

**JEF-151-14.27 STRUCTURE
REPLACEMENT
ODOT PID NO. 115933
JEFFERSON COUNTY, OHIO**

**STRUCTURE FOUNDATION
EXPLORATION REPORT**

Prepared For:

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Prepared By:

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Rii Project No. W-23-045

November 2025

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November 24, 2025

Mr. Jason Centers, P.E.
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**Re: Structure Foundation Exploration Report
JEF-151-14.27 Structure Replacement
SR 151 over Georges Run
ODOT PID No. 115933
Jefferson County, Ohio
Rii Project No. W-23-045**

Mr. Centers:

Resource International, Inc. (Rii) is pleased to submit this structure foundation exploration report for the above-referenced project. Engineering logs have been prepared and are attached to this report along with results of laboratory testing. This report includes geotechnical recommendations for the proposed JEF-151-14.27 structure replacement carrying SR 151 over Georges Run in Jefferson County, Ohio.

We sincerely appreciate the opportunity to be of service to you on this project. If you have any questions regarding the structure foundation exploration or this report, please contact us.

Sincerely,

RESOURCE INTERNATIONAL, INC.

Ashok Gaire, P.E.
Project Manager

Daniel E. Karch, P.E.
Director – Geotechnical Services

Enclosure: Structure Foundation Exploration Report

ISO 9001: 2015 QMS

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EXECUTIVE SUMMARY

Resource International, Inc. (Rii) has completed a structure foundation exploration for the proposed JEF-151-14.27 structure replacement carrying SR 151 over Georges Run in Jefferson County, Ohio. Based on the available plans, the existing structure is planned to be replaced with a four-sided precast reinforced concrete box (RCB) structure measuring 73.0 feet in length, 20 feet in span and 7 feet in rise. Additionally, headwalls and wingwalls are planned to be constructed at the inlet and outlet of the proposed structure. The invert elevations of the proposed structure are 877.97 and 876.87 feet mean sea level (msl) at the inlet and outlet, respectively

Exploration and Findings

On April 18, 2023, two (2) structure borings, designated as B-001-0-23 and B-002-0-23, were drilled in the vicinity of proposed structure replacement and were advanced to depths of 25.0 and 30.5 feet, respectively below the existing ground surface. Additionally, one (1) pavement core, designated as X-001-0-23, was obtained from the eastbound approach to the existing bridge to determine existing pavement composition and condition. At the ground surface borings B-001-0-23 and B-002-0-23 encountered 19.5 and 20.0 inches of asphalt pavement overlying 2.5 and 3.0 inches of aggregate base, respectively. Pavement core X-001-0-23 was obtained within the driving lane of SR 151 and encountered 20.0 inches of asphalt overlying granular soils. Beneath the surficial materials, the soil borings predominantly encountered natural cohesive soils with intermittent layers of granular soils to the top of bedrock. The natural cohesive soils encountered were described as silt and clay and clay (ODOT A-6a, A-7-6) with varying amounts of sand and gravel. The granular soils encountered were described as gravel with sand, silt, and clay (ODOT A-2-6).

Borings B-001-0-23 and B-002-0-23 encountered top of the bedrock at depths of 16.0 and 13.5 feet below the existing grade, corresponding to elevations 873.6 and 874.8 feet msl, respectively. The upper augerable bedrock was described as brown, highly to severely weathered sandstone. Rock coring was performed in the borings upon encountering more competent bedrock. The recovered bedrock core samples were described as highly weathered to slightly weathered, highly fractured to moderately fractured, brown to gray, weak to moderately strong sandstone.

The top of the bedrock surface elevations encountered in historic borings were 876.8 and 877.9 feet msl. The encountered bedrock was described as brown or gray sandstone.

Seepage was encountered in borings B-001-0-23 and B-002-0-23 at depths of 6.0 and 11.0 feet below the existing grade, respectively. Groundwater was not encountered prior to rock coring in either boring. Upon the completion of drilling groundwater was not recorded due to the influence of water added during rock coring process.

Conclusions and Recommendations

4-Sided Precast Reinforced Box Culvert Support

It is understood that the invert elevations of the proposed RCB structure at inlet and outlet are 877.97 and 876.87 feet msl, respectively. Medium dense to very dense granular soils (at inlet) and very stiff cohesive soils (at outlet) were encountered at the proposed culvert invert elevations in borings B-001-023 and B-002-0-23. The historic borings encountered weathered bedrock above or near the proposed culvert bearing elevation. The encountered soils and bedrock encountered at and below the proposed culvert bearing elevations are suitable to support the proposed RCB structure.

Rii understands that the proposed structure is planned to be extended beyond the footprint of the existing structure, and will be founded at or near the existing stream bed elevation. These soils, at or near the existing stream bed elevations, generally tend to exhibit soft, loose and/or wet conditions and provide inadequate support to the proposed structure. Therefore, Rii recommends that 12 inches of soils below the proposed culvert bearing elevations be over-excavated and backfilled with properly placed and compacted granular materials (such as ODOT CMS Granular material Type B or Granular material Type C). Where bedrock is encountered at or above the bearing elevation, the 12 inches of bedrock below the culvert support elevation should be over-excavated and replaced with ODOTCMS Granular Material Type B. All bearing surfaces should be observed and approved by a geotechnical engineer or his/or representative.

Headwall / Wingwall Foundation Support

Since the bedrock surface elevation is at or very near the proposed foundation bearing elevations, Rii recommends that all the foundation be supported on the bedrock. Following the recommendations provided in this report, spread foundation bearing on sandstone bedrock may be proportioned for the bearing resistance values not exceeding those provided below:

- Nominal bearing resistance of $q_n = 31.2$ ksf at the strength limit state.
- LRFD Bearing Resistance Factor of $\phi = 0.55$ at the strength limit state.

Settlement of footings bearing on bedrock is negligible, in accordance with the Ohio Department of Transportation (ODOT) Bridge Design Manual (BDM) 2020 Section 305.

Please note that this executive summary does not contain all the information presented in the report. The unabridged subsurface exploration report should be read in its entirety to obtain a more complete understanding of the information presented.

1.0 INTRODUCTION

This report is a presentation of the structure foundation exploration performed for the proposed JEF-151-14.27 structure replacement carrying SR 151 over Georges Run in Jefferson County, Ohio.

Based on the available plans, the existing structure is planned to be replaced with a four-sided precast reinforced concrete box (RCB) structure measuring 73.0 feet in length, 20 feet in span and 7 feet in rise. Additionally, headwalls and wingwalls are planned to be constructed at the inlet and outlet of the proposed structure. The invert elevations of the proposed structure are 877.97 and 876.87 feet mean sea level (msl) at the inlet and outlet, respectively.

The exploration was performed within general accordance of the Ohio Department of Transportation's (ODOT) Specifications for Geotechnical Explorations (SGE), dated January 2023. The project site and general location of the proposed culvert are as shown on the vicinity map and boring plan presented in Appendix I.

2.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1 Site Geology

Physiographically, the project site lies in the Little Switzerland Plateau physiographic region within the Appalachian Plateau Province where the rocks are relatively flat-lying and comprised of Pennsylvanian strata. The plateaus are covered with a thin to non-existent layer of colluvium over bedrock. The top of bedrock generally follows the ground surface and is typically shallower on the ridges and slopes, and deeper in the valley zones where more overburden soils or thicker colluvium has accumulated. Colluvium is the weathered rock, weathered debris, and scattered residuum that forms due primarily to gravity and erosion.

Based on the Ohio Department of Natural Resources (ODNR), the bedrock at the proposed project site consists of Pennsylvanian age sandstone, siltstone, shale, and lesser amount of limestone and coal. The rock types within these zones can change rapidly both horizontally and vertically and commonly intertongue and intergrade. Based on the available mapping, the bedrock surface elevation in the vicinity of this project site is approximately 875 feet msl.



2.2 Existing Site Conditions

The existing structure carrying SR 151 (New Alexandria Road) over Georges Run is located approximately 1.06 miles east of Scott Featner Road and 0.29 miles west of Cool Springs Road (SLM 14.27), in the southeast section of Jefferson County, Ohio. Based on the available plans and the information obtained from national bridge inventory data, the existing structure is a single-span, reinforced concrete cast-in-place slab superstructure with wingwalls, and has the span length of approximately 20 feet, a clear roadway width of 32.0 feet between the curbs and a total length of approximately 37.5 feet. The existing structure was built in 1986.

Rii performed site reconnaissance on April 3, 2023. Deterioration of the existing slab along with exposed rebar were observed on the existing bridge. SR 151 (New Alexandria Road) within the project limits is a two-lane, undivided, bi-directional, asphalt paved roadway with partial width (less than 2.0 feet wide) shoulders. Guard rails are present in the vicinity of the bridge structure. Land use immediately surrounding the project area consists predominantly wooded and brush areas along with residential properties. The available plans indicate that an underground watermain is located along the roadway within the project limits.

3.0 EXPLORATION

On April 18, 2023, two (2) structure borings, designated as B-001-0-23 and B-002-0-23, were drilled in the vicinity of proposed structure replacement and were advanced to depths 25.0 and 30.5 feet below the existing ground surface, respectively. Additionally, one (1) pavement core, designated as X-001-0-23, was obtained from the eastbound approach to the existing bridge to determine existing pavement composition and its condition. The borings and pavement core were drilled at locations illustrated on the boring plan presented in Appendix I of this report, and a summary of boring and pavement core information is provided in Table 1.

