GEOTECHNICAL ENGINEERING INVESTIGATION

VERNON HISTORIC TRAIL

CITY OF VERNON
JENNINGS COUNTY, INDIANA
Atlas Project No. 170GC01588

PREPARED FOR:

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July 14, 2023

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Subject: **Geotechnical Engineering Investigation**
Vernon Historic Trail
Vernon, Indiana
Jennings County
Atlas Project No. 170GC01588

Dear Mr. Gill:

Submitted herewith is the report of the geotechnical engineering investigation performed by Atlas Technical Consultants LLC (Atlas) for the referenced project. This study was authorized in accordance with Atlas Client Services Agreement dated April 5th, 2023 and Atlas Proposal 23-03708 dated April 4, 2023.

This report contains the results of the field and laboratory testing program, an engineering interpretation of this data with respect to the currently available project characteristics and recommendations to aid design and construction of the foundations and other earth-connected elements of this project. We wish to remind you that we will store the samples for 90 days after which time they will be discarded unless you request otherwise.

We appreciate the opportunity to be of service to you on this project. If we can be of any further assistance, or if you have any questions regarding this report, please do not hesitate to contact either of the undersigned.

Respectfully submitted,
Atlas Technical Consultants LLC

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1. INTRODUCTION

This report presents the results of the geotechnical engineering evaluation performed by Atlas Technical Consultants, LLC (Atlas) for the proposed Vernon Historic Trail. The general location of the project site is shown on the Vicinity Map (see Figure 1 in Appendix).

This geotechnical evaluation was performed to characterize and evaluate the existing subsurface conditions at the site and to develop recommendations necessary for the design and construction of the soil supported elements of the proposed project. The evaluation consisted of a site reconnaissance, an exploratory drilling and sampling program, laboratory testing of soil samples obtained from the test borings, a review of the provided information, engineering analyses and preparation of this report.

2. PROJECT CHARACTERISITCS

It is our understanding that the project will be comprised of four project areas consisting of the construction of a compacted aggregate/earthen mountain bike/hiking trail (Project Area 1), a paved pedestrian trail (Project Area 2), a scenic river overlook (Project Area 3), and a public restroom area (Project Area 4).

Project Area 1 is described as a proposed hiking trail to be located at the west end of the project at the trailhead to the Muscatatuck Park Vinegar Mill Overlook and River Hiking Trail system. The proposed hiking trail will head east along the north bank of the Muscatatuck River and terminate at a proposed trailhead to be located near the intersection of Jackson Street with Water Street in the Town of Vernon. At the time of this report the process to secure Right of Entry had not completed, therefore this report does not include any assessment or evaluation for Project Area 1.

Project Area 2 is described as a proposed paved pedestrian trail that will begin at the intersection of Jackson Street with Water Street and travel east along Jackson Street to Montgomery Drive. The paved trail will then travel south along Montgomery Drive to the Vernon Gym. The paved pedestrian trail will then head east from the proposed river overlook to South Pike Street. The trail will then travel north on South Pike Street to Jackson Street. The trail will then extend east along Jackson Street to Commons Street, terminating at Vernon’s Commons Park.

Project Area 3 is a proposed river overlook on the south side of the Vernon Gym and will feature a stone seating wall approximately 25 feet in length. It is anticipated that the overlook structure will be constructed as wooden deck-like structure and overlook the riverside slope. Specific details regarding this structure is not available at this time.

Project Area 4 is comprised of a new public restroom and parking area located within the existing Vernon’s Commons Park.

3. PURPOSE AND SCOPE

The purpose of this study was to determine the general subsurface conditions at the project site by drilling five test borings and to evaluate this data with respect to foundation concept and design for the proposed pedestrian trail, overlook structure, and public restroom. Also included is an evaluation of the site with respect to potential construction problems and recommendations dealing with earthwork and quality control during construction.

It is important to note that the results of this investigation should be used for preliminary planning purposes only and that additional test borings, laboratory testing and engineering analyses may be needed in conjunction with more detailed project characteristics in order to develop final geotechnical design and construction recommendations for specific projects or facilities.

3.1 Field Investigation

A total of five test borings were drilled at the approximate locations shown on the Boring Plan (Figure 3 in the Appendix). The test borings were extended to depths of 7.5 to 15 ft below the existing grade. Split-barrel samples were obtained by the Standard Penetration Test procedures (ASTM D1586) at 2.5 to 5.0 ft intervals.

