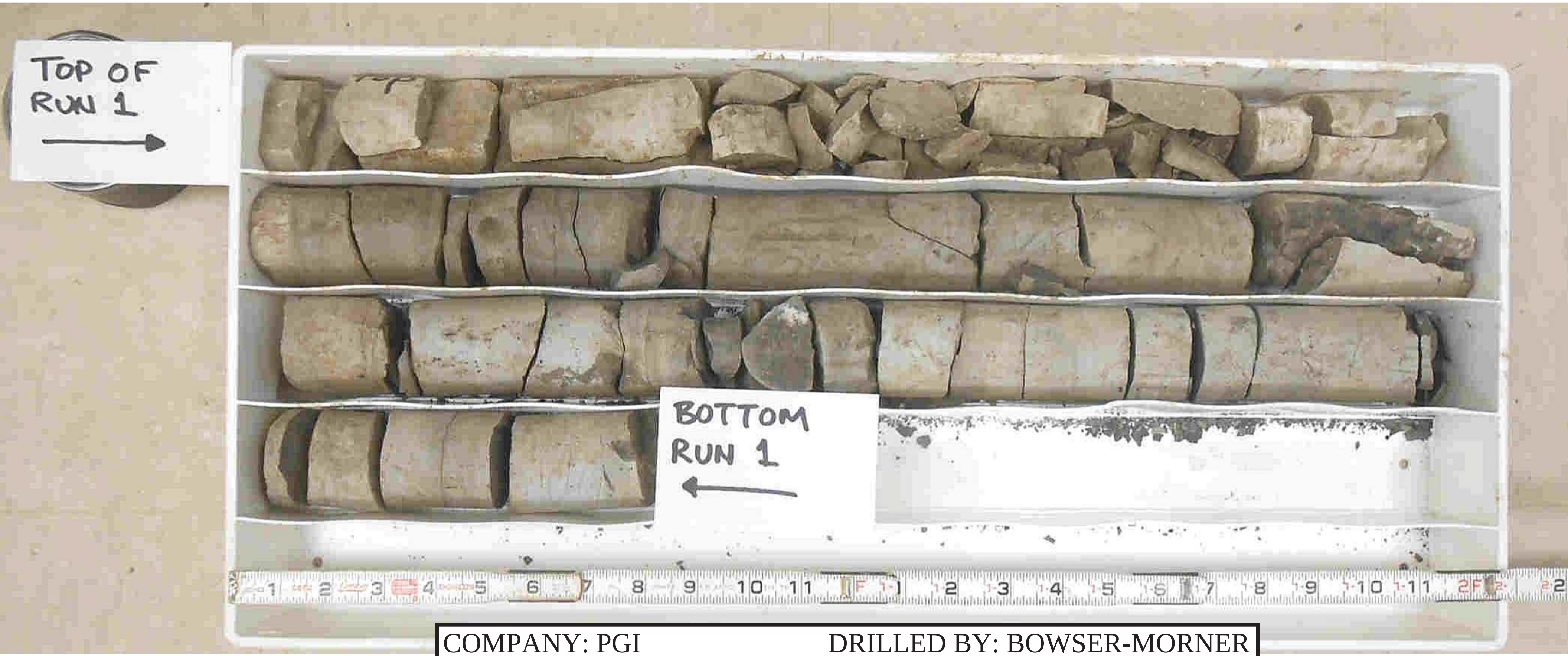
MAXIMUM LOAD (LBS)	46313
COMPRESSIVE STRENGTH (PSI)	14151
TIME OF TEST (MINUTES)	2:40
LOADING DIRECTION	PERPENDICULAR TO BEDDING
TECHNICIAN	F. BUSHER





BEFORE TESTING

AFTER FAILURE



COMPANY: PGI DRILLED BY: BOWSER-MORNER
PROJECT: HAN-75-14.39
BRIDGE NO.: HAN-68-1668 over US 68 RAMP A & NS RAILROAD
BORING: B-126-0-13 BOX 1/2
DATE of CORING: 7/22/13
RUN-1: 7.5' - 14.0'
REC: 100% RQD: 10%

COMPANY: PGI DRILLED BY: BOWSER-MORNER
PROJECT: HAN-75-14.39
BRIDGE NO.: HAN-68-1668 over US 68 RAMP A & NS RAILROAD
BORING: B-126-0-13 BOX 2/2
DATE of CORING: 7/22/13
RUN-2: 14.0' - 19.0'
REC: 100% RQD: 43%

COMPANY: PGI DRILLED BY: BOWSER-MORNER
PROJECT: HAN-75-14.39
BRIDGE NO.: HAN-68-1668 over US 68 RAMP A & NS RAILROAD
BORING: B-128-0-13 BOX 1/3
DATE of CORING: 7/22/13
RUN-1: 7.5' - 9.5' RUN-2: 9.5' - 14.0'
REC: 100% RQD: 30% REC: 100% RQD: 37%

COMPANY: PGI	DRILLED BY: BOWSER-MORNER

PROJECT: HAN-75-14.39
BRIDGE NO.: HAN-68-1668 over US 68 RAMP A & NS RAILROAD
BORING: B-128-0-13 BOX 2/3
DATE of CORING: 7/22/13
RUN-3: 14.0' - 24.0'
REC: 100% RQD: 60%

COMPANY: PGI DRILLED BY: BOWSER-MORNER
PROJECT: HAN-75-14.39
BRIDGE NO.: HAN-68-1668 over US 68 RAMP A & NS RAILROAD
BORING: B-128-0-13 BOX 3/3
DATE of CORING: 7/22/13
RUN-4: 24.0' - 29.0'
REC: 100% RQD: 6%

COMPANY: PGI DRILLED BY: BOWSER-MORNER
PROJECT: HAN-75-14.39
BRIDGE NO.: HAN-68-1668 over US 68 RAMP A & NS RAILROAD
BORING: B-129-0-13 BOX 1/2
DATE of CORING: 7/17/13

RUN-1: 7.0' - 8.0'	RUN-2: 8.0' -17.0'
REC: 96% RQD: 0%	REC: 69% RQD: 33%

COMPANY: PGI DRILLED BY: BOWSER-MORNER
PROJECT: HAN-75-14.39
BRIDGE NO.: HAN-68-1668 over US 68 RAMP A & NS RAILROAD
BORING: B-129-0-13 BOX 2/2
DATE of CORING: 7/17/13
RUN-3 17.0' - 19.0' RUN-4: 19.0' - 27.0'
REC: 100% RQD: 38% REC: 83% RQD: 50%

COMPANY: PGI DRILLED BY: BOWSER-MORNER
PROJECT: HAN-75-14.39
BRIDGE NO.: HAN-68-1668 over US 68 RAMP A & NS RAILROAD
BORING: B-130-0-13 BOX 1/1
DATE of CORING: 7/18/13

RUN-1: 6.0' - 7.2'	RUN-2: 7.2' - 8.3'
REC: 97% RQD: 0%	REC: 98% RQD: 0%
RUN -3: 8.3' - 9.9'	RUN -4: 9.9' - 14.0'
REC: 100% RQD: 0%	REC: 100% RQD: 63%

COMPANY: PGI DRILLED BY: BOWSER-MORNER
PROJECT: HAN-75-14.39
BRIDGE NO.: HAN-68-1668 over US 68 RAMP A & NS RAILROAD
BORING: B-131-0-13 BOX 1/1
DATE of CORING: 8/16/13
RUN-1: 14.0' - 24.0'
REC: 100% RQD: 60%

COMPANY: PGI DRILLED BY: OHIO TESTBOR
PROJECT: HAN-75-14.39
BRIDGE NO.: HAN-68-1668 over US 68 RAMP A & NS RAILROAD
BORING: B-131-1-13 BOX 1/2
DATE of CORING: 6/25/13
RUN-1: 9.0' - 19.0'
REC: 100% RQD: 30%