Table 1. Test Boring and Pavement Core Summary

Boring Number	Station ¹	Offset ¹	Latitude	Longitude	Ground Elevation ² (feet msl)	Boring Depth (feet)
B-001-0-23	501+82	7' Rt	40.293809	-80.651112	889.6	25.0
B-002-0-23	502+43	8' Lt	40.293807	-80.650943	888.3	30.5
X-001-0-23	501+28	7' Rt	40.293845	-80.651342	890.8	---

1. Stations, offsets were interpolated from basemaps provided by Jacobs.

2. Ground surface elevation were determined from survey performed by Rii.

The boring and pavement core locations were determined and located in the field by Rii personnel. Rii utilized a GPS unit to obtain latitude and longitude coordinates of the boring and core locations. Station, offsets at the borings and pavement core locations were interpolated from basemaps provided by Jacobs. Ground surface elevations at boring locations were surveyed by Rii.

The borings were drilled and sampled with a Diedrich D-50 track mounted utilizing 3.25-inch inside diameter hollow stem augers to advance the boreholes. Standard penetration test (SPT) and split spoon sampling were performed at 2.5-foot intervals to the top of bedrock. The pavement core was obtained with a 4.0-inch diameter thin-walled pavement core bit.

The SPT, per the American Society for Testing and Materials (ASTM) designation D1586, is conducted using a 140-pound hammer free falling 30.0 inches to drive a 2.0-inch outside diameter split spoon sampler 18.0 inches. Rii utilized a calibrated automatic drop hammer to generate consistent energy transfer to the sampler. Driving resistance is recorded on the boring logs in terms of blows per 6.0-inch interval of the driving distance. The second and third intervals are added to obtain the number of blows per foot (N). SPT blow counts aid in determining soil properties applicable in foundation system design and settlement calculation of foundation soil. Measured blow count (N_m) values are corrected to an equivalent (60 percent) energy ratio, N_{60} , by the following equation. Both values are represented on boring logs in Appendix III.

$$N_{60} = N_m \cdot (ER/60)$$

Where:

N_m = measured N value

ER = drill rod energy ratio, expressed as a percent, for the system used

The hammer utilized in Diedrich D-50 rig used for this project was calibrated on March 21, 2022 and had a drill rod energy ratio of 86.4 percent.

The depth to bedrock was determined by split spoon sampler refusal and an increased resistance in the auger advancement. Both borings were extended into the bedrock and an NQ-2 double-tube diamond bit core barrel (utilizing wire line equipment) was used to core the bedrock. The rock cores obtained from the boring were logged in the field and visually classified in the laboratory. The retrieved core was analyzed to identify the type of rock, color, mineral content, bedding planes and other geological and mechanical features of interest in this project. The Rock Quality Designation (RQD) for each rock core run was calculated according to the following equation:

$$RQD = \frac{\sum \text{segments equal to or longer than 4.0 inches}}{\text{core run length}} \times 100$$

The RQD value aids in estimating the general quality of the rock and is used in conjunction with other parameters to designate the quality of the rock mass.

Upon completion of drilling, the borings were sealed with a cement-bentonite grout in accordance with ODOT standards. Pavement was patched with equivalent thickness of cold patch asphalt.

Hand penetrometer readings, which provide a rough estimate of the unconfined compression strength (UCS) of the soil, were reported on the boring logs in units of tons per square foot (tsf) and were utilized to classify the consistency of the cohesive soil in each layer. An indirect estimate of the unconfined compressive strength of the cohesive split spoon samples can also be made from a correlation with the blow counts (N_{60}). Please note that split spoon samples are considered to be disturbed and the laboratory determination of their shear strengths may vary from undisturbed conditions.

During drilling, field personnel prepared field logs showing the encountered subsurface conditions. Soil samples obtained from the drilling operation were preserved in sealed glass jars and rock cores were preserved in core boxes and were delivered to the soil laboratory. In the laboratory, the soil and rock core samples were visually classified and select samples were tested, as noted in Table 2.

Table 2. Laboratory Test Schedule

Laboratory Test	Test Designation	Number of Tests
Natural Moisture Content	AASHTO T265	11
Plastic and Liquid Limits	AASHTO T89, T90	4
Sieve/Hydrometers	AASHTO T88	4
Uniaxial Compressive Strength Test	ASTM D 7012	1
Point Load Strength Test	ASTM D 5731	1

The tests performed are necessary to classify existing soil according to the Ohio Department of Transportation (ODOT) classification system and to estimate engineering properties of importance in determining foundation design and construction recommendations. A description of the soil terms used throughout this report is presented in Appendix II. Results of the laboratory testing are presented on the boring logs in Appendix III and the results of special laboratory testing are also presented in Appendix IV .

4.0 FINDINGS

Interpreted engineering logs have been prepared from field logs, visual examination of samples and laboratory testing. Classification follows the current version of the ODOT Specifications of Geotechnical Exploration (SGE). The following is a generalization of what was found in the test borings and what is represented on the boring logs.

4.1 Surface Materials

At the ground surface borings B-001-0-23 and B-002-0-23 encountered 19.5 and 20.0 inches of asphalt pavement overlying 2.5 and 3.0 inches of aggregate base, respectively.

Pavement core X-001-0-23 was obtained within the driving lane of SR 151 and encountered 20.0 inches of asphalt overlying granular soils. The granular soils were too thick to excavate with hand tools, and therefore a thickness measurement was not able to be taken. Detailed information of the pavement core is provided in Appendix VI.

4.2 Subsurface Soils

Beneath the surficial materials, the soil borings predominantly encountered natural cohesive soils with intermittent layers of granular soils to the top of bedrock. The natural cohesive soils encountered were described as silt and clay and clay (ODOT A-6a, A-7-6) with varying amounts of sand and gravel. The granular soils encountered were described as gravel with sand, silt, and clay (ODOT A-2-6).

The shear strength and consistency of the cohesive soils are primarily derived from the hand penetrometer values (HP). Where HP values could not be determined, the consistency of cohesive soils were correlated with SPT blow counts (N_{60}) values. The consistency of the cohesive soils encountered ranged from medium stiff ($0.5 \text{ tsf} \leq \text{HP} \leq 1.0 \text{ tsf}$) to very stiff ($2.0 \text{ tsf} \leq \text{HP} \leq 4.0 \text{ tsf}$). The unconfined compressive strength of the cohesive soil samples tested, obtained from the hand penetrometer, ranged from 1.0 to 2.0 tsf. SPT- N_{60} values within the cohesive soil layers ranged from 6 to 23 blows per foot (bpf).

The relative density of granular soils is primarily derived from SPT blow counts (N_{60}). Based on the SPT blow counts obtained, the relative density of granular material encountered ranged from medium dense ($10 \text{ bpf} \leq N_{60} \leq 30 \text{ bpf}$) to very dense ($N_{60} \geq 50 \text{ bpf}$). The SPT- N_{60} values within the granular soils ranged from 19 bpf to 65 bpf.

Moisture contents of the cohesive soil samples tested ranged from 8 to 32 percent and the moisture contents of the granular soil samples tested ranged from 8 to 12 percent. The natural moisture contents of the cohesive soil samples tested for plasticity ranged from 1 to 9 percent above their corresponding plastic limits values, indicating that the

moisture content to be moderately to significantly above their estimated optimum moisture levels.

4.3 Bedrock

Bedrock was encountered at depths as summarized in Table 3.

Table 3. Summary of Bedrock Depths

Boring	Ground Elevation (feet msl) ¹	Top of Bedrock		Top of Bedrock Core	
		Depth (feet)	Elevation (feet, msl)	Depth (feet)	Elevation (feet, msl)
B-001-0-23	889.6	16.0	873.6	19.0	870.6
B-002-0-23	888.3	13.5	874.8	16.5	871.8

Borings B-001-0-23 and B-002-0-23 encountered top of the bedrock at depths 16.0 and 13.5 feet below the existing grade, corresponding to elevations of 873.6 and 874.8 feet msl, respectively. The upper augerable bedrock was described as brown, highly to severely weathered sandstone.

Rock coring was performed in the borings upon encountering more competent bedrock. The recovered bedrock core samples were described as highly weathered to slightly weathered, highly fractured to moderately fractured, brown to gray, weak to moderately strong sandstone. The recovered rock core samples exhibited core recovery values ranging from 58 to 97 percent and rock quality designation (RQD) values ranging from 0 of 48 percent. The uniaxial compressive strength and point load strength testing performed on selected core samples exhibited unconfined compressive strength (Qu) values of 4,522 and 7,718 psi. A detailed information of rock core samples along with rock core photographs are provided in Appendix III.

4.4 Groundwater

Seepage was encountered in borings B-001-0-23 and B-002-0-23 at depths of 6.0 and 11.0 feet below the existing grade, respectively. Groundwater was not encountered prior to rock coring in either boring. Upon the completion of drilling groundwater was not recorded due to the influence of water added during rock coring process.

Please note that short-term water level readings, especially in cohesive materials, are not necessarily an accurate indication of the actual groundwater level. In addition, groundwater levels and the presence of groundwater are considered to be dependent on seasonal fluctuations in precipitation and groundwater levels in nearby bodies of water at the time of the investigation.

4.5 Historic Boring Information

Based on ODOT Transportation Information Mapping System (TIMS), historic borings designated as B-1 and B-2, were previously drilled (November 1983) in the immediate vicinity of this project site. The historic borings encountered layers of cohesive and granular soils consisting of silt and clay, sandy silt, gravel with sand and, gravel with sand silt and clay (ODOT A-6a, A-4a, A-1-a and A-2-6) extending to the top of bedrock. The top of the bedrock was encountered approximately at a depth of 13.0 feet below grade at the time of performance of the historic borings. The top of the bedrock surface elevations encountered in historic borings were 876.8 and 877.9 feet msl. The encountered bedrock was described as brown or gray sandstone.

