Geotechnical Exploration Report

MCE-CR 52 Bridge #449 Replacement
Marion Township, Morgan County, Ohio

Prepared for

Morgan County Engineer
155 East Main Street
Room 208
McConnelsville, Ohio 43756

Prepared by

Professional Service Industries, Inc.
4960 Vulcan Avenue
Columbus, Ohio 43228

May 10, 2018

PSI Project No. 01021274

John Xu, P.E.
Senior Project Manager

Paul S. Hundley, P.E.
Regional Engineer/Principal
Consultant

Professional Service Industries, Inc.
4960 Vulcan Ave.
Columbus, OH 43228
Phone: (614) 876-8000

May 10, 2018

Mr. Stevan Hook, P.E., P.S.
Morgan County Engineer
155 East Main Street, Room 208
McConnelsville, OH 43756

Re: **Geotechnical Exploration Report**
MRG-CR 52 Bridge #449 Replacement
Marion Township, Morgan County, Ohio
PSI Project No. 01021274

Dear Mr. Hook:

Thank you for choosing Professional Service Industries, Inc. (PSI), an Intertek company, as your consultant for the CR 52 Bridge project in Morgan County, Ohio. The information you requested is attached.

PSI performed the geotechnical engineering study that you requested in general accordance with our agreement dated December 28th, 2017. PSI transmits one (1) copy with this letter.

We thank you for your business and we look forward to finding ways to grow our partnership, expand our services, and continue Building Better Together.

Respectfully submitted,
PROFESSIONAL SERVICE INDUSTRIES, INC.

Daniel Karch
Staff Engineer
EI # EI.00583

John Xu
Senior Project Manager
OH, PE #60483, Exp. 12/31/19

Paul Hundley
Regional Engineer/Principal
OH, PE #43741, Exp. 12/31/19

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STATE OF OHIO
JOHN
XU
60483
REGISTERED
PROFESSIONAL ENGINEER

MM/JX/PSH/mm

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1 EXECUTIVE SUMMARY

The bridge site is located along County Road 52 (CR-52) in Marion Township, Morgan County, Ohio. The project bridge is located along a 2-lane roadway section. The existing bridge spans Goshen Creek. It is estimated that the water level is approximately 11 to 12 feet below the bridge deck.

The Morgan County Engineer's staff selected the number of test borings and the test boring locations were marked in the field by PSI's engineering staff prior to field exploration. Two (2) test borings, B-001 and B-002, were advanced in the vicinity of the proposed rear and forward abutments of bridge CR-52 #449. Both B-001 and B-002 were drilled in the shoulder approximately 10 feet and 14.5 feet, respectively, south of the centerline of the roadway. B-001 was drilled approximately 16 feet west of the of the western side of the bridge, while B-002 was drilled approximately 14 feet east of the eastern side of the bridge.

Boring B-001 encountered cohesive soil until approximately 9.5 feet before transitioning to non-cohesive soils that continued reaching shale bedrock at a depth of approximately 13.5 feet below surface grade. The cohesive soils consisted of A-6a classifications and the non-cohesive soils consisted of A-2-4. Based on the boring information, soils between approximately 3.5 to 9.0 feet in boring B-001 consisted of generally medium stiff soil. Below these depths, "medium dense" granular soils and "very weak" to "slightly strong" bedrock were found.

Test boring B-002 encountered cohesive soil until approximately 10.5 before transitioning to a thin layer of silt above shale bedrock. The shale bedrock was encountered to the termination depth in boring B-002. The cohesive soils consisted of A-7-6, A-6a, and A-6b classifications.

Standard Penetration Testing (SPT) was performed at each sampling interval. SPT "N" values varied at different sampling depths, but high "N_{60}" values were encountered below approximately 13.5 feet in borings B-001 and B-003. Auger refusal in rock was not encountered in either test borings. Both test borings were terminated after drilling to depths of 45 and 34 feet, respectively.

Free water was encountered during drilling activities at depths of approximately 10.3 and 10.7 feet below surface grades in borings B-001 and B-002, respectively. Free water was observed upon completion of drilling at depths of 10.3 and 6.0 feet, respectively, in borings B-001 and B-002. After removing the augers from the boreholes, the measured water depths were 9.5 feet in B-001 and 8.2 feet in B-002.

Spread footings or drilled shafts are recommended for this project. Approximate shaft lengths of 18.5 feet were estimated for the project.

This summary should be used in conjunction with the entire exploration report since this summary sheet cannot include all details of the preliminary exploration findings.

2 PROJECT STRUCTURE INFORMATION

2.1 PROJECT STRUCTURE DESCRIPTION

According to information provided by the Morgan County Engineer on May 8, 2018, the project involves design and re-construction of a new county road bridge over the Goshen Creek in the Marion Township of Morgan County, Ohio. The existing bridge is a two-span steel girder bridge constructed in 1981 with and overall length of 57 feet and width of 14 feet and 7 inches. The existing bridge also underwent major reconstruction in 1990 in which the pier and stringers were added. The condition of the existing bridge is poor, and deterioration of the steel members can be seen on the existing bridge. A detailed design plan was not available to PSI at the time the report was prepared.

The following table lists the material and information provided for this project:

DESCRIPTION OF MATERIAL	DATE	PROVIDED BY
Bridge Inventory (MR449)	05/08/2018	Morgan County Engineer
Preliminary Structure Information	05/08/2018	Morgan County Engineer
MR449 Location Map	11/21/2017	Morgan County Engineer
Bridge Photos	11/21/2017	Morgan County Engineer

The following table lists the structural loads and site features that are provided or estimated for the design basis for the conclusions of this report:

STRUCTURAL LOAD / PROPERTY	REQUIREMENT / REPORT BASIS		
Proposed Bridge		R*	B*
Type of the Proposed Bridge	(TBD)	☒	☐
Bridge Span	Single Span	☒	☐
Width of the Bridge	(TBD)	☒	☐
Abutment Type	(TBD)	☒	☐
Loading	(TBD)	☒	☐
Coordinates: Latitude, Longitude	39.512670, -81.864878	☒	☐
Grading Change	Minimum	☐	☒

*"R" = Requirement indicates specific design information was supplied.
"B" = Report Basis indicates specific design information was not supplied; therefore, this report is based on this parameter.

The geotechnical recommendations presented in this report are based on the available project information for the proposed CR 52 Bridge #449 located in Marion Township, Morgan County, Ohio and the subsurface materials described in this report. If any of the information noted above is incorrect, please inform PSI in writing so that we may amend the recommendations presented in this report if appropriate and if desired by the client. PSI will not be responsible for the implementation of its recommendations when it is not notified of changes in the project.

2.2 PURPOSE AND SCOPE OF SERVICES

The purpose of this geotechnical study was to explore the subsurface conditions at the site in order to develop foundation design recommendations for proposed bridge abutment foundations. PSI's contracted scope of services included drilling two (2) soil test borings, select laboratory testing, and bridge foundations estimation using the information obtained from PSI's field exploration.

The scope of services in the geotechnical exploration report did not include an environmental assessment for determining the presence/absence of wetlands, hazardous/toxic materials in the soil, bedrock, surface water, groundwater, or air on, below, or around this site. Any statements in this report or on the boring logs regarding odors, colors, and unusual or suspicious items or conditions are strictly for informational purposes.

3 SITE RECONNAISANCE AND PLANNING

3.1 SITE LOCATION AND DESCRIPTION

PSI conducted reconnaissance of the site which included discussions with Morgan County Engineer's Office and information from our site visit. The bridge site is located along County Road 52 (CR-52) in Marion Township, Morgan County, Ohio. The project bridge is located along a 2-lane roadway. The existing bridge spans the Goshen Creek. It is estimated that the water level is approximately 11 to 12 feet below the bridge deck, with a water depth of approximately 2 feet during the summer months.

3.2 SITE GEOLOGY

PSI estimated top of rock depths using available information published by the Ohio Department of Natural Resources (ODNR). Based on the geologic map published by the Ohio Geological Survey, the site lies in the Marietta Plateau. Soils consist of Pennsylvanian-age Upper Conemaugh Group through Permian-age Dunkard Group cyclic sequences of red and gray shale and siltstone, sandstone, limestone and coal; Pleistocene (Teays)-age Minford Clay; red and brown silty-clay loam colluvium; and landslide deposits.

PSI's preliminary research indicated bedrock was found between depths of 10 to 34 feet in the vicinity (few thousand feet to a mile) of the site based on several water well logs published by Ohio Department of Natural Resources (ODNR).

Information obtained from the Ohio Department of Natural Resources (ODNR) website also indicated that no known abandoned mine was recorded in the vicinity of the site area. "Known and Probable Karst in Ohio" map published by ODNR indicates that no Karst (sink hole) is recorded in the vicinity of the project site.

Water well Log and drilling report data published by ODNR were researched prior to preparation of this report. According to PSI's preliminary research, top of bedrock in the vicinity of the site area was anticipated at a depth of approximately 12 feet below surface grades according to well log ODNR 939775. If bedrock were to be found in the vicinity in the site area, it most likely would have been shale, sandstone, or limestone according to nearby water well logs published by ODNR.

3.3 PLANNING

Morgan County Engineer's office selected the number of test borings and the test boring locations were marked in the field by PSI's engineering staff prior to field exploration. Two (2) planned test borings, B-001 (behind rear bridge abutment) and B-002 (behind forward abutment), were advanced along the CR-52. Both B-001 and B-002 were drilled in the shoulder approximately 10 feet and 14.5 feet, respectively, south of the centerline of the roadway. B-001 was drilled approximately 16 feet west of the of the western side of the bridge, while B-002 was drilled approximately 14 feet east of the eastern side of the bridge. The bridge structure borings were to be terminated at depths where 30 feet of hard or dense soils were penetrated or after rock coring. The drilled test boring locations are shown on the attached figure - "Boring Location Map" in the appendix of this report.
PSI personnel contacted Ohio Utility Protection Services (OUPS) and the project owner prior to commencing test-boring operations. PSI notified Morgan County Engineer's Office prior to field exploration. During field

exploration, the Morgan County Engineer's Office provided traffic control to temporarily close one traffic lane at a time during drilling operations.