COMPANY: PGI DRILLED BY: OHIO TESTBOR
PROJECT: HAN-75-14.39
BRIDGE NO.: HAN-68-1668 over US 68 RAMP A & NS RAILROAD
BORING: B-131-1-13 BOX 2/2
DATE of CORING: 6/25/13
RUN-1: 19.0' - 24.0'
REC: 100% RQD: 63%

ROCK MASS RATING From Table 10.4.6.4-1	
Project: HAN-75-14.39	**Project No.:** G13011G
Structure: Bridge No. HAN-68-1668 over NS Railroad and US 68 Ramp A	
Boring No.: B-126-0-13	**Substructure Unit:** FW AB MSE Wall
Strength of Intact Rock Material	
Uniaxial Compressive Strength	2499 ksf
Relative Rating	8
Drill Core Quality RQD	
RQD	24%
Relative Rating	4
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	19
Groundwater Conditions	
Relative Rating	10
Strike & Dip Orientation of Joint	
Relative Rating	0
Total Mass Rating	49
Class No	III
Description	Fair Rock
Boring No.: B-128-0-13	**Substructure Unit:** Forward Abutment
Strength of Intact Rock Material	
Uniaxial Compressive Strength	1530 ksf
Relative Rating	5
Drill Core Quality RQD	
RQD	40%
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	19
Groundwater Conditions	
Relative Rating	10
Strike & Dip Orientation of Joint	
Relative Rating	0
Total Mass Rating	48
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-68-1668 over NS Railroad and US 68 Ramp A

Boring No.: B-129-0-13 **Substructure Unit:** Pier

Strength of Intact Rock Material	
Uniaxial Compressive Strength	1828 ksf
Relative Rating	5
Drill Core Quality RQD	
RQD	39%
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	19
Groundwater Conditions	
Relative Rating	10
Strike & Dip Orientation of Joint	
Relative Rating	0
Total Mass Rating	48
Class No	III
Description	Fair Rock

Boring No.: B-131-0-13 **Substructure Unit:** Rear Abutment

Strength of Intact Rock Material	
Uniaxial Compressive Strength	3655 ksf
Relative Rating	10
Drill Core Quality RQD	
RQD	60%
Relative Rating	13
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	19
Groundwater Conditions	
Relative Rating	10
Strike & Dip Orientation of Joint	
Relative Rating	0
Total Mass Rating	60
Class No	II
Description	Good Rock

ROCK MASS RATING From Table 10.4.6.4-1

Project: HAN-75-14.39 **Project No.:** G13011G

Structure: Bridge No. HAN-68-1668 over NS Railroad and US 68 Ramp A

Boring No.: B-131-1-13 **Substructure Unit:** Rear AB MSE Wall

Strength of Intact Rock Material	
Uniaxial Compressive Strength	2037 ksf
Relative Rating	7
Drill Core Quality RQD	
RQD	41%
Relative Rating	7
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	19
Groundwater Conditions	
Relative Rating	10
Strike & Dip Orientation of Joint	
Relative Rating	0
Total Mass Rating	51
Class No	III
Description	Fair Rock

Boring No.: B-130-0-13 **Substructure Unit:** Rear AB MSE Wall

Strength of Intact Rock Material	
Uniaxial Compressive Strength	1913 ksf
Relative Rating	7
Drill Core Quality RQD	
RQD	32%
Relative Rating	7
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	19
Groundwater Conditions	
Relative Rating	10
Strike & Dip Orientation of Joint	
Relative Rating	0
Total Mass Rating	51
Class No	III
Description	Fair Rock

Bearing Resistance and Settlement Analyses of Footing on Jointed Rock

Project:	HAN-75-14.39-HAN-68-1668	**Project No.:**	G13011G
Boring No.:	B-129-0-13	**Substructure Unit:**	Pier

Rock Parameters	
Rock Mass Rating (RMR) (From AASHTO LRFD Table 10.4.6.4.1)	48
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))	1828
Presumptive Bearing Resistance for Spread Footing at Service Limit State (ksf) (From AASHTO LRFD Table C10.6.2.6.1-1)	30
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)	0.14
Average Elastic Modulus for Intact Rock, E_i (ksi) (From Load vs Displacement from Lab Test, ASTM D 7012)	2100
Elastic Modulus of Rock Mass, E_m (ksi) (From AASHTO LRFD Eq 10.4.6.5-1)	1292
Reduction Factor (E_m/E_i) (From AASHTO LRFD Table 10.4.6.5-1)	0.11
Elastic Modulus of Rock Mass (E_m) (ksi) (From AASHTO LRFD Eq 10.4.6.5-2)	231
Assumed Em (ksi)	200
Nominal Bearing Resistance (Carter and Kulhawy (1988))	qult=(√s+(m√s+s)$^{0.5}$)qu (**At the Strength Limit State**)
Effective Length of Footing, L (feet)	33.5
Effective Width of Footing, B (feet)	12.5
L/B	2.7
Type of Footing	Spread, Rectangular
Depth of Footing Below Ground, D (feet)	3.8
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))	79
Resistance Factor (From LRFD Table 10.5.5.2.2-1)	0.45
Factored Resistance (ksf)	35
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)	1.27
Influence Coefficient, I_p = L/B)$^{1/2}$/B_z (From AASHTO LRFD Eq 10.6.2.4.4-4)	1.289
Nominal Bearing Rsistance (ksf)	30
Elastic Settlement p (inches)	0.197

BEARING CAPACITY ANALYSIS	
AASHTO Article 10.6.3.1.2 and Munfakh (2001)	
Project	**HAN-75-14.39-Bridge No. HAN-75-1668**
Project#	**G13011G**
Bore#	**B-131-0-13 (Rear MSE Wall)**
Method	**AASHTO 10.6.3.1.2**
Foundation Dimension	
Width of Footing (B_f-2e) (ft) (Per AASHTO LRFD Article 10.6.1.3)	24.0
Length of Footing (L_f) (feet)	175.0
Length (L_f)/Width (B_f) (>5 is continous footing)	7.3
Type of Footing	Strip
Footing Bearing Elevation (feet)	779.6
Depth of Footing (D_f) Feet below Proposed Grade	4.0
Depth of Groundwater Table below Footing (ft)	8.8
Height of Slope (Hs) (feet)	Flat Ground
Soil Parameters	
Undrained Shear Strength/Cohesion (psf)	0
Angle of internal friction (Phi) Degrees	30
Unit Weight of soil above base of footing (pcf)	125
Unit Weight of soil below base of footing (pcf)	125
Bearing Capacity Factors	
N_c	30.14
N_q	18.40
N_γ	22.40
Shape Correction Factors	
s_c	1.00
s_q	1.00
s_γ	0.70
Load Inclination Factors	
ic	1.0
iq	1.0
i_γ	1.0
Correction for Water Table	
D_f+1.5B_f	40.0
C_{wq}	0.5
$C_{w\gamma}$	0.5
Embedment Depth Correction Factor	
Df/Bf	0.2
d_q	1.0
Bearing Capacity Terms	
Cohesion Term	0
Surcharge Term	4600
Unit Weight Term	11760
Nominal Bearing Resistence (psf)	16360
Factored Bearing Resistence (psf)	10634

