Pro Geotech, Inc.

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April 28, 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-75-1697 L&R over Blanchard River
Findlay, Hancock County, Ohio
PID No. 87005
PGI Project No. G13011G

Dear Mr. Hussein:

Enclosed please find our Final Structure Foundation Exploration Report for the above referenced project. Our services included a geotechnical field exploration, laboratory testing, engineering analysis, and related design and construction recommendations. These services have been provided in accordance with our proposal dated January 16, 2013. It is important that the items under "**Limitations**" be precisely followed and complied with.

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

Respectfully,

PRO GEOTECH, INC.

S. Siva

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

Walid Najjar

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

Enclosure
G13011Grpt/SS/4/28/2015

FINAL
STRUCTURE FOUNDATION EXPLORATION REPORT
FOR HAN-75-14.39
BRIDGE NO. HAN-75-1697 L&R OVER BLANCHARD RIVER

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

PREPARED FOR:

PARSONS BRINCKERHOFF

PREPARED BY:

PRO GEOTECH, INC.

APRIL 28, 2015

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1.0 EXECUTIVE SUMMARY

This report has been prepared for the HAN-75-14.39 project which calls for replacement of the existing Interstate Route 75 (IR-75) mainline Bridge No. HAN-75-1697 Left & Right over Blanchard River, TR. 88, and CR. 223 in Findlay, Hancock County, Ohio. Four (4) historic test borings identified as B-1 (B-001-1-87), B-2 (B-002-1-87), B-3 (B-003-1-87), and B-4 (B-004-1-87) were obtained from the subsurface geotechnical exploration completed on April 1987. A total of two (2) project test borings identified as B-046-0-13 and B-047-0-13 were advanced for bridge foundations design purposes. These project test borings were advanced to approximate depths ranging from 21.5 to 27.5 feet below the existing ground surface. Historic test borings B-001-1-87 and B-004-1-87 were advanced in the vicinity of the proposed rear and forward abutments, respectively. Project test boring B-046-0-13 and historic test boring B-002-1-87 were advanced in the vicinity of proposed Pier 1 while project test boring B-047-0-13 and historic test boring B-003-1-87 were advanced in the vicinity of proposed Pier 2.

<u>Findings:</u> The subsurface soil conditions in the vicinity of this proposed bridge were determined from the soil information obtained from project test borings B-046-0-13 and B-047-0-13 and historic test borings B-001-1-87 through B-004-1-87.

The subsurface soils encountered in project test borings were primarily cohesive in nature and consisted of both fill materials and natural soils. Test boring B-046-0-13 consisted of fill material and natural soil and B-047-0-13 consisted entirely of fill material above the bedrock. The fill material consisted of silty clay (A-6b) and clay (A-7-6) and was encountered to an approximate depth of 3.5 feet in test boring B-046-0-13. Natural soils encountered above bedrock in test boring B-046-0-13 consisted of plastic sandy silt (A-4a), plastic silt (A-4b), and clay (A-7-6). Bedrock was encountered in project test boring B-046-0-13 at an approximate depth of 16.0 feet below the ground surface while bedrock was encountered in project test boring B-047-0-13 at an approximate depth of 11.0 feet below the ground surface. The consistency of the cohesive soils ranged from "medium stiff" to "hard", but was predominately “medium stiff”.

The subsurface soils encountered in all historic test borings were generally cohesive soils, but non-cohesive soils were also encountered above bedrock in test borings B-001-1-87 and B-002-1-87. The cohesive soils encountered consisted of silt and clay (A-6a), sandy silt (A-4a), plastic silt (A-4b), and silty clay (A-6b), and the non-cohesive soils encountered consisted of non-plastic sandy silt (A-4a) and non-plastic silt (A-4b). Bedrock was encountered in historic boring B-001-1-87 at an approximate depth of

33.5 feet below the ground surface while bedrock was encountered in historic boring B-004-1-87 at an approximate depth of 39.0 feet below the ground surface. Bedrock was encountered in project test boring B-002-1-87 at an approximate depth of 12.5 feet below the ground surface while bedrock was encountered in project test boring B-003-1-87 at an approximate depth of 13.5 feet below the ground surface. The consistency of cohesive soils ranged from "soft" to "hard", but was predominately “very stiff” while the relative density was ranged from “medium dense” to “dense”.

Bedrock was encountered in all project and historic test borings. The core samples encountered in project test borings consisted of dolomite of the Tymochtee/Greenfield Group. The dolomite was light gray, and slightly weathered. Bedding within the dolomite was generally very thin to medium and was fractured to moderately fractured. No slickensides were observed and the fractures were typically tight and slightly rough. The compressive strength of the core specimens ranged from 6,888 psi in test boring B-046-0-13 to 9,765 psi in test boring B-047-0-13 which characterizes them as “moderately strong” to “strong”, respectively. The Rock Quality Designation (RQD) for the core samples ranged from 21% to 48%. The Rock Mass Rating obtained for the bedrock core samples according to LRFD Table 10.4.6.4-1 varied from 36 to 44 and is classified as “Poor Rock” to “Fair Rock”.

<u>Recommendations:</u> Site plans provided by PB personnel indicate that the proposed superstructure design loads will be transferred to the underlying bedrock by means of piles at the rear and forward abutment locations and by means of drilled shafts at the proposed Pier 1 and Pier 2 locations. Since the top of bedrock at the abutment locations was encountered at approximate depths ranging from 33.5 feet to 39.0 feet below the existing pavement, the proposed superstructure loads may be transferred to the underlying bedrock by means of end bearing piles at the abutment locations. Since the top of bedrock at the pier locations was encountered at approximate depths ranging from 11.0 feet to 16.0 feet below the existing ground surface, the proposed superstructure loads may be transferred to the underlying bedrock by means of drilled shafts at the pier locations.

Design information provided by PB personnel indicates that the maximum compression factored loads along a vertical axial direction at the Strength and Service Limit will be 1230 kips per shaft and 1025 kips, respectively and lateral loads will control the drilled shaft design at Pier 1 and Pier 2 locations. Drilled shaft foundations can be reinforced concrete columns designed to carry their maximum factored load at the Strength Limit State. The unit side and unit tip resistances were calculated using equations 10.8.3.5.4b-1 and 10.8.3.5.4c-1 ($q_p = 2.5q_n$), respectively in the AASHTO LRFD Bridge Design Specifications. Based on these equations, unit side resistance of 10.0 ksf was estimated for the bedrock at test borings B-046-0-13, B-002-1-87, B-047-0-13, and B-003-1-87. Unit tip resistance of 2480 ksf was estimated for the bedrock at structural test borings B-046-0-13, B-002-1-87, B-047-0-13, and B-003-1-87. Table 6.1.1 summarizes total factored resistance for the selected diameter of 3.0 feet and socket length of 4.5 feet at the pier boring locations. Based on the factored axial compression resistance for the selected shaft socket length and diameter, the estimated maximum total settlement and differential settlement will not exceed one inch and one half inch, respectively. The shaft factored resistance and settlement calculation spreadsheets are included in Appendix B.

Table 6.1.1 – Estimated Design Parameters for Drilled Shafts

Boring No.	Top of Bedrock Elevation (feet)	Shaft Tip Depth (feet)	Socket Diameter (feet)	Socket Length (feet)	Total Factored Resistance (kips)
Pier 1					
B-046-0-13	752.8±	4.5	3.0	4.5	8764
B-002-1-87	751.6±	4.5	3.0	4.5	8764
Pier 2					
B-047-0-13	754.9±	4.5	3.0	4.5	8764
B-003-1-87	753.9±	4.5	3.0	4.5	8764

The drilled shaft supported piers may experience horizontal movement caused by lateral loads and overturning moments. A lateral load analysis should be performed using LPILE computer program by Ensoft or similar computer program for selected shaft diameter and socket length to check whether lateral resistance is adequate to support lateral loads and overturning moments. Table 6.1.2 summarizes the weak rock parameters to perform lateral load analyses by PB personnel. In lateral load analysis, the bedrock socket length used in vertical axial compression capacity analyses should be optimized to find the minimum length necessary to resist the applied lateral load based on serviceability and structural requirements and selected the maximum bedrock socket length between above value and 1.5 times the bedrock socket diameter.

Table 6.1.2 - Estimated Weak Rock Parameters for Lateral Load Analyses

Boring No.	Top Bedrock Elevation(ft)	Effective Unit Weight * (pci)	Youngs's Modulus (psi)	Compressive Strength (psi)	RQD (%)	k_rm
Piers						
B-046-0-13	752.8±	0.059	200000	6888	31	0.0005
B-047-0-13	754.9±	0.059	200000	6888	48	0.0005

*Below the water

Design information provided by PB personnel indicates that the maximum factored loads along a vertical direction will be 179 kips per pile. The end bearing piles must be steel H-piles driven to refusal on the underlying dolomite bedrock. H-pile sizes HP-10X42 may be selected for abutment foundation design. The estimated pile parameters for end bearing piles at each boring location are summarized in Table 6.1.3. The pile cut-off elevations at the abutments were extracted from the final structure site plan provided by PB personnel.

Table 6.1.3 - Estimated Design Parameters for H-Piles

Boring No.	Pile Cut-off Elevation (ft)	Pile Tip Elevation (ft)	Estimated Effective Pile Length (ft)	Pile Type	Pile Size	Maximum Factored Structural Resistance/pile
Abutments						
B-001-1-87	778.1	751.9	30.0	H-Pile	10X42	310 kips
B-004-1-87	783.5	753.7	30.0	H-Pile	10X42	310 kips

At the rear abutment, consolidation of the foundation soils caused by construction of the proposed embankment will be more than 0.6 inches. Therefore negative skin friction will develop along the pile section between the bottom of the proposed embankment and the top of bedrock due to the consolidation of the foundation soils caused by construction of the proposed embankment. The piles should be designed in accordance with section 202.2.3.2.c – “Down Drag Forces on Piles” of the *ODOT Bridge Design Manual* issued in January 2007. Nominal down drag load of 65 kips per pile may be assumed for pile sizes HP-10X42 at the rear abutment location.

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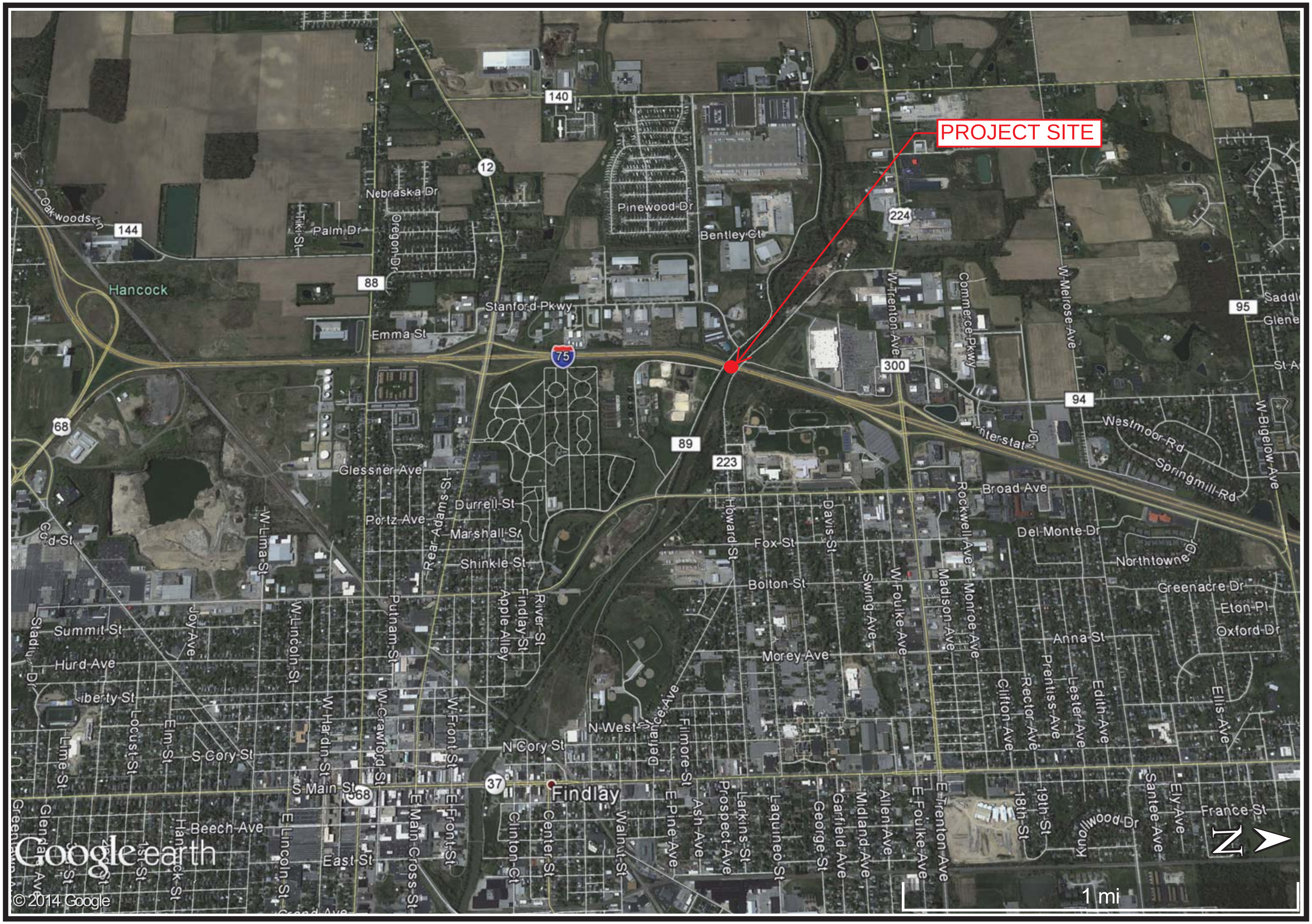
2.0 INTRODUCTION