3.4 EXPLORATORY TEST BORING

The field explorations were performed in accordance with applicable ODOT and ASTM Specifications. A CME-45C track-mounted drilling rig was mobilized to advance the test borings between March 15th and 16th, 2018. Representative disturbed samples of soil were collected at center-to-center intervals of 1.5, 2.5 and 5.0 feet. The Standard Penetration Test (ASTM D-1586) was performed at each sampling interval.

The test borings were terminated after 45 and 34 feet of drilling depth was reached, respectively. A 30 feet layer of high density materials (defined by SPT blow counts greater than 30 blow/ft) was encountered in test boring B-001, while rock coring was performed in B-002. One (1) rock core run was performed. Rock coring was performed using NQ2 size diamond core barrel in accordance with ASTM D-2113 standard. A 10 foot rock core run was performed in test boring B-002. Water was used as a cooling medium and to extract cuttings from the borehole during rock coring operations. The rock coring run was taken between depths of 24 and 34 feet in boring B-002.

PSI's drilling crew monitored the water levels in the borehole for the presence of groundwater during drilling operations. Long term groundwater monitoring was not planned for this project. The typed drilling log, included in the appendix of this report, show the SPT resistance (N_{60}) values for each soil sample obtained in the test boring, and present the classification and description of soil and rock encountered at various depths in the test boring. PSI includes the test boring logs for development of bridge foundation recommendations in this report. Since no other test borings were advanced for the structure this year prior to drilling the two test borings, B-001 and B-002 will be used to describe the exploration findings in the following report sections.

3.5 SAMPLE ANALYSIS

Limited laboratory analyses were performed for the test borings. The test results are included in the test boring logs. All recent laboratory tests were performed in general accordance with ASTM, AASHTO or other standards listed in the appendix. In the recently drilled borings, the soils were classified in accordance with the ODOT Soil Classification System (OSCS). A description of the classification system and the results of the laboratory tests are included in the appendix.

4 SITE AND SUBSURFACE CONDITIONS

4.1 SUBSURFACE CONDITIONS

Boring B-001 encountered cohesive soil until approximately 9.5 feet before transitioning to non-cohesive soils that continued reaching shale bedrock at a depth of approximately 13.5 feet below surface grade. The cohesive soils consisted of A-6a classifications and the non-cohesive soils consisted of A-2-4. Based on the boring information, soils between approximately 3.5 to 9.0 feet in boring B-001 consisted of generally medium stiff soil. Below these depths, "medium dense" granular soils and "very weak" to "slightly strong" bedrock were found.

Test boring B-002 encountered cohesive soil until approximately 10.5 before transitioning to a thin layer of silt above shale bedrock. The shale bedrock was encountered to the termination depth in boring B-002. The cohesive soils consisted of A-7-6, A-6a, and A-6b classifications.

Standard Penetration Testing (SPT) was performed at each sampling interval. SPT "N" values varied at different sampling depths, but high "N_{60}" values were encountered below approximately 13.5 feet in borings B-001 and B-003. Auger refusal in rock was not encountered in either test borings. Both test borings were terminated after drilling to depths of 45 and 34 feet, respectively.

4.2 WATER LEVEL MEASUREMENTS

Free water was encountered during drilling activities at depths of approximately 10.3 and 10.7 feet below surface grades in borings B-001 and B-002, respectively. Free water was observed upon completion of drilling at depths of 10.3 and 6.0 feet, respectively, in borings B-001 and B-002. After removing the augers from the boreholes, the measured water depths were 9.5 feet in B-001 and 8.2 feet in B-002. PSI estimated the water and creek channel depths below the existing bridge deck (near the middle of the bridge span). The depths of the water surface in the creek and bottom of the creek channel were estimated to be 11 and 12 feet, respectively, below the top of the bridge deck (top pf the asphalt road surface). PSI estimates that the bridge deck elevation is approximately 733 feet.

The groundwater level at the site, as well as perched water levels and volumes, will fluctuate based on variations in rainfall, snowmelt, evaporation, surface run-off and other related hydrogeologic factors. The water level measurements presented in this report are the levels that were measured at the time of PSI's field activities.

4.3 D_{50} SIZES FOR SCOUR ANALYSIS

Soil samples for D_{50} determination were obtained in B-001 at sample depths of 7.5 – 9.0, 9.0 -- 10.5, 10.5 – 12.0, and 12.0 – 13.5 feet. Four samples from B-001 were used for scour analysis. According to the plan, the scour sample should be taken at the pier location. However, since the planned bridge will be single span, a pier is not planned for this project. PSI utilized samples from B-001 to provide a representative sample for scour analysis. The end of scour sampling occurred in weathered shale. Therefore, past local experience should be considered for the scour depth in rock at this site. The results for the grain size D_{50} and D_{95} are shown in the table on the following page. The test results are also included in the appendix of this report. The project design engineer should choose soil parameters based on requirements of scour analysis.

Particle Size D_{50} and D_{95} for Scour Depth Analysis

Boring Location	Sample Depths (ft)	Top of Sample Elevation (ft)	D_{50} (mm)	D_{95} (mm)	ODOT Classification
B-001	9.0 – 10.5	724.0	1.1599	15.8766	A-2-4
B-001	10.5 – 12.0	722.5	1.1735	15.2417	A-2-4
B-001	12.0 – 13.5	721.0	0.0068	0.3392	A-2-4
B-001	13.5 -- 15.0	719.5	0.0068	0.3392	Rock

The above subsurface description is of a generalized nature to highlight the major subsurface stratification features and material characteristics. The boring logs included in the appendix should be reviewed for specific information at individual boring locations. These records include soil/rock descriptions, stratifications, penetration resistances, and locations of the samples and laboratory test data. The stratifications shown on the boring logs represent the conditions only at the actual boring locations. Variations may occur and should be expected between boring locations. The stratifications represent the approximate boundary between subsurface materials and the actual transition may be gradual. Water level information obtained during field operations is also shown on these boring logs. The samples that were not altered by laboratory testing will be retained for 60 days from the date of this report and then will be discarded.

5 GEOTECHNICAL RECOMMENDATIONS

PSI estimates the bottom of the proposed bridge abutment will be at approximately 10 feet below the roadway grade (elevation of 723 feet). Following are PSI's foundation recommendations for this project.

5.1 DESIGN CONSIDERATIONS FOR PROPOSED BRIDGE FOUNDATIONS

Spread Footings. When a bridge abutment is located in the waterway and are not constructed directly on bedrock, they may have potential for scour damage. Normally, conventional spread foundations are not recommended to be constructed on soils (to support proposed bridge super-structures). On this site, bedrock is located at relatively shallow depths. Therefore, conventional spread footings are considered a suitable foundation system for the future bridge. It is also economical to construct conventional spread footings.

Drilled Shaft Foundations. Drilled shafts are considered an alternative economical foundation system for this project because of the relatively shallow depth to bedrock. PSI recommends the proposed drilled shafts' bases be set at minimum depth of 18.5 feet below the existing grade into the weathered shale. Due to the relatively shallow depth to rock, PSI recommends embedding the drilled shaft foundations into the rock and design base on rock parameters only. The drilled shafts should have a minimum diameter of 36 inches.

Reuse of The Existing Gravity Bridge Abutment Foundations. If practical, reuse of the old foundation can be more economical than complete foundation reconstruction. The existing gravity reinforced concrete bridge abutments may be reused if the proposed bridge has the same span, width and similar or lighter loading. A detailed sub-structure inspection must be performed, and the substructures must be acceptable for reuse after some repair or modifications. Sometimes, the project owner may keep the existing sub-structures partially in place as shoring structures for new bridge construction. It is the project owner's decision for reuse of the existing substructures.

5.2 SPREAD FOOTINGS

A conventional spread footing should be considered a suitable foundation system for both the rear and forward bridge abutments. Conventional spread footings may be constructed directly on the slightly weathered shale or un-weathered siltstone. The following table is prepared to estimate bedrock bearing resistance.

Geotechnical Resistance and Factors for Spread Foundations (Service Limit)

Abutment Location	***Bearing Materials***	***Estimated Bearing Elevations (ft)***	***Nominal Bearing Resistance, Rn (ksf)***	***Sliding Resistance Parameter, Friction Angle (ψ)***	***Resistance Factors, φ***
Rear	Weathered Shale	719.5	20	-	0.45
	Weathered Shale	719.5	-	20	0.55
Forward	Weathered Shale	720.5	20	-	0.45
	Weathered Shale	720.5	-	20	0.55

A resistance factor of 0.65 (according to Reference Manual LRFD for Highway Bridge Structures, FHWA Contract No. DTFH61-94-00098) may be used to assess foundation settlement, horizontal movement, overall stability under anticipated poor conditions to ensure the design satisfies the structure's Service Limit State.

It is anticipated that the shale bedrock near the surface may be more weathered than the rock further in depth. If the bridge foundation is constructed in shale, it is recommended concrete should be filled all sides of the excavation areas to prevent further shale weathering and decomposing. Some rock excavation may be necessary so that the foundations bear on competent rock. Conditions of the bearing surface should be determined in the field at the time of construction. According to PSI's estimation according to the condition of the rock cores, full height reinforced concrete abutments can be founded on bedrock at elevations of 793 feet or deeper.

Based on the known subsurface conditions and site geology, laboratory testing and past experience, PSI anticipates that properly designed foundations, should experience negligible total settlements.

If footings are constructed below the ground water or the creek water levels, PSI recommends that dewatering the site prior to and during foundation construction. Cofferdams or other shoring installations should be utilized to protect the foundation construction since the bottom of the river channel is in rock and the river water can rise quickly after each storm. Dewatering and cofferdam construction should conform to ODOT Item 503.03 "Cofferdams and Excavation Bracing". It is the project contractor's responsibility to determine the method of construction of dewatering system and cofferdams.

5.3 DRILLED SHAFT FOUNDATIONS

Drilled shafts can also be utilized for the foundations of the proposed bridge where spread footings are not feasible or groundwater or surface water infiltration become an issue. The vertical and lateral load resistance of the drilled shafts can be calculated with the parameters given in the following tables. These parameters are for analytical programs such as AllPile or COM624P which will analyze both the applied load verses strain resistance of the soil and the deflection of the structural element. PSI recommends the proposed drilled shaft base be set at minimum depth of 18.5 feet below the existing grade into the weathered shale. Due to the relatively shallow depth to rock, PSI recommends embedding the drilled shaft foundations into the rock and design based on rock parameters only.