AASHTO Eqn 10.6.3.1.2a

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

AASHTO Article 10.6.3.1.2 and Munfakh (2001)	
Project	**HAN-75-14.39-Bridge No. HAN-75-1668**
Project#	**G13011G**
Bore#	**B-128-0-13 (Forward MSE Wall)**
Method	**AASHTO 10.6.3.1.2**
Foundation Dimension	
Width of Footing (B_f -2e) (ft) (Per AASHTO LRFD Article 10.6.1.3)	21.5
Length of Footing (L_f) (feet)	255.0
Length (L_f)/Width (B_f) (>5 is continous footing)	11.9
Type of Footing	Strip
Footing Bearing Elevation (feet)	774.7
Depth of Footing (D_f) Feet below Proposed Grade	3.2
Depth of Groundwater Table below Footing (ft)	2.5
Height of Slope (Hs) (feet)	Flat Ground
Soil Parameters	
Undrained Shear Strength/Cohesion (psf)	0
Angle of internal friction (Phi) Degrees	30
Unit Weight of soil above base of footing (pcf)	125
Unit Weight of soil below base of footing (pcf)	125
Bearing Capacity Factors	
N_c	30.14
N_q	18.40
$N\gamma$	22.40
Shape Correction Factors	
s_c	1.00
s_q	1.00
s_γ	0.70
Load Inclination Factors	
ic	1.0
iq	1.0
i_γ	1.0
Correction for Water Table	
D_f+1.5B_f	35.5
C_{wq}	0.5
$C_{w\gamma}$	0.5
Embedment Depth Correction Factor	
Df/Bf	0.1
d_q	1.0
Bearing Capacity Terms	
Cohesion Term	0
Surcharge Term	3680
Unit Weight Term	10535
Nominal Bearing Resistence (psf)	14215
Factored Bearing Resistence (psf)	9240

AASHTO Eqn 10.6.3.1.2a

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

Pro Geotech, Inc.

AASHTO 2007-2010 (LRFD)
HAN-75-14.39-Bridge No. HAN-68-1668

MSEW(3.0): Update # 14.93

PROJECT IDENTIFICATION

Title:	HAN-75-14.39-Bridge No. HAN-68-1668
Project Number:	PID 87005
Client:	PB
Designer:	SS
Station Number:	802+25

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.....
.....Rear MSE Wallrev.BEN

Original date and time of creating this file: Jan 23, 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 $N\gamma$= 22.40

SEISMICITY

Not Applicable

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

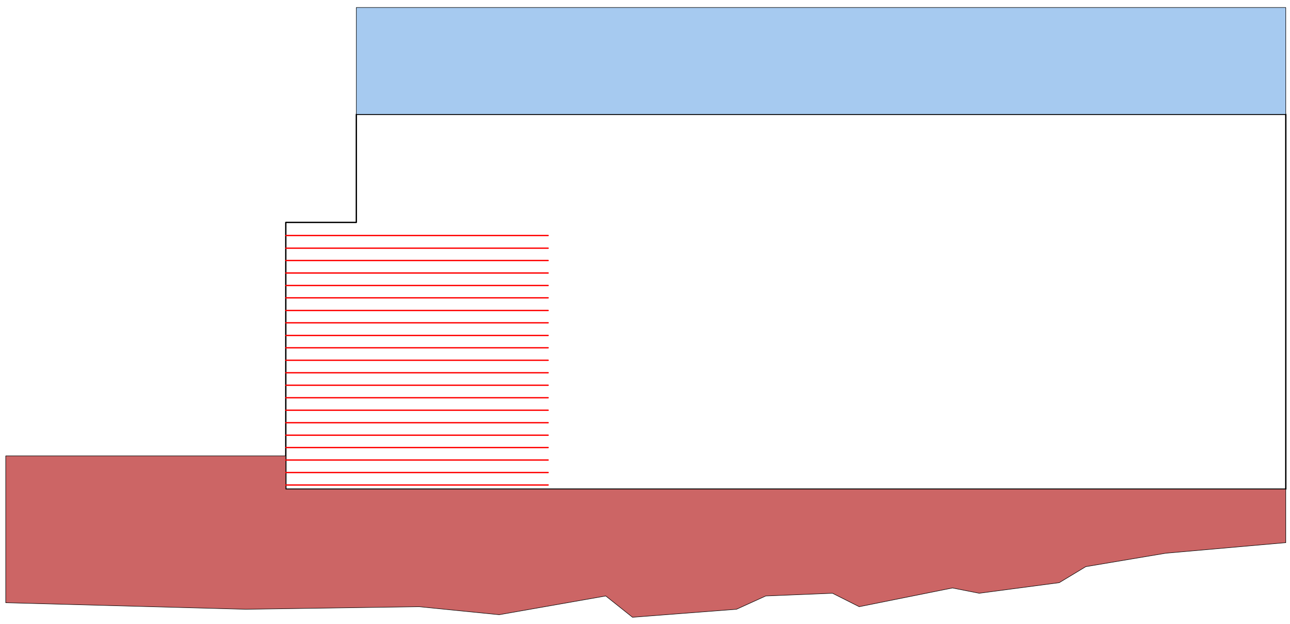
Design height, Hd	32.10	[ft]	{ Embedded depth is E = 4.00 ft, and height above top of finished bottom grade is H = 28.10 ft }
Batter, ω	0.0	[deg]	
Backslope, β	0.0	[deg]	
Backslope rise	0.0	[ft]	Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

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 250.0 [lb/ft ²]

ABUTMENT GEOMETRY (On pile foundation.)
Abutment's width, bf = 6.50 at distance from back of wall, cf = 2.00 [ft].
Footing's dimension: height, h' = 13.00, width, b = 1.75, and thickness, t = 3.25 [ft].
Dimensions of bridge bearing plate: height, fh = 0.38, width, fw = 0.83 [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)

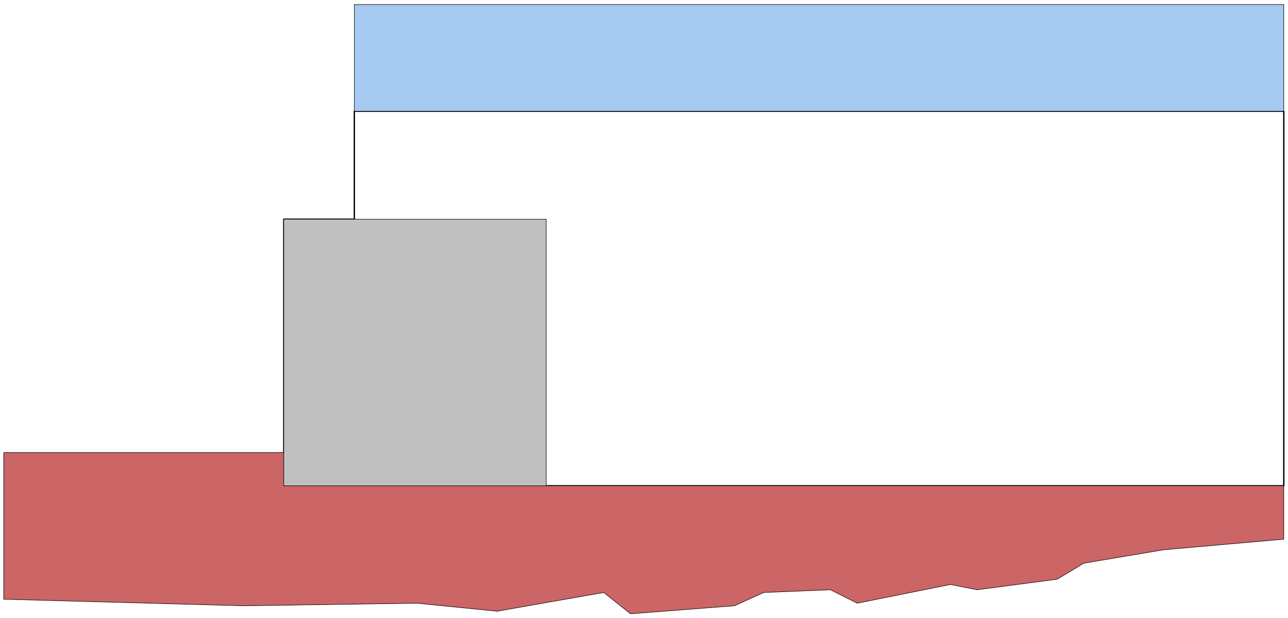
Bearing capacity, CDR = 1.17, factored bearing load = 9366 lb/ft².
Foundation Interface: Direct sliding, CDR = 1.356, Eccentricity, e/L = 0.1865, CDR-overturning = 2.44