This report has been prepared for the HAN-75-14.39 project which calls for replacement of the existing Interstate Route 75 (IR-75) mainline Bridge No. HAN-75-1697 Left & Right over Blanchard River, TR. 88, and CR. 223 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 replacement of Bridge No. HAN-75-1697 Left & Right which carry IR-75 vehicular traffic over Blanchard River, TR. 88, and CR. 223. The proposed replacement bridges will be three span structures each with a total length of 338 feet. The superstructures for the proposed bridges are expected to be continuous wide flanged pre-stressed concrete I-beams with a reinforced concrete deck on integral abutments and piers. The sub-structure units will be supported on reinforced concrete spill-through abutments on capped piles and cap and column piers on drilled shafts. The bridges are to be designed based on HL-93 loading criteria and the ODOT Bridge Design Manual, issued in 2007 which includes LRFD Bridge Design Specifications. Also, existing IR 75 profile grade will be realigned vertically in the vicinity of the replacement bridges and the proposed profile grade of IR 75 will rise approximately 4.2 feet at the rear abutment and 1.4 feet at the forward abutment. 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-75-1697 OVER BLANCHARD RIVER, TR. 88, AND CR. 223
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 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 two (2) test borings in the vicinity of existing Bridge No. HAN-75-1697 Left & Right over Blanchard River for structural foundation design purposes. The two (2) structural test borings for the bridge were to be advanced to approximate depth 30.0 feet each below the existing ground, and included obtaining 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 bridge including shallow and deep foundations
- Preparation of ODOT Geotechnical Design Checklists
- Geotechnical Exploration Plans are included in our scope of services for this project

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

3.1 Geology

Based on information obtained from the Physiographic Regions of Ohio, the project site lies on the Huron-Erie Lake Plains and Till Plains Sections of the Central Lowland Province. The project site is located within the Central Ohio Clayey Till Plain Region of the Till Plains Section. The Columbus Escarpment separates the Findlay Embayment District from the Central Ohio Clayey Till Plain Region. The project site is located at approximate elevations ranging from 755 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 (largely till) ranging from 5 feet to 100 feet in thickness. The main geologic deposit of the project site consists of clayey, high-lime Wisconsinan-age till; lake-planed moraine, very flat, planed by waves in glacial lakes; small patches of sand, silt, or clay 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 soils in the vicinity of the project area consist primarily of layers of silt loam, clay loam, silty clay, and silty clay loam. 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 occur 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 wells which are active and abandoned are located within the project site. According to ODNR's Ohio Mines Locator website, no abandoned underground or surface mines are present in the immediate vicinity of the project site. Based on the Ohio Division of Geological Survey Interactive Map of Ohio Mineral Industries, an active limestone industrial quarry is located approximately 0.4 miles southwest of the project site. According to ODNR, the project site is located outside of the "Probable Karst Regions" of Ohio and outside of the "Landslide-Prone Areas" of Ohio. According to ODNR website, two (2) earthquakes occurred within the Hancock County; one in 1990 with magnitude of 2.3 Richter Scale and another in 2011 with magnitude of 2.4 Richter Scale. Their epicenters were located approximately 8.8 miles to the northeast in Big Lick Township and 14.2 miles to the south in Delaware Township.

3.2 Observation of the Project

The reconnaissance of the project site was performed by one of PGI's geotechnical engineers in July and August 2013. The project site is located in a commercial area and includes buildings that are located within an approximate distance of 300 feet from the bridge site. The existing left and right structures are three-span continuous steel girder reinforced concrete deck on abutments and piers. The total span length of each bridge is approximately 306 feet. The structures are supported on capped piles at the abutment locations and spread footing at the pier locations and carries IR 75 vehicular traffic over Blanchard River, TR. 88, and CR. 223. The concrete pier columns and walls generally appeared to be in good condition. Surface cracks, light in frequency were observed on exposed abutment surfaces. An asphalt overlay was placed on the top of the concrete deck and appeared to be in good condition. A pothole (spalling) approximately 4X100 feet in size which was patched using a modified asphalt mixture was observed on the top surface of the concrete deck. The concrete pier walls and columns generally appeared to be in good condition. Some surface cracks were observed on both abutments. Surface cracks, very light in frequency, were visible along the bottom deck concrete surface. Rust was observed in very few places on the steel girders. The embankment section at the existing IR 75 mainline bridge approach generally appeared to be in good condition. No visible signs of embankment slope instability were observed and embankment settlement was not observed. The steel girders at the south end of the bridges were damaged because these girders below the deck were getting hit by the moving traffic due to low clearance.

4.0 EXPLORATION

4.1 Historic and Project Exploration Program

Historical records of a geotechnical exploration performed in December 1987 were available for this bridge from the ODOT Geotechnical Documents Management System ftp site. These records consist of Structure Foundation Investigation sheets which included four (4) historic test borings identified as B-1 (B-001-1-87), B-2 (B-002-1-87), B-3 (B-003-1-87), and B-4 (B-004-1-87) from the subsurface geotechnical exploration completed on April 1987. These historic records are included in Appendix B.

In order to explore the subsurface conditions at the project site, drilling, sampling, and field testing operations were performed during August 2013. A total of two (2) project test borings identified as B-046-0-13 and B-047-0-13 were advanced for bridge foundations design purposes. Test boring B-046-0-13 was advanced in the vicinity of the proposed south pier, on the west side of the bridge while test boring B-047-0-13 was advanced in the vicinity of the proposed north pier, on the east side of the bridge. These project test borings were advanced to approximate depths ranging from 21.5 to 27.5 feet below the existing ground surface. No scour samples were obtained from the test borings because bedrock was encountered just below the riverbed.

The test borings were marked in the field by PGI based on boring location plans developed by PGI personnel and after obtaining approval from PB and ODOT 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. A Diedrich D-90 ATV-mounted drilling rig was used to advance the test borings. Both test 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, both test borings were advanced and the rock was sampled using a type NX series core barrel, water method. Both test borings were monitored for the presence of groundwater during drilling operations. All test borings were backfilled with compacted soil cuttings at the end of drilling operations for safety purposes.

Northing and 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 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. A Soil Profile and Boring Location Map are also included in Appendix A.

4.2 Laboratory Testing Program

All soil 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 rock core samples. These tests consisted of Compressive Strength of Rock Core Specimens. 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 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 project test borings B-046-0-13 and B-047-0-13 and historic test borings B-001-1-87 through B-004-1-87. Project test borings B-046-0-13 and B-047-0-13 were advanced through 4.0 inches and 5.0 inches of topsoil, respectively. The subsurface soils encountered in these test borings were primarily cohesive in nature and consisted of both fill materials and natural soils. Test boring B-046-0-13 consisted of fill material and natural soil and B-047-0-13 consisted entirely of fill material above the bedrock. The fill material consisted of silty clay (A-6b) and clay (A-7-6) and was encountered to an approximate depth of 3.5 feet in test boring B-046-0-13. Natural soils encountered above bedrock in test boring B-046-0-13 consisted of plastic sandy silt (A-4a), plastic silt (A-4b), and clay (A-7-6). Bedrock was encountered in project test boring B-046-0-13 at an approximate depth of 16.0 feet below the ground surface while bedrock was encountered in project test boring B-047-0-13 at an approximate depth of 11.0 feet below the ground surface. The laboratory test results indicated that the moisture contents of the tested soil samples obtained from the structure test borings ranged from 8% to 25% and the consistency ranged from "medium stiff" to "hard", but was predominately “medium stiff”.

The subsurface soils encountered in all historic test borings were generally cohesive soils, but non-cohesive soils were also encountered above bedrock in test borings B-001-1-87 and B-002-1-87. The cohesive soils encountered consisted of silt and clay (A-6a), sandy silt (A-4a), plastic silt (A-4b), and silty clay (A-6b), and the non-cohesive soils encountered consisted of non-plastic sandy silt (A-4a) and non-plastic silt (A-4b). Bedrock was encountered in historic boring B-001-1-87 at an approximate depth of 33.5 feet below the ground surface while bedrock was encountered in historic boring B-004-1-87 at an approximate depth of 39.0 feet below the ground surface. Bedrock was encountered in project test boring B-002-1-87 at an approximate depth of 12.5 feet below the ground surface while bedrock was encountered in project test boring B-003-1-87 at an approximate depth of 13.5 feet below the ground surface. The laboratory test results indicated that the moisture contents of the tested soil samples obtained from the structure test borings ranged from 13% to 28% and the consistency ranged from "soft" to "hard", but was predominately “very stiff”. The moisture contents of the tested non-cohesive soils ranged from 12% to 19% and the relative density was ranged from “medium dense” to “dense”.

For specific conditions of the project and historic test borings at various depths, please refer to the individual test boring logs located in Appendix A of this report. For complete moisture contents for project test borings B-046-0-13 and B-047-0-13, refer to the laboratory test results located in Appendix B.

5.2 Bedrock Conditions

Bedrock was encountered in all project and historic test borings. Bedrock encountered in the project test borings was split spoon sampled until little or no penetration or recovery was encountered. Bedrock core samples were then obtained using an NX diamond impregnated core barrel. The coring operations were performed in accordance with the procedure for Diamond Core Drilling for Site Investigations (ASTM D 2113). The core samples consisted of dolomite of the Tymochtee/Greenfield Group. The dolomite was light gray, and slightly weathered. Bedding within the dolomite was generally very thin to medium and was fractured to moderately fractured. No slickensides were observed and the fractures were typically tight and slightly rough. The compressive strength of the core specimens ranged from 6,888 psi in test boring B-046-0-13 to 9,765 psi in test boring B-047-0-13 which characterizes them as “moderately strong” to “strong”, respectively.

The Rock Quality Designation (RQD) for the core samples ranged from 21% to 48%. 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 36 to 44 and is classified as “Poor Rock” to “Fair Rock”. The Rock Mass Rating spreadsheets are included in Appendix B. Refer to the drilling logs in Appendix A and rock core photos in Appendix B for additional bedrock information. Also refer to “Bedrock Descriptions” in Appendix B for general bedrock information.

Table 5.2.1 – Bedrock Information

Boring Number	Rock Core Run No.	Top of Bedrock Elevations (ft)	Rock Core Run Elevations (ft)	Length of Core Run (ft)	Recovery (%)	RQD (%)
B-046-0-13	Run-1	752.8	752.3	4.0	94	44
	Run-2		748.3	7.0	96	21
B-047-0-13	Run-1	754.9	754.4	10.0	100	48
B-001-1-87	Run-1	753.4	751.9	5.0	100	NA
B-002-1-87	Run-1	751.6	751.1	5.0	100	NA
	Run-2		746.1	3.5	94	NA
	Run-3		742.6	4.0	98	NA
B-003-1-87	Run-1	753.9	753.9	5.0	100	NA
B-004-1-87	Run-1	754.7	753.7	5.0	92	NA

Elevations were provided by PB personnel for project test borings, NA – Not Available

Table 5.2.2 –Compressive Strength Test Results of Rock Core Specimens

Boring Number	Specimen Depth (ft)	Rock Type	Unit Weight (pcf)	Compressive Strength (psi)
B-046-0-13	20.9	Dolomite	177.52	6,888
B-047-0-13	16.2	Dolomite	172.12	9,765

5.3 Groundwater Conditions

Groundwater was not encountered during drilling in either of the project test borings. 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. All test borings were backfilled immediately upon completion for safety purposes; therefore an extended groundwater level reading was not taken.