Recommended Soil Parameters for Drilled Shaft Pile Design											
Soil Layer	Soil Description	Depth Range Below Grade (ft)	Average SPT N Value (blows/ft)	Total Unit Weight, Y' (pcf)	Soil Cohesion, C (psf)	Soil Internal Angle of Friction, φ (Deg)	Static Lateral Modulus, K (pci)	Strain Factor, E_{50} (%)	Downward Average Unit Side Resistance (ksf)	Uplift Average Unit Side Resistance (ksf)	Average Unit End Bearing Resistance (ksf)
1	Clay (A-7-6), Silt and Clay (A-6a), Gravel with Sand and Silt (A-2-4)	0 to 13.5	14	Ignore	Ignore	Ignore	Ignore	-	Ignore	Ignore	Ignore
2	Weathered Shale	13.5 to 45	125	-	-	-	-	-	Ignore	Ignore	15

Note: The Table was prepared using simplified soil profiles and a water level of 10 feet. Some adjustments may be necessary by the design engineers.

Exclusion zones are the upper 13.5 feet. Average Unit End Bearing Resistance with F. S. = 3.0.

Settlement can be estimated according to FHWA-HI-88-042 Manual – Drilled Shafts", for short term settlement. Once the drilled shaft design is completed the settlement can be estimated based on diameter of base and loading ratio (Actual load/Ultimate load) of the drilled shafts.

The caissons should have a minimum diameter of 36 inches. The construction of the caissons should be specified in keeping with ACI 336.1-01, "Specification for the Construction of Drilled Piers". In addition, the caissons should be constructed following the guidelines in ACI 336.3 Chapter 4. A qualified representative of PSI should assess that the caissons are founded on competent bearing materials and that the caisson installation procedures comply with the specifications.

Prior to the placement of the reinforcement cage or concrete, the bottom of the caisson excavation should be thoroughly cleaned and free of all loose or soft materials. This is critical for end-bearing shafts. For end bearing shafts a final bottom check should be performed just before concrete placement to check for small cave-ins or accumulated sediment. Reinforcing should be placed after the Geotechnical Engineer has approved the shaft for concrete placement. The reinforcement should be centered using spacers as needed and should not contact the side walls of the shaft. PSI recommends that the drilled shaft have a minimum diameter of 48 inches to facilitate cleaning and inspection before placement of the reinforcing cage.

Concrete should be placed inside the casing as soon as possible after the completion of drilling operations. We recommend placing the caisson concrete on the same day that the caisson is drilled. During the placement of concrete we recommend that the slump be between 5 to 7 inches; this will allow for proper distribution of the concrete and limit the potential for segregation and air pockets.

A temporary casing, which is removed when concrete is placed, should be utilized to prevent collapse of the overburden and infiltration of the groundwater. When temporary casing is utilized, extreme care should be given as to limit the amount of disturbance along the sides of the drilled shaft. The contractor must fill any voids or enlargements in the shaft excavations with concrete at the time of concrete placement. When removing the casing, a minimum head of 5 feet of concrete should be maintained above the bottom of the casing at all times, and in some instances, a much higher head is required. The volume of concrete placed in the excavation should be checked to confirm that no substantial sloughing of the excavation occurred during concrete placement or removal of the casing. If any discrepancies are noted, the geotechnical engineer should be notified immediately.

5.4 BRIDGE ABUTMENT AND WING WALLS

The bridge abutments and wing walls will be expected to retain backfill materials to some height. In addition to the lateral earth pressures, the abutments will be expected to resist the reaction of the superstructure of the bridge and the increase in earth pressures due to wheel loads on the backfill adjacent to the abutments. Consideration should be given to the following factors in connection with the design of the abutments and wing walls:

The abutment walls should be designed for at-rest loading conditions assuming a triangular load distribution and an equivalent fluid pressure of 60 pounds per square foot (psf), per foot of abutment depth. In the event provisions are made in the design of the superstructure of the bridge to permit sufficient lateral movement of the abutment to develop active earth pressure conditions, then an equivalent fluid pressure of 40 psf per foot of abutment depth may be employed. The wing walls are to be designed for active earth pressure conditions.

The influence of the wheel loads should be considered in the design of the abutments by representing them as an additional 24-inch layer of backfill. For the at-rest condition, an equivalent fluid pressure of 120 psf per foot of abutment depth should be utilized. For the active condition, the equivalent pressure can be reduced to 80 psf per foot of wall depth. Since this pressure is assumed to be uniformly distributed, the resultant force should be assumed at mid-height of the abutment wall.

The abutments and wing walls should include an adequate drainage system in the form of weep holes and/or perforated drainpipes to preclude the possibility of any water buildup against the back face of these members. A well-graded granular material is to be employed as backfill around tile members or weep holes to avoid any clogging and to ensure positive drainage. A non-woven geotextile wrap for the pipe or a portion of the granular fill can also be utilized. It is further recommended that free draining materials or proprietary wall drain panels be placed against the entire face of the walls along their full length. The drainage blanket should have a minimum thickness of 18 inches and is to terminate approximately 24 inches below the finish subgrade elevation for the approach slabs or surface grades. A cohesive fill cap is recommended for the top 24 inches in order to prevent direct surface water infiltration.

During high water periods, the backfill behind the abutments and wing walls may become saturated by water. This will result in additional lateral pressures on the retaining structures during the period of receding water and until the drainage of the granular backfill is accomplished. Therefore, in addition to the previously discussed lateral earth pressures, the unbalanced water pressure should also be considered in the retaining wall design.

The appropriate safety factors should be considered in the stability analysis assuming that the earth pressures, water pressures, and highway surcharge loads could occur coincidently.

Once the abutments and wing walls are in place, over-compaction of the materials against these structures should be avoided under all circumstances, so as to prevent undue lateral earth pressures. Further, it is recommended that backfilling of cut excavations at the bridge abutments be undertaken only subsequent to installation of structural members of the new superstructure.

5.5 SLOPES

The benched placement of engineered structural fill on natural slopes steeper than five (5) horizontal to one (1) vertical where the final area will be uncontained is recommended. The placement of fill should begin at the base of the natural slope with benches or terraces. The benches or terraces should be a minimum of eight (8) feet wide laterally, and should be cut into the slope every five (5) feet of vertical rise. The naturally occurring existing soils should be prepared and fill placed in accordance with the previously described structural fill guidelines. A representative of the geotechnical engineer should monitor the benching and fill placement operations.

Unless specifically designed, temporary slopes shall not exceed steeper than a ratio of two (2) horizontal to one (1) vertical where workers or equipment will occupy space at the toe or movement of the excavated slope will jeopardize the stability of an adjacent structure. Temporary slopes exceeding ten (10) feet in vertical height should have a slope stability analysis. Temporary slopes exceeding twenty (20) feet in vertical height should have shear strength testing performed to assess the in-situ strength characteristics.

Permanent cut slopes shall not be excavated to a final grade steeper than a ratio of three (3) horizontal to one (1) vertical without a specific slope stability analysis. Specific shear strength testing should be performed to assess the in-situ strength characteristics for permanent slopes steeper than four (4) horizontal to one (1) vertical.

Special consideration must also be given to the stability of the natural cut ground when supporting substantial fills, to structural fills themselves, and to cut surfaces in natural soil and rock excavations. The evaluation of slope stability aspects of this site and the proposed development is beyond the scope of this exploration. Relatively detailed grading plans will have to be developed before meaningful evaluation of slope stability can be accomplished. All slope stability evaluations should be performed by qualified geotechnical engineering personnel prior to the initiation of any significant grading activities at this site.

5.6 BRIDGE ABUTMENT AND WING WALLS

The bridge abutments and wing walls will be expected to retain backfill materials to some height. In addition to the lateral earth pressures, the abutments will be expected to resist the reaction of the superstructure of the bridge and the increase in earth pressures due to wheel loads on the backfill adjacent to the abutments. Consideration should be given to the following factors in connection with the design of the abutments and wing walls:

The following table, Typical Values for Equivalent Fluid Unit Weight of Soil, (listed in AASHTO LRFD Bridge Design Specifications, 2016) can be used in the lateral earth pressure design. These soil parameters can be used for leveled fill or backfill with typical (25 degree) slope.

Table 3.11.5.5-1—Typical Values for Equivalent Fluid Unit Weights of Soils

Type of Soil	Level Backfill		Backfill with β = 25 degrees	
	At-Rest γ_{eq} (kcf)	Active Δ/H = 1/240 γ_{eq} (kcf)	At-Rest γ_{eq} (kcf)	Active Δ/H = 1/240 γ_{eq} (kcf)
Loose sand or gravel	0.055	0.040	0.065	0.050
Medium dense sand or gravel	0.050	0.035	0.060	0.045
Dense sand or gravel	0.045	0.030	0.055	0.040

The following typical values for Equivalent Height of Soil for Vehicular Loading on Abutments Perpendicular to Traffic (listed in AASHTO LRFD Bridge Design Specifications, 2016) can be used in the lateral earth pressure design.

Table 3.11.6.4-1—Equivalent Height of Soil for Vehicular Loading on Abutments Perpendicular to Traffic

Abutment Height (ft)	h_{eq} (ft)
5.0	4.0
10.0	3.0
≥20.0	2.0

The abutments and wing walls should include an adequate drainage system in the form of weep holes and/or perforated drainpipes to preclude the possibility of any water buildup against the back face of these members. A well-graded granular material is to be employed as backfill around tile members or weep holes to avoid any clogging and to ensure positive drainage. A non-woven geotextile wrap for the pipe or a portion of the granular fill can also be utilized. It is further recommended that free draining materials or proprietary wall drain panels be placed against the entire face of the walls along their full length. The drainage blanket should have a minimum thickness of 18 inches and is to terminate approximately 24 inches below the finish subgrade elevation for the approach slabs or surface grades. A cohesive fill cap is recommended for the top 24 inches in order to prevent direct surface water infiltration.

During high water periods, the backfill behind the abutments and wing walls may become saturated by water. This will result in additional lateral pressures on the retaining structures during the period of receding water and until the drainage of the granular backfill is accomplished. Therefore, in addition to the previously discussed lateral earth pressures, the unbalanced water pressure should also be considered in the retaining wall design.

The appropriate safety factors should be considered in the stability analysis assuming that the earth pressures, water pressures, and highway surcharge loads could occur coincidently.