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	31.60	1	N/A	1.26	1.40	1.402	3.796	1.599	0.1819	---
2	2.00	31.60	1	N/A	1.08	1.20	1.203	3.072	1.644	0.1683	---
3	3.50	31.60	1	N/A	1.12	1.24	1.244	2.992	1.692	0.1550	---
4	5.00	31.60	1	N/A	1.16	1.29	1.287	2.914	1.743	0.1419	---
5	6.50	31.60	1	N/A	1.20	1.33	1.334	2.836	1.797	0.1291	---
6	8.00	31.60	1	N/A	1.25	1.38	1.384	2.749	1.854	0.1165	---
7	9.50	31.60	1	N/A	1.29	1.44	1.439	2.676	1.914	0.1041	---
8	11.00	31.60	1	N/A	1.35	1.50	1.497	2.606	1.979	0.0919	---
9	12.50	31.60	1	N/A	1.39	1.54	1.545	2.530	2.048	0.0799	---
10	14.00	31.60	1	N/A	1.41	1.57	1.568	2.699	2.121	0.0679	---
11	15.50	31.60	1	N/A	1.43	1.58	1.584	2.777	2.200	0.0561	---
12	17.00	31.60	1	N/A	1.43	1.58	1.584	2.785	2.284	0.0443	---
13	18.50	31.60	1	N/A	1.41	1.56	1.562	2.718	2.374	0.0324	---
14	20.00	31.60	1	N/A	1.38	1.53	1.532	2.607	2.471	0.0204	---
15	21.50	31.60	1	N/A	1.35	1.50	1.499	2.466	2.575	0.0081	---
16	23.00	31.60	1	N/A	1.32	1.47	1.471	2.347	2.687	-0.0045	---
17	24.50	31.60	1	N/A	1.30	1.45	1.449	2.299	2.807	-0.0177	---
18	26.00	31.60	1	N/A	1.29	1.44	1.437	2.248	2.935	-0.0318	---
19	27.50	31.60	1	N/A	1.29	1.44	1.437	2.193	3.072	-0.0472	---
20	29.00	31.60	1	N/A	1.29	1.44	1.436	2.117	3.217	-0.0643	---
21	30.50	31.60	1	N/A	0.82	0.91	0.913	1.286	3.367	-0.0841	---

BEARING CAPACITY for GIVEN LAYOUT

	STATIC	SEISMIC	UNITS
(Water table is at wall base elevation)			
Factored bearing resistance. q-n	10946	N/A	[lb/ft ²]
Factored bearing load, σ_V	9366.3	N/A	[lb/ft ²]
Eccentricity, e	3.78	N/A	[ft]
Eccentricity, e/L	0.120	N/A	
CDR calculated	1.17	N/A	
Base length	31.60	N/A	[ft]

Unfactored applied bearing pressure = (Unfactored R) / [L - 2 * (Unfactored e)] =
Unfactored R = 165035.60 [lb/ft], L = 31.60, Unfactored e = 3.13 [ft], and Sigma = 6515.35 [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 = 1.356

#	Metal strip Elevation [ft]	Metal strip Length [ft]	CDR Static	CDR Seismic	Metal strip Type #	Product name
1	0.50	31.60	1.599	N/A	1	---
2	2.00	31.60	1.644	N/A	1	---
3	3.50	31.60	1.692	N/A	1	---
4	5.00	31.60	1.743	N/A	1	---
5	6.50	31.60	1.797	N/A	1	---
6	8.00	31.60	1.854	N/A	1	---
7	9.50	31.60	1.914	N/A	1	---
8	11.00	31.60	1.979	N/A	1	---
9	12.50	31.60	2.048	N/A	1	---
10	14.00	31.60	2.121	N/A	1	---
11	15.50	31.60	2.200	N/A	1	---
12	17.00	31.60	2.284	N/A	1	---
13	18.50	31.60	2.374	N/A	1	---
14	20.00	31.60	2.471	N/A	1	---
15	21.50	31.60	2.575	N/A	1	---
16	23.00	31.60	2.687	N/A	1	---
17	24.50	31.60	2.807	N/A	1	---
18	26.00	31.60	2.935	N/A	1	---
19	27.50	31.60	3.072	N/A	1	---
20	29.00	31.60	3.217	N/A	1	---
21	30.50	31.60	3.367	N/A	1	---

ECCENTRICITY for GIVEN LAYOUT (for Simplified Method)

At interface with foundation: e/L static = 0.1865; Overturning: CDR-static = 2.44

#	Metal strip Elevation [ft]	Metal strip Length [ft]	e / L Static	e / L Seismic	Metal strip Type #	Product name
1	0.50	31.60	0.1819	N/A	1	---
2	2.00	31.60	0.1683	N/A	1	---
3	3.50	31.60	0.1550	N/A	1	---
4	5.00	31.60	0.1419	N/A	1	---
5	6.50	31.60	0.1291	N/A	1	---
6	8.00	31.60	0.1165	N/A	1	---
7	9.50	31.60	0.1041	N/A	1	---
8	11.00	31.60	0.0919	N/A	1	---
9	12.50	31.60	0.0799	N/A	1	---
10	14.00	31.60	0.0679	N/A	1	---
11	15.50	31.60	0.0561	N/A	1	---
12	17.00	31.60	0.0443	N/A	1	---
13	18.50	31.60	0.0324	N/A	1	---
14	20.00	31.60	0.0204	N/A	1	---
15	21.50	31.60	0.0081	N/A	1	---
16	23.00	31.60	-0.0045	N/A	1	---
17	24.50	31.60	-0.0177	N/A	1	---
18	26.00	31.60	-0.0318	N/A	1	---
19	27.50	31.60	-0.0472	N/A	1	---
20	29.00	31.60	-0.0643	N/A	1	---
21	30.50	31.60	-0.0841	N/A	1	---

AASHTO 2007-2010 (LRFD)
HAN-75-14.39-Bridge No. HAN-68-1668

MSEW(3.0): Update # 14.93

PROJECT IDENTIFICATION

Title:	HAN-75-14.39-Bridge No. HAN-68-1668
Project Number:	87005
Client:	PB
Designer:	SS
Station Number:	805+00

Description:

External Stability Analysis of the Forward 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.....
.....ward MSE Wallrev.BEN

Original date and time of creating this file: Jan 23, 2015

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)