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 IR 75 Mainline Bridge No. HAN-75-1697 L&R over Blanchard River, TR. 88, and CR. 223. Site plans provided by PB personnel indicate that the proposed superstructure design loads will be transferred to the underlying bedrock by means of piles at the rear and forward abutment locations and by means of drilled shafts at the proposed Pier 1 and Pier 2 locations. Elevations of the bottom of the proposed pile caps at the rear and forward abutment locations will be 777.1 and 782.5 feet, respectively. Additional embankment fill, 4.1 feet in thickness at the rear abutment and 1.4 feet in thickness at the forward abutment will be placed on top of existing IR 75 embankment to the raise the existing grade to the proposed profile grade due to vertical realignment of IR 75. The foundation recommendations are provided in accordance with the ODOT *Bridge Design Manual* issued in 2007 using *LRFD Bridge Design Specifications*

6.1 Bridge Foundation Systems

Soil and rock information obtained from project test borings B-046-0-13, B-047-0-13 and historic project test borings B-001-1-87 through B-004-1-87 was used to provide foundation recommendations for this proposed replacement bridge. Historic test borings B-001-1-87 and B-004-1-87 were advanced in the vicinity of the proposed rear and forward abutments, respectively. Project test boring B-046-0-13 and historic test boring B-002-1-87 were advanced in the vicinity of proposed Pier 1 while project test boring B-047-0-13 and historic test boring B-003-1-87 were advanced in the vicinity of proposed Pier 2. As outlined in Section 5.1 - "Subsurface Soil Conditions", the top of bedrock was encountered in the vicinity of bridge site at depths ranging from 11.0 feet to 39.0 feet below the existing ground surface. Bedrock at these boring locations consists of dolomite and was encountered to termination depth in all historic and project test borings. The Rock Mass Rating obtained for the bedrock core samples according to LRFD Table 10.4.6.4-1 varied from 36 to 44 and is considered as “Fair Rock” to “Poor Rock”. Therefore the proposed bridge superstructure loads may be transferred to the underlying bedrock by means of piles or drilled shafts foundations. Since the top of bedrock at the abutment locations was encountered at approximate depths ranging from 33.5 feet to 39.0 feet below the existing pavement, the proposed superstructure loads may be transferred to the underlying bedrock by means of end bearing piles at the abutment locations. Since the top of bedrock at the pier locations was encountered at approximate depths ranging from 11.0 feet to 16.0 feet below the existing ground surface, the proposed superstructure loads may be transferred to the underlying bedrock by means of drilled shafts at the pier locations.

Drill Shaft: Drilled shaft foundation systems may be used to transfer the proposed superstructure loads to the underlying bedrock at the pier locations. Design information provided by PB personnel indicates that the maximum compression factored loads along a vertical axial direction at the Strength and Service Limit will be 1230 kips per shaft and 1025 kips, respectively and lateral loads will control the drilled shaft design at Pier 1 and Pier 2 locations. Drilled shaft foundations can be reinforced concrete columns designed to carry their maximum factored load at the Strength Limit State. The unit side resistance and unit tip resistance were calculated using equations 10.8.3.5.4b-1 and 10.8.3.5.4c-1 (q_p = $2.5q_n$), respectively in the AASHTO LRFD Bridge Design Specifications. Based on these equations, unit side resistance of 10.0 ksf was estimated for the bedrock at test borings B-046-0-13, B-002-1-87, B-047-0-13, and B-003-1-87. Unit tip resistance of 2480 ksf was estimated for the bedrock at structural test borings B-046-0-13, B-002-1-87, B-047-0-13, and B-003-1-87. The nominal shaft tip resistance can be calculated for the selected shaft diameter from the unit tip resistance by multiplying the shaft cross-sectional area. The nominal shaft side resistance can be calculated for the selected shaft diameter and socket length from the unit side resistance by multiplying the shaft length surface area. The tip resistance portion of the factored axial compression resistance is calculated from the nominal shaft tip resistance by multiplying a resistance factor of 0.50. The side resistance portion of the factored axial compression resistance is calculated from the nominal shaft side resistance by multiplying a resistance factor of 0.55. Table 6.1.1 summarizes total factored resistance for the selected diameter of 3.0 feet and socket length of 4.5 feet at the pier boring locations. Side resistance from the soil overburden and upper two (2) feet of the shallow bedrock can be ignored. Based on the factored axial compression resistance for the selected shaft socket length and diameter, the estimated maximum total settlement and differential settlement will not exceed one inch and one half inch, respectively. The shaft factored resistance and settlement calculation spreadsheets are included in Appendix B.

Table 6.1.1 – Estimated Design Parameters for Drilled Shafts

Boring No.	Top of Bedrock Elevation (feet)	Shaft Tip Depth (feet)	Socket Diameter (feet)	Socket Length (feet)	Total Factored Resistance (kips)
Pier 1					
B-046-0-13	752.8±	4.5	3.0	4.5	8765
B-002-1-87	751.6±	4.5	3.0	4.5	8765
Pier 2					
B-047-0-13	754.9±	4.5	3.0	4.5	8765
B-003-1-87	753.9±	4.5	3.0	4.5	8765

Drilled shaft socket diameters less than 36 inches are not recommended. The drilled shafts should be spaced at a minimum of 2.5 shaft diameters on center. If drilled shafts are spaced less than four (4) shaft diameters on center, the group effect between shafts must be evaluated in accordance with Article 10.8.1.2 of the *AASHTO LRFD Bridge Design Specifications*. However, if drilled shafts are socketed into bedrock, group effect between shafts may be neglected. The diameter of bedrock sockets must be 6 inches less than the diameter of the shaft above bedrock elevation in accordance with Section 303.4.3 of the *2007 ODOT Bridge Design Manual*. The drilled shaft supported piers may experience horizontal movement caused by lateral loads and overturning moments. A lateral load analysis should be performed using LPILE computer program by Ensoft or similar computer program for selected shaft diameter and socket length to check whether lateral resistance is adequate to support lateral loads and overturning moments. Table 6.1.2 summarizes the weak rock parameters to perform lateral load analyses by PB personnel. In lateral load analysis, the bedrock socket length used in vertical axial compression capacity analyses should be optimized to find the minimum length necessary to resist the applied lateral load based on serviceability and structural requirements and selected the maximum bedrock socket length between above value and 1.5 times the bedrock socket diameter.

Table 6.1.2 - Estimated Weak Rock Parameters for Lateral Load Analyses

Boring No.	Top Bedrock Elevation(ft)	Effective Unit Weight * (pci)	Youngs's Modulus (psi)	Compressive Strength (psi)	RQD (%)	k_rm
Piers						
B-046-0-13	752.8±	0.059	200000	6888	31	0.0005
B-047-0-13	754.9±	0.059	200000	6888	48	0.0005

*Below the water

Selecting the construction method for installing the drilled shafts is the responsibility of the contractor. Seepage of water into the drilled shaft holes will occur within the soil overburden during installation. If water is encountered in the hole due to seepage, care should be taken to remove all water before placing concrete or the tremie method may be utilized to place the concrete. The successful performance of a drilled shaft depends on the construction method used as well as the quality of workmanship during installation. Therefore, qualified geotechnical personnel should be present during construction for inspection in order to assure the quality of the drilled shafts and to verify that the rock conditions are as per the boring logs. Drilled shaft bottoms should be free of all loose material prior to placement of concrete. For detailed drilled shaft construction, refer to Item 524 – "Drilled Shafts" of the ODOT *Construction and Material Specifications* issued in January 2013.

H-Piles: Driven piles consisting of end bearing steel piles may be used to transfer the proposed superstructure loads to the underlying bedrock at the abutment locations. The end bearing piles must be driven through the existing embankment. Design information provided by PB personnel indicates that the maximum factored loads along a vertical direction will be 179 kips per pile. The end bearing piles must be steel H-piles driven to refusal on the underlying dolomite bedrock. H-pile sizes HP-10X42 may be selected for abutment foundation design. The total factored load on each HP-10X42 pile should not exceed the corresponding maximum structural resistance of 310 kips as per the ODOT *Bridge Design Manual* Section 202.2.3.2.a. Note that the above mentioned structural resistance values can be used only on the axial loaded piles that have a negligible bending moment. The estimated pile parameters for end bearing piles at each boring location are summarized in Table 6.1.3. The pile cut-off elevations at the abutments were extracted from the final structure site plan provided by PB personnel.

Table 6.1.3 - Estimated Design Parameters for H-Piles

Boring No.	Pile Cut-off Elevation (ft)	Pile Tip Elevation (ft)	Estimated Effective Pile Length (ft)	Pile Type	Pile Size	Maximum Factored Structural Resistance/pile
Abutments						
B-001-1-87	778.1	751.9	30.0	H-Pile	10X42	310 kips
B-004-1-87	783.5	753.7	30.0	H-Pile	10X42	310 kips

It is recommended that the piles be spaced a minimum of three (3) pile diameters on center. At the rear abutment, consolidation of the foundation soils caused by construction of the proposed embankment will be more than 0.6 inches. Therefore negative skin friction will develop along the pile section between the bottom of the proposed embankment and the top of bedrock due to the consolidation of the foundation soils caused by construction of the proposed embankment. The piles should be designed in accordance with section 202.2.3.2.c – "Down Drag Forces on Piles" of the *ODOT Bridge Design Manual* issued in January 2007. Nominal down drag load of 65 kips per pile may be assumed for pile sizes HP-10X42 at the rear abutment location. Pre-boring of holes may be required through compacted embankment at the pile locations in order to drive the piles to refusal on bedrock. If required, the depth of the hole should be a maximum of 10 feet below the bottom of the abutment pile cap at the rear and forward abutment locations. Pre-drilling of holes should be performed in accordance with ODOT Item 507.11 – *Prebored Holes*. 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.

The pile supported abutments may experience horizontal movement caused by lateral loads. In order to prevent damage caused by lateral loads, the piles should be installed in accordance with Section 303.4.2.4 - "Piles Battered", of the 2007 *ODOT Bridge Design Manual.* During pile driving operations, damage could be caused to existing buildings within approximately 500 feet of the proposed pile driving location due to induced vibrations. Therefore, pile hammer, and pile installation techniques should be selected in such a way to minimize the induced vibrations. The public often tends to claim that their building(s) have been damaged due to pile driving operations even though the damage was caused by something else. Therefore, PGI recommends performing a structure survey before the pile driving and monitoring vibrations during pile driving.

6.2 Lateral Earth Pressures and Abutment Drainage

The bridge abutments must be designed to resist lateral pressures exerted by both dead and live loads. The active lateral earth pressures exerted behind the bridge abutments may be approximated by an equivalent fluid weighing 40 pcf above the water table and 80 pcf below the water table; provided that level ground exists behind the abutments and that no surcharge loads are placed behind the walls. Freely draining material must be placed behind the culvert wing walls 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 these 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 do not migrate into the voids of the porous backfill.

6.3 Approach Slab Design Parameters

During construction of the project, the proposed pavement will be constructed on fill materials. Therefore, the soil parameters derived from the fill materials must be used for the pavement design. Representative samples of proposed fill materials should be tested and CBR values should be derived prior to construction.

6.4 Groundwater Management

Based on the groundwater conditions described in Section 5.3, "Groundwater Conditions," groundwater was not encountered during drilling at the project boring locations. Groundwater level at the site will be controlled by the adjacent Blanchard River water level. If the bottom depth of the excavation for the pier foundations extends below the water level of Blanchard River, water infiltration is anticipated. Low to moderate volume pumping or dewatering may required use of the sump pumps at the pier locations. It must be noted that the groundwater levels during construction may vary due to seasonal fluctuations, and groundwater may occur where not encountered previously.

6.5 Earthwork and Construction Monitoring

All excavation and backfilling operations should be conducted in accordance with ODOT's "Construction and Materials Specifications," Item 503 - "Excavation for Structures" issued in January 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). 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 above the water table and a three (3) horizontal to one (1) vertical slope for excavation below the water table. Prior to any backfill placement against the abutments, exposed subgrade under the approach slabs should be subjected to inspection under the direction of competent geotechnical personnel. Any areas that exhibit an unacceptable subgrade reaction, local soft/loose soil zones, and areas of unacceptable material must be undercut to a minimum depth of two (2) feet below the elevation of the soil being inspected. All removed soils should be replaced with compacted, engineered fill materials.

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

7.0 LIMITATIONS

This report is subject to the following conditions and limitations:

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

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

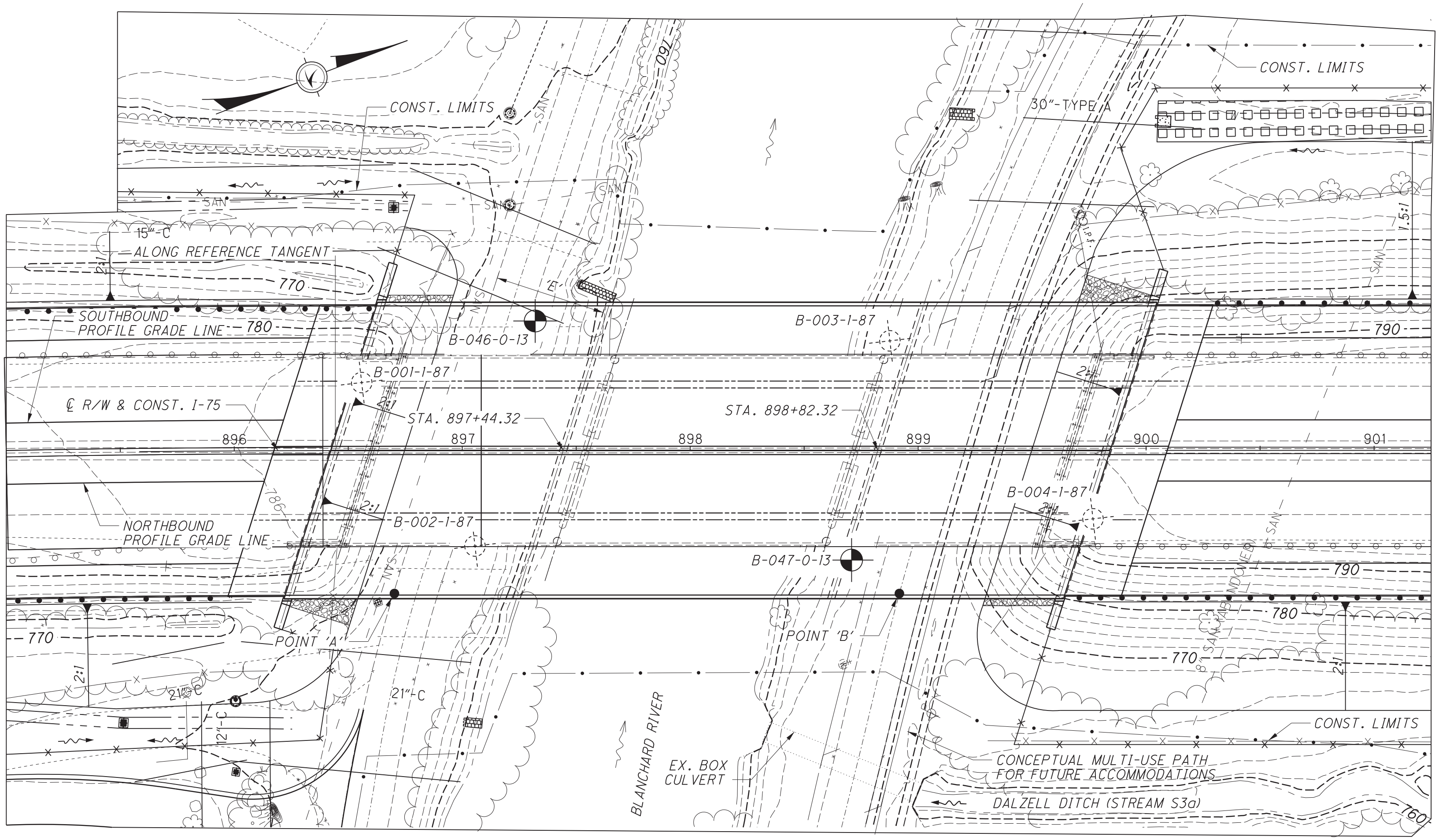
7.3 All earthwork and foundation construction must be performed under the supervision of a Professional Engineer in accordance with ODOT Construction Specifications.

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

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

APPENDICES

APPENDIX A



BORING LOCATION PLAN

I-75 BRIDGE NO. HAN-075-1697 L/R OVER BLANCHARD RIVER, TR. 88, AND CR. 223

HAN-75-14.39

PID No. 87005

PROJECT: HAN-75-14.39	DRILLING FIRM / OPERATOR: B-M / JOSH DEAN	DRILL RIG: DIEDRICH D-90 ATV	STATION / OFFSET: 897+32.0, 56.8' LT	EXPLORATION ID B-046-0-13
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: PGI / W. NAJJAR	HAMMER: DIEDRICH AUTOMATIC	ALIGNMENT: IR-75 BASELINE	
PID: 87005 BR ID: HAN-75-1697	DRILLING METHOD: 3.25" HSA / NX	CALIBRATION DATE: 9/18/12	ELEVATION: 768.8 (MSL) EOB: 27.5 ft.	PAGE 1 OF 1
START: 8/2/13 END: 8/2/13	SAMPLING METHOD: SPT/NX	ENERGY RATIO (%): 80.2	COORD: 41.051691190, 83.672110910	

MATERIAL DESCRIPTION AND NOTES	ELEV. 768.8	DEPTHS
TOPSOIL (4' THICK)	768.5	
MEDIUM STIFF, BROWN, **SILTY CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, FILL, DAMP	765.3	
MEDIUM STIFF, BROWN, **CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, MOIST	762.8	
VERY STIFF TO HARD, BROWN AND GRAY, **SANDY SILT**, SOME CLAY, TRACE STONE FRAGMENTS, MOIST TO DAMP @8.5'; HARD, DAMP @11.0'; NO SPLIT SPOON RECOVERY	755.3	
HARD, GRAY, **PLASTIC SILT**, SOME CLAY, TRACE STONE FRAGMENTS, DAMP @16.0'; NO SPLIT SPOON RECOVERY	752.8	TR
POSSIBLE DOLOMITE BEDROCK NOTE: AUGERED TO 16.5', BEGAN CORING BEDROCK	752.3	
DOLOMITE, LIGHT GRAY, SLIGHTLY WEATHERED, MODERATELY STRONG, VERY THIN TO THIN BEDDED, FRACTURED TO MODERATELY FRACTURED WITH FEW ANGULAR FRACTURES, TIGHT APERTURE WIDTH, SLIGHTLY ROUGH. NOTE: VUGGY FROM 16.5' TO 18.0'.	748.3	
DOLOMITE, LIGHT GRAY, SLIGHTLY WEATHERED, MODERATELY STRONG, VERY THIN TO THIN BEDDED, FRACTURED TO MODERATELY FRACTURED WITH FEW ANGULAR FRACTURES, TIGHT APERTURE WIDTH, SLIGHTLY ROUGH. @20.9'; COMPRESSIVE STRENGTH = 6,888 PSI	741.3	EOB

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
2 3 3	8	78	SS-1	2.00	-	-	-	-	-	-	-	-	18	A-6b (V)	
2 2 3	7	100	SS-2	2.00	-	-	-	-	-	-	-	-	25	A-7-6 (V)	
3 7 10	23	94	SS-3	4.00	-	-	-	-	-	-	-	-	16	A-4a (V)	
8 10 13	31	100	SS-4	4.5+	-	-	-	-	-	-	-	-	13	A-4a (V)	
6 6 8	19	56	SS-5	-	-	-	-	-	-	-	-	-	-		
16 19 17	48	89	SS-6	4.5+	-	-	-	-	-	-	-	-	8	A-4b (V)	
50/1"	-	0	SS-7	-	-	-	-	-	-	-	-	-	-		
44		94	NX-1											CORE	
21		96	NX-2											CORE	

NOTES: NO GROUNDWATER WAS ENCOUNTERED DURING DRILLING AND NO READING WAS TAKEN UPON COMPLETION OF DRILLING DUE TO ROCK CORING OPERATIONS.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH SOIL CUTTINGS

PROJECT: HAN-75-14.39	DRILLING FIRM / OPERATOR: B-M / JOSH DEAN	DRILL RIG: DIEDRICH D-90 ATV	STATION / OFFSET: 898+68.8, 52.4' RT	EXPLORATION ID B-047-0-13
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: PGI / W. NAJJAR	HAMMER: DIEDRICH AUTOMATIC	ALIGNMENT: IR-75 BASELINE	
PID: 87005 BR ID: HAN-75-1697	DRILLING METHOD: 3.25" HSA / NX	CALIBRATION DATE: 9/18/12	ELEVATION: 765.9 (MSL) EOB: 21.5 ft.	PAGE 1 OF 1
START: 8/2/13 END: 8/2/13	SAMPLING METHOD: SPT/NX	ENERGY RATIO (%): 80.2	COORD: 41.051939580, 83.671567690	

MATERIAL DESCRIPTION AND NOTES	ELEV. 765.9	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 (5" THICK)	765.5																	
MEDIUM STIFF, BROWN, **SILTY CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, FILL, MOIST		1–2	3 3 3	8	89	SS-1	2.75	-	-	-	-	-	-	-	-	19	A-6b (V)	
		4–5	2 2 3	7	67	SS-2	2.50	-	-	-	-	-	-	-	-	21	A-6b (V)	
	759.9																	
MEDIUM STIFF, BROWN, **CLAY**, LITTLE SAND, TRACE STONE FRAGMENTS, FILL, MOIST		6–7	2 2 3	7	94	SS-3	2.00	-	-	-	-	-	-	-	-	25	A-7-6 (V)	
		9–10	1 2 3	7	94	SS-4	1.50	-	-	-	-	-	-	-	-	25	A-7-6 (V)	
@11.0'; NO SPLIT SPOON RECOVERY	754.9	TR 11	50/1"	-	0	SS-5	-	-	-	-	-	-	-	-	-	-		
POSSIBLE DOLOMITE BEDROCK NOTE: AUGERED TO 11.5', BEGAN CORING BEDROCK	754.4																	
DOLOMITE, LIGHT GRAY, SLIGHTLY WEATHERED, STRONG, VERY THIN TO THIN BEDDED, FRACTURED TO MODERATELY FRACTURED, TIGHT APERTURE WIDTH, SLIGHTLY ROUGH. NOTE: VUGGY FROM 11.5' TO 18.0'. @16.2'; COMPRESSIVE STRENGTH = 9,765 PSI		12–21	48		100	NX-1											CORE	
	744.4	EOB																

NOTES: NO GROUNDWATER WAS ENCOUNTERED DURING DRILLING AND NO READING WAS TAKEN UPON COMPLETION OF DRILLING DUE TO ROCK CORING OPERATIONS.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH SOIL CUTTINGS

MICROFILMED

HANCOCK COUNTY
HAN-75-14.39

8/10

Historic Boring B-001-1-87
896+55.64, 29' LT

LOG OF BORING

Date Completed 5/6/87
Boring No. B-1
Station & Offset 897+50, 29' LT. (REAR ABUTMENT)
Surface Elev. 786.9'

Elev.	Depth	Std. Pen. (N)	Rec. ft.	Loss ft.	Description	Sample No.	% Agg.	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	SHTL Class.
786.9	0														
786.2		AUGERED			ASPHALT										VISUAL
784.2	2	AUGERED			LARGE GRAVEL (DRILLER'S DESCRIPTION)	-	-	-	-	-	-	-	-	-	VISUAL
781.9	6	1/2/2			BROWN SANDY CLAY	1	9	6	17	24	44	32	15	22	A-6a
776.9	12	2/5/9			BROWN AND GRAY SANDY CLAY	2	8	5	17	24	46	32	14	18	A-6a
771.9	16	6/8/13			BROWN AND GRAY SILTY CLAY	3	0	4	14	26	56	35	16	19	A-6b
766.9	22	11/15/24			BROWN SILT	4	0	1	16	57	26	NP	NP	23	A-4b
761.9	26	11/14/19			BROWN SANDY GRAVELLY SILT	5	40	5	10	27	18	18	4	12	A-4a
756.9	32	11/15/45			GRAY GRAVELLY SILT	6	17	5	7	37	34	19	1	14	A-4a
753.4	34				TOP OF ROCK; BROKEN DOLOMITIC LIMESTONE	-	-	-	-	-	-	-	-	-	VISUAL
751.9	36–38		5.0	0.0	DOLOMITIC-LIMESTONE, GRAY, HARD, DENSE, SOMEWHAT LEACHED, SLIGHTLY VUGGY, BROKEN AND JOINTED. NO CORE LOSS.										
746.9	40				BOTTOM OF BORING										

Historic Boring B-002-1-87
897+05.60, 42' RT

LOG OF BORING

Date Completed 5/6/87
Boring No. B-2
Station & Offset 898+00, 42' RT. (SOUTH PIER)
Surface Elev. 765.1'

Elev.	Depth	Std. Pen. (N)	Rec. ft.	Loss ft.	Description	Sample No.	% Agg.	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	SHTL Class.
765.1	0														
764.1		AUGERED			TOPSOIL	-	-	-	-	-	-	-	-	-	VISUAL
760.1	6	6/10/13			BROWN SILT	1	0	1	4	61	34	26	2	19	A-4b
755.1	12	7/12/30			GRAY SANDY SILT	2	9	4	7	40	40	20	3	14	A-4a
751.6	14				TOP OF ROCK; BROKEN DOLOMITIC LIMESTONE	-	-	-	-	-	-	-	-	-	VISUAL
751.1	16–18		5.0	0.0											
	20–22		3.2	0.3	DOLOMITIC-LIMESTONE, GRAY, HARD, DENSE, SOMEWHAT LEACHED, SLIGHTLY VUGGY, BROKEN AND JOINTED. CORE LOSS 3%.										
	24–26		3.9	0.1											
738.6					BOTTOM OF BORING										

Historic Boring B-003-1-87
898+87.60, 48' LT

LOG OF BORING

Date Started 4/15/87
Date Completed 4/15/87
Boring No. B-3
Sampler Type SS Dia. 1 3/8"
Casing: Length ______ Dia. ______
Station & Offset 899+82', 48' LT. (NORTH PIER)
Water Elev. ______
Surface Elev. 766.4'

Elev.	Depth	Std. Pen. (N)	Rec. ft.	Loss ft.	Description	Sample No.	% Agg.	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	SHTL Class.
766.4	0														
766.0		AUGERED			SOD AND TOPSOIL	-	-	-	-	-	-	-	-	-	VISUAL
763.9	4	2/2/4			BROWN SANDY CLAY	1	0	1	36	29	34	28	11	24	A-6a
761.4	6	3/4/6			BROWN SANDY CLAY	2	0	0	21	40	39	33	14	23	A-6a
758.9	8	2/2/4			BROWN SILTY CLAY	3	0	1	16	42	41	35	16	27	A-6b
756.4	10	2/2/7			BROWN SANDY SILT	4	11	10	34	24	21	23	6	28	A-4a
753.9	12				TOP OF ROCK										
	14–16		5.0	0.0	DOLOMITIC-LIMESTONE, GRAY, HARD, DENSE, SOMEWHAT LEACHED, EXTREMELY VUGGY AND VERY BADLY BROKEN AND JOINTED. NO CORE LOSS.										
748.9	18				BOTTOM OF BORING										