The locations of the historic borings are shown on boring plan in Appendix I, and the historic boring logs are attached in Appendix V of this report.

5.0 CONCLUSIONS AND RECOMMENDATIONS

Data obtained from the drilling and testing program have been used to determine the foundation support capabilities and the settlement potential for the soil and bedrock conditions encountered at the site. These parameters have been used to provide guidelines for the design of foundation system for the subject structure as well as the construction specifications related to the placement of foundation systems and general earthwork recommendations, all of which are discussed in the following paragraphs.

5.1 Foundation Recommendations

5.1.1 4-Sided Precast Reinforced Box Culvert Support

It is understood that the invert elevations of the proposed RCB structure at inlet and outlet are 877.97 and 876.87 feet msl, respectively. Medium dense to very dense granular soils (at inlet) and very stiff cohesive soils (at outlet) were encountered at the proposed culvert invert elevations in borings B-001-0-23 and B-002-0-23. The historic borings encountered weathered bedrock above or near the proposed culvert bearing elevation.

Note that, historic boring B-2 drilled in the vicinity of culvert inlet encountered bedrock at elevation 876.8 feet msl and historic boring B-1 drilled in the vicinity of culvert outlet encountered bedrock at elevation 877.9 feet msl. Therefore, a few inches of bedrock excavation should be anticipated to achieve the proposed invert elevations at the outlet of the proposed structure.

Rii understands that the proposed structure is planned to be extended beyond the footprint of the existing structure, and will be founded at or near the existing stream bed elevation. These soils, at or near the existing stream bed elevations, generally tend to exhibit soft, loose and/or wet conditions and provide inadequate support to the proposed

structure. Therefore, Rii recommends that 12 inches of soils below the proposed culvert bearing elevations be over-excavated and backfilled with properly placed and compacted granular materials (such as ODOT CMS Granular material Type B or Granular material Type C). The granular materials will provide stable working platform and also provide uniform support for the proposed structure. Where bedrock is encountered at or above the bearing elevation, the 12 inches of bedrock below the culvert support elevation should be over-excavated and replaced with CMS Granular Material Type B. All bearing surfaces should be observed and approved by a geotechnical engineer or his/or representative.

Rii understands that the footprint of the proposed culvert will be extended beyond the footprint of existing culvert and the proposed roadway grade will approximately match the existing roadway grade after construction of the proposed structure. Based on the existing grade in the vicinity of proposed extension and the proposed grade, up to approximately 11 feet of the fill will be required to bring the site to the proposed roadway grade.

Rii performed settlement analysis utilizing the estimated surcharge load from the proposed fill, general soil profile obtained from the field exploration and soil parameters based on results of the field and laboratory tests. It is estimated that the maximum total settlement of the soils due to the placement of the proposed fill will be approximately 0.9 inches. However, the maximum total settlement of the soils below the proposed culvert bearing elevation due to the placement of the proposed fill is estimated to be approximately 0.4 inches.

5.1.2 Headwall / Wingwall Foundation Support

Based on the available plans provided by Jacobs, the proposed headwalls/wingwalls are planned to be founded at elevations of 876.37 and 874.22 feet msl at the inlet and outlet, respectively. Boring B-001-0-23, drilled in the vicinity of inlet, encountered very dense granular soils at the proposed bearing elevation immediately underlain by weathered bedrock. Boring B-002-0-23, drilled in the vicinity of the outlet, encountered weathered bedrock immediately below the proposed bearing elevation. As stated, historic encountered bedrock at elevations 876.8 and 877.9 feet msl in the vicinity of inlet and outlet, respectively.

Since the bedrock surface is at or very near the proposed foundation bearing elevations, Rii recommends that all the foundation be supported on the bedrock. Following the recommendations provided in this report, spread foundation bearing on sandstone bedrock may be proportioned for the bearing resistance values not exceeding those provided below:

- Nominal bearing resistance of $q_n = 31.2$ ksf at the strength limit state.
- LRFD Bearing Resistance Factor of $\phi = 0.55$ at the strength limit state.

Settlement of footings bearing on bedrock is negligible, in accordance with the Ohio Department of Transportation (ODOT) Bridge Design Manual (BDM) 2020 Section 305.

Based on the bedrock surface elevations determined from structure borings and historic borings and the proposed foundation bearing elevation, up to approximately 3.7 feet of bedrock excavation may be required to achieve the proposed foundation bearing elevation.

The bearing rock surface should be free of loose materials and should be in the relatively dry condition. Footings should be placed a minimum of 2.0 feet below the predicted scour elevation.

5.2 Lateral Earth Pressure Parameters

For the soil types encountered in the borings, the “in-situ” unit weight (γ), cohesion (c), effective angle of friction (ϕ'), and lateral earth pressure coefficients for at-rest conditions (k_o), active conditions (k_a), and passive conditions (k_p) have been estimated and are provided in Table 4 and Table 5.

Table 4. Estimated Undrained Soil Parameters for Design

Soil Type	γ (pcf) ¹	c (psf)	ϕ	k_a	k_o	k_p
Medium Stiff to Stiff Cohesive Soils	125	1,000	0°	N/A	N/A	N/A
Very Stiff to Hard Cohesive Soils	125	2,000	0°	N/A	N/A	N/A
Medium Dense Very Dense Granular Soil	130	0	32°	0.31	0.47	3.25
Compacted Cohesive Engineered Fill	125	2,000	0°	N/A	N/A	N/A
Compacted Granular Engineered Fill	130	0	33°	0.29	0.46	3.39

1. When below groundwater table, use effective unit weight, $\gamma' = \gamma - 62.4$ pcf and add hydrostatic water pressure.

Table 5. Estimated Drained Soil Parameters for Design

Soil Type	γ (pcf) ¹	c (psf)	ϕ'	k_a	k_o	k_p
Medium Stiff to Stiff Cohesive Soils	125	0	24°	0.42	0.59	2.37
Very Stiff Cohesive Soils	125	0	26°	0.39	0.56	2.56
Medium Dense to Very Dense Granular Soil	130	0	32°	0.31	0.47	3.25
Compacted Cohesive Engineered Fill	125	0	28°	0.36	0.53	2.76
Compacted Granular Engineered Fill	130	0	33°	0.29	0.46	3.39

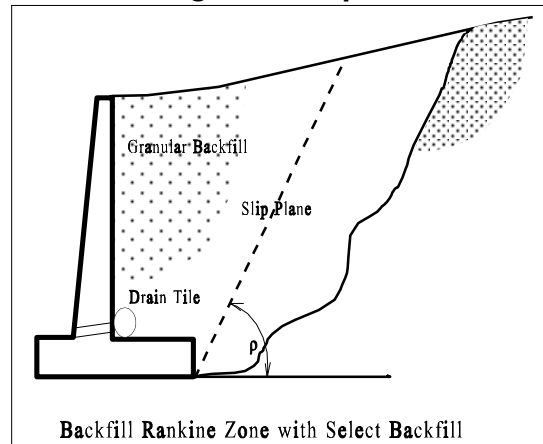
1. When below groundwater table, use effective unit weight, $\gamma' = \gamma - 62.4$ pcf and add hydrostatic water pressure.

These parameters are considered appropriate for the design of subsurface walls, wing walls, headwalls and excavation support systems. Subsurface structures (where the top of the structure is restrained from movement) should be designed based on at-rest conditions. For proposed wing walls or temporary retaining structures (where the top of the structure is allowed to move), earth pressure distributions should be based on active conditions (k_a) and passive pressure (k_p). The values in tables above have been estimated from correlation charts based on minimum standards specified for compacted engineered fill materials. These recommendations do not take into consideration the effect of any surcharge loading or a sloped ground surface (a flat surface is assumed). Earth pressures on excavation support systems will be dependent on the type of sheeting and method of bracing or anchorage.

In order to alleviate the build-up of hydrostatic pressure above the flow line of the stream behind the walls, a minimum of 2.0 feet of clean free-draining granular fill (i.e., No. 57 gravel) should be placed full depth behind the walls. If granular fill other than No. 57 gravel is used, it should not have more than 8 percent (by weight) passing the No. 200 screen, and should be compacted to 95 percent of the maximum dry density as determined by the Standard Proctor Test (ASTM D698). A perforated, corrugated drain tile, wrapped with filter fabric, should be placed along the perimeter at the base of the walls or at the design flow line/flood line for drainage purposes. A clay cap (minimum 1.0-foot thick) should be placed overtop the granular backfill to deter inflow of the surface water. The drainage system should properly outlet to creek/river or to a properly sized sump pump system.

The 2.0 feet of free draining material placed behind the wall prevents the formation of hydrostatic pressures as noted above. However, unless the free draining granular backfill is placed beyond the slip plane, it has no influence on the equivalent fluid weight of the soil. If free draining granular fill (meeting the requirements listed above) is to be placed beyond the slip plane ($\rho=45^\circ$ for at-rest conditions; $\rho=45^\circ+\phi/2$ for active conditions), the values presented for the compacted granular engineered fill can be employed, consequently lowering the pressures on the wall.