Once the abutments and wing walls are in place, over-compaction of the materials against these structures should be avoided under all circumstances, so as to prevent undue lateral earth pressures. Further, it is recommended that backfilling of cut excavations at the bridge abutments be undertaken only subsequent to installation of structural members of the new superstructure.

5.7 SLOPES

The benched placement of engineered structural fill on natural slopes steeper than five (5) horizontal to one (1) vertical where the final area will be uncontained is recommended. The placement of fill should begin at the base of the natural slope with benches or terraces. The benches or terraces should be a minimum of eight (8) feet wide laterally, and should be cut into the slope every five (5) feet of vertical rise. The naturally occurring existing soils should be prepared and fill placed in accordance with the previously described structural fill guidelines. A representative of the geotechnical engineer should monitor the benching and fill placement operations.

Unless specifically designed, temporary slopes shall not exceed steeper than a ratio of two (2) horizontal to one (1) vertical where workers or equipment will occupy space at the toe or of the movement of the excavated slope will jeopardize the stability of an adjacent structure. Temporary slopes exceeding ten (10) feet in vertical height should have a slope stability analysis. Temporary slopes exceeding twenty (20) feet in vertical height should have shear strength testing performed to assess the in-situ strength characteristics.

Permanent cut slopes shall not be excavated to a final grade steeper than a ratio of three (3) horizontal to one (1) vertical without a specific slope stability analysis. Specific shear strength testing should be performed to assess the in-situ strength characteristics for permanent slopes steeper than four (4) horizontal to one (1) vertical.

Special consideration must also be given to the stability of the natural cut ground when supporting substantial fills, to structural fills themselves, and to cut surfaces in natural soil and rock excavations. The evaluation of slope stability aspects of this site and the proposed development is beyond the scope of this exploration. Relatively detailed grading plans will have to be developed before meaningful evaluation of slope stability can be accomplished. All slope stability evaluations should be performed by qualified geotechnical engineering personnel prior to the initiation of any significant grading activities at this site.

6 GEOTECHNICAL RISK

The concept of risk is an important aspect of the geotechnical evaluation. The primary reason for this is that the analytical methods used to develop geotechnical recommendations do not comprise an exact science. The analytical tools which geotechnical engineers use are generally empirical and must be used in conjunction with engineering judgment and experience. Therefore, the solutions and recommendations presented in the geotechnical evaluation should not be considered risk-free and, more importantly, are not a guarantee that the interaction between the soils and the proposed structure will perform as planned. Based on the information generated and referenced during this evaluation, and PSI's experience in working with these conditions, the engineering recommendations presented in the preceding section constitutes PSI's professional estimate of those measures that are necessary for the proposed structure to perform according to the proposed design.

7 REPORT LIMITATIONS

The recommendations submitted are based on the available subsurface information obtained by PSI and design details furnished by Morgan County Engineer's Office. If there are revisions to the plans for this project or if deviations from the subsurface conditions noted in this report are encountered during construction, PSI should be notified immediately to determine if changes in the foundation recommendations are required. If PSI is not retained to perform these functions, PSI will not be responsible for the impact of those conditions on the project.

The geotechnical engineer warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed.

After the plans and specifications are more complete, the geotechnical engineer should be retained and provided the opportunity to review the final design plans and specifications to check that our engineering recommendations have been properly incorporated into the design documents. At that time, it may be necessary to submit supplementary recommendations. This report has been prepared for the exclusive use of the project owner (Morgan County Engineer) for the specific application to the proposed CR 52 bridge #449 replacement project in Marion Township, Morgan County, Ohio.

Appendix

Note: Base map provided by client; altered for PSI use.

intertek psi Professional Services Industries, Inc. 4960 Vulcan Ave. Suite C Columbus, OH 43228 Telephone: (614) 876-8000	**Site Vicinity Plan**	**MCE—MRG CR-52 Bridge #449 Replacement** Marion Township, Morgan County, Ohio PSI Project No.: 01021274

Note: Base map provided by client; altered for PSI use.

intertek psi
Professional Services Industries, Inc.
4960 Vulcan Ave. Suite C
Columbus, OH 43228
Telephone: (614) 876-8000

Boring Location Plan

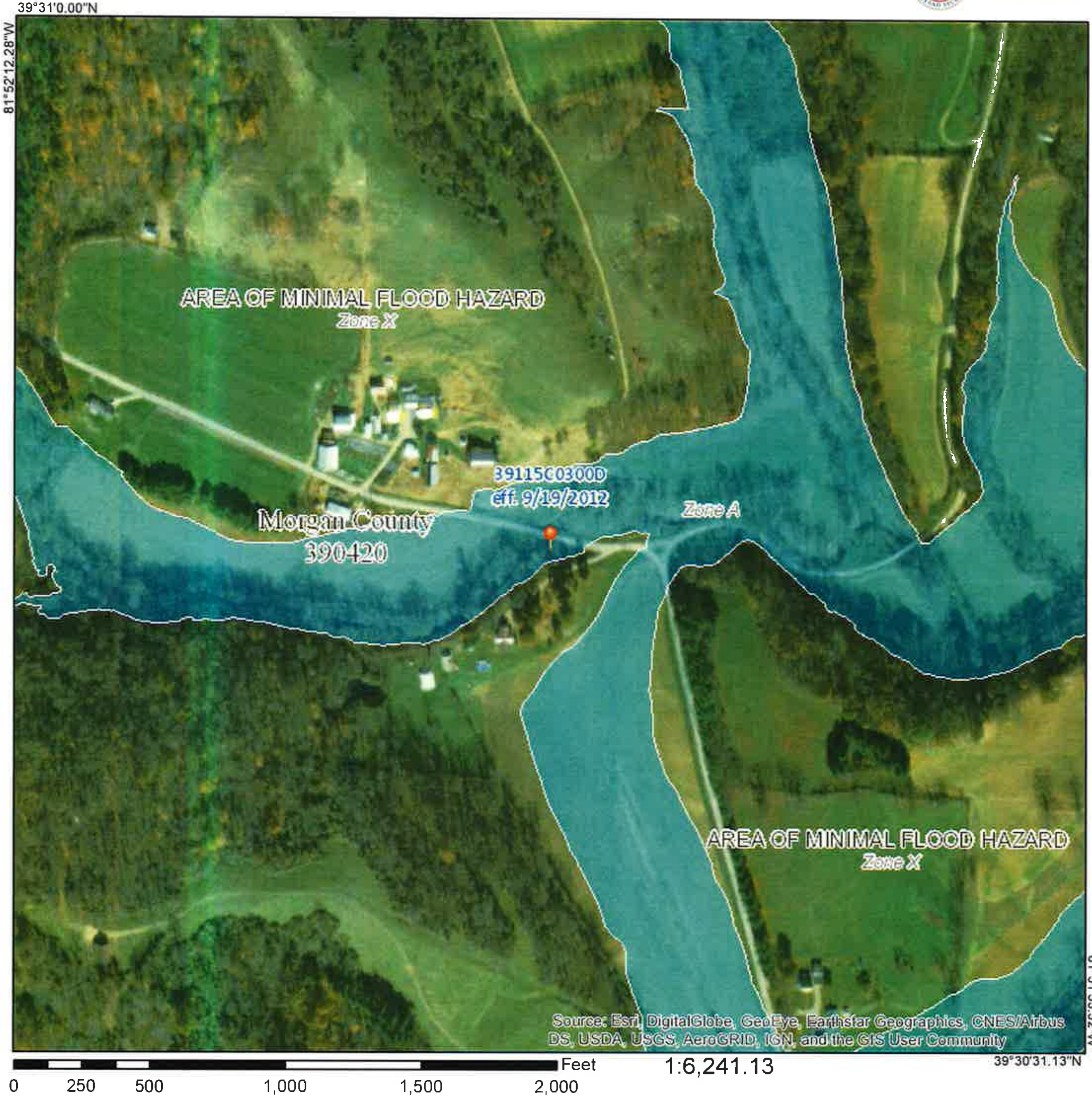
MCE—MRG CR-52 Bridge #449 Replacement

Marion Township, Morgan County, Ohio

PSI Project No.: 01021274

National Flood Hazard Layer FIRMette

FEMA



1:6,241.13

Legend

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR FIRM PANEL LAYOUT

SPECIAL FLOOD HAZARD AREAS		Without Base Flood Elevation (BFE) *Zone A, V, A99*
		With BFE or Depth
		Regulatory Floodway *Zone AE, AO, AH, VE, AR*
OTHER AREAS OF FLOOD HAZARD		0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
		Future Conditions 1% Annual Chance Flood Hazard *Zone X*
		Area with Reduced Flood Risk due to Levee. See Notes. *Zone X*
		Area with Flood Risk due to Levee *Zone D*
OTHER AREAS	NO SCREEN	Area of Minimal Flood Hazard *Zone X*
		Effective LOMRs
		Area of Undetermined Flood Hazard *Zone D*
GENERAL STRUCTURES		Channel, Culvert, or Storm Sewer
		Levee, Dike, or Floodwall
OTHER FEATURES	20.2 / 17.5	Cross Sections with 1% Annual Chance Water Surface Elevation
		Coastal Transect
	513	Base Flood Elevation Line (BFE)
		Limit of Study
		Jurisdiction Boundary
		Coastal Transect Baseline
		Profile Baseline
		Hydrographic Feature
MAP PANELS		Digital Data Available
		No Digital Data Available
		Unmapped

This map complies with FEMA's standards for the use of digital flood maps if it is not void as described below. The base map shown complies with FEMA's base map accuracy standards

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 5/9/2018 at 9:02:36 AM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

This map image is void if the one or more of the following map elements do not appear: base map imagery, flood zone labels, legend, scale bar, map creation date, community identifiers, FIRM panel number, and FIRM effective date. Map images for unmapped and unmodernized areas cannot be used for regulatory purposes.