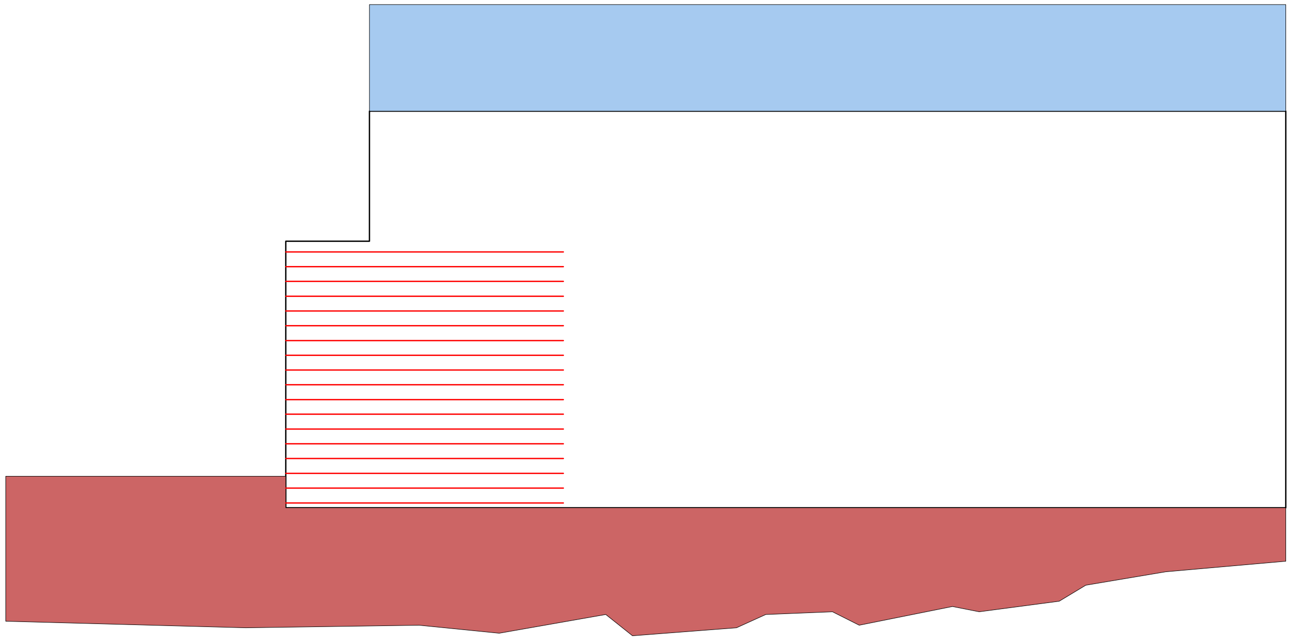
Design height, Hd	27.10	[ft]	{ Embedded depth is E = 3.20 ft, and height above top of finished bottom grade is H = 23.90 ft }
Batter, ω	0.0	[deg]	
Backslope, β	0.0	[deg]	
Backslope rise	0.0	[ft]	Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

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 250.0 [lb/ft ²]

ABUTMENT GEOMETRY (On pile foundation.)
Abutment's width, bf = 6.50 at distance from back of wall, cf = 2.00 [ft].
Footing's dimension: height, h' = 13.20, width, b = 1.75, and thickness, t = 3.25 [ft].
Dimensions of bridge bearing plate: height, fh = 0.50, width, fw = 0.83 [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)

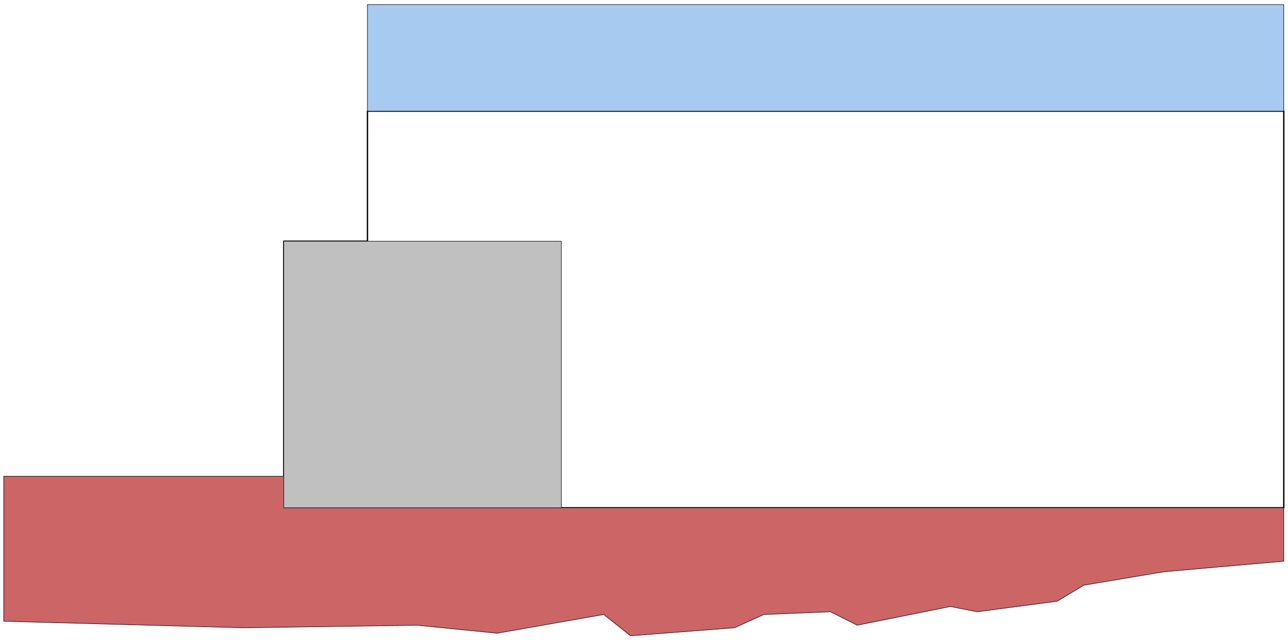
Bearing capacity, CDR = 1.20, factored bearing load = 8194 lb/ft².
Foundation Interface: Direct sliding, CDR = 1.308, Eccentricity, e/L = 0.1891, CDR-overturning = 2.36

	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	28.20	1	N/A	1.40	1.56	1.560	3.292	1.543	0.1837	---
2	2.00	28.20	1	N/A	1.21	1.34	1.344	2.650	1.590	0.1678	---
3	3.50	28.20	1	N/A	1.26	1.39	1.395	2.566	1.640	0.1522	---
4	5.00	28.20	1	N/A	1.30	1.45	1.450	2.494	1.693	0.1369	---
5	6.50	28.20	1	N/A	1.36	1.51	1.510	2.415	1.749	0.1218	---
6	8.00	28.20	1	N/A	1.39	1.55	1.549	2.413	1.809	0.1070	---
7	9.50	28.20	1	N/A	1.41	1.56	1.563	2.543	1.872	0.0923	---
8	11.00	28.20	1	N/A	1.42	1.57	1.573	2.624	1.940	0.0777	---
9	12.50	28.20	1	N/A	1.41	1.57	1.566	2.647	2.012	0.0631	---
10	14.00	28.20	1	N/A	1.39	1.54	1.539	2.630	2.089	0.0486	---
11	15.50	28.20	1	N/A	1.36	1.51	1.506	2.504	2.172	0.0338	---
12	17.00	28.20	1	N/A	1.33	1.47	1.473	2.358	2.260	0.0188	---
13	18.50	28.20	1	N/A	1.30	1.45	1.446	2.202	2.354	0.0033	---
14	20.00	28.20	1	N/A	1.28	1.42	1.425	2.039	2.454	-0.0129	---
15	21.50	28.20	1	N/A	1.27	1.42	1.415	1.981	2.559	-0.0302	---
16	23.00	28.20	1	N/A	1.27	1.42	1.415	1.929	2.669	-0.0491	---
17	24.50	28.20	1	N/A	1.27	1.41	1.412	1.854	2.783	-0.0703	---
18	26.00	28.20	1	N/A	1.03	1.14	1.139	1.424	2.896	-0.0951	---

BEARING CAPACITY for GIVEN LAYOUT

	STATIC	SEISMIC	UNITS
(Water table is at wall base elevation)			
Factored bearing resistance. q-n	9800	N/A	[lb/ft ²]
Factored bearing load, σ_V	8193.6	N/A	[lb/ft ²]
Eccentricity, e	3.34	N/A	[ft]
Eccentricity, e/L	0.118	N/A	
CDR calculated	1.20	N/A	
Base length	28.20	N/A	[ft]

Unfactored applied bearing pressure = (Unfactored R) / [L - 2 * (Unfactored e)] =
Unfactored R = 129136.32 [lb/ft], L = 28.20, Unfactored e = 2.71 [ft], and Sigma = 5667.10 [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 = 1.308