Figure 1. Slip Plane



5.3 Groundwater Considerations

Seepage was encountered in borings B-001-0-23 and B-002-0-23 at depths 6.0 and 11.0 feet below existing grade, respectively. Groundwater was not encountered during drilling. However, seepage water may be encountered during excavations within trapped/perched zones. These trapped/perched zones are generally the layer(s) of granular soils that are isolated within the fine-grained soil layers and may or may not be identified in boring logs. Additionally, groundwater should be anticipated within the granular layers at or near the normal pool elevation of George's Run. If excavation encounter such layers, temporary dewatering may be accomplished by placing localized sumps and pumps within and beyond the excavation. Seepage rates from these layers are difficult to predict and flow rate could be significant.

Where groundwater is encountered, proper groundwater control measures should be implemented to prevent disturbance to excavation bottoms consisting of cohesive soil, and to prevent the possible development of a quick or "boiling" condition if soft/loose silts and/or fine sands are encountered. It is preferable that the groundwater level, if encountered, be maintained at least 36.0 inches below the deepest excavation. Note that determining and maintaining actual groundwater levels during construction is the responsibility of the contractor.

5.4 Construction Considerations

All site work shall conform to local codes and to the latest ODOT Construction and Material Specifications (CMS).

5.5 Excavation Considerations

All excavations should be shored / braced or laid back at a safe angle in accordance to Occupational Safety and Health Administration (OSHA) guidelines. During excavation, if slopes cannot be laid back to OSHA Standards due to adjacent structures or other obstructions, temporary shoring may be required. The following table should be utilized as a general guide for implementing OSHA guidelines when estimating excavation back slopes at the various boring locations. Actual excavation back slopes must be field verified by qualified personnel at the time of excavation in strict accordance with OSHA guidelines.

Table 6. Excavation Back Slopes

Soil	Maximum Back Slope (H:V)	Notes
Soft to Medium Stiff Cohesive	1.5 : 1.0	Above Ground Water Table and No Seepage
Stiff Cohesive	1.0 : 1.0	Above Ground Water Table and No Seepage
Very Stiff to Hard Cohesive	0.75 : 1.0	Above Ground Water Table and No Seepage
All Granular & Cohesive Soil Below Ground Water Table or with Seepage	1.5 : 1.0	None
Upper 3 feet of Bedrock	0.75 : 1.0	None
Competent Bedrock	Vertical	None

6.0 LIMITATIONS OF STUDY

The recommendations presented in this report are predicated upon construction inspection by a qualified soil technician under the direct supervision of a professional geotechnical engineer. Adequate testing and inspection during construction are considered necessary to assure an adequate foundation system.

The recommendations for this project were developed utilizing soil and bedrock information obtained from the test borings that were made at the proposed site. At this time we would like to point out that soil borings only depict the soil and bedrock conditions at the specific locations and time at which they were made. The conditions at other locations on the site may differ from those occurring at the boring locations.

The conclusions and recommendations herein have been based upon the available soil information and the preliminary design details furnished by a representative of the owner of the proposed project. Any revision in the plans for the proposed construction from those anticipated in this report should be brought to the attention of the geotechnical engineer to determine whether any changes in the foundation or earthwork recommendations are necessary. If deviations from the noted subsurface conditions are encountered during construction, they should also be brought to the attention of the geotechnical engineer.

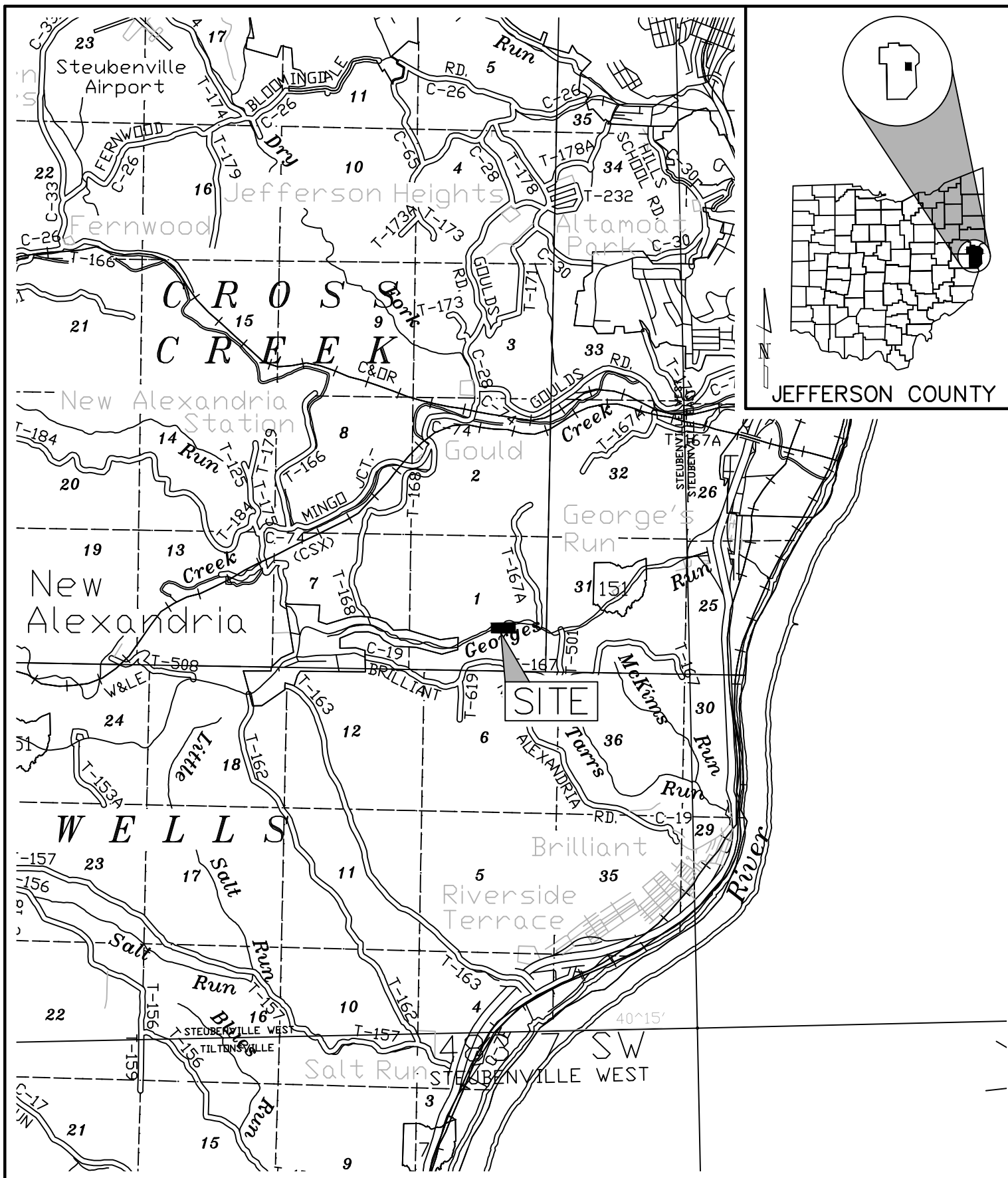
The scope of our services does not include any environmental assessment or investigation for the presence or absence of hazardous or toxic materials in the soil, groundwater or surface water within or beyond the site studied. Any statements in this report or on the test boring logs regarding odors, staining of soils or other unusual conditions observed are strictly for the information of our client.

Our professional services have been performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices. Resource International is not responsible for the conclusions, opinions or recommendations made by others based upon the data included.



APPENDIX I

VICINITY MAP AND BORING PLAN



VICINITY MAP

JEF-151-14.27 - SR 151 OVER GEORGES
RUN BRIDGE REPLACEMENT
JEFFERSON COUNTY, OHIO

RII PROJECT NO.
W-23-045

SCALE: 1"=5000'