Mines of Ohio

May 9, 2018

Current

Air Shaft

Drift Entry

Vertical Mine Shaft

Slope Entry

Past

Air Shaft

Drift Entry

Vertical Mine Shaft

Slope Entry

Locations

N

0 0.0175 0.035 0.07 mi

0 0.03 0.06 0.12 km

1:2,257

Ohio Dept. of Natural Resources

D-50 Values

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT MRG CR 52 BRIDGE #449 REPLACEMENT **PID**

OGE NUMBER 01021274 **PROJECT TYPE** BRIDGE REPLACEMENT

BORING NUMBER	SAMPLE NUMBER	ELEVATION	D-50 VALUE (mm)	D-95 VALUE (mm)
B-001	SS	725.5	-	-
	SS	724.0	0.4748	15.9578
	SS	722.5	1.1599	15.8766
	SS	721.0	1.1735	15.2417
	SS	719.5	0.0068	0.3392

PROJECT: MRG CR 52 BRIDGE #449	DRILLING FIRM / OPERATOR: PSI / J.E.	DRILL RIG: CME 45 C ATV 2007	STATION / OFFSET:
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: PSI / K.P.	HAMMER: CME AUTOMATIC	ALIGNMENT:
PID: SFN:	DRILLING METHOD: 3.25" HSA / NQ2	CALIBRATION DATE: 10/2/12	ELEVATION: 733.0 (MSL) EOB: 45.0 ft.
START: 3/15/18 END: 3/15/18	SAMPLING METHOD: SPT / NQ2	ENERGY RATIO (%): 92	LAT / LONG: 39.512670, -81.864878

EXPLORATION ID B-001

MATERIAL DESCRIPTION AND NOTES	ELEV. 733.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	ABAN-DONED
TOPSOIL (6")	732.5																	
HARD, BROWN, **SILT AND CLAY**, WITH SAND, SOME SANDSTONE FRAGMENTS, MOIST		2–3	5 11 17	43	78		2.00	4	4	32	40	20	30	17	13	18	A-6a (6)	
	729.3																	
MEDIUM STIFF, BROWN, **SILT AND CLAY**, WITH SAND, MOIST		4–5	5 2 2	6	22		-	-	-	-	-	-	-	-	-	9	A-6a (V)	
		8–9	1 1 2	5	89		0.75	-	-	-	-	-	-	-	-	27	A-6a (V)	
	723.5																	
MEDIUM DENSE, BROWN, **GRAVEL WITH SAND AND SILT**, TRACE CLAY, WET		9–10 (0 hr)	1 3 5	12	89		-	38	13	20	20	9	-	-	-	14	A-2-4 (V)	
		11–12	3 2 3	8	83		-	44	17	16	15	8	-	-	-	17	A-2-4 (V)	
		12–13	2 3 5	12	39		-	45	15	16	16	8	-	-	-	21	A-2-4 (V)	
	719.5																	
SHALE, BROWN AND RED, HIGHLY WEATHERED, VERY WEAK.		14–15	2 9 28	57	100		-	1	3	8	47	41	-	-	-	11	Rock (V)	
		16–17	15 25 50/3"	-	100		-	-	-	-	-	-	-	-	-	10	Rock (V)	
	714.7																	
SHALE, RED AND GRAY, HIGHLY WEATHERED, SLIGHTLY STRONG.		18–19	50/5"	-	100		-	-	-	-	-	-	-	-	-	14	Rock (V)	

PID:	SFN:	PROJECT: MRG CR 52 BRIDGE #449	STATION / OFFSET:	START: 3/15/18	END: 3/15/18	PG 2 OF 3	B-001

MATERIAL DESCRIPTION AND NOTES	ELEV. 713.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	ABAN-DONED
SHALE, RED AND GRAY, HIGHLY WEATHERED, SLIGHTLY STRONG. *(continued)*		21–22	15 26 50	117	100		-	-	-	-	-	-	-	-	-	14	Rock (V)	
		24	53	-	100		-	-	-	-	-	-	-	-	-	10	Rock (V)	
	707.5	25																
SHALE, GRAY, HIGHLY WEATHERED, SLIGHTLY STRONG.		26	50/5"	-	100		-	-	-	-	-	-	-	-	-	7	Rock (V)	
	704.5	27–28																
SHALE, GRAY, BROWN, AND PURPLE, HIGHLY WEATHERED, WEAK.		29	51	-	100		-	-	-	-	-	-	-	-	-	12	Rock (V)	
		34	42 52	-	100		-	-	-	-	-	-	-	-	-	20	Rock (V)	
		39–40	25 37 54	140	100		-	-	-	-	-	-	-	-	-	16	Rock (V)	
		41																

PID:	SFN:	PROJECT: MRG CR 52 BRIDGE #449	STATION / OFFSET:	START: 3/15/18	END: 3/15/18	PG 3 OF 3	B-001

MATERIAL DESCRIPTION AND NOTES	ELEV. 691.6	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	ABAN- DONED
SHALE, GRAY, BROWN, AND PURPLE, HIGHLY WEATHERED, WEAK. *(continued)*		42 43																
	688.0	44 EOB 45	18 26 29	84	100		-	-	-	-	-	-	-	-	-	13	Rock (V)	

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH 1 BAG CEMENT; BACKFILLED WITH 1 BAG HOLE PLUG

PROJECT: MRG CR 52 BRIDGE #449	DRILLING FIRM / OPERATOR: PSI / J.E.	DRILL RIG: CME 45 C ATV 2007	STATION / OFFSET:
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: PSI / K.P.	HAMMER: CME AUTOMATIC	ALIGNMENT:
PID: SFN:	DRILLING METHOD: 3.25" HSA / NQ2	CALIBRATION DATE: 10/2/12	ELEVATION: 734.0 (MSL) EOB: 34.0 ft.
START: 3/15/18 END: 3/15/18	SAMPLING METHOD: SPT / NQ2	ENERGY RATIO (%): 92	LAT / LONG: 39.512584, -81.864523

EXPLORATION ID B-002

MATERIAL DESCRIPTION AND NOTES	ELEV. 734.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	ABAN-DONED
TOPSOIL (8")	733.3																	
STIFF, BROWN, **CLAY**, LITTLE SAND, TRACE GRAVEL, MOIST	730.5	2–3	2 3 3	9	78		1.75	2	3	8	45	42	50	22	28	25	A-7-6 (17)	
MEDIUM STIFF, BROWN, **SILT AND CLAY**, LITTLE SAND, TRACE GRAVEL, MOIST	725.5	4–5; ▼ 0 hr (6)	3 3 2	8	39		0.75	1	2	13	51	33	37	22	15	23	A-6a (10)	
MEDIUM STIFF, BROWN, **SILTY CLAY**, SOME SAND, MOIST	723.3	9–10; W	1 1 2	5	89		0.25	0	2	24	43	31	39	21	18	30	A-6b (11)	
LOOSE, BROWN, **SANDY SILT**, WET	720.5	11–12	0 1 2	5	0		-	-	-	-	-	-	-	-	-	-	A-6a (V)	
SHALE, BROWN, HIGHLY WEATHERED, VERY WEAK.	718.5	14–15	12 25 42	103	89		-	-	-	-	-	-	-	-	-	12	Rock (V)	
SHALE, RED AND GRAY, HIGHLY WEATHERED, WEAK.		16–17	33 35 50	130	78		-	-	-	-	-	-	-	-	-	9	Rock (V)	
		18.5	50/3"	-	100		-	-	-	-	-	-	-	-	-	5	Rock (V)	

PID:	SFN:	PROJECT: MRG CR 52 BRIDGE #449	STATION / OFFSET:	START: 3/15/18	END: 3/15/18	PG 2 OF 2	B-002

MATERIAL DESCRIPTION AND NOTES	ELEV. 714.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	ABAN-DONED
SHALE, RED AND GRAY, HIGHLY WEATHERED, WEAK. *(continued)*		21–22	19 41 50/4"	-	100		-	-	-	-	-	-	-	-	-	8	Rock (V)	
	710.0	24	50/4"	-	100		-	-	-	-	-	-	-	-	-	4	Rock (V)	
SHALE, GRAY, BROWN, AND PURPLE, MODERATELY WEATHERED, SLIGHTLY TO MODERATELY STRONG, SLIGHTLY FRACTURED; RQD 78.3%, REC 99.2%.		24–34 (EOB 34)	78		99	NQ2-10											CORE	

NOTES: NONE

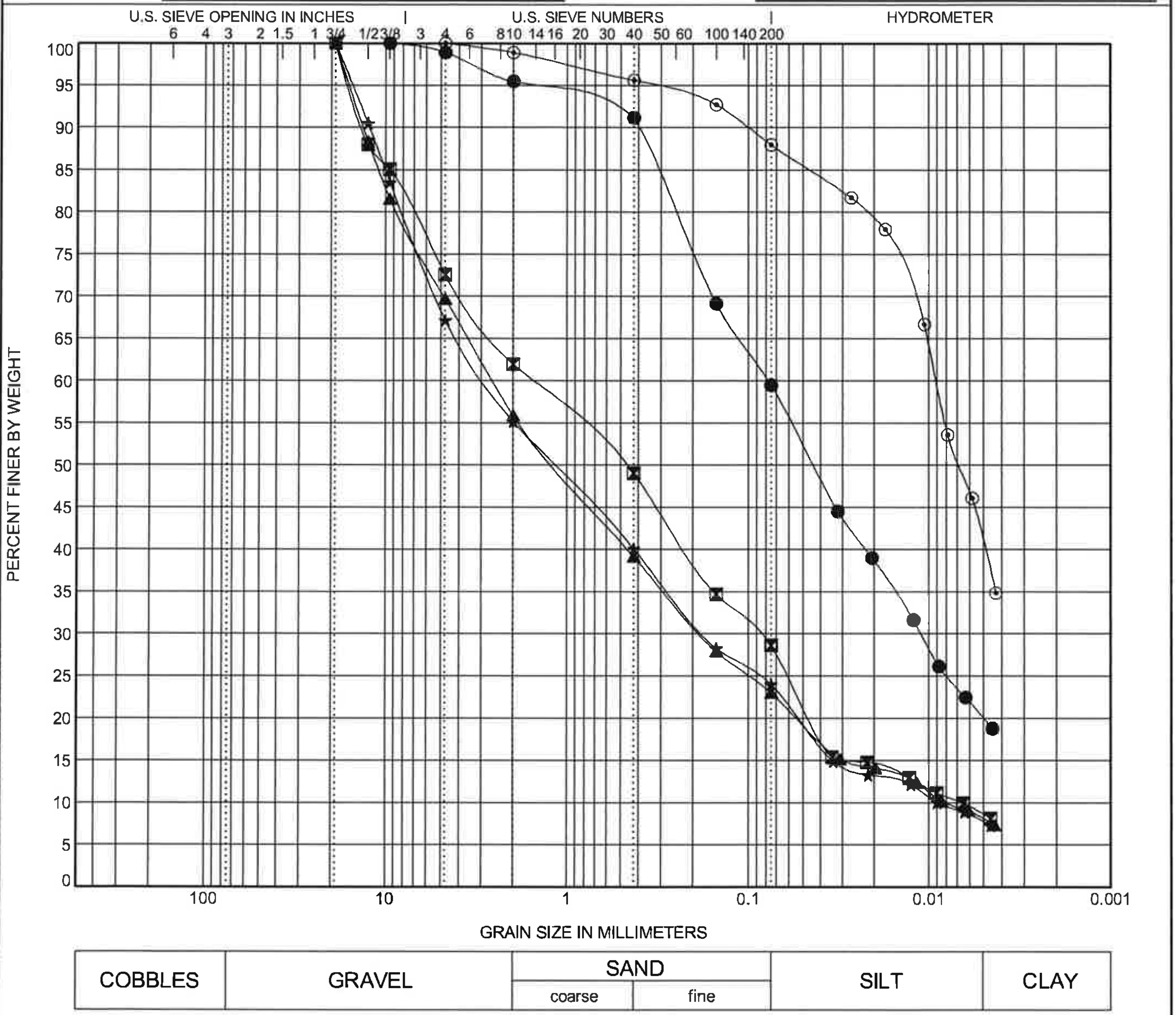
ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH 1 BAG CEMENT; BACKFILLED WITH 3 BAGS HOLE PLUG