#	Metal strip Elevation [ft]	Metal strip Length [ft]	CDR Static	CDR Seismic	Metal strip Type #	Product name
1	0.50	28.20	1.543	N/A	1	---
2	2.00	28.20	1.590	N/A	1	---
3	3.50	28.20	1.640	N/A	1	---
4	5.00	28.20	1.693	N/A	1	---
5	6.50	28.20	1.749	N/A	1	---
6	8.00	28.20	1.809	N/A	1	---
7	9.50	28.20	1.872	N/A	1	---
8	11.00	28.20	1.940	N/A	1	---
9	12.50	28.20	2.012	N/A	1	---
10	14.00	28.20	2.089	N/A	1	---
11	15.50	28.20	2.172	N/A	1	---
12	17.00	28.20	2.260	N/A	1	---
13	18.50	28.20	2.354	N/A	1	---
14	20.00	28.20	2.454	N/A	1	---
15	21.50	28.20	2.559	N/A	1	---
16	23.00	28.20	2.669	N/A	1	---
17	24.50	28.20	2.783	N/A	1	---
18	26.00	28.20	2.896	N/A	1	---

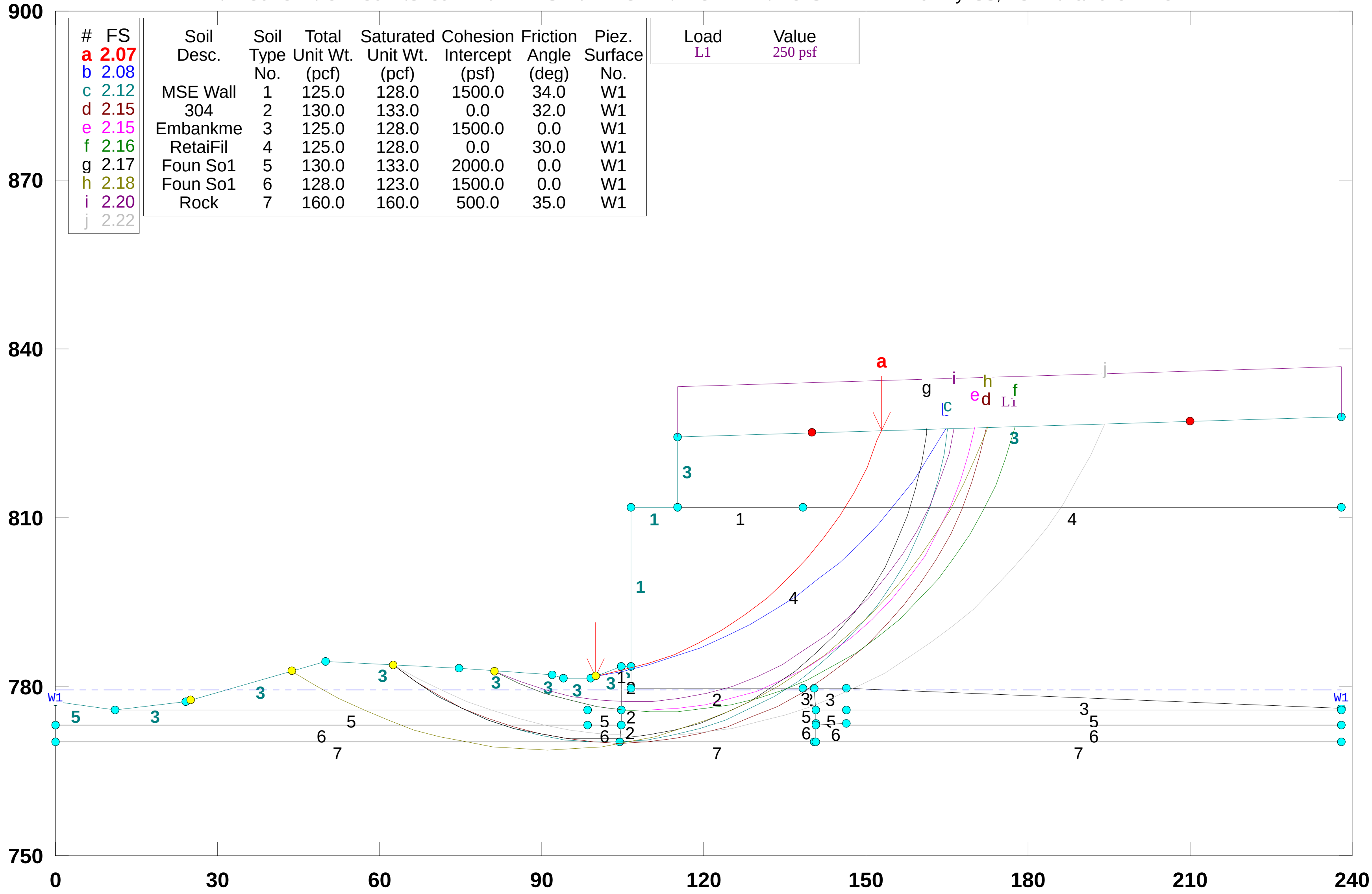
ECCENTRICITY for GIVEN LAYOUT (for Simplified Method)

At interface with foundation: e/L static = 0.1891; Overturning: CDR-static = 2.36

#	Metal strip Elevation [ft]	Metal strip Length [ft]	e / L Static	e / L Seismic	Metal strip Type #	Product name
1	0.50	28.20	0.1837	N/A	1	---
2	2.00	28.20	0.1678	N/A	1	---
3	3.50	28.20	0.1522	N/A	1	---
4	5.00	28.20	0.1369	N/A	1	---
5	6.50	28.20	0.1218	N/A	1	---
6	8.00	28.20	0.1070	N/A	1	---
7	9.50	28.20	0.0923	N/A	1	---
8	11.00	28.20	0.0777	N/A	1	---
9	12.50	28.20	0.0631	N/A	1	---
10	14.00	28.20	0.0486	N/A	1	---
11	15.50	28.20	0.0338	N/A	1	---
12	17.00	28.20	0.0188	N/A	1	---
13	18.50	28.20	0.0033	N/A	1	---
14	20.00	28.20	-0.0129	N/A	1	---
15	21.50	28.20	-0.0302	N/A	1	---
16	23.00	28.20	-0.0491	N/A	1	---
17	24.50	28.20	-0.0703	N/A	1	---
18	26.00	28.20	-0.0951	N/A	1	---