The number, locations and depths of the test borings were selected by Atlas with direction from FPBH, Inc. The test boring locations were staked in the field by Atlas personnel using a handheld GPS, with test boring elevations estimated from preliminary plans prepared by FPBH, Inc. The test borings were drilled at the approximate locations noted on the Boring Logs in Appendix II and as shown on the Boring Plan in Figure 3 of the Appendix.

Logs of all test borings, which show visual descriptions of the pavement section and all soil strata encountered using the AASHTO classification system (AASHTO M145), are included in Appendix II. Sampling information and other pertinent field data and observations are also included on the Test Boring Logs. In addition, a sheet defining the terms and symbols used on the Test Boring Logs and explaining the SPT procedure is provided immediately preceding the Boring Logs in Appendix II.

3.2 Laboratory Investigation

The soil samples were visually classified by a geotechnical engineer in accordance with the AASHTO Soil Classification System (AASHTO M145). The visual classifications were subsequently verified or modified based upon the results of laboratory tests. Boring Logs were subsequently prepared and are included in Appendix II and summary sheets of classification testing are included in Appendix III. Soil index property tests including natural moisture content tests (AASHTO T265), grain size analyses (AASHTO T88), and Atterberg limits tests (AASHTO T89 and T90) were performed on representative samples. In addition to classification tests, calibrated hand penetrometer tests (“pocket penetrometer” tests) were performed on intact cohesive samples. The results of all laboratory tests are included on the Boring Logs in Appendix II and summary sheets in Appendix III.

Table No. 1 - Laboratory Testing Program

Laboratory Test Description	Test Method Designation
AASHTO Soil Classification System	AASHTO M145
Moisture Content Test of Soils	AASHTO T265
Atterberg Limits Tests	AASHTO T89 AASHTO T90
Grain Size Analysis	AASHTO T88
Calibrated Hand Penetrometer Test ("Pocket Penetrometer Test")	NA

NA- No standardized test method available.

4. GENERAL SUBSURFACE CONDITIONS

The general subsurface conditions were investigated by drilling five test borings to depths of 7.5 to 15 ft at the approximate locations shown on the Boring Plan (Figure 3 in the Appendix). The subsurface conditions disclosed by the field investigation are summarized in the following paragraphs. Detailed descriptions of the subsurface conditions encountered in each test boring are presented on the Boring Logs in Appendix II. It should be noted that the stratification lines shown on the Boring Logs represent approximate transitions between material types. In-situ stratum changes could occur gradually or at slightly different depths.

4.1 Subsurface Soil and Bedrock Conditions

Test Borings PR-B-201 and PRC-202 were advanced within Project Area 4 (Proposed Public Restroom) and generally encountered approximately 6 inches of topsoil. Beneath the topsoil, these borings generally encountered soft to medium stiff Sandy Loam (A-2-6) extending to approximate depths of 5.5 feet to 13.5 feet.

Test Boring RB-103 was completed within Project Area 2 (Pedestrian Trail) while test borings RO-B-1A and RO-B-2A were completed in Project Area 3 (River Overlook). These test borings generally encountered a pavement section consisting of about 4 inches to 5 inches of asphalt pavement; with the exception of RB-103, no aggregate subbase was encountered underlying the pavement. Table No. 2 summarizes the existing pavement sections encountered at the boring locations.

Table No. 2 – Summary of Existing Pavement Section

Boring ID	Project Area	Asphalt Pavement Thickness (in)	Aggregate Base Thickness (in)
RB-103	Trail	5.0	4.0
RO-B-1A	Overlook	4.0	--
RO-B-2A	Overlook	5.0	--

Below the existing asphalt section described above, the test borings encountered mostly highly plastic soils consisting of clay (A-7-6), sandy clay loam (A-6), and silty clay loam (A-7-6). Logs for all of the soil test borings are included in Appendix II. Cohesive soils were primarily very soft to medium stiff consistency.

The cohesive soils encountered in the test borings exhibited Liquid Limit (LL) values ranging from about 24 to 66 percent and Plasticity Index (PI) values ranging from 10 to 48 percent.