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT MRG CR 52 BRIDGE #449 REPLACEMENT PID

OGE NUMBER 01021274 PROJECT TYPE BRIDGE REPLACEMENT



Specimen Identification			ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
●	B-001	1.5	A-6a ~ SANDY LEAN CLAY(CL)	30	17	13
⊠	B-001	9.0	~			
▲	B-001	10.5	~			
★	B-001	12.0	~			
⊙	B-001	13.5	~			

Specimen Identification			D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
●	B-001	1.5	0.402	0.044	0.011		4	4	32	40	20		
⊠	B-001	9.0	13.403	0.475	0.088	0.007	38	13	20	20	9	0.74	238.26
▲	B-001	10.5	13.267	1.16	0.182	0.008	44	17	16	15	8	1.57	317.42
★	B-001	12.0	12.256	1.174	0.174	0.009	45	15	16	16	8	1.19	312.64
⊙	B-001	13.5	0.101	0.007			1	3	8	47	41		

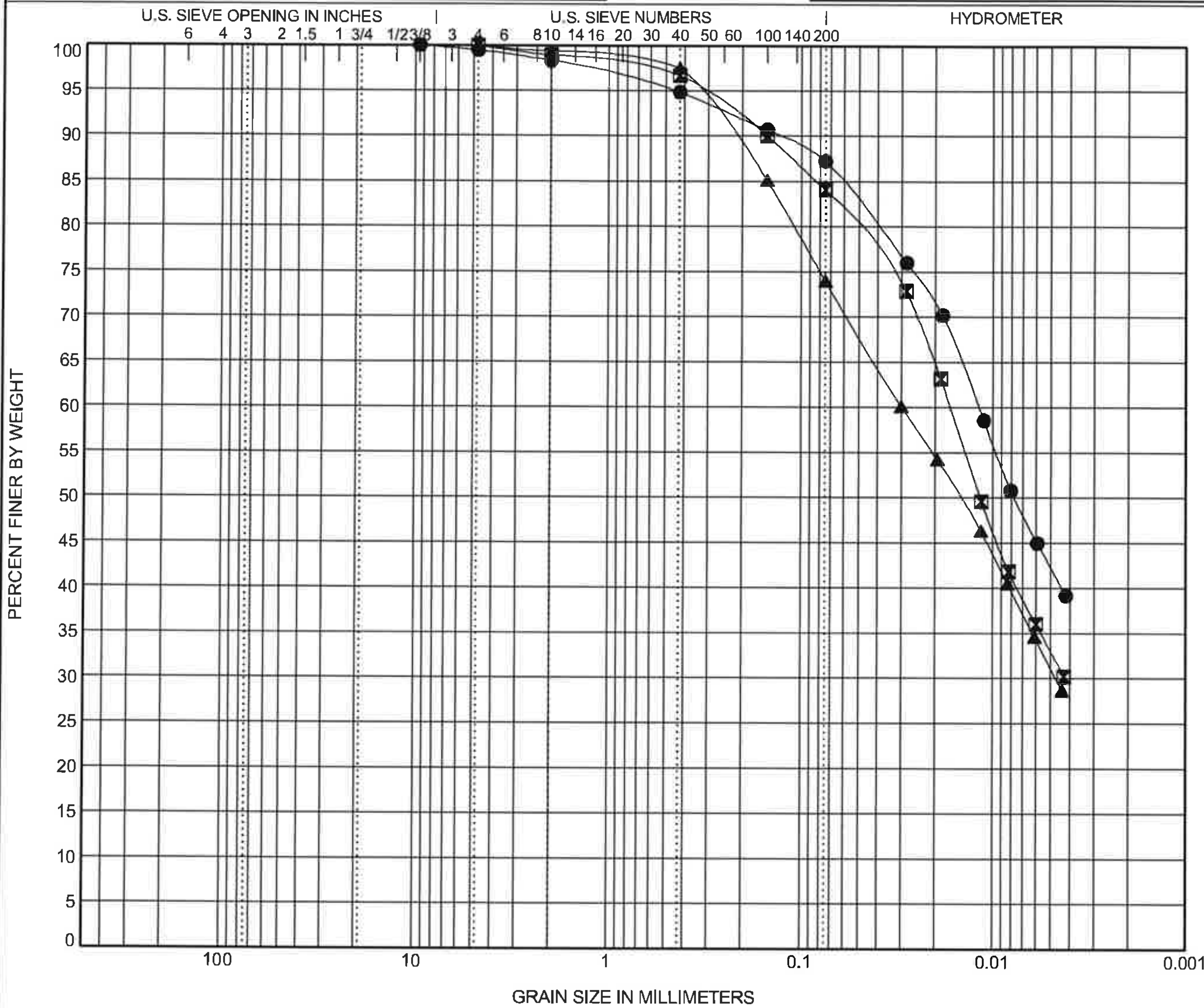
GRAIN SIZE - OH DOT.GDT - 4/20/18 12:33 - \\PSIPRODDBW02\BENTLEY_GINT\PROJECTS\ODOT 0102\01021274 ODOT.GPJ

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT MRG CR 52 BRIDGE #449 REPLACEMENT **PID**

OGE NUMBER 01021274 **PROJECT TYPE** BRIDGE REPLACEMENT



COBBLES	GRAVEL	SAND		SILT	CLAY
		coarse	fine		

Specimen Identification			ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
●	B-002	1.5	**A-7-6 ~ FAT CLAY(CH)**	50	22	28
⊠	B-002	3.5	**A-6a ~ LEAN CLAY with SAND(CL)**	37	22	15
▲	B-002	8.5	**A-6b ~ LEAN CLAY with SAND(CL)**	39	21	18

Specimen Identification			D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
●	B-002	1.5	0.131	0.008			2	3	8	45	42		
⊠	B-002	3.5	0.149	0.012			1	2	13	51	33		
▲	B-002	8.5	0.226	0.015	0.005		0	2	24	43	31		

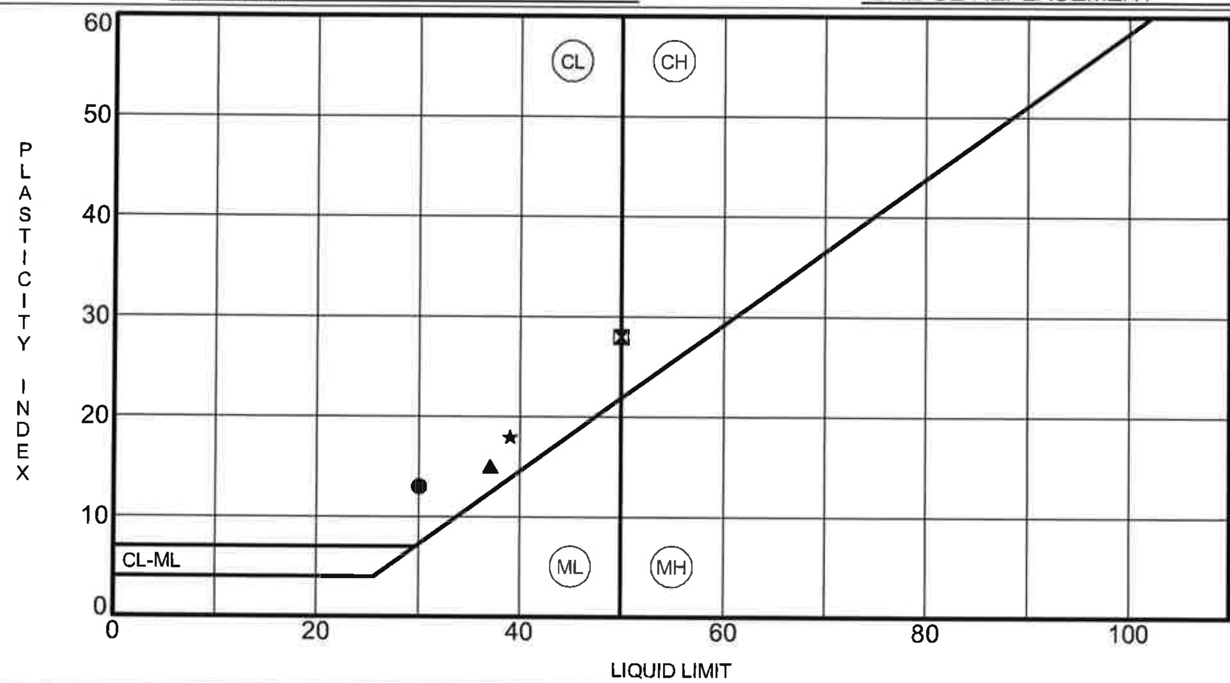
GRAIN SIZE - OH DOT.GDT - 4/20/18 12:33 - \\PSIPRODDBW02\BENTLEY_GINT\PROJECTS\ODOT 0102\01021274 ODOT.GPJ