HAN-75-14.39-Bridge No. HAN-68-1668-RearMSE Wall, Global Stability Analysis-ST

M:\PROJEC~2\13PROJ~1\G13011~1\ANALYS~1\BRIDGE~1\RROADF~1\B131STNR.PL2 Run By: SS, PGI 1/26/2015 11:57AM



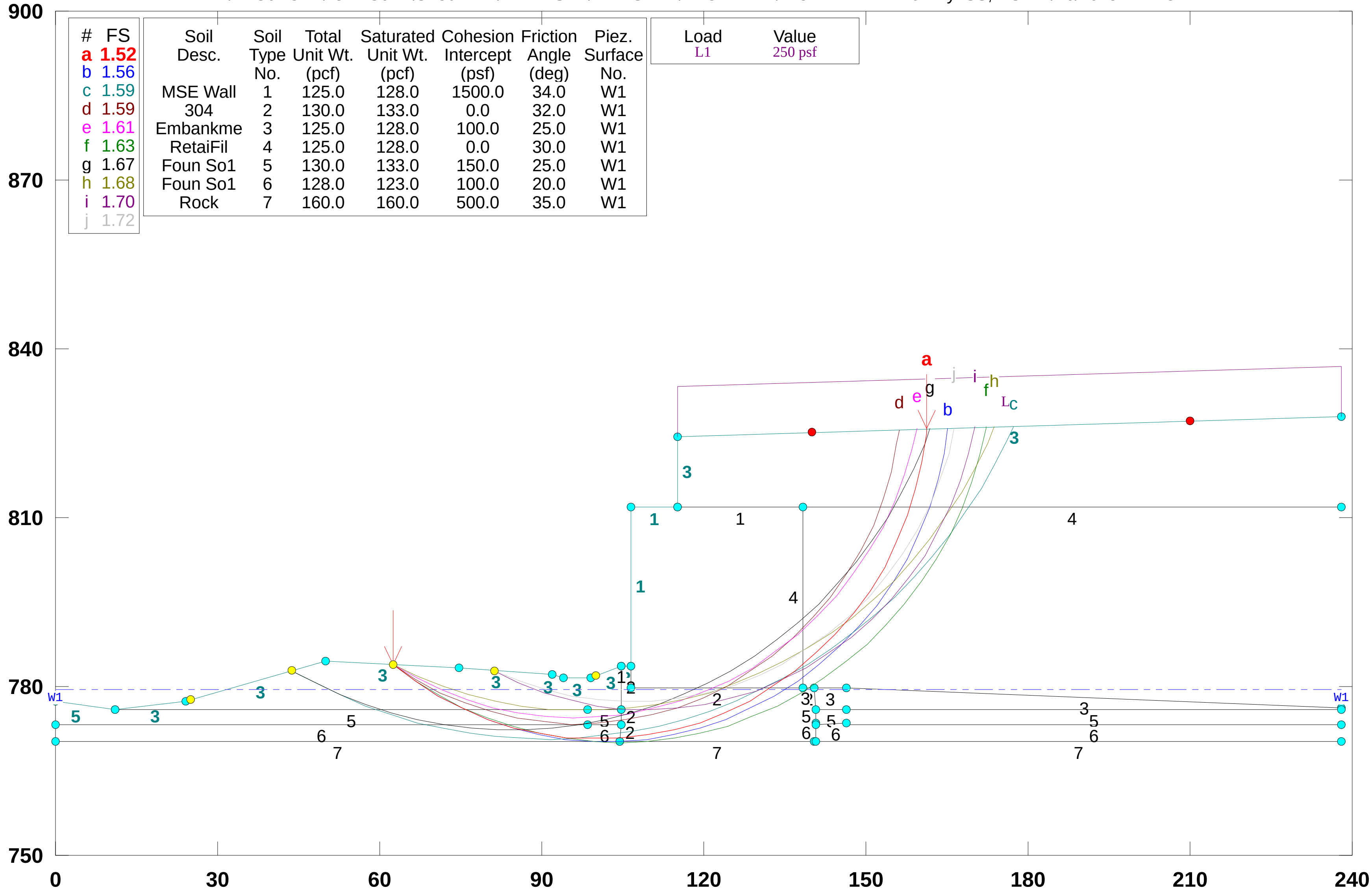
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
MSE Wall	1	125.0	128.0	1500.0	34.0	W1
304	2	130.0	133.0	0.0	32.0	W1
Embankme	3	125.0	128.0	1500.0	0.0	W1
RetaiFil	4	125.0	128.0	0.0	30.0	W1
Foun So1	5	130.0	133.0	2000.0	0.0	W1
Foun So1	6	128.0	123.0	1500.0	0.0	W1
Rock	7	160.0	160.0	500.0	35.0	W1

GSTABL7 v.2 FSmin=2.07

Safety Factors Are Calculated By The Modified Bishop Method

HAN-75-14.39-Bridge No. HAN-68-1668-RearMSE Wall, Global Stability Analysis-LT

M:\PROJEC~2\13PROJ~1\G13011~1\ANALYS~1\BRIDGE~1\RROADF~1\B131LTR.PL2 Run By: SS, PGI 1/26/2015 12:18PM



GSTABL7 v.2 FSmin=1.52

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

HAN-75-14.39-Bridge No. HAN-68-1668-ForwMSE Wall, Global Stability Analysis-ST

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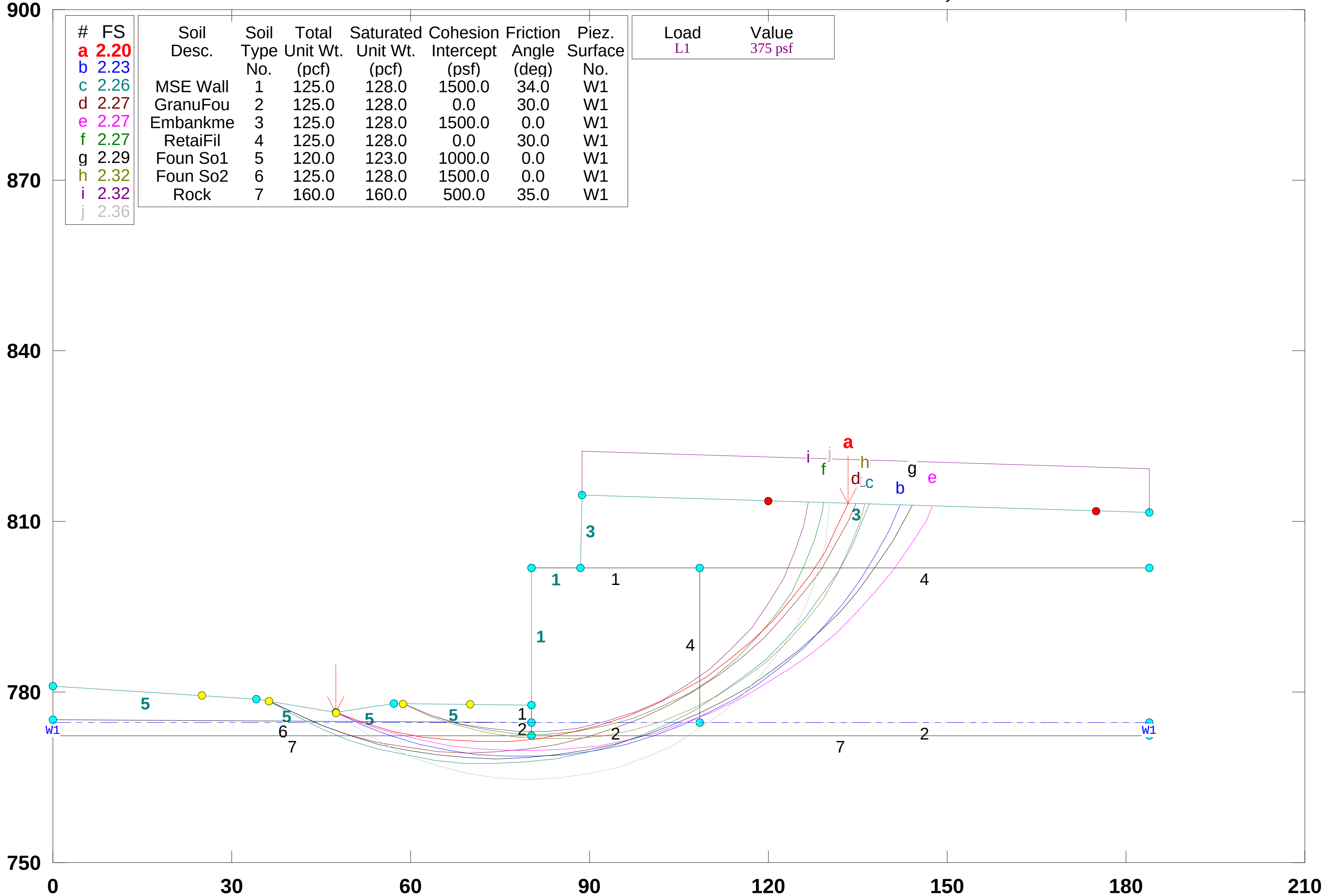


#	FS
a	2.20
b	2.23
c	2.26
d	2.27
e	2.27
f	2.27
g	2.29
h	2.32
i	2.32
j	2.36

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
MSE Wall	1	125.0	128.0	1500.0	34.0	W1
GranuFou	2	125.0	128.0	0.0	30.0	W1
Embankme	3	125.0	128.0	1500.0	0.0	W1
RetaiFil	4	125.0	128.0	0.0	30.0	W1
Foun So1	5	120.0	123.0	1000.0	0.0	W1
Foun So2	6	125.0	128.0	1500.0	0.0	W1
Rock	7	160.0	160.0	500.0	35.0	W1