Historic Boring B-004-1-87
899+72.60, 30' RT

LOG OF BORING

Date Started 4/28/87
Date Completed 4/29/87
Boring No. B-4
Sampler Type SS Dia. 1 3/8"
Casing: Length ______ Dia. ______
Station & Offset 900+67, 30' RT. (FORWARD ABUTMENT)
Water Elev. ______
Surface Elev. 793.7'

Elev.	Depth	Std. Pen. (N)	Rec. ft.	Loss ft.	Description	Sample No.	% Agg.	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	SHTL Class.
793.7	0														
793.2		AUGERED			ASPHALT	-	-	-	-	-	-	-	-	-	VISUAL
788.7	6	4/5/9			BROWN SANDY CLAY	1	6	5	38	15	36	28	13	16	A-6a
783.7	12	5/7/12			BROWN CLAYEY SILT	2	4	0	1	60	35	23	9	19	A-4b
778.7	16	5/8/10			BROWN SANDY CLAY	3	6	6	22	26	40	28	12	13	A-6a
773.7	22	8/12/12			GRAY AND BROWN SANDY SILT	4	4	5	40	20	31	21	7	15	A-4a
768.7	26	18/38/28			BROWN AND GRAY SANDY GRAVELLY CLAY W/BLDRS.	5	34	4	19	16	27	26	11	15	A-6a
763.7	32	4/8/10			BROWN SILT AND CLAY	6	0	0	14	17	69	35	15	23	A-6a
758.7	36	2/2/3			BROWN AND GRAY SANDY CLAY	7	0	0	20	40	40	34	16	26	A-6b
754.7					TOP OF ROCK										
753.7	40				BROKEN DOLOMITIC LIMESTONE	-	-	-	-	-	-	-	-	-	VISUAL
	42–44		4.6	0.4	DOLOMITIC LIMESTONE, GRAY, HARD, DENSE, SOMEWHAT LEACHED, EXTREMELY VUGGY, AND VERY BADLY BROKEN AND JOINTED. CORE LOSS 8%.										
744.7					BOTTOM OF BORING										

SFN-3203034

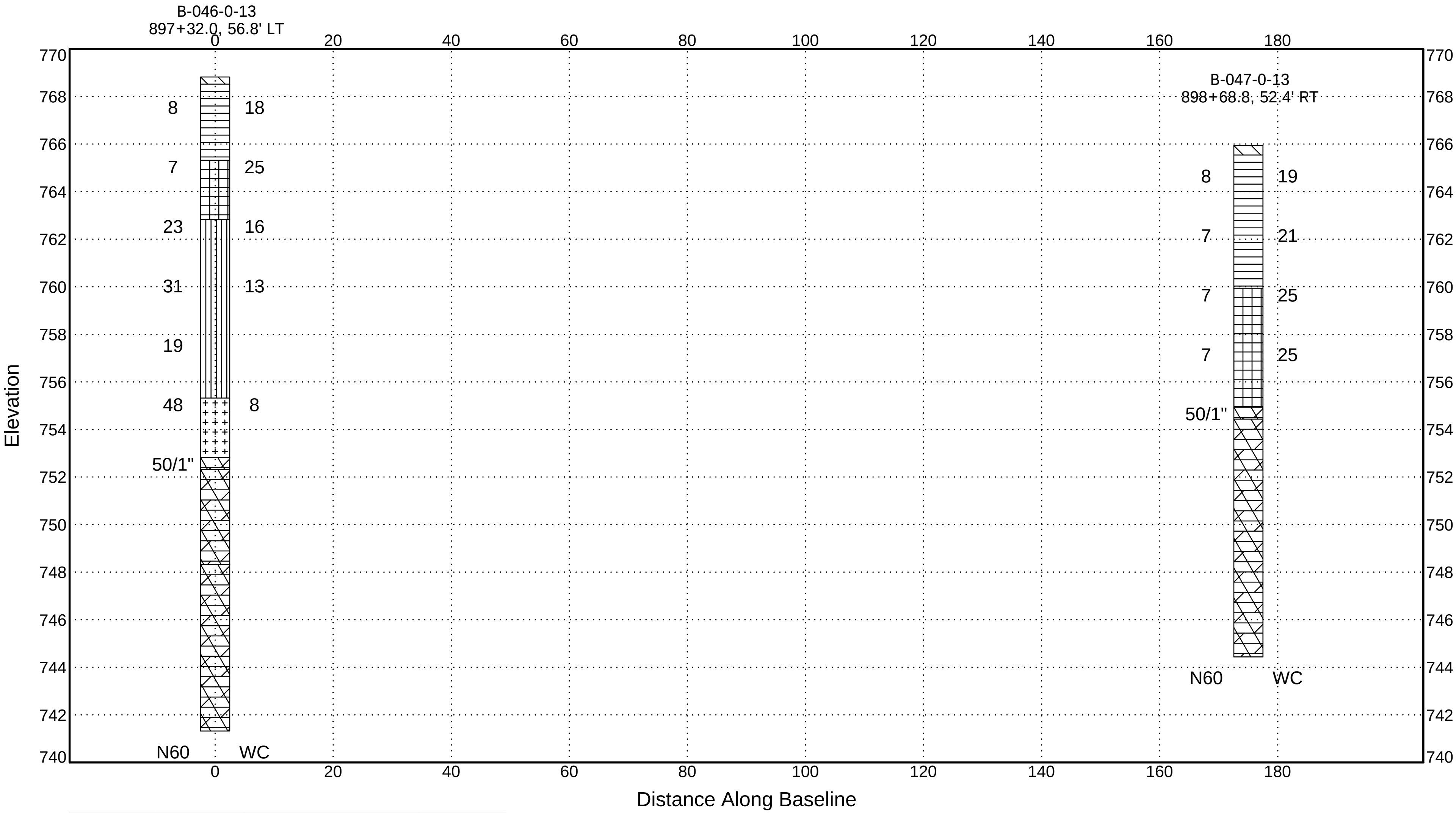
Revised 7/24/87

OHIO DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS - TESTING LABORATORY
1600 WEST BROAD STREET COLUMBUS, OHIO 43223

STRUCTURE FOUNDATION INVESTIGATION
BRIDGE NO. HAN-75-1700
OVER RIVER RD. AND BLANCHARD RIVER
SEC. HAN-75-14.39

BORING DATA

TYPED BY	CHECKED BY	REVIEWED BY	DATE
S.M.G.	L.N.L.	R.D.R	5/27/87



Borehole	North	East	Elev.	Depth
B-046-0-13	506769	1645240	768.8	27.5
B-047-0-13	506857	1645391	765.9	21.5

DISTANCES:
Beginning 0
Ending 180
VIEWING ANGLES (degrees):
Horizontal 0.0
Vertical 0.0

Position	North	East
Left, Front	506769	1645240
Right, Front	506860	1645396
Left, Back	506769	1645240
Right, Back	506860	1645396

SOIL BORINGS PROFILE
BRIDGE NO. HAN-75-1697 L & R

HAN-75-14.39

FINDLAY, HANCOCK COUNTY, OHIO

PROJECT #	DATE	PLATE
87005	Jul 14	1

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-046-0-13	SS-1	1.0	18											BROWN SILTY CLAY, LITTLE SAND, TRACE STONE FRAGMENTS (FILL)	A-6b (V)
B-046-0-13	SS-2	3.5	25											BROWN CLAY, LITTLE SAND, TRACE STONE FRAGMENTS	A-7-6 (V)
B-046-0-13	SS-3	6.0	16											BROWN AND GRAY SANDY SILT, SOME CLAY, TRACE STONE FRAGMENTS	A-4a (V)
B-046-0-13	SS-4	8.5	13											BROWN AND GRAY SANDY SILT, SOME CLAY, TRACE STONE FRAGMENTS	A-4a (V)
B-046-0-13		11.0												NO RECOVERY	
B-046-0-13	SS-5	13.5	8											GRAY PLASTIC SILT, SOME CLAY, TRACE STONE FRAGMENTS	A-4b (V)
B-047-0-13	SS-1	1.0	19											BROWN SILTY CLAY, LITTLE SAND, TRACE STONE FRAGMENTS (FILL)	A-6b (V)
B-047-0-13	SS-2	3.5	21											BROWN SILTY CLAY, LITTLE SAND, TRACE STONE FRAGMENTS (FILL)	A-6b (V)
B-047-0-13	SS-3	6.0	25											BROWN CLAY, LITTLE SAND, TRACE STONE FRAGMENTS (FILL)	A-7-6 (V)
B-047-0-13	SS-4	8.5	25											BROWN CLAY, LITTLE SAND, TRACE STONE FRAGMENTS (FILL)	A-7-6 (V)

Pro Geotech, Inc.

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

Summary of Laboratory Results

Client: PARSONS BRINKERHOFF
Project: HAN-75-14.39 - BRIDGE NO.: HAN-75-1697
Location: FINDLAY, HANCOCK COUNTY, OHIO
PID Number: 87005

Pro Geotech, Inc.

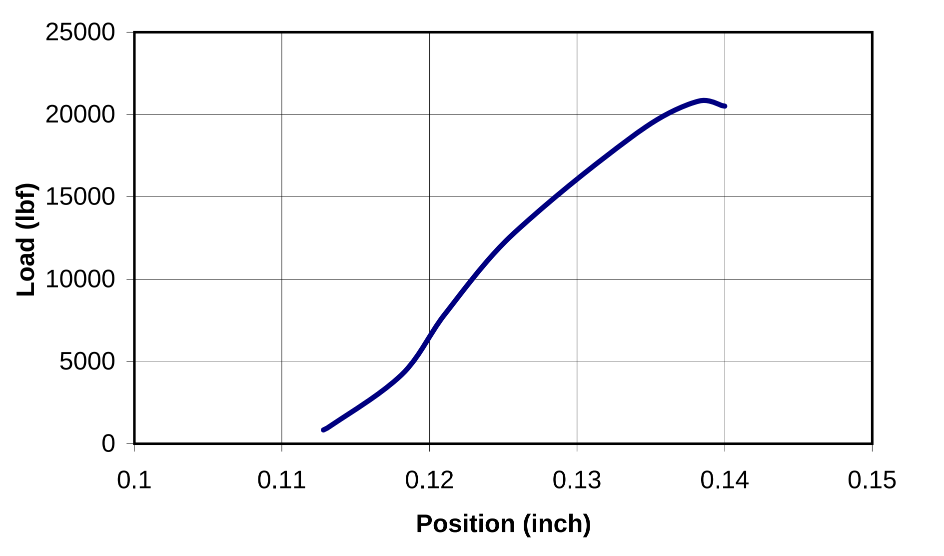
Compressive Strength of Rock
ASTM D 7012

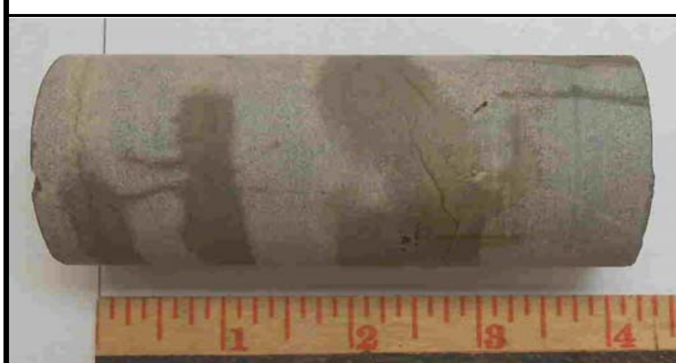
PROJECT	HAN-75-14.39	PGI PROJECT NO.	G13011G	DATE	9/16/2013
STRUCTURE	IR 75 Bridge No. HAN-75-1697 over Blanchard River				
BORING NUMBER	B-046-0-13	TOP DEPTH (FT)	20.9	BOTTOM DEPTH (FT)	21.2
SAMPLE NUMBER	NX-2	DISTRICT	1	PID NO.	87005
COUNTY	HANCOCK	ROUTE	75	SECTION	1697
STATION	897+32.0	OFFSET	56.8'	OFFSET DIRECTION	LT

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

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

MAXIMUM LOAD (LBS)	20789
COMPRESSIVE STRENGTH (PSI)	6888
TIME OF TEST (MINUTES)	3:20
LOADING DIRECTION	PERPENDICULAR TO BEDDING
TECHNICIAN	FBUSHER





BEFORE TESTING

AFTER FAILURE

Pro Geotech, Inc.