ATTERBERG LIMITS' RESULTS

OHIO DEPARTMENT OF TRANSPORTION
OFFICE OF GEOTECHNICAL ENGINEERING

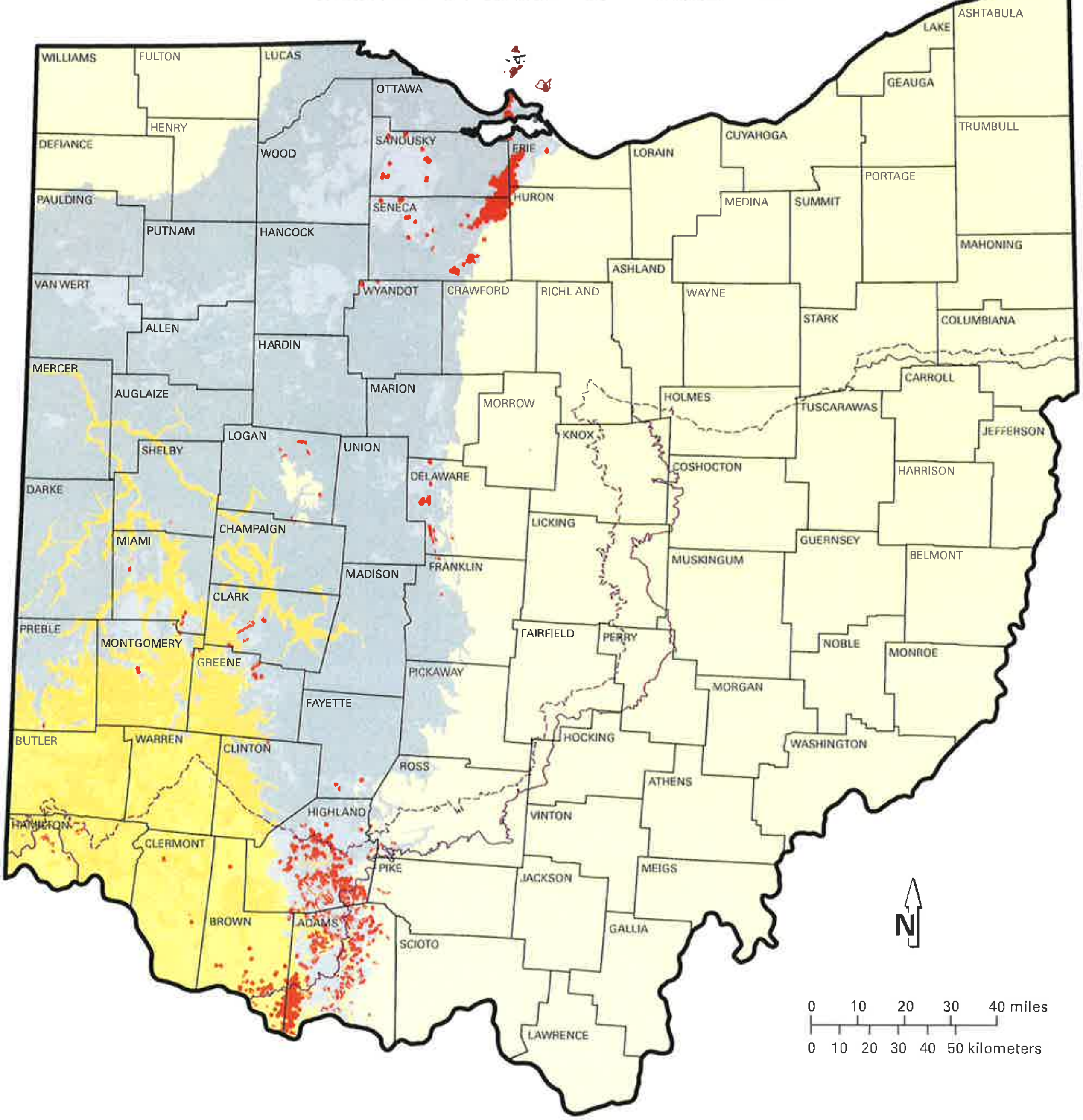
PROJECT MRG CR 52 BRIDGE #449 REPLACEMENT **PID**

OGE NUMBER 01021274 **PROJECT TYPE** BRIDGE REPLACEMENT



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**B-001**	**1.5**	**30**	**17**	**13**	**60**	**SANDY LEAN CLAY(CL)**
⊠	**B-002**	**1.5**	**50**	**22**	**28**	**87**	**FAT CLAY(CH)**
▲	**B-002**	**3.5**	**37**	**22**	**15**	**84**	**LEAN CLAY with SAND(CL)**
★	**B-002**	**8.5**	**39**	**21**	**18**	**74**	**LEAN CLAY with SAND(CL)**

OHIO KARST AREAS



EXPLANATION

- Silurian- and Devonian-age carbonate bedrock overlain by less than 20 feet of glacial drift and/or alluvium
- Silurian- and Devonian-age carbonate bedrock overlain by more than 20 feet of glacial drift and/or alluvium
- Interbedded Ordovician-age limestone and shale overlain by less than 20 feet of glacial drift and/or alluvium
- Interbedded Ordovician-age limestone and shale overlain by more than 20 feet of glacial drift and/or alluvium
- Probable karst areas
- Area not known to contain karst features
- Wisconsinan Glacial Margin
- Illinoian Glacial Margin

Recommended citation: Ohio Division of Geological Survey, 1999 (rev. 2002, 2006), Known and probable karst in Ohio: Ohio Department of Natural Resources, Division of Geological Survey Map EG-1, generalized page-size version with text, 2 p., scale 1:2,000,000.

OHIO KARST AREAS

Karst is a landform that develops on or in limestone, dolomite, or gypsum by dissolution and that is characterized by the presence of characteristic features such as sinkholes, underground (or internal) drainage through solution-enlarged fractures (joints), and caves. While karst landforms and features are commonly striking in appearance and host to some of Ohio's rarest fauna, they also can be a significant geologic hazard. Sudden collapse of an underground cavern or opening of a sinkhole can cause surface subsidence that can severely damage or destroy any overlying structure such as a building, bridge, or highway. Improperly backfilled sinkholes are prone to both gradual and sudden subsidence, and similarly threaten overlying structures. Sewage, animal wastes, and agricultural, industrial, and ice-control chemicals entering sinkholes as surface drainage are conducted directly and quickly into the ground-water system, thereby posing a severe threat to potable water supplies. Because of such risks, many of the nation's state geological surveys, and the U.S. Geological Survey, are actively mapping and characterizing the nation's karst regions.

The five most significant Ohio karst regions are described below.

BELLEVUE-CASTALIA KARST PLAIN

The Bellevue-Castalia Karst Plain occupies portions of northeastern Seneca County, northwestern Huron County, southeastern Sandusky County, and western Erie County. Adjacent karst terrain in portions of Ottawa County, including the Marblehead Peninsula, Catawba Island, and the Bass Islands, is related in geologic origin to the Bellevue-Castalia Karst Plain. The area is underlain by up to 175 feet of Devonian carbonates (Delaware Limestone, Columbus Limestone, Lucas Dolomite, and Amherstburg Dolomite) overlying Silurian dolomite, anhydrite, and gypsum of the Bass Islands Dolomite and Salina Group.

The Bellevue-Castalia Karst Plain is believed to contain more sinkholes than any of Ohio's other karst regions. Huge, irregularly shaped, closed depressions up to 270 acres in size and commonly enclosing smaller, circular-closed depressions 5 to 80 feet in diameter pockmark the land between the village of Flat Rock in northeastern Seneca County and Castalia in western Erie County. Surface drainage on the plain is very limited, and many of the streams which are present disappear into sinkholes called swallow holes.

Karst in the Bellevue-Castalia and Lake Erie islands region is due to collapse of overlying carbonate rocks into voids created by the dissolution and removal of underlying gypsum beds. According to Verber and Stansbery (1953, Ohio Journal of Science), ground water is introduced into Salina Group anhydrite ($CaSO_4$) through pores and fractures in the overlying carbonates. The anhydrite chemically reacts with the water to form gypsum ($CaSO_4 \cdot 2H_2O$), undergoing a 33 to 62 percent increase in volume in the process. This swelling lifts overlying strata, thereby opening fractures and creating massive passageways for conduction of greater volumes of ground water through the Silurian Bass Islands Dolomite and into underlying Salina Group strata. Gypsum, being readily soluble in water, is dissolved, creating huge voids. Overlying carbonates then collapse or break down, leaving surface depressions similar to those resulting from roof failure of an underground mine.

DISSECTED NIAGARA ESCARPMENT

The dissected Niagara Escarpment of southwestern Ohio includes the largest single area of karst terrain in the state and the greatest number of surveyed caves. It also is estimated to include the second-largest number of sinkholes in the state. The area is underlain by Silurian rocks of the Peebles Dolomite, Lilley Formation, Bisher Formation, Estill Shale, and Noland Formation in Adams, Highland, and Clinton Counties and the Cedarville Dolomite, Springfield Dolomite, Euphemia Dolomite, Massie Shale, Laurel Dolomite, Osgood Shale, and Dayton Formation in Greene, Clark, Miami, Montgomery, and Preble Counties. The Peebles-Lilley-Bisher sequence and the Cedarville-Springfield-Euphemia sequence constitute the Lockport Group.

Most karst features along the Niagara Escarpment in southwestern Ohio are developed in Lockport Group strata. More than 100 sinkholes and caves developed in the Lockport have been documented in the field, and more than 1,000 probable sinkholes in the Lockport have been identified on aerial photographs, soils maps, and topographic maps. As with most karst terrain, sinkholes developed on the Niagara Escarpment commonly show linear orientations aligned with prevailing joint trends in the area. The greatest concentration of sinkholes on the escarpment is south of the Wisconsinan glacial border in southern Highland and Adams Counties, where highly dissected ridges capped by Silurian carbonate rocks rise 150 to 200 feet above surrounding drainage. Illinoian till in these areas is thin to absent, and soils are completely leached with respect to calcium and calcium-magnesium carbonate. Such geologic settings are ideal for active karst processes, as downward-percolating, naturally acidic rain water is not buffered until it has dissolved some of the underlying carbonate bedrock. Other significant karst features of the Niagara Escarpment include small caves in escarpment re-entrants created by the valleys of the Great Miami and Stillwater Rivers in Miami County.