Three of the test borings drilled for this project were drilled to auger refusal. Auger refusal is defined herein as the depth at which a conventional test drill rig cannot advance the hollow-stem-augers or continuous-flight-augers. It is important to understand that auger refusal is not necessarily coincident with the bedrock surface since the augers can penetrate the upper weathered or fractured bedrock in some cases, or can encounter refusal on objects above the bedrock surface (such as concrete slabs, rubble, etc.). The following table summarizes the depths and elevations at which auger refusal was encountered in the test borings drilled for this investigation.

Table No. 3 – Summary of Auger Refusal Depth

Boring ID	Approximate Auger Refusal Depth (ft)	Approximate Auger Refusal Elevation (ft)
PR-B-201	13.5	617
PR-B-202	8.5	615
RB-103	7.5	685

The qualitative strengths or consistencies of the cohesive soils and the qualitative densities of the granular soils as described above and on the test boring logs were estimated based on the results of the standard penetration test (ASTM D-1586) and based on the definitions as described on the Classification System for Soil Exploration contained in the Appendix of this report. Most of the soils described as “soft” on the basis of this criterion appear to be somewhat stronger based on examination of the samples and the results of calibrated hand penetrometer testing.

4.2 Ground Water Conditions

Ground water observations were made during the drilling operations by noting the depth of free ground water (if any) on the drilling tools and in the open boreholes (if any) immediately after withdrawal of the drilling augers. No free ground water was noted during or at completion of drilling in any of the borings. However, it must be noted that fluctuations in the level of the ground water will occur due to variations in rainfall and other factors not evident at the time of our investigation.

5. DESIGN RECOMMENDATIONS

The following design recommendations have been developed on the basis of the previously described project characteristics (Section 2) and subsurface conditions (Section 4). If there are any changes in these project criteria, including the project location on the site, the finish ground floor elevation, site grading, structure type, loading conditions, etc., a review should be made by this office.

It is important to note that the test boring coverage, and in particular the project information, are not sufficient at this time to develop final geotechnical engineering design recommendations for any specific facility. A final geotechnical engineering evaluation must be performed specifically for the proposed facilities within the development, which may require additional test borings and should be based upon detailed project characteristics, grading plans and specific locations and characteristics of the proposed facilities.

The design recommendations presented herein are contingent upon the assumption that all earth related elements of the project will be carefully and continuously observed, tested and evaluated by a representative of Atlas Technical Consultants LLC to confirm that the earth related elements of the project are compatible and consistent with the conditions upon which the design recommendations are based. The careful and thorough field testing and observations of the soil related aspects of the project are a critical and essential component of the design recommendations.

5.1 Seismic Parameters

Based on geologic mapping, the results of the test borings and our experience, it is our opinion that the subsurface conditions at this site meet the criteria for Site Class C based on Chapter 20 of ASCE 7-16, "Minimum Design Loads and Associated Criteria for Buildings and Other Structures". The recommended seismic design parameters are summarized in the following table:

Table No. 4 – Recommended Seismic Design Parameters

Seismic Design Parameter	Recommended Class/Value
Seismic Site Class[1]	C
Site Modified Peak Ground Acceleration, PGA_M	0.129g
Design Spectral Response Acceleration at Short Periods[2], S_{DS}	0.168g
Design Spectral Response Acceleration at 1-Second Period[2], S_{D1}	0.089g

1. **Based upon Chapter 20 of ASCE 7-16 "Minimum Design Loads and Associated Criteria for Buildings and Other Structures"**
2. **Based upon Chapter 11 of ASCE 7-16 "Minimum Design Loads and Associated Criteria for Buildings and Other Structures"**

There is virtually no probability of “liquefaction”, a phenomenon whereby ground shaking due to earthquake activity causes a severe loss of soil strength of granular soils, or “cyclic softening”, significant strength reduction of cohesive soils due to earthquake activity, of the soils at this project site under any reasonably anticipated seismic event.

5.2 Project Area 2 - Walking Trail

Project Area 2 will be a proposed paved pedestrian trail that will begin at the intersection of Jackson Street with Water Street and travel east along Jackson Street to Montgomery Drive. The paved trail will then travel south along Montgomery Drive to the Vernon Gym. The paved pedestrian trail will then head east from the proposed river overlook to South Pike Street. The trail will then travel north on South Pike Street to Jackson Street. The trail will then extend east along Jackson Street to Commons Street, terminating at Vernon’s Commons Park.