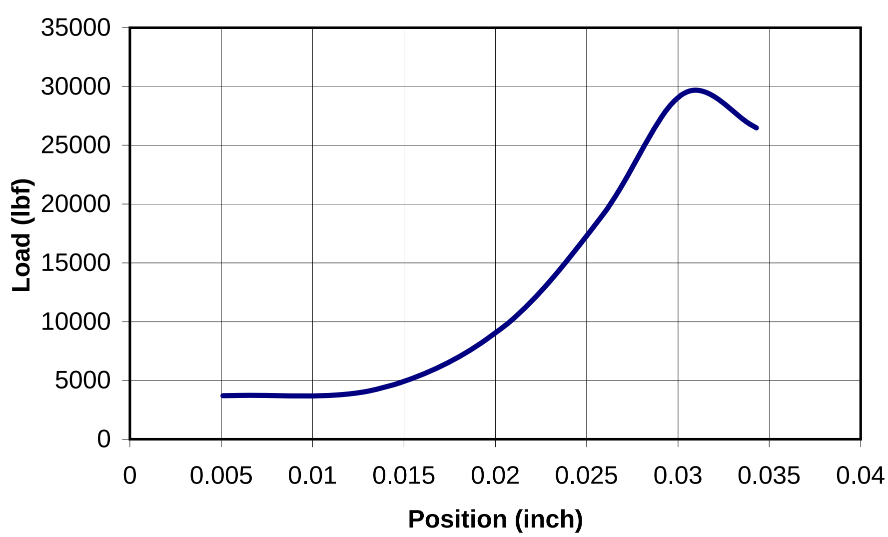
Compressive Strength of Rock
ASTM D 7012

PROJECT	HAN-75-14.39	PGI PROJECT NO.	G13011G	DATE	9/6/2013
STRUCTURE	IR 75 Bridge No. HAN-75-1697 over Blanchard River				
BORING NUMBER	B-047-0-13	TOP DEPTH (FT)	16.2	BOTTOM DEPTH (FT)	16.5
SAMPLE NUMBER	NX-1	DISTRICT	1	PID NO.	87005
COUNTY	HANCOCK	ROUTE	75	SECTION	1697
STATION	898+68.8	OFFSET	52.4'	OFFSET DIRECTION	RT

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

MEASUREMENT	LENGTH (INCH)	DIAMETER (INCH)		
1	4.012	1.958	LENGTH/DIAMETER	2.05
2	4.009	1.957	CORRECTION FACTOR	1.00
3	4.021	1.959	AREA (SQ. INCH)	3.011
			MASS (GRAMS)	546.08
AVERAGE	4.014	1.958	UNIT WEIGHT (LBS/FT^3)	172.12

MAXIMUM LOAD (LBS)	29403
COMPRESSIVE STRENGTH (PSI)	9765
TIME OF TEST (MINUTES)	2:45
LOADING DIRECTION	PERPENDICULAR TO BEDDING
TECHNICIAN	FBUSHER



BEFORE TESTING

AFTER FAILURE

COMPANY: PGI DRILLED BY: B-M
PROJECT: HAN-75-14.39
BRIDGE NO.: HAN-75-1697 over Blanchard River
BORING: B-046-0-13 BOX 1/1
DATE of CORING: 8/2/13
RUN-1: 16.5' - 20.5' RUN-2: 20.5' - 27.5'
REC: 94% RQD: 44% REC: 96% RQD: 21%

COMPANY: PGI DRILLED BY: B-M
PROJECT: HAN-75-14.39
BRIDGE NO.: HAN-75-1697 over Blanchard River
BORING: B-047-0-13 BOX 1/1
DATE of CORING: 8/2/13
RUN-1: 11.5' - 21.5'
REC: 100% RQD: 48%

ROCK MASS RATING From Table 10.4.6.4-1	
Project: HAN-75-14.39	**Project No.:** G13011G
Structure: IR-75 Mainline Bridge No. HAN-75-1697 L&R over Blanchard River	
Boring No.: B-046-0-13	**Substructure Unit:** South Pier
Strength of Intact Rock Material	
Uniaxial Compressive Strength	992 ksf
Relative Rating	4
Drill Core Quality RQD	
RQD	31%
Relative Rating	4
Joint Conditions	
Spacing of Joints	2" to 1'
Relative Rating	7
Conditions of Joints	Slightly Rough Surfaces, Separation < 0.05", and Hard Joint Wall
Relative Rating	17
Ground water Conditions	
Relative Rating	4
Strike & Dip Orientation of Joint	
Relative Rating	0
Total Mass Rating	36
Class No	III
Description	Poor Rock
Boring No.: B-047-0-13	**Substructure Unit:** North Pier
Strength of Intact Rock Material	
Uniaxial Compressive Strength	1406 ksf
Relative Rating	5
Drill Core Quality RQD	
RQD	48%
Relative Rating	8
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
Ground water Conditions	
Relative Rating	4
Strike & Dip Orientation of Joint	
Relative Rating	0
Total Mass Rating	44
Class No	III
Description	Fair Rock

EMBANKMENT SETTLEMENT ANALYSIS - Rear Abutment						
Project:	HAN-75-14.39 - Bridge No. HAN-68-1697	**Project #**	G13011G	**Test Boring #**	B-001-1-87	
Type of Foundation	Compression Index (Cc) (From Lab Test)			Depth of Ground Water Level below footing (feet)	33.5	
Shallow Foundation (Strip)	Recompression Index (Cr) (From Lab Test)			Unit Weight of Water (pcf)	62.4	
Length =	Depth of Footing (D_f) below ground (feet)	4.1		Specific Gravity of Soil Solids (G)		
Width = 160.0'	Applied Design Pressure (psf)	550		Unit Weight of Soil above the base of foundation (pcf)	125	

Depth Below the Foundation (Z)	**AVERAGE PROPERTIES**		**CALCULATIONS**		**Total**
D_f=0.0' & Z=0.0'	Thickness of Layer (feet)	16	OB Pressure at the top Layer(psf)	0	**Setlement**
	Ave. Corrected SPT Value (N_{60})	9	OB Pressure at the center Layer (psf)	960	**(inches)**
	Specific Gravity of Soil Solids (G)	2.7	Excess Pressure At Center Due to appliedLoad	524	
(above the Water Table)	Moisture content (%)	20	Compression Index (C_c)	0.2	
Z=8.0' (At Centre of Layer)	Liquid Limit (%)	32	Recompression Index (C_r)	0.02	0.02
	Plastic Limit (%)	19	Initial Void Ratio (e_0)	0.68	
	Plasticity Index (%)	13	Settlement due to compression (inches)	4.31	
	Unit Weight of soil (pcf)	120	Settlement due to recompression (inches)	0.43	**0.43**
D_f=16.0' & Z=16.0'	Submerged Unit Weight of Soil (pcf)		OB Pressure at the bottom Layer (psf)	1920	
D_f=16.0' & Z=16.0'	Thickness of Layer (feet)	5	OB Pressure at the top Layer(psf)	1920	**Setlement**
	Ave. Corrected SPT Value (N_{60})	21	OB Pressure at the center Layer (psf)	2233	**(inches)**
	Specific Gravity of Soil Solids (G)	2.75	Excess Pressure At Center Due to appliedLoad	493	
(above the Water Table)	Moisture content (%)	19	Compression Index (C_c)	0.19	
Z=18.5' (At Centre of Layer)	Liquid Limit (%)	35	Recompression Index (C_r)	0.019	0.019
	Plastic Limit (%)	19	Initial Void Ratio (e_0)	0.63	
	Plasticity Index (%)	16	Settlement due to compression (inches)	0.60	
	Unit Weight of soil (pcf)	125	Settlement due to recompression (inches)	0.06	**0.06**
D_f=21.0' & Z=21.0'	Submerged Unit Weight of Soil (pcf)		OB Pressure at the bottom Layer (psf)	2545	
D_f=21.0' & Z=21.0'	Thickness of Layer (feet)	4	OB Pressure at the top Layer(psf)	2545	**Setlement**
	Ave. Corrected SPT Value (N_{60})	39	OB Pressure at the center Layer (psf)	2815	**(inches)**
	Specific Gravity of Soil Solids (G)	2.65	Excess Pressure At Center Due to appliedLoad	481	
(above the Water Table)	Moisture content (%)	23	Bearing Capacity Index (C)	70	
Z=23.0' (At Centre of Layer)	Liquid Limit (%)	NP	Immediate Settlement in Foundation Soil (inches)	0.05	**0.05**
	Plastic Limit (%)	NP	Initial Void Ratio (e_0)	0.51	
	Plasticity Index (%)	NP			
	Unit Weight of soil (pcf)	135			
D_f=25.0' & Z=25.0'	Submerged Unit Weight of Soil (pcf)		OB Pressure at the bottom Layer (psf)	3085	

Project:	HAN-75-14.39 - Bridge No. HAN-68-1697		Project #	G13011G	Test Boring #	B-001-1-87
D_f=25.0' & Z=25.0'	Thickness of Layer (feet)	12.5	OB Pressure at the top Layer(psf)		3085	**Setlement**
	Ave. Corrected SPT Value (N_{60})	45	OB Pressure at the center Layer (psf)		3960	**(inches)**
	Specific Gravity of Soil Solids (G)	2.65	Excess Pressure At Center Due to appliedLoad		465	
(above the Water Table)	Moisture content (%)	13	Bearing Capacity Index (C)		110	
Z=29.25' (At Centre of Layer)	Liquid Limit (%)	NP	Immediate Settlement in Foundation Soil (inches)		0.07	**0.07**
	Plastic Limit (%)	NP	Initial Void Ratio (e_0)		0.33	
	Plasticity Index (%)	NP				
	Unit Weight of soil (pcf)	140				
D_f=33.5' & Z=33.5'	Submerged Unit Weight of Soil (pcf)		OB Pressure at the bottom Layer (psf)		4835	

Total Settlement:	**0.61**
Consolidation Settlement:	**0.49**
Immediate Settlement:	**0.12**

HAN-75-14.39 - BRIDGE NO. HAN-68-1617 REAR MSE WALL
Stress Distribution using 2 V : 1 H Slope Method for Strip Footing

Boring No.: B-001-1-87

Width of the footing B (feet)	**160**	**Applied Design Pressure (psf)**				**550**			
Depth (Z) below the footing (feet)	8	18.5	23	29.25					
Vertical Stress Intensity at Z q (psf)	524	493	481	465					

Estimation of Drilled Shaft Resistence and Settlement in Jointed Rock

Project: HAN-75-14.39 - HAN-75-1697 **Project No.:** G13011G

Structure: IR-75 Mainline Bridge over Blanchard River

Boring No.: B-046-0-13 **Substructure Unit:** Pier 1 & Pier 2

Unit Side Resistence (q_s): $0.65*(\text{Reduction Factor } \alpha_E)*P_a*\text{Sqrt}(q_u/P_a) < 7.8*Pa*\text{Sqrt}(f'_c/P_a)$ (Eq. 10.8.3.5.4b-1)

Uniaxial Comp.Strength of Intact Rock, q_u (ksf): 992 Atmospheric Pressure P_a(ksf): 2.12

Reduction Factor α_E: 0.55 (Table 10.8.3.5.4b-1) Concrete Compressive Strength f'_c(ksf): 576

Unit Side Resistence, qs (ksf): 12.49 <272.57 ksf (From Eq 10.8.3.5.4b-1

Unit Side Resistence (ksf): 10.00

Unit Tip Resistence (q_p): (Sq.root(s)+Sq.root(m*Sq.root(s)+s))*qu (Eq. 10.8.3.5.4c-2)

Fractured Rock Mass Parameters "s" and "m" m = s =

(From Table 10.4.6.4-4)

Unit Tip Resistence, q_p (ksf):

Unit Tip Resistence (q_p): 2.5*qu (Eq. 10.8.3.5.4c-1)

Unit Tip Resistence, q_p (ksf): 2480

Calculation of Nominal Resistence of Side and Tip

Shaft Socket Diameter, Br (feet):	**3**	**4**	**5**	**6**
Length of Socket, Dr (feet) :	4.5	6	7.5	9
Perimeter Area of Socket As (Sq. ft)	23.56	50.27	86.39	131.95
Cross-Sectional Area of Socket, Ap (Sq. ft)	7.07	12.57	19.63	28.27
Nominal Shaft Side Resistence, Rs (kips):	240.8	513.8	883.1	1348.7
Nominal Shaft Tip Resistence, Rp (kips):	17530.1	31164.6	48694.7	70120.3
Resistence Factor for Side from T. 10.5.5.2.4-1	0.55	0.55	0.55	0.55
Resistence Factor for Tip from T. 10.5.5.2.4-1	0.50	0.50	0.50	0.50
Factored Resistance from Side (kips)	132.5	282.6	485.7	741.8
Factored Resistance from Tip (kips)	8765.0	15582.3	24347.3	35060.2

Butt settlement of drilled Shaft : Q((Dr/Ap*Ec)+(Ips/Br*Em))

Note: Applied Axial load per shaft is obtained by limiting factored resistence to 0.4 inch of elastic settlement

Applied Axial Load on Top of Socket, Q (kips)	1025	1025	1025	1025
Concrete Young's Modulus, Ec (kci)	3800	3800	3800	3800
Shortening of Drilled Shaft (Inches)	0.172	0.129	0.103	0.086
Rock Mass Modulus, Em (kci)	200.0	200.0	200.0	200.0
Ec/Em	19.0	19.0	19.0	19.0
Dr/Br	1.50	1.50	1.50	1.50
Influence Coefficient (Ips) from Fig 4.6.5.5.2A	0.24	0.24	0.24	0.24
(Modified after Pells and Turner (1979))				
Settlement of Base (inches)	0.410	0.308	0.246	0.205
Total Butt Settlement of Shaft (inches)	0.582	0.436	0.349	0.291

GEOLOGY OF THE SITE

THE STRUCTURE SITE IS LOCATED IN THE RELATIVELY FLAT GLACIATED PORTION OF THE MISSISSIPPI VALLEY PLAIN REGION AT THE EDGE OF THE OLD LAKE MAUMEE BEACH RIDGE, ON THE BROAD FLOODPLAIN OF AND OVER THE BLANCHARD RIVER, IN AN AREA WHERE RELATIVELY THIN TO MODERATELY DEEP GLACIAL AND ALLUVIAL DEPOSITS OVERLIE DOLOMITIC LIMESTONE BEDROCK, OF THE MONROE FORMATION.