BELLEFONTAINE OUTLIER

The Bellefontaine Outlier in Logan and northern Champaign Counties is an erosionally resistant "island" of Devonian carbonates capped by Ohio Shale and surrounded by a "sea" of Silurian strata. Though completely glaciated, the outlier was such an impediment to Ice Age glaciers that it repeatedly separated advancing ice sheets into two glacial lobes—the Miami Lobe on the west and the Scioto Lobe on the east. Most Ohioans recognize the outlier as the location of Campbell Hill—the highest point in the state at an elevation of 1,549 feet above mean sea level.

Although it is not known for having an especially well-developed karst terrain, the outlier is the location of Ohio's largest known cave, Ohio Caverns. The greatest sinkhole concentrations are present in McArthur and Rushcreek Townships of Logan County, where the density of sinkholes in some areas approaches 30 per square mile. Sinkholes here typically occur in upland areas of Devonian Lucas Dolomite or Columbus Limestone that are 30 to 50 feet or more above surrounding drainage and are covered by less than 20 feet of glacial drift and/or Ohio Shale.

SCIOTO AND OLENTANGY RIVER GORGES

The uplands adjacent to the gorges of the Scioto and Olentangy Rivers in northern Franklin and southern Delaware Counties include areas of well-developed, active karst terrain. These uplands also are among the most rapidly developing areas of the state, which means karst should be a consideration in site assessments for commercial and residential construction projects.

The Scioto River in this area has been incised to a depth of 50 to 100 feet into underlying bedrock, creating a shallow gorge. The floor, walls, and adjacent uplands of the gorge consist of Devonian Delaware and Columbus Limestones mantled by up to 20 feet of Wisconsinan till. Sinkhole concentrations up to 1 sinkhole per acre are not uncommon in Concord, Scioto, and Radnor Townships of Delaware County. The sinkholes range in diameter from about 10 to 100 feet and commonly are aligned linearly along major joint systems.

The Olentangy River is approximately 5 miles east of the Scioto River in southern Delaware County and occupies a gorge that is narrower and up to 50 feet deeper than the Scioto River gorge. The floor and the lower half of the walls along the Olentangy gorge are composed of Delaware and Columbus Limestones, the upper half of the walls is composed of Devonian Ohio and Olentangy Shales mantled by a thin veneer of glacial drift. Karst terrain has developed along portions of the gorge in a manner similar to karst terrain along the Scioto River.

ORDOVICIAN UPLANDS

The Ordovician uplands of southwestern Ohio are the location of surprisingly well-developed karst terrain despite the large component of shale in local bedrock. Numerous sinkholes are present in Ordovician rocks of Adams, Brown, Clermont, and Hamilton Counties.

The carbonate-rich members of the Grant Lake Formation (Bellevue and Mount Auburn), Grant Lake Limestone (Bellevue and Straight Creek), and the upper portion of the Arnheim formation are the Ordovician units most prone to karstification; however, the shale-rich (70 percent shale, 30 percent limestone) Waynesville Formation also has been subjected to a surprising amount of karst development in southeastern Brown and southwestern Adams Counties, just north of the Ohio River.

ACKNOWLEDGMENT

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GENERAL NOTES

SAMPLE IDENTIFICATION

The Unified Soil Classification System (USCS), AASHTO 1988 and ASTM designations D2487 and D-2488 are used to identify the encountered materials unless otherwise noted. Coarse-grained soils are defined as having more than 50% of their dry weight retained on a #200 sieve (0.075mm); they are described as: boulders, cobbles, gravel or sand. Fine-grained soils have less than 50% of their dry weight retained on a #200 sieve; they are defined as silts or clay depending on their Atterberg Limit attributes. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size.

DRILLING AND SAMPLING SYMBOLS

SFA: Solid Flight Auger - typically 4" diameter flights, except where noted.
HSA: Hollow Stem Auger - typically 3¼" or 4¼ I.D. openings, except where noted.
M.R.: Mud Rotary - Uses a rotary head with Bentonite or Polymer Slurry
R.C.: Diamond Bit Core Sampler
H.A.: Hand Auger
P.A.: Power Auger - Handheld motorized auger

SS: Split-Spoon - 1 3/8" I.D., 2" O.D., except where noted.
ST: Shelby Tube - 3" O.D., except where noted.
BS: Bulk Sample
PM: Pressuremeter
CPT-U: Cone Penetrometer Testing with Pore-Pressure Readings

SOIL PROPERTY SYMBOLS

N: Standard "N" penetration: Blows per foot of a 140 pound hammer falling 30 inches on a 2-inch O.D. Split-Spoon.
N_{60}: A "N" penetration value corrected to an equivalent 60% hammer energy transfer efficiency (ETR)
Q_u: Unconfined compressive strength, TSF
Q_p: Pocket penetrometer value, unconfined compressive strength, TSF
w%: Moisture/water content, %
LL: Liquid Limit, %
PL: Plastic Limit, %
PI: Plasticity Index = (LL-PL),%
DD: Dry unit weight, pcf
▼,▽,▼ Apparent groundwater level at time noted

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Relative Density	N - Blows/foot
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	50 - 80
Extremely Dense	80+

ANGULARITY OF COARSE-GRAINED PARTICLES

Description	Criteria
Angular:	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular:	Particles are similar to angular description, but have rounded edges
Subrounded:	Particles have nearly plane sides, but have well-rounded corners and edges
Rounded:	Particles have smoothly curved sides and no edges

GRAIN-SIZE TERMINOLOGY

Component	Size Range
Boulders:	Over 300 mm (>12 in.)
Cobbles:	75 mm to 300 mm (3 in. to 12 in.)
Coarse-Grained Gravel:	19 mm to 75 mm (¾ in. to 3 in.)
Fine-Grained Gravel:	4.75 mm to 19 mm (No.4 to ¾ in.)
Coarse-Grained Sand:	2 mm to 4.75 mm (No.10 to No.4)
Medium-Grained Sand:	0.42 mm to 2 mm (No.40 to No.10)
Fine-Grained Sand:	0.075 mm to 0.42 mm (No. 200 to No.40)
Silt:	0.002 mm to 0.075 mm
Clay:	<0.002mm to <0.005 mm depending on agency

PARTICLE SHAPE

Description	Criteria
Flat:	Particles with width/thickness ratio > 3
Elongated:	Particles with length/width ratio > 3
Flat & Elongated:	Particles meet criteria for both flat and elongated

RELATIVE PROPORTIONS OF FINES

Descriptive Term	% Dry Weight
Trace:	< 5%
With:	5% to 12%
Modifier:	>12%

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GENERAL NOTES
(Continued)

CONSISTENCY OF FINE-GRAINED SOILS

Q_U - TSF	N - Blows/foot	Consistency
0 - 0.25	0 - 2	Very Soft
0.25 - 0.50	2 - 4	Soft
0.50 - 1.00	4 - 8	Firm (Medium Stiff)
1.00 - 2.00	8 - 15	Stiff
2.00 - 4.00	15 - 30	Very Stiff
4.00 - 8.00	30 - 50	Hard
8.00+	50+	Very Hard

MOISTURE CONDITION DESCRIPTION

Description	Criteria
Dry:	Absence of moisture, dusty, dry to the touch
Moist:	Damp but no visible water
Wet:	Visible free water, usually soil is below water table

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term	% Dry Weight
Trace:	< 15%
With:	15% to 30%
Modifier:	>30%

STRUCTURE DESCRIPTION

Description	Criteria
Stratified:	Alternating layers of varying material or color with layers at least ¼-inch (6 mm) thick
Laminated:	Alternating layers of varying material or color with layers less than ¼-inch (6 mm) thick
Fissured:	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided:	Fracture planes appear polished or glossy, sometimes striated

Description	Criteria
Blocky:	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed:	Inclusion of small pockets of different soils
Layer:	Inclusion greater than 3 inches thick (75 mm)
Seam:	Inclusion 1/8-inch to 3 inches (3 to 75 mm) thick extending through the sample
Parting:	Inclusion less than 1/8-inch (3 mm) thick

SCALE OF RELATIVE ROCK HARDNESS

Q_U - TSF	Consistency
2.5 - 10	Extremely Soft
10 - 50	Very Soft
50 - 250	Soft
250 - 525	Medium Hard
525 - 1,050	Moderately Hard
1,050 - 2,600	Hard
>2,600	Very Hard

ROCK BEDDING THICKNESSES

Description	Criteria
Very Thick Bedded	Greater than 3-foot (>1.0 m)
Thick Bedded	1-foot to 3-foot (0.3 m to 1.0 m)
Medium Bedded	4-inch to 1-foot (0.1 m to 0.3 m)
Thin Bedded	1¼-inch to 4-inch (30 mm to 100 mm)
Very Thin Bedded	½-inch to 1¼-inch (10 mm to 30 mm)
Thickly Laminated	1/8-inch to ½-inch (3 mm to 10 mm)
Thinly Laminated	1/8-inch or less "paper thin" (<3 mm)

ROCK VOIDS

Voids	Void Diameter
Pit	<6 mm (<0.25 in)
Vug	6 mm to 50 mm (0.25 in to 2 in)
Cavity	50 mm to 600 mm (2 in to 24 in)
Cave	>600 mm (>24 in)

GRAIN-SIZED TERMINOLOGY
(Typically Sedimentary Rock)

Component	Size Range
Very Coarse Grained	>4.76 mm
Coarse Grained	2.0 mm - 4.76 mm
Medium Grained	0.42 mm - 2.0 mm
Fine Grained	0.075 mm - 0.42 mm
Very Fine Grained	<0.075 mm

ROCK QUALITY DESCRIPTION

Rock Mass Description	RQD Value
Excellent	90 -100
Good	75 - 90
Fair	50 - 75
Poor	25 -50
Very Poor	Less than 25

DEGREE OF WEATHERING

Slightly Weathered:	Rock generally fresh, joints stained and discoloration extends into rock up to 25 mm (1 in), open joints may contain clay, core rings under hammer impact.
Weathered:	Rock mass is decomposed 50% or less, significant portions of the rock show discoloration and weathering effects, cores cannot be broken by hand or scraped by knife.
Highly Weathered:	Rock mass is more than 50% decomposed, complete discoloration of rock fabric, core may be extremely broken and gives clunk sound when struck by hammer, may be shaved with a knife.

SOIL CLASSIFICATION CHART

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

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