5.2.1 Pavement Design Considerations

The pavement subgrades are anticipated to consist primarily of naturally-occurring, medium to high plasticity cohesive soils; or engineered fill similar to the near-surface soils observed at the test boring locations. Although the soils encountered in the test borings appear to be suitable for support of the multi-use trail (the existing pavement is currently supported directly upon these soils), it must be noted that even those soils that may currently be relatively firm can become unstable during construction when exposed to precipitation and construction traffic. Our experience indicates that most subgrade soils beneath existing pavements will be soft or yielding once the existing pavement section is removed, regardless of the presence of the existing pavement and apparently firm soils in the test borings.

Given the urban environment and potential for shallow utilities, a Type V subgrade treatment is recommended for use along the proposed Multi-Use Trail. Subgrade treatment Type V shall be in accordance with ISS 207.04 consisting of 3 inches of the subgrade excavated and replaced with coarse aggregate No. 53. No additional foundation improvement is required. A resilient modulus value of 3,000 lbs/sq.in. is recommended for use in pavement design for the natural subgrade soil. A resilient modulus value of 4,500 lbs/sq.in. is recommended for use in pavement design in conjunction with Type V subgrade treatment for the Multi-use trail. The table on the following page summarizes the recommended parameters for the design of the pavements for the bridge rehabilitation maintenance of traffic.

Table No. 5 - Pavement Design Parameters

Untreated Subgrade Soil Resilient Modulus Value, lbs/sq.in.	3,000
Modified/Prepared Subgrade Soil Resilient Modulus Value, lbs/sq.in.	4,500
Predominant Subgrade Soil	CLAY A-7-6
Percent Passing #200	66
Percent Silt	36
Liquid Limit, percent	66
Plastic Limit, percent	18
Plasticity Index, percent	48
Approximate Depth to Ground Water, ft.	>5
Natural Dry Density of Untreated Subgrade (pcf)	100
Natural Moisture of Untreated Subgrade, percent	17
Subgrade Treatment Type (ISS Section 207.04)	Type V

5.3 Project Area 3 - Proposed Overlook Structures

Project Area 3 is a proposed river overlook on the south side of the Vernon Gym and will feature a stone seating wall approximately 25 feet in length. It is anticipated that the overlook structure will be constructed as wooden deck-type structure however, specific design details regarding this structure are not available at this time.

The subsurface conditions at the project site generally consist of an upper stratum of medium stiff cohesive soils to a depth of approximately 10.5 ft below the existing ground surface. Underlying these soils, the test borings generally encountered weathered shale extending to the termination depth of the borings at 15 feet. It should be noted that auger refusal was not encountered at these boring locations. The riverside slope, which will likely not be modified as part of this project, is relatively steep, and is likely to be susceptible to sloughing and sliding during and immediately after elevated water level events. It is recommended that measures be taken to protect the proposed overlook structure foundations from potential sliding or sloughing.

5.3.1 Drilled Pier Foundations

Drilled pier foundations that extend to bear in the shale can be used to support the overlook structure and resist the axial compression and tension loading. It is estimated that 2 ft diameter drilled piers that are properly reinforced for the loading conditions, filled with concrete and extend to a minimum depth of 13 ft below the existing ground surface can support loads of up to 10 kips/drilled pier in axial compression and 2 kips/drilled pier in axial tension.

The drilled piers should be designed and constructed in general accordance with ACI 336.3R "Design and Construction of Drilled Piers". In order to attain the design capacities, the drilled piers must not be spaced closer than 3 pier diameters (center-to-center). The drilled piers will require temporary casing to prevent side caving and to help minimize ground water infiltration into the drilled pier excavations. It is recommended that only straight shaft drilled piers be used and that belling or underreaming of the piers should not be attempted.

5.4 Project Area 4 - Proposed Public Restroom

Project Area 4 is comprised of a new public restroom and parking area to be located in Vernon's Commons Park. It is anticipated the proposed public restroom building will be a lightly loaded structure constructed with conventional shallow spread footing foundations.

The subsurface conditions at the project site generally consist of an upper stratum of medium stiff cohesive soils to a depths ranging from approximately 5.5 feet to approximately 13 feet below the existing ground surface. Underlying these soils, the test borings generally encountered weathered shale extending to the termination depth of the borings.