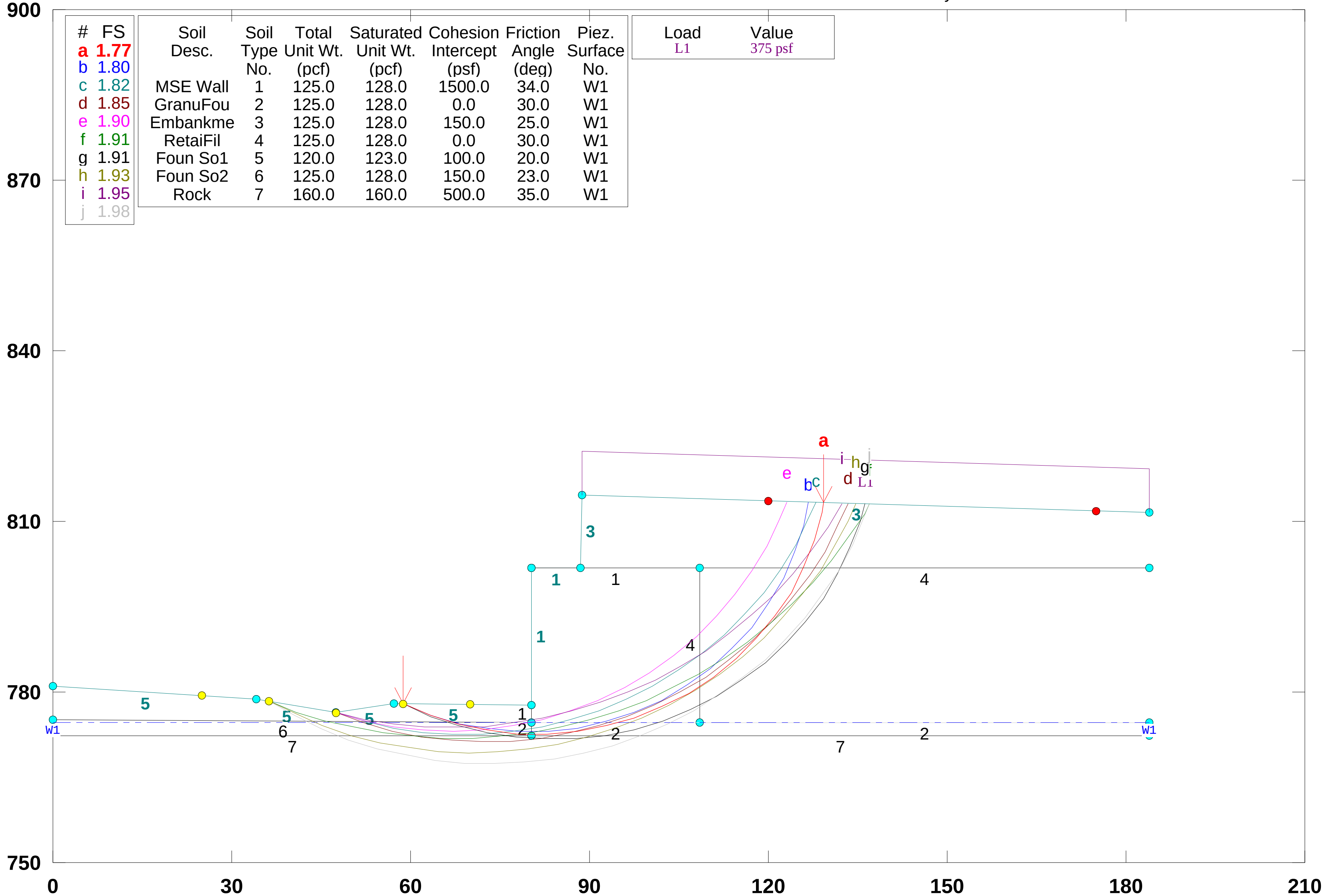
Load	Value
L1	375 psf

GSTABL7 v.2 FSmin=2.20

Safety Factors Are Calculated By The Modified Bishop Method

HAN-75-14.39-Bridge No. HAN-68-1668-ForwMSE Wall, Global Stability Analysis-LT

M:\PROJEC~2\13PROJ~1\G13011~1\ANALYS~1\BRIDGE~1\RROADF~1\B-128LTN.PL2 Run By: SS 3/15/2014 5:17AM



#	FS
a	1.77
b	1.80
c	1.82
d	1.85
e	1.90
f	1.91
g	1.91
h	1.93
i	1.95
j	1.98

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
MSE Wall	1	125.0	128.0	1500.0	34.0	W1
GranuFou	2	125.0	128.0	0.0	30.0	W1
Embankme	3	125.0	128.0	150.0	25.0	W1
RetaiFil	4	125.0	128.0	0.0	30.0	W1
Foun So1	5	120.0	123.0	100.0	20.0	W1
Foun So2	6	125.0	128.0	150.0	23.0	W1
Rock	7	160.0	160.0	500.0	35.0	W1

Load	Value
L1	375 psf

GSTABL7 v.2 FSmin=1.77

Safety Factors Are Calculated By The Modified Bishop Method

VI.D. Geotechnical Reports

C-R-S: HAN-75-14.39-Bridge No. HAN-68-1668	PID:87005	Reviewer:SS	Date:1/23/2015

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-68-14.39- Bridge No. HAN-68-1668	PID:87005	Reviewer:SS	Date:1/23/2015

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	[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?	
[Y]	N	X	3	Has the shear strength of the foundation bedrock been determined? Check method used: ☐ laboratory shear tests [☐] other List Other items:	Unconfined Compression Strength of Bedrock

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) $\boxed{□}$ 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 $\boxed{□}$ 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?	
Y N $\boxed{X}$		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?	
Y N $\boxed{X}$	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

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

2) COLOR :

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

3) PRIMARY COMPONENT

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

Cohesive (fine grained) Soils - Consistency

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

4) COMPONENT MODIFIERS:

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

5) Soil Organic Content

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

6) Relative Visual Moisture

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

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

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

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

3) **WEATHERING**

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

4) **RELATIVE STRENGTH**

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

5) **TEXTURE**

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

6) **BEDDING**

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

7) DESCRIPTORS

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

8) DISCONTINUITIES

a) Discontinuity Types

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

b) Degree of Fracturing

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

c) Aperture Width

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

d) Surface Roughness

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

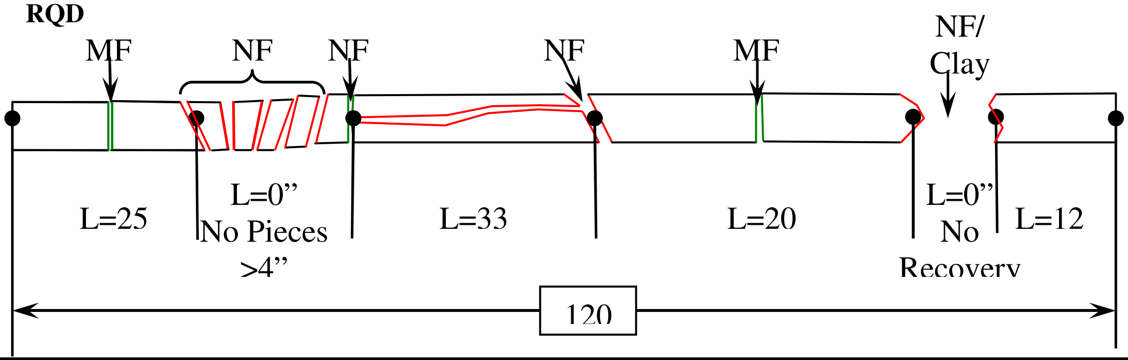
10) LOSS

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

L_R=Run Length R_R=Run Recovery

L_U=Rock Unit Length R_U=Rock Unit Recovery

9) RQD



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

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