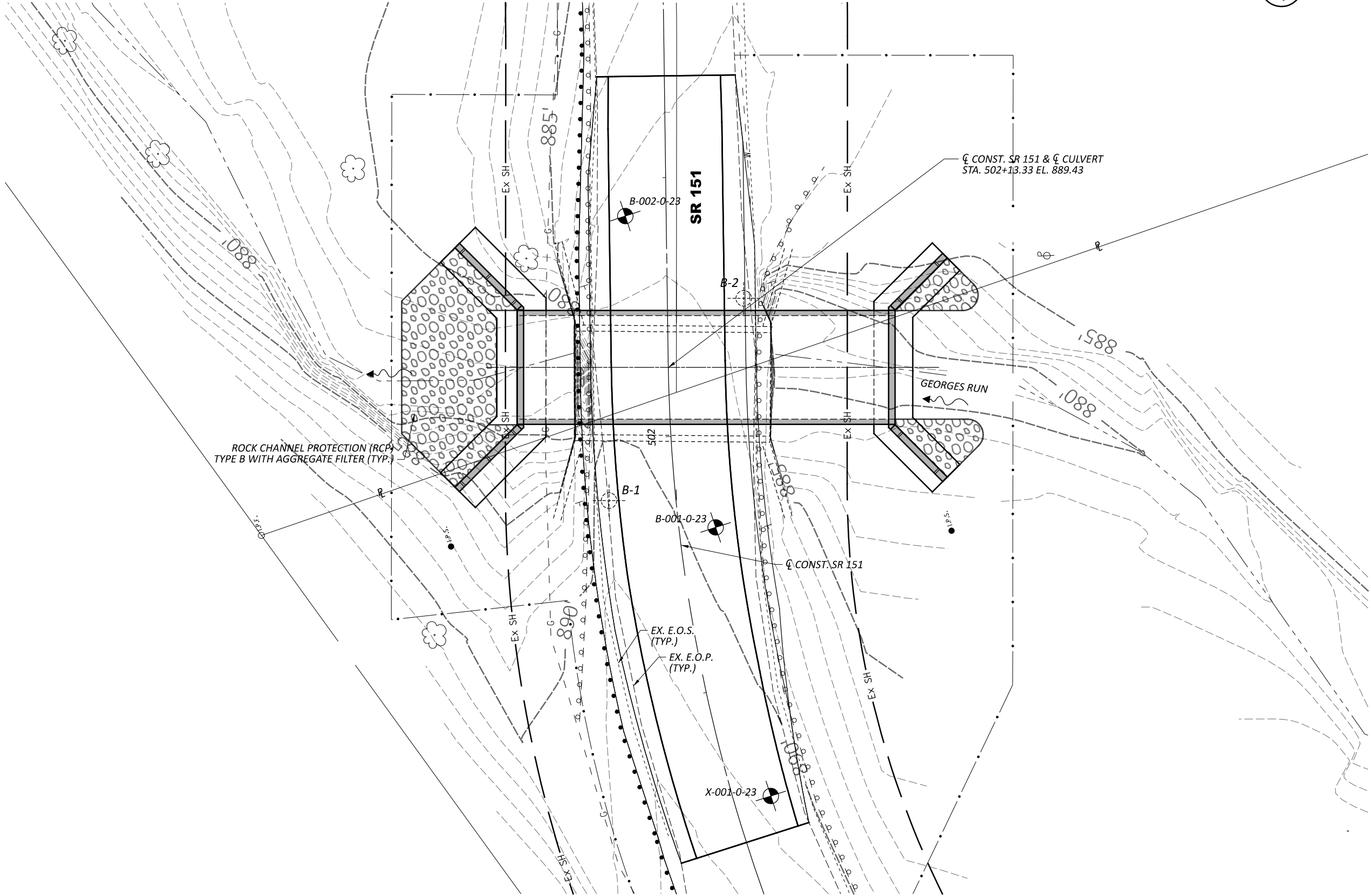
0 2500 5000

DRAWN
JAS

REVIEWED
DEK

DATE
5/25/2023





DESIGN AGENCY



6350 PRESIDENTIAL GATEWAY
 COLUMBUS, OHIO 43231
 (614) 823-4949

DESIGNER

JAS

REVIEWER

AG MM-DD-YY

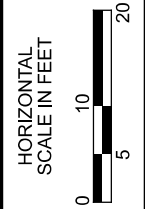
PROJECT ID

115933

SUBSET	TOTAL
0	0

SHEET	TOTAL
P.0	0

JEF-151-14.27 - SR 151 BRIDGE REPLACEMENT



APPENDIX II

DESCRIPTION OF SOIL TERMS



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	Classification		LL _O /LL x 100*	% Pass #40	% Pass #200	Liquid Limit (LL)	Plastic Index (PI)	Group Index Max.	REMARKS
		AASHTO	OHIO							
	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	
	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			Uncontrolled Fill (Describe)			Bouldery Zone			Peat
	Pavement or Base									

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

DESCRIPTION OF SOIL TERMS

The following terminology was used to describe soils throughout this report and is generally adapted from ASTM 2487/2488 and ODOT Specifications for Geotechnical Explorations.

Granular Soils - The relative compactness of granular soils is described as:

ODOT A-1, A-2, A-3, A-4 (non-plastic) or USCS GW, GP, GM, GC, SW, SP, SM, SC, ML (non-plastic)

<u>Description</u>	<u>Blows per foot – SPT (N₆₀)</u>		
Very Loose	Below		5
Loose	5	-	10
Medium Dense	11	-	30
Dense	31	-	50
Very Dense	Over		50

Cohesive Soils - The relative consistency of cohesive soils is described as:

ODOT A-4, A-5, A-6, A-7, A-8 or USCS ML, CL, OL, MH, CH, OH, PT

<u>Description</u>	<u>Unconfined Compression (tsf)</u>		
Very Soft	Less than		0.25
Soft	0.25	-	0.5
Medium Stiff	0.5	-	1.0
Stiff	1.0	-	2.0
Very Stiff	2.0	-	4.0
Hard	Over		4.0

Gradation - The following size-related denominations are used to describe soils:

<u>Soil Fraction</u>	<u>USCS Size</u>	<u>ODOT Size</u>
Boulders	Larger than 12"	Larger than 12"
Cobbles	12" to 3"	12" to 3"
Gravel coarse	3" to ¾"	3" to ¾"
fine	¾" to 4.75 mm (¾" to #4 Sieve)	¾" to 2.0 mm (¾" to #10 Sieve)
Sand coarse	4.75 mm to 2.0 mm (#4 to #10 Sieve)	2.0 mm to 0.42 mm (#10 to #40 Sieve)
medium	2.0 mm to 0.42 mm (#10 to #40 Sieve)	-
fine	0.42 mm to 0.074 mm (#40 to #200 Sieve)	0.42 mm to 0.074 mm (#40 to #200 Sieve)
Silt	0.074 mm to 0.005 mm (#200 to 0.005 mm)	0.074 mm to 0.005 mm (#200 to 0.005 mm)
Clay	Smaller than 0.005 mm	Smaller than 0.005 mm

Modifiers of Components - Modifiers of components are as follows:

<u>Term</u>	<u>Range</u>		
Trace	0%	-	10%
Little	10%	-	20%
Some	20%	-	35%
And	35%	-	50%

Moisture Table - The following moisture-related denominations are used to describe cohesive soils:

<u>Term</u>	<u>Range - USCS</u>	<u>Range - ODOT</u>
Dry	0% to 10%	Well below Plastic Limit
Damp	>2% below Plastic Limit	Below Plastic Limit
Moist	2% below to 2% above Plastic Limit	Above PL to 3% below LL
Very Moist	>2% above Plastic Limit	
Wet	≥ Liquid Limit	3% below LL to above LL

Organic Content – The following terms are used to describe organic soils:

<u>Term</u>	<u>Organic Content (%)</u>
Slightly organic	2-4
Moderately organic	4-10
Highly organic	>10

Bedrock – The following terms are used to describe the relative strength of bedrock:

<u>Description</u>	<u>Field Parameter</u>
Very Weak	Can be carved with knife and scratched by fingernail. Pieces 1 in. thick can be broken by finger pressure.
Weak	Can be grooved or gouged with knife readily. Small, thin pieces can be broken by finger pressure.
Slightly Strong	Can be grooved or gouged 0.05 in deep with knife. 1 in. size pieces from hard blows of geologist hammer.
Moderately Strong	Can be scratched with knife or pick. 1/4 in. size grooves or gouges from blows of geologist hammer.
Strong	Can be scratched with knife or pick with difficulty. Hard hammer blows to detach hand specimen.
Very Strong	Cannot be scratched by knife or pick. Hard repeated blows of geologist hammer to detach hand specimen.
Extremely Strong	Cannot be scratched by knife or pick. Hard repeated blows of geologist hammer to chip hand specimen.

DESCRIPTION OF ROCK TERMS

The following terminology was used to describe the rock throughout this report and is generally adapted from ASTM D5878 and the ODOT Specifications for Geotechnical Explorations.

Weathering – Describes the degree of weathering of the rock mass:

<u>Description</u>	<u>Field Parameter</u>
Unweathered	No evidence of any chemical or mechanical alteration of the rock mass. Mineral crystals have a right appearance with no discoloration. Fractures show little or not 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.

Strength of Bedrock – The following terms are used to describe the relative strength of bedrock:

<u>Description</u>	<u>Field Parameter</u>
Very Weak	Can be carved with knife and scratched by fingernail. Pieces 1 in. thick can be broken by finger pressure.
Weak	Can be grooved or gouged with knife readily. Small, thin pieces can be broken by finger pressure.
Slightly Strong	Can be grooved or gouged 0.05 in deep with knife. 1 in. size pieces from hard blows of geologist hammer.
Moderately Strong	Can be scratched with knife or pick. 1/4 in. size grooves or gouges from blows of geologist hammer.
Strong	Can be scratched with knife or pick with difficulty. Hard hammer blows to detach hand specimen.
Very Strong	Cannot be scratched by knife or pick. Hard repeated blows of geologist hammer to detach hand specimen.
Extremely Strong	Cannot be scratched by knife or pick. Hard repeated blows of geologist hammer to chip hand specimen.