EXPLORATION

THE EXPLORATION CONSISTED OF FOUR DRIVE SAMPLE-CORE BORINGS MADE BY MEANS OF A MECHANICALLY-POWERED HOLLOW STEM ROTARY AUGER MOUNTED ON A MOBILE PLATFORM, PERFORMED BETWEEN APRIL 15 AND MAY 6, 1987.

INVESTIGATIONAL FINDINGS AND OBSERVATIONS

THE TEST BORINGS DISCLOSED THAT INTERVALS OF EXTREMELY LOOSE TO EXTREMELY DENSE UNSTRATIFIED BASIC SILTS AND CLAYS MODIFIED WITH SAND, GRAVEL AND VARYING PERCENTAGES OF EACH OTHER THAT GRADUALLY INCREASE IN DENSITY WITH INCREASE IN DEPTH OVERLIE GENTLY SLOPING BEDROCK SURFACE. TEST BORING NO. B-1 (MADE IN THE GENERAL VICINITY OF THE REAR ABUTMENT) ENCOUNTERED BEDROCK SURFACE AT 33.5-FOOT DEPTH, ELEVATION 753.4 FEET AND CONTINUED TO ADVANCE TO A TOTAL DEPTH OF 40.0 FEET, ELEVATION 746.9 FEET WHERE THE BORING WAS TERMINATED AFTER HAVING PENETRATED 6.5 FEET BELOW BEDROCK SURFACE. TEST BORING NO. B-2 (MADE IN THE GENERAL VICINITY OF THE REAR PIER) ENCOUNTERED BEDROCK SURFACE AT 13.5-FOOT DEPTH, ELEVATION 751.6 FEET AND CONTINUED TO ADVANCE TO A TOTAL DEPTH OF 26.5 FEET, ELEVATION 738.6 FEET WHERE THE BORING WAS TERMINATED AFTER HAVING PENETRATED 13.0 FEET BELOW BEDROCK SURFACE. TEST BORING NO. B-3 (MADE IN THE GENERAL VICINITY OF THE FORWARD PIER) ENCOUNTERED BEDROCK SURFACE AT 12.5-FOOT DEPTH, ELEVATION 753.9 FEET AND CONTINUED TO ADVANCE TO A TOTAL DEPTH OF 17.5 FEET, ELEVATION 748.9 FEET WHERE THE BORING WAS TERMINATED AFTER HAVING PENETRATED 5.0 FEET BELOW BEDROCK SURFACE. TEST BORING NO. B-4 (MADE IN THE GENERAL VICINITY OF THE FORWARD ABUTMENT) ENCOUNTERED BEDROCK SURFACE AT 39.0-FOOT DEPTH, ELEVATION 754.7 FEET AND CONTINUED TO ADVANCE TO A TOTAL DEPTH OF 45.0 FEET, ELEVATION 748.7 FEET WHERE THE BORING WAS TERMINATED AFTER HAVING PENETRATED 6.0 FEET BELOW BEDROCK SURFACE. A BOULDER-LADEN INTERVAL WAS ENCOUNTERED IN TEST BORING NO. B-4 AT 25.0-FOOT DEPTH, ELEVATION 768.7 FEET.

CRUSHING STRENGTH TESTS PERFORMED ON THE UNDERLYING DOLOMITIC LIMESTONE THROUGHOUT THE AREA AVERAGED 13,300.50 POUNDS PER SQUARE INCH.

NO FREE WATER OBSERVATIONS WERE MADE IN ANY OF THE TEST BORINGS PERFORMED DURING, OR AT THE CONCLUSION OF DRILLING OPERATIONS.

LEGEND

- Auger Boring Location - Plan View.
- Press and / or Drive Sample and / or Core Boring Location - Plan View.
- Drive Rod Penetration Resistance Sounding Location - Plan View.
- Capped Pile
- Footing
- Footing on Pile
- TR — Top of Rock
- Horizontal Bar on Boring Log Indicates the Depth the Sample Was Taken.
- X/Y/Z — Figures Beside the Boring Log in Profile Indicate the Number of Blows for Standard Penetration Test.
  - X = Number of Blows for First 6 inches.
  - Y = Number of Blows for Second 6 inches.
  - Z = Number of Blows for Third 6 inches.

Drive Rod Penetration Resistance Sounding Log - Profile

- Casing
- Resistance "R" < 10,000 lbs.
- Resistance "R" > 10,000 lbs.
- Z — Indicates Final Measurement of Penetration, in Inches.
- W — Indicates Free Water Elevation.
- Indicates Static Water Elevation.

SYMBOLS OF ROCK TYPES

- Coal
- Weathered Mudstone or Claystone
- Mudstone or Claystone
- Weathered Shale
- Shale
- Weathered Siltstone
- Siltstone
- Weathered Sandstone
- Sandstone
- Leached Dolomite
- Dolomite
- Leached Limestone
- Limestone
- Boulders or Cobbles

GENERAL INFORMATION

Drive Rod Penetration Sounding Tests

Drive rod penetration resistance tests constitute driving a 1.315-inch diameter steel rod, with a 45° cone point, into the ground, using a 122-pound drop-hammer with a free fall of five feet. At one or two-foot depth intervals, a measurement is taken to determine the amount of penetration achieved in three hammer drops. This reading is converted to an empirical value for capacity "R", in thousands of pounds (which is a measure of both the point resistance and frictional resistance on the rod), by using charts prepared by the Ohio Department of Highways, Bureau of Bridges, on the basis of correlation study of rod penetration with past performance of pile driving. For interpretation, a graph is prepared by plotting the value "R" against the depth at which the reading was taken, and connecting the plotted points. The curve so obtained reflects the density of subsurface materials in a manner that can be readily compared with data from similar tests at other locations on the structure site. From this comparison, the overall uniformity of subsurface condition may be evaluated.

Drive Sample Borings - Drive-Press Sample Borings

Drive sample borings are made by means of a rotary-type drill rig, employing a 2" O.D., 1-3/8" I.D. sampler, at 2-1/2 and / or 5-foot depth intervals, driven by means of a 140-pound drop-hammer with a free fall of 30 inches. The number of blows required to drive the sampler 18 inches is considered the standard penetration test.

Drive-press sample borings are made by means of a rotary-type drill rig, employing a 2" O.D., 1-3/8" I.D. drive sampler, and 3" O.D. thin-wall press sampler. The press sampler is advanced by continuous uniform pressure, applied by the drill rig.

The boring log sheets show a graphic plot of the information obtained, including depth and elevation of the sample, [illegible] of blows for the standard penetration tests in three 6-inch increments, depth of press sampl[illegible] field sample number, sample description - based on laboratory tests and the Casagrande AC classification system - and gradation, plasticity, and moisture content determinations. Results of strength and consolidation testing, if performed, appear on separate enclosures.

At depths where materials are bouldery or gravelly to the extent that the sampler can not be driven, a wash sample is procured for visual classification, in order to determine the general character of the material. These samples are not considered sufficiently representative to warrant laboratory testing.

Particle Size Definitions

Boulders	12"	Cobbles	3"	Gravel	2.0mm (No. 10 sieve)	Coarse Sand	0.42mm (No. 40 sieve)	Fine Sand	0.074mm (No. 200 sieve)	Silt	0.005mm	Clay

NOTE - ALL AVAILABLE SOIL AND BEDROCK INFORMATION WHICH CAN BE CONVENIENTLY SHOWN ON THE STRUCTURE FOUNDATION INVESTIGATION SHEETS HAS BEEN SO REPORTED. ADDITIONAL SUBSURFACE INVESTIGATIONS MAY HAVE BEEN MADE TO STUDY SOME SPECIAL ASPECT OF THE PROJECT. COPIES OF THIS DATA, IF ANY, MAY BE INSPECTED IN THE DISTRICT DEPUTY DIRECTOR'S OFFICE, THE BUREAU OF TESTS AT 1600 WEST BROAD STREET, THE PAVEMENT AND SOILS SECTION OF THE BUREAU OF LOCATION AND DESIGN OR IN THE BRIDGE BUREAU AT 25 SOUTH FRONT STREET.

Revised 7/24/87

NOTE: [illegible]

OHIO DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS — TESTING LABORATORY
1600 WEST BROAD STREET, COLUMBUS, OHIO 43223

STRUCTURE FOUNDATION INVESTIGATION
BRIDGE NO. HAN-75-1700
OVER RIVER RD. AND BLANCHARD RIVER
SEC. HAN-75-14.39

CHECKED BY	REVIEWED BY	DATE
L.N.L.	R.D.R.	5/27/87

SFN-3203034

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HANCOCK COUNTY
HAN-75-14.39

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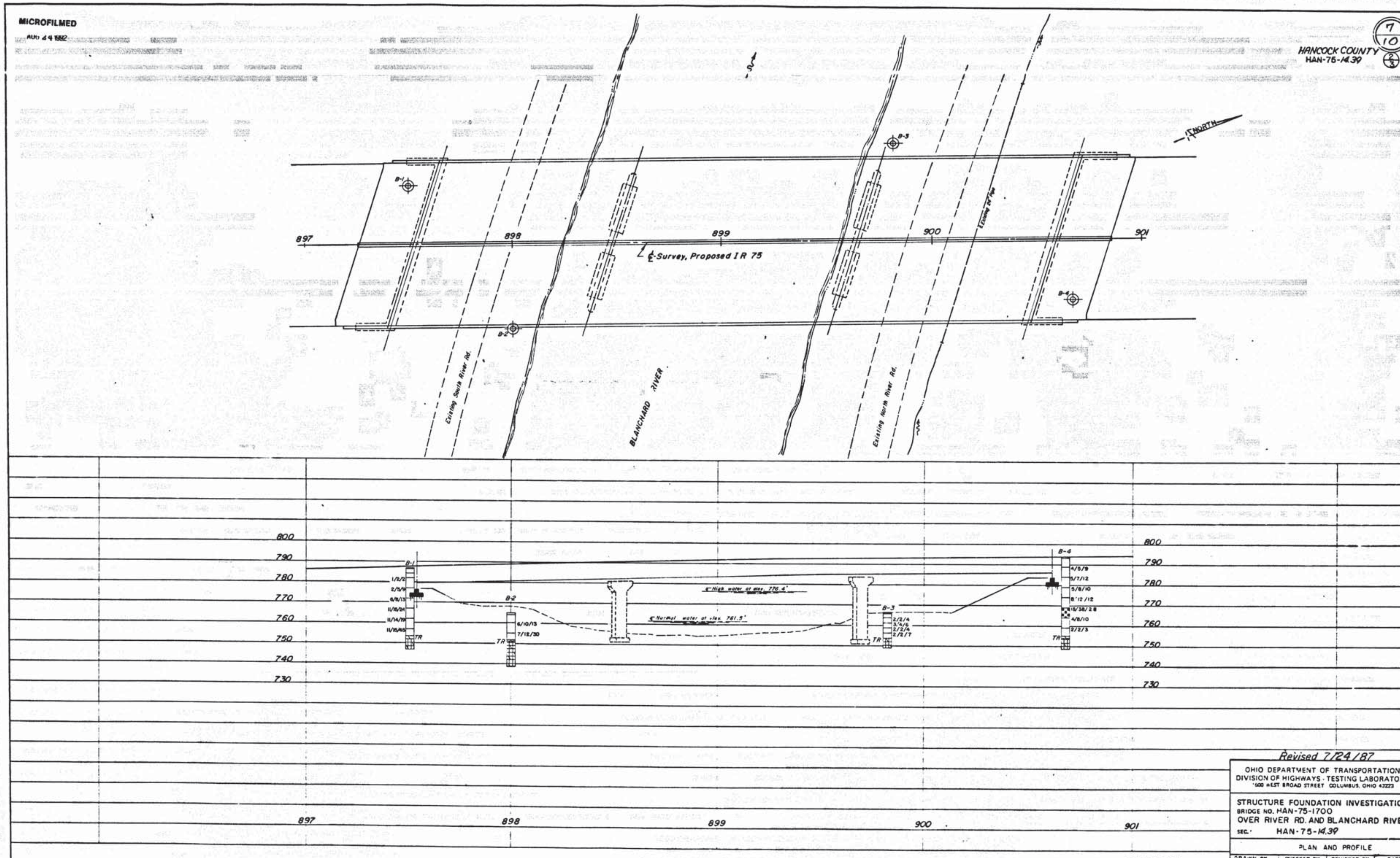

Revised 7/24/87

OHIO DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS - TESTING LABORATORY
1600 WEST BROAD STREET COLUMBUS, OHIO 43223