Bedding Thickness – Description of bedding thickness as the average perpendicular distances between bedding surfaces:

<u>Description</u>	<u>Thickness</u>
Very Thick	Greater than 36 inches
Thick	18 to 36 inches
Medium	10 to 18 inches
Thin	2 to 10 inches
Very Thin	0.4 to 2 inches
Laminated	0.1 to 0.4 inches
Thinly Laminated	Less than 0.1 inches

Fracturing – Describes the degree and condition of fracturing (fault, joint, or shear):

Degree of Fracturing

<u>Description</u>	<u>Spacing</u>
Unfractured	Greater than 10 feet
Intact	3 to 10 feet
Slightly Fractured	1 to 3 feet
Moderately Fractured	

Aperture Width

<u>Description</u>	<u>Width</u>
Open	Greater than 0.2 inches
Narrow	0.05 to 0.2 inches
Tight	Less than 0.05 inches

Surface Roughness

<u>Description</u>	<u>Criteria</u>
Very Rough	Near vertical steps and ridges occur on surface
Slightly Rough	Asperities on the surfaces distinguishable
Slickensided	Surface has smooth, glassy finish, evidence of Striations

RQD – Rock Quality Designation (calculation shown in report) and Rock Quality (ODOT, GB 3, January 13, 2006):

<u>RQD %</u>	<u>Rock Index Property Classification (based on RQD, not slake durability index)</u>
0 – 25%	Very Poor
26 – 50%	Poor
51 – 70%	Fair
71 – 85%	Good
86 – 100%	Very Good

APPENDIX III

BORING LOGS:

B-001-0-23 and B-002-0-23

ROCK CORE PHOTOGRAPHS

BORING LOGS

Definitions of Abbreviations

AS	=	Auger sample
GI	=	Group index as determined from the Ohio Department of Transportation classification system
HP	=	Unconfined compressive strength as determined by a hand penetrometer (tons per square foot)
LL _o	=	Oven-dried liquid limit as determined by ASTM D4318. Per ASTM D2487, if LL _o /LL is less than 75 percent, soil is classified as "organic".
LOI	=	Percent organic content (by weight) as determined by ASTM D2974 (loss on ignition test)
PID	=	Photo-ionization detector reading (parts per million)
QR	=	Unconfined compressive strength of intact rock core sample as determined by ASTM D2938 (pounds per square inch)
QU	=	Unconfined compressive strength of soil sample as determined by ASTM D2166 (pounds per square foot)
RC	=	Rock core sample
REC	=	Ratio of total length of recovered soil or rock to the total sample length, expressed as a percentage
RQD	=	Rock quality designation – estimate of the degree of jointing or fracture in a rock mass, expressed as a percentage:

$$\frac{\sum \text{segments equal to or longer than 4.0 inches}}{\text{core run length}} \times 100$$

S	=	Sulfate content (parts per million)
SPT	=	Standard penetration test blow counts, per ASTM D1586. Driving resistance recorded in terms of blows per 6-inch interval while letting a 140-pound hammer free fall 30 inches to drive a 2-inch outer diameter (O.D.) split spoon sampler a total of 18 inches. The second and third intervals are added to obtain the number of blows per foot (N _m).
N ₆₀	=	Measured blow counts corrected to an equivalent (60 percent) energy ratio (ER) by the following equation: N ₆₀ = N _m *(ER/60)
SS	=	Split spoon sample
2S	=	For instances of no recovery from standard SS interval, a 2.5 inch O.D. split spoon is driven the full length of the standard SS interval plus an additional 6.0 inches to obtain a representative sample. Only the final 6.0 inches of sample is retained. Blow counts from 2S sampling are not correlated with N ₆₀ values.
3S	=	Same as 2S, but using a 3.0 inch O.D. split spoon sampler.
TR	=	Top of rock
W	=	Initial water level measured during drilling
▼	=	Water level measured at completion of drilling

Classification Test Data

Gradation (as defined on Description of Soil Terms):

GR	=	% Gravel
SA	=	% Sand
SI	=	% Silt
CL	=	% Clay

Atterberg Limits:

LL	=	Liquid limit
PL	=	Plastic limit
PI	=	Plasticity Index
WC	=	Water content (%)



PROJECT: JEF-151-14.27
 TYPE: STRUCTURE
 PID: 115933 BR ID:
 START: 4/18/23 END: 4/18/23

DRILLING FIRM / OPERATOR: RII / TG
 SAMPLING FIRM / LOGGER: RII / MJ
 DRILLING METHOD: 3.25" HSA / NQ
 SAMPLING METHOD: SPT / NQ

DRILL RIG: DIEDRICH D-50 (# 313)
 HAMMER: AUTOMATIC
 CALIBRATION DATE: 3/21/22
 ENERGY RATIO (%): 86.4

STATION / OFFSET: 501+81.5 / 7' RT.
 ALIGNMENT: CL SR 151
 ELEVATION: 889.6 (MSL) EOB: 25.0 ft.
 COORD: 40.293809, -80.651112

EXPLORATION ID
B-001-0-23

PAGE
 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	HOLE SEALED
								GR	CS	FS	SI	CL	LL	PL	PI			
1.7'- ASPHALT (19.5")	889.6																	
0.2'- AGGREGATE BASE (2.5")	887.9	1																
STIFF TO VERY STIFF, BROWN TO DARK BROWN SILT AND CLAY , "AND" COARSE TO FINE SAND, LITTLE FINE GRAVEL, MOIST.	887.7	2	7															
		3	6	14	72	SS-1	2.00	-	-	-	-	-	-	-	-	19	A-6a (V)	
		4	3	10	61	SS-2	1.50	11	20	16	31	22	34	20	14	21	A-6a (5)	
		5	3															
		6	3															
		7	3	9	39	SS-3	-	-	-	-	-	-	-	-	-	13	A-6a (V)	
		8																
		9	3															
		10	5	19	89	SS-4	-	-	-	-	-	-	-	-	-	18	A-6a (V)	
	878.6	11	13															
MEDIUM DENSE TO VERY DENSE, DARK BROWN GRAVEL WITH SAND, SILT, AND CLAY , MOIST.		12	9	22	61	SS-5	-	28	24	15	20	13	28	17	11	8	A-2-6 (0)	
		13																
		14	5	65	100	SS-6	-	-	-	-	-	-	-	-	-	11	A-2-6 (V)	
		15	25															
		16	20															
SANDSTONE : BROWN, SEVERELY TO HIGHLY WEATHERED, VERY WEAK.	873.6	16	60/3"	-	40	SS-7	-	-	-	-	-	-	-	-	-	-	Rock (V)	
		17																
		18																
	870.6	19																
SANDSTONE : BROWN TO GRAY, MODERATELY WEATHERED, MODERATELY STRONG, MEDIUM GRAINED, THICK BEDDED, FRACTURED TO MODERATELY FRACTURED, NARROW APERTURES, SLIGHTLY ROUGH SURFACES, BLOCKY, GOOD SURFACE CONDITION. -NQ2-2: Qu @ 20.5' = 7,718 PSI		20	0		58	NQ2-1											CORE	
		21																
		22																
		23	28		83	NQ2-2											CORE	
		24																
	864.6	25																

NOTES: SEEPAGE @ 6.0'; GROUNDWATER WAS NOT ENCOUNTERED DRILLING AND WAS NOT RECORDED AT COMPLETION DUE TO THE USE OF WASH WATER DURING ROCK CORING.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PUMPED 47 LBS CEMENT / 25 LBS BENTONITE POWDER / 40 GAL WATER. PAVEMENT PATCHED WITH ASPHALT COLD PATCH .

	PROJECT: JEF-151-14.27	DRILLING FIRM / OPERATOR: RII / TG	DRILL RIG: DIEDRICH D-50 (# 313)	STATION / OFFSET: 502+42.6 / 8' LT.	EXPLORATION ID B-002-0-23
	TYPE: STRUCTURE	SAMPLING FIRM / LOGGER: RII / MJ	HAMMER: AUTOMATIC	ALIGNMENT: CL SR 151	
	PID: 115933 BR ID:	DRILLING METHOD: 3.25" HSA / NQ	CALIBRATION DATE: 3/21/22	ELEVATION: 888.3 (MSL) EOB: 30.5 ft.	PAGE 1 OF 2
	START: 4/18/23 END: 4/18/23	SAMPLING METHOD: SPT / NQ	ENERGY RATIO (%): 86.4	COORD: 40.293807, -80.650943	

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTH	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	HOLE SEALED
								GR	CS	FS	SI	CL	LL	PL	PI			
1.7'- ASPHALT (20.0")	888.3																	
0.3'- AGGREGATE BASE (3.0")	886.6	1																
MEDIUM DENSE, GRAY GRAVEL WITH SAND, SILT, AND CLAY, MOIST.	886.3	2	7	19	61	SS-1	-	37	30	18	11	4	29	18	11	12	A-2-6 (0)	
	884.8	3	8 5															
MEDIUM STIFF TO VERY STIFF, GRAY TO DARK BROWNISH GRAY CLAY, SOME SILT, SOME COARSE TO FINE SAND, LITTLE FINE GRAVEL, MOIST TO DAMP.		4	3	9	39	SS-2	1.00	-	-	-	-	-	-	-	-	17	A-7-6 (V)	
		5																
		6	1															
		7	1 3	6	89	SS-3	1.00	13	17	14	33	23	41	23	18	32	A-7-6 (8)	
		8																
		9	3															
		10	5 7	17	33	SS-4	-	-	-	-	-	-	-	-	-	15	A-7-6 (V)	
		11																
		12	5 7 9	23	81	SS-5	-	-	-	-	-	-	-	-	-	8	A-7-6 (V)	
	874.8	13																
SANDSTONE : BROWN, HIGHLY TO SEVERELY WEATHERED, VERY WEAK.		14	29 50/4"	-	100	SS-6	-	-	-	-	-	-	-	-	-	-	Rock (V)	
		15																
	871.8	16																
SANDSTONE : BROWN, HIGHLY WEATHERED TO SLIGHTLY WEATHERED, WEAK TO MODERATELY STRONG, MEDIUM GRAINED, MEDIUM TO THICK BEDDED, HIGHLY FRACTURED TO MODERATELY FRACTURED, NARROW TO OPEN APERTURES, SLIGHTLY ROUGH SURFACES, BLOCKY/DISTURBED/SEAMY, FAIR SURFACE CONDITION, TRACE COAL LAMINAE.		17																
-NQ2-1: Estimated Qu @ 16.5' 17.0'= 4,522 psi		18	0		63	NQ2-1											CORE	
-17'-25.8' Highly Weathered, Highly fractured, Weak		19																
		20																
		21																
		22																
		23	0		68	NQ2-2											CORE	
		24																

0-2018-ODOT BORING LOG RII - OH DOT.GDT - 11/24/25 16:22 - U:\GIS\PROJECTS\2023\W-23-045.GPJ

PID: 115933 BR ID:		PROJECT: JEF-151-14.27		STATION / OFFSET: 502+42.6 / 7.88 LT		START: 4/18/23		END: 4/18/23		PG 2 OF 2		B-002-0-23							
MATERIAL DESCRIPTION AND NOTES		ELEV. 863.3	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	HOLE SEALED
									GR	CS	FS	SI	CL	LL	PL	PI			
SANDSTONE : BROWN, HIGHLY WEATHERED TO SLIGHTLY WEATHERED, WEAK TO MODERATELY STRONG, MEDIUM GRAINED, MEDIUM TO THICK BEDDED, HIGHLY FRACTURED TO MODERATELY FRACTURED, NARROW TO OPEN APERATURES, SLIGHTLY ROUGH SURFACES, BLOCKY/DISTURBED/SEAMY, FAIR SURFACE CONDITION, TRACE COAL LAMINAE. <i>(same as above)</i> 16.5' -17.0' & 25.8' - 30.5' Slightly Weathered, Fractured to Moderately fractured, Moderately Strong		857.8	EOB	26	48	97	NQ2-3										CORE		
				27															
				28															
				29															
				30															
NOTES: SEEPAGE @ 11.0'; GROUNDWATER WAS NOT ENCOUNTERED DRILLING AND WAS NOT RECORDED AT COMPLETION DUE TO THE USE OF WASH WATER DURING ROCK CORING.																			
ABANDONMENT METHODS, MATERIALS, QUANTITIES: PUMPED 47 LBS CEMENT / 25 LBS BENTONITE POWDER / 40 GAL WATER. PAVEMENT PATCHED WITH ASPHALT COLD PATCH .																			

APPENDIX IV

LABORATORY TEST RESULTS



RESOURCE INTERNATIONAL, INC.
Engineering Consultants

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Columbus, OH 43231

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9885 Rockside Road

Cleveland, OH 44125

Phone (216) 573-0955

4480 Lake Forest Drive

Cincinnati, Ohio 45242

Phone (513) 769-6998

**Unconfined Compressive Strength
of Intact Rock Core Specimens**
(ASTM D 7012-14)

Project: JEF-151-14.27

Project No.: W-23-045

Date of Testing: 5/18/2023

Test Performed by: E.M.

Rock Description: Gray Sandstone

Boring No.: B-001-0-23

Sample No.: NQ2-2

Depth (ft): 20.5-21.0

Moisture condition: As received

Testing Temperature: 21°C

Rate of Loading: 55.3 lbs/sec

Testing Time: 252 sec (Rate 2-15 minutes to failure)

Average Length: 4.065 in

Average Diameter: 1.981 in

Length to diameter ratio: 2.052

Cross Sectional Area: 3.082 in²

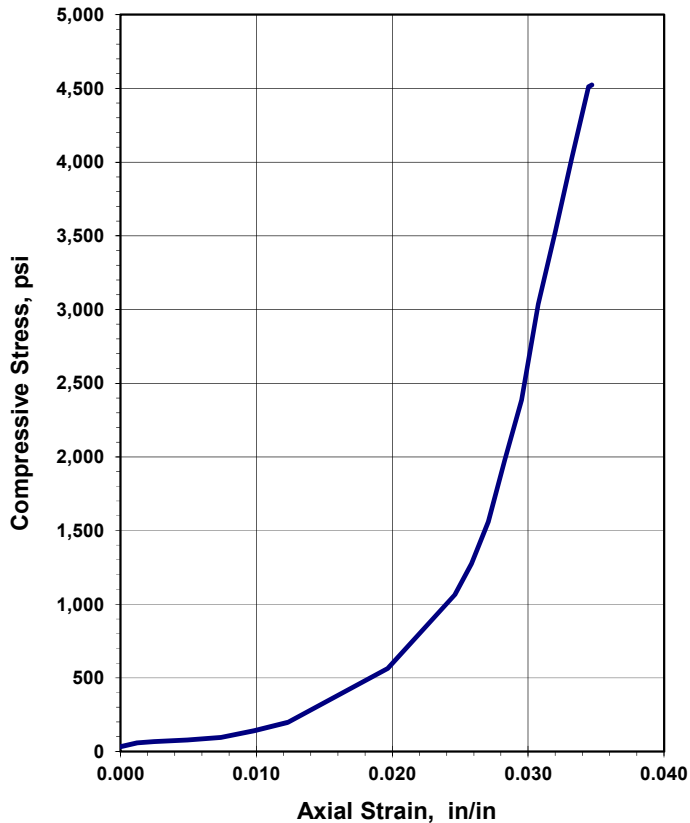
Volume: 0.0073 ft³

Failure Load: 13,939 lbs

Axial Strain at Failure: 0.0347 in/in

Stress: 4,522 psi

Unconfined Compression Test



Before Testing



After Failure



REMARKS: _____



RESOURCE INTERNATIONAL, INC.
Engineering Consultants

**Point Load Strength Index
of Rock Specimens
(ASTM D 5731-08)**

6350 Presidential Gatew.
Columbus, OH 43231
Phone (614) 823-4949

9885 Rockside Road
Cleveland, OH 44125
Phone (216) 573-0955

4480 Lake Forest Drive
Cincinnati, Ohio 45242
Phone (513) 769-6998

Project: JEF-151-14.27
Project No.: W-23-045
Date of Testing: 5/15/2023
Test Performed by: EM

Rock Description: Sandstone

Boring No.: B-002-0-23
Station / Offset: NA
Sample No. / Depth: RC-1

Test Apparatus: Forney-LA 0080
Serial Number: A125/AZ/0014
Date of Calibration: 6/12/2022

Sample No.	Test Type	Depth (ft)	W (mm)	D (mm)	Load (N)	D_e^2 (mm ²)	D_e (mm)	F	Is (MPa)	Is ₍₅₀₎ (MPa)	σ_c (MPa)
1	a $\perp$	16.5-17.0	50.1	26.4	3,497.700	1,685	41.1	0.92	2.08	1.90	49.81
2	a $\perp$	16.5-17.0	50.1	29.9	4,049.500	1,908	43.7	0.94	2.12	2.00	50.95
3	a $\perp$	16.5-17.0	50.1	30.3	3,524.400	1,936	44.0	0.94	1.82	1.72	43.70
3	a $\perp$	16.5-17.0	50.1	25.3	3,982.800	1,616	40.2	0.91	2.46	2.23	59.14
3	a $\perp$	16.5-17.0	50.2	23.4	3,893.800	1,496	38.7	0.89	2.60	2.32	62.47

STATISTICS

Mean Is ₍₅₀₎ $\perp$	2.03 MPa (295 psi)
Mean Is ₍₅₀₎ $\parallel$	
la ₍₅₀₎	

Specific Specimen Shape:

d = diametrical
a = axial
b = block
i = irregular lump
 $\perp$ = perpendicular to bedding plane
 $\parallel$ = parallel to bedding plane

Estimated Uniaxial Compression, $\sigma_c = K \cdot Is$

K = 24
*Per Section 206.1.3 of 2011 ODOT
Rock Slope Design Guide

Mean σ_c = **53.21 MPa (7,718 psi)**

Remarks: _____

APPENDIX V

HISTORIC BORING LOGS

B-1 and B-2

LOG OF BORING

Date Started 10-15-83

Sensor Type

Dia

 $1 \frac{3}{8} \frac{3}{4}$

Water Elev. _____

Date Completed 11-17-83

Dio

Boring No. **B-11**

Station & Offset 7764+922, 131 E.F.

Surface Elev 890.4'

[illegible]

LOG OF BORING

Date Started 11/18/83Sampler Type SSS Dia. 1 3/8"