STRUCTURE FOUNDATION INVESTIGATION
BRIDGE NO. HAN-75-1700
OVER RIVER RD. AND BLANCHARD RIVER
SEC. HAN-75-14.39

PLAN AND PROFILE

DRAWN BY	CHECKED BY	REVIEWED BY	DATE
A.F.	L.N.L.	R.D.R.	5/27/87

SFN 3203034

HANCOCK COUNTY
HAN-75-14.39

8/10

3/3

LOG OF BORING

Date Completed 5/6/87 | Casing: Length ____ Dia ____
Boring No. B-1 | Station & Offset 897+50, 29' LT. (REAR ABUTMENT) | Surface Elev 786.9'

Elev.	Depth	Std. Pen. (N)	Rec. ft.	Loss ft.	Description	Sample No.	% Agg.	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	SHTL Class.
786.9	0														
786.2		AUGERED			ASPHALT	-	-	-	-	-	-	-	-	-	VISUAL
784.2	2	AUGERED			LARGE GRAVEL (DRILLER'S DESCRIPTION)	-	-	-	-	-	-	-	-	-	VISUAL
781.9	6	1/2/2			BROWN SANDY CLAY	1	9	6	17	24	44	32	15	22	A-6a
776.9	12	2/5/9			BROWN AND GRAY SANDY CLAY	2	8	5	17	24	46	32	14	18	A-6a
771.9	18	6/8/13			BROWN AND GRAY SILTY CLAY	3	0	4	14	26	56	35	16	19	A-6b
766.9	22	11/15/24			BROWN SILT	4	0	1	16	57	26	NP	NP	23	A-4b
761.9	26	11/14/19			BROWN SANDY GRAVELLY SILT	5	40	5	10	27	18	18	4	12	A-4a
756.9	32	11/15/45			GRAY GRAVELLY SILT	6	17	5	7	37	34	19	1	14	A-4a
753.4	34				TOP OF ROCK										
751.9					BROKEN DOLOMITIC LIMESTONE	-	-	-	-	-	-	-	-	-	VISUAL
	38		5.0	0.0	DOLOMITIC-LIMESTONE, GRAY, HARD, DENSE, SOMEWHAT LEACHED, SLIGHTLY VUGGY, BROKEN AND JOINTED. NO CORE LOSS.										
746.9	40				BOTTOM OF BORING										

LOG OF BORING

Date Completed 5/4/87 | Casing: Length ____ Dia ____
Boring No. B-2 | Station & Offset 898+00, 42' RT. (SOUTH PIER) | Surface Elev 765.1'

Elev.	Depth	Std. Pen. (N)	Rec. ft.	Loss ft.	Description	Sample No.	% Agg.	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	SHTL Class.
765.1	0														
764.1		AUGERED			TOPSOIL	-	-	-	-	-	-	-	-	-	VISUAL
760.1	6	6/10/13			BROWN SILT	1	0	1	4	61	34	26	2	19	A-4b
755.1	12	7/12/30			GRAY SANDY SILT	2	9	4	7	40	40	20	3	14	A-4a
751.6	14				TOP OF ROCK										
751.1					BROKEN DOLOMITIC LIMESTONE	-	-	-	-	-	-	-	-	-	VISUAL
	18		5.0	0.0	DOLOMITIC-LIMESTONE, GRAY, HARD, DENSE, SOMEWHAT LEACHED, SLIGHTLY VUGGY, BROKEN AND JOINTED. CORE LOSS 3%.										
	22		3.2	0.3											
	26		3.9	0.1											
738.6					BOTTOM OF BORING										

LOG OF BORING

Date Started 4/15/87 | Sampler Type SS Dia 1 3/8" | Water Elev. ____
Date Completed 4/15/87 | Casing: Length ____ Dia ____
Boring No. B-3 | Station & Offset 899+82', 48' LT. (NORTH PIER) | Surface Elev 766.4'

Elev.	Depth	Std. Pen. (N)	Rec. ft.	Loss ft.	Description	Sample No.	% Agg.	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	SHTL Class.
766.4	0														
766.0		AUGERED			SOD AND TOPSOIL	-	-	-	-	-	-	-	-	-	VISUAL
763.9	4	2/2/4			BROWN SANDY CLAY	1	0	1	36	29	34	28	11	24	A-6a
761.4	6	3/4/6			BROWN SANDY CLAY	2	0	0	21	40	39	33	14	23	A-6a
758.9	8	2/2/4			BROWN SILTY CLAY	3	0	1	16	42	41	35	16	27	A-6b
756.4	12	2/2/7			BROWN SANDY SILT	4	11	10	34	24	21	23	6	28	A-4a
753.9					TOP OF ROCK										
	16		5.0	0.0	DOLOMITIC-LIMESTONE, GRAY, HARD, DENSE, SOMEWHAT LEACHED, EXTREMELY VUGGY AND VERY BADLY BROKEN AND JOINTED. NO CORE LOSS.										
748.9	18				BOTTOM OF BORING										

LOG OF BORING

Date Started 4/28/87 | Sampler Type SS Dia 1 3/8" | Water Elev. ____
Date Completed 4/29/87 | Casing: Length ____ Dia ____
Boring No. B-4 | Station & Offset 900+67, 30' RT. (FORWARD ABUTMENT) | Surface Elev 793.7'

Elev.	Depth	Std. Pen. (N)	Rec. ft.	Loss ft.	Description	Sample No.	% Agg.	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	SHTL Class.
793.7	0														
793.2		AUGERED			ASPHALT	-	-	-	-	-	-	-	-	-	VISUAL
788.7	6	4/5/9			BROWN SANDY CLAY	1	6	5	38	15	36	28	13	16	A-6a
783.7	12	5/7/12			BROWN CLAYEY SILT	2	4	0	1	60	35	23	9	19	A-4b
778.7	18	5/8/10			BROWN SANDY CLAY	3	6	6	22	26	40	28	12	13	A-6a
773.7	22	8/12/12			GRAY AND BROWN SANDY SILT	4	4	5	40	20	31	21	7	15	A-4a
768.7	26	18/38/28			BROWN AND GRAY SANDY GRAVELLY CLAY W/BLDRS.	5	34	4	19	16	27	26	11	15	A-6a
763.7	32	4/8/10			BROWN SILT AND CLAY	6	0	0	14	17	69	35	15	23	A-6a
758.7	36	2/2/3			BROWN AND GRAY SANDY CLAY	7	0	0	20	40	40	34	16	26	A-6b
754.7					TOP OF ROCK										
753.7	40				BROKEN DOLOMITIC LIMESTONE	-	-	-	-	-	-	-	-	-	VISUAL
	42		4.6	0.4	DOLOMITIC LIMESTONE, GRAY, HARD, DENSE, SOMEWHAT LEACHED, EXTREMELY VUGGY, AND VERY BADLY BROKEN AND JOINTED. CORE LOSS 8%.										
748.7	44				BOTTOM OF BORING										

SFN-3203034

Revised 7/24/87

OHIO DEPARTMENT OF TRANSPORTATION
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STRUCTURE FOUNDATION INVESTIGATION
BRIDGE NO. HAN-75-1700
OVER RIVER RD. AND BLANCHARD RIVER
SEC. HAN-75-14.39

BORING DATA

TYPED BY	CHECKED BY	REVIEWED BY	DATE
S.M.G.	L.N.L.	R.D.R.	5/27/87

VI.D. Geotechnical Reports

C-R-S: HAN-75-14.39-Bridge No. HAN-75-1697	PID: 87005	Reviewer: SS	Date: 7/14/2014

General	
Y $\boxed{N}$ 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?	
$\boxed{Y}$ N X 4 Have all geotechnical reports included each of the sections as described in Sections 705.2 through 705.8.4 of the SGE?	

Notes:

IV.A Foundations/Structures - Non-bridge Applications

C-R-S: HAN-68-14.39- Bridge No. HAN-75-1697	PID: 87005	Reviewer: SS	Date: 7/14/2014

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

Soil and Bedrock Strength Data					
Y	N	$\boxed{X}$	1	Has the shear strength of the foundation soils been determined? Check method used: 9 laboratory shear tests 9 estimation from SPT or field tests	Foundation on bedrock
Y	N	$\boxed{X}$	2	Have sufficient soil shear strength, consolidation, and other parameters been determined so that the required allowable loads for the foundation/structure can be designed?	
$\boxed{Y}$	N	X	3	Has the shear strength of the foundation bedrock been determined? Check method used: 9 laboratory shear tests $\boxed{9}$ other List Other items:	Compression Strength of Bedrock

Notes:

Stage 1:

IV.A Foundations/Structures - Non-bridge Applications

Spread Footings				
Y $\boxed{N}$	4		Are there spread footings on the project?	
			If no, go to Question **11**	
Y N X	5		Has the recommended bottom of footing elevation and reason for this recommendation been provided?	
Y N 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:	
Y N X		a	bearing capacity?	
Y N X		b	sliding?	
Y N X		c	Overturning?	
Y N X		d	settlement?	
Y N X	7		Has the need for a shear key been evaluated?	
Y N X		a	If needed, have the details been included in the plans?	
Y N X	8		If special conditions exist (e.g. geometry, sloping rock, varying soil conditions), was the bottom of footing "stepped" to accommodate them?	
Y N X	9		Has the recommended allowable soil or rock bearing pressure been provided?	
Y N 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 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 - Bridge			
[Y] N	11	Are there piles on the project? If no, go to Question **17**	
[Y] N	12	Has an appropriate pile type been selected? Check the type selected: [9] H-pile (driven) 9 H-pile (drilled) 9 Cast In-place Concrete 9 other List Other items:	
[Y] N X	13	Have the estimated pile length or tip elevation and section (diameter) been specified? Check method used: [9] SPILE, DRIVEN, PICAP3 or equivalent software 9 hand calculations	
	14	If required for design, have sufficient soil parameters been provided and calculations performed to evaluate the:	
Y [N] X		a Lateral load capacity and maximum deflection of the piles?	To be determined by PB
[Y] N X		b Vertical load capacity and maximum settlement of the piles?	
[Y] N X		c Negative skin friction on piles driven through new embankment or soft foundation layers?	
Y N [X]		d Potential for and impact of lateral squeeze from soft foundation soils?	
Y N [X]	15	If piles are to be driven <u>to bedrock, have "pile points"</u> been recommended to assure secure contact with the rock surface, as per BDM 202.2.3.2.a?	
[Y] N 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				
$\boxed{Y}$ N	17	Are there drilled shafts on the project?		
		If no, go to the next checklist.		
$\boxed{Y}$ N X	18	Have the drilled shaft diameter and embedment length been specified?		
$\boxed{Y}$ 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:	To be determined by PB	
Y $\boxed{N}$ X		a. maximum lateral shear		
Y $\boxed{N}$ X		b. maximum bending moment		
Y $\boxed{N}$ X		c. maximum deflection		
Y $\boxed{N}$ X		d. reinforcement design		
$\boxed{Y}$ 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 $\boxed{N}$ 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?	To be estimated by PB	
$\boxed{Y}$ N X	23	Has the site been assessed for groundwater influence?	Adjacent to Blanchard River	
Y $\boxed{N}$ X		a If yes, if artesian flow is a potential concern, does the design address control of groundwater flow during construction?		
Y N $\boxed{X}$	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

Cohesive (fine grained) Soils - Consistency

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

2) COLOR :

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

3) PRIMARY COMPONENT

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

4) COMPONENT MODIFIERS:

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

5) Soil Organic Content

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

6) Relative Visual Moisture

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

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

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

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

3) **WEATHERING**

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

4) **RELATIVE STRENGTH**

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

5) **TEXTURE**

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

6) **BEDDING**

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

7) DESCRIPTORS

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

8) DISCONTINUITIES

a) Discontinuity Types

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

b) Degree of Fracturing

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

c) Aperture Width

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

d) Surface Roughness

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

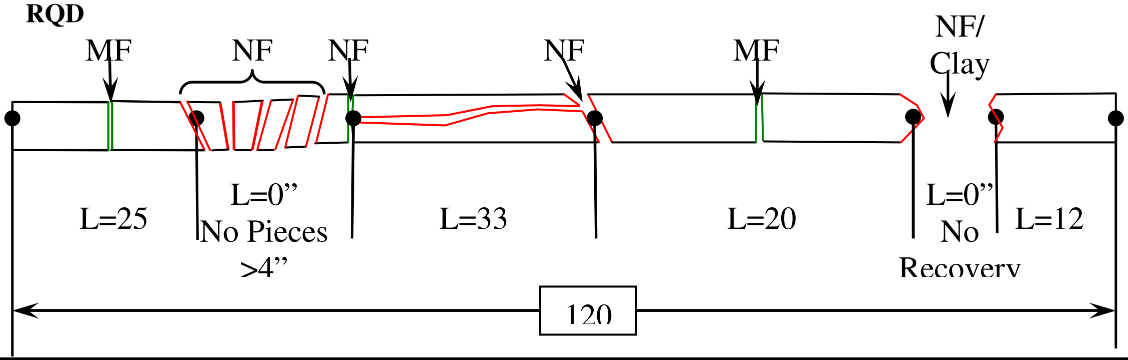
10) LOSS

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

L_R=Run Length R_R=Run Recovery

L_U=Rock Unit Length R_U=Rock Unit Recovery

9) RQD



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

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