Water Elev. _____

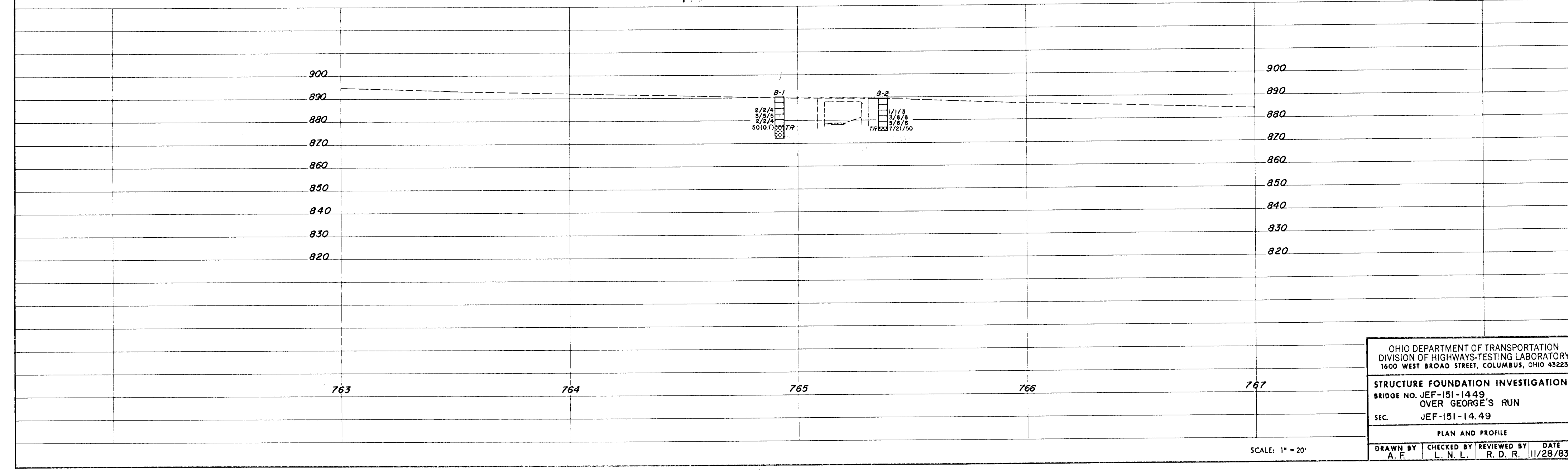
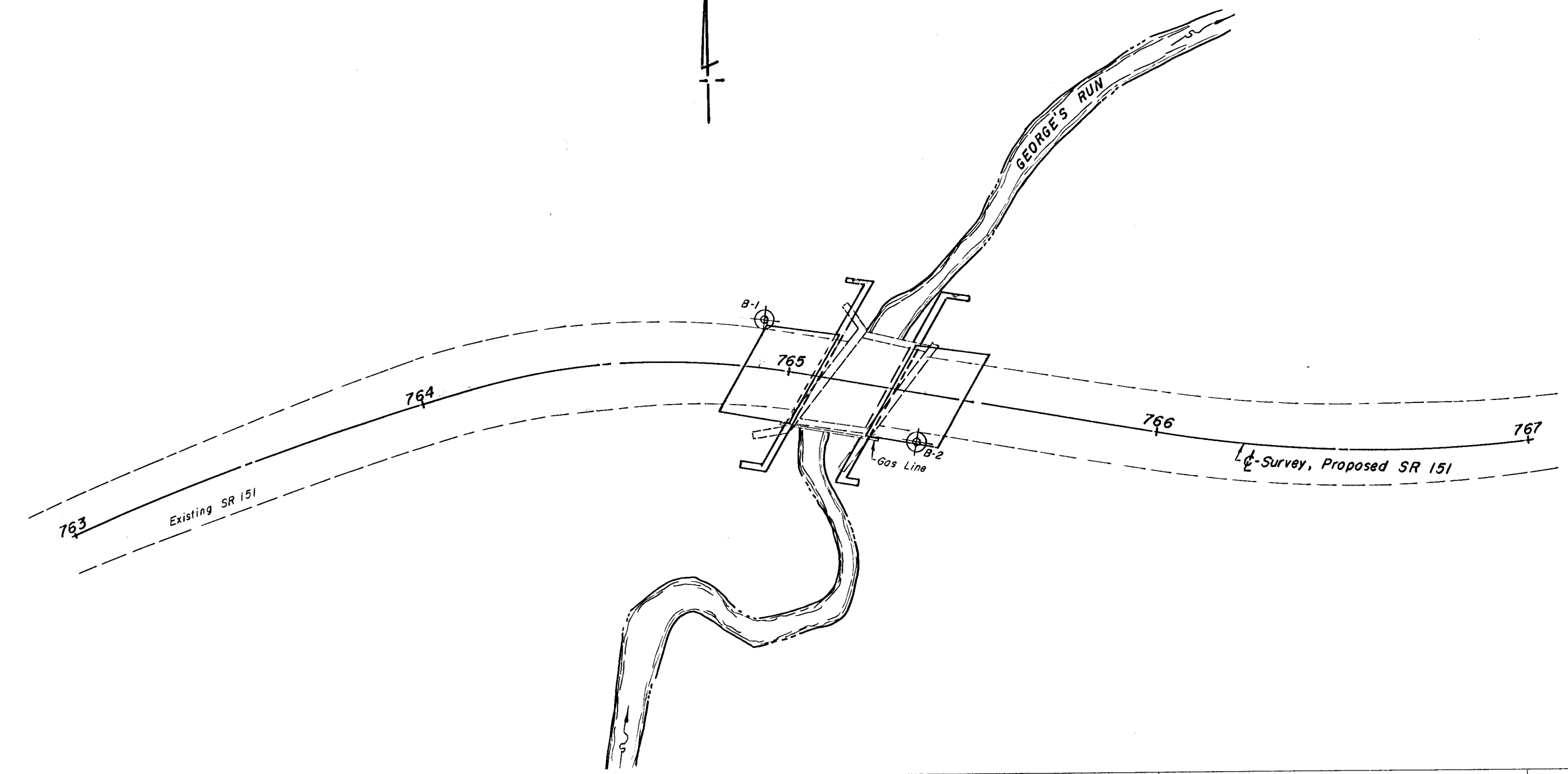
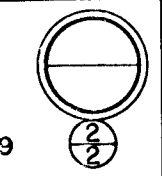
Date Completed 11/21/83

Casing: Length _____ Dia. _____

Boring No. C-B-2Station & Offset 76755+377, 113' RT.Surface Elev. 889.33'

Elev.	Depth	Std. Pen. (N)	Rec. ft.	Loss ft.	Description	Sample No.	Physical Characteristics								SHTL Class.
							% Agg	% C.S.	% F.S.	% Silt	% Clay	LL	PI	W.C	
889.3	0														
886.8	2				BERM MATERIAL AND BOULDERS										
884.3	4				BROWN SANDY SILT WITH GRAVEL										
881.8	6	1/1/3			GRAY SANDY GRAVELLY SILT	5	24	8	13	26	29	35	10	27	A-4-a
879.3	8	3/6/8			BROWN SILTY SANDY GRAVEL	6	69	8	10	8	7	NP	NP	12	A-4-a
876.8	10	5/6/8			BROWN SILTY SANDY GRAVEL	7	55	10	10	17	14	30	11	13	A-2-b
875.3	12				TOP OF ROCK										
875.3	14	7/21/50			GRAY BROKEN SANDSTONE	8	47	7	15	27	10	NP	NP	8	VISUAL

C.B. BOTTOM OF BORING



OHIO DEPARTMENT OF TRANSPORTATION DIVISION OF HIGHWAYS-TESTING LABORATORY 1600 WEST BROAD STREET, COLUMBUS, OHIO 43223			
STRUCTURE FOUNDATION INVESTIGATION			
BRIDGE NO. JEF-151-14.49 OVER GEORGE'S RUN			
SEC. JEF-151-14.49			
PLAN AND PROFILE			
DRAWN BY A. F.	CHECKED BY L. N. L.	REVIEWED BY R. D. R.	DATE 11/28/83

SCALE: 1" = 20'

APPENDIX VI

PAVEMENT CORE REPORT



9885 Rockside Road
Suite 145
Cleveland, OH 44125
Telephone: (216) 573-0955

Pavement Core Data Summary

PROJECT	JEF-151-1427
JOB No.	W-23-045
Location:	40.293845°
	-80.65143°
BORING/CORE No.	X-001-0-23
DATE CORE OBTAINED	4/18/2023
CORE OBTAINED BY	TG, IS, MJ

Core Composition										Comments/Remarks
Core Number	Layer Thickness (in.)	Pavement Layer Number	Asphalt			Concrete	Aggregate/Granular Base		Other	
			Surface Binder	Intermediate Binder	Base Binder					
X-001-0-23	1.50	10	✓							- Layers 7, 8 & 10 have a trace of voids. - Layers 1, 3, 4 & 5 are slag mixes. - Layer 2 is a slag mix, it is broken @ 16.00". - The core separated between layers 2, 3, 4, 5 & 6.
	1.50	9	✓							
	2.50	8	✓							
	1.50	7		✓						
	1.00	6	✓							
	3.00	5	✓							
	2.00	4	✓							
	2.50	3	✓							
	2.00	2	✓							
	2.50	1			✓					

Total Pavement Thickness = 20.00 in. Total Asphalt Thickness = 20.00 in. Total Concrete Thickness = 0.00 in. Total Base Thickness = ND



APPENDIX VII

ANALYSIS CALCULATIONS

W-23-045; JEF-151-14.27 **115933**

Wingwall/Headwall -Shallow Foundation Bearing Resistance

Borings B-001-0-23 and B-002-0-23

Proposed headwalls/wingwalls foundation bearing elevations are 876.37 and 874.22 feet msl at the inlet and outlet, respectively

At and near the estimated wall bearing elevation severely to highly weathered bedrock is encountered

B =	2.0	ft	(Assumed)	
L =	51.5	ft	(Assumed)	
Su=	6,000	psf	(Shear strength of foundation soil/rock)	Qu estimated from FHWA-ICT-017-018 (Stark et al, 2017)
γ =	130	pcf	(Unit weight)	
D _f =	2.0	ft	(Minimum foundation embedment depth)	
φ =	0	deg	(Friction angle of foundation soil)	
D _w =	0.0	ft		

Nominal Bearing Resistance (q_n)

$$q_n = cN_{cn} + \gamma D_f N_{qm} C_{wq} + \frac{1}{2} \gamma B N_{\gamma m} C_{w\gamma} = 31.20 \text{ ksf}$$

$$N_{cn} = N_c s_c i_c = 5.18 \quad N_{qm} = N_q s_q d_q i_q = 1.00 \quad N_{\gamma m} = N_\gamma s_\gamma i_\gamma = 0.00$$

N _c =	5.14
N _q =	1.00
N _γ =	0.00

s _c =	1.008
s _q =	1.000
s _γ =	0.984

i _c =	1.000
i _q =	1.000
i _γ =	1.000

d _q =	1.000
C _{wq} =	0.500
C _{wγ} =	0.500

Factored Bearing Resistance (q_R)

$$q_R = q_n \cdot \phi_b = 17.2 \text{ ksf}$$

$$\phi_b = 0.55$$