5.4.1 Spread Footings

Our findings show that the proposed public restroom building can be supported on conventional shallow spread footings provided that all unsuitable soils (i.e., miscellaneous or uncontrolled fill soils, soils containing marl or other organic materials, and any softer or looser natural soils) are completely removed from beneath the proposed spread footings. Spread footings that bear on firm existing natural soil, or on well-compacted engineered fill that is placed directly over firm existing natural soil after first removing all unsuitable materials, can be designed for a net allowable soil bearing pressure of 2,500 lbs/sq.ft for both column (square type) and wall (strip type) footings. The net allowable soil bearing pressure can be increased by a factor of 1.33 for transient loading conditions, such as wind gusts and earthquake loads.

Zones or pockets of unsuitable bearing materials in the form of miscellaneous or uncontrolled fill or softer or looser natural soils may be encountered at some spread footing locations to a depth of approximately 3.5 ft below the existing ground surface and may extend deeper at isolated locations. Where pockets of unsuitable materials are encountered below the base of a spread footing, it will be necessary to remove the unsuitable materials at the footing locations (see Section 6.3 for undercutting details) in order for the spread footings to bear on suitable soils. The need for removal and replacement of unsuitable soils should be determined based on careful field observations at the time of construction, and it is suggested that the contract documents include provisions for the removal and replacement of unsuitable materials as determined to be necessary based on these field observations.

It is important that the soil at the base of each spread footing excavation be carefully observed and evaluated as described in Section 6.3 so that all unsuitable materials, such as miscellaneous or uncontrolled fill, soils containing marl or other organic materials, and any softer or looser natural soils, can be identified and removed and to verify that the footings will bear on suitable soils. The careful and thorough field testing and observation of the soils at the bases of the spread footing excavations are critical and essential components of the foundation design.

In using net pressure, the weight of the footing and backfill over the footing, including the weight of the floor slab, need not be considered; hence, only loads applied at or above the finished floor need to be used for dimensioning the footings. Wall footings should be at least 18 in. wide, and column footings should be at least 3 ft wide for bearing capacity considerations.

All exterior footings and footings in unheated areas should be located at a depth of at least 3 ft below the final exterior grade for frost protection. Although the Indiana Building Code requires only 2.5 ft of foundation embedment below the exterior grade in Jennings County, our experience indicates that the actual frost depths in this region can occur deeper. Interior footings can be located at nominal depths below the finished floor, provided all undesirable materials (i.e., miscellaneous or uncontrolled fill materials, remnants from previous construction, soils containing organics and/or marl, softer or looser natural soils, etc.) are removed at the footing locations.

Provided the footings are designed as prescribed herein and the foundation soils are observed and evaluated as outlined in Section 6.3, it is estimated that the total and differential foundation settlements should not exceed about 1 in. and ¾ in., respectively. Careful field control will contribute substantially to minimizing the settlements.

Uplift forces on the spread footings can be resisted by the weight of the footings and the soil backfill material that is placed over the footings. It is recommended that the soil backfill weight considered to resist uplift loads be limited to that immediately above and within the perimeter of the footings unless a much higher factor of safety is used. A total soil unit weight of 110 lbs/cu.ft can be used for the backfill material placed above the footings, provided it is compacted as recommended in Section 6.2. It is also recommended that a factor of safety of at least 1.3 be used for calculating uplift resistance from the footings, provided only the weight of the footing and the soil immediately above it are used to resist uplift forces.

Lateral loads imparted upon shallow spread footings can be resisted by the passive lateral earth pressure against the sides of the footings and by friction between the foundation soils and the bases of the footings. If passive lateral earth pressure is to be used to resist lateral loads imparted on the spread footings, it is essential that the soil that is relied upon to provide the passive lateral earth pressure resistance is not excavated or otherwise disturbed at any time in the future. If it is possible that disturbance or an excavation could be made in any portion of the passive zone, including not only soils immediately beside the spread footings but also the soils that exist above the top of the footing elevation, since the passive resistance is dependent upon the weight of the overburden soils, then passive lateral earth pressure resistance should not be considered for resistance of lateral loads.

Based upon the soils encountered at this site, an allowable passive lateral earth pressure (allowable “equivalent fluid pressure”) of 110 lbs/sq.ft per foot of depth below the ground surface can be used for that portion of the footing that is below a depth of 3 ft below the final exterior grade or 2.5 ft below the interior floor slab; no portion of the footing above these depths should be used for lateral resistance. Since significant displacement is required to mobilize passive resistance, a factor of safety of 2.5 has been used to determine the allowable equivalent fluid pressure for the passive condition in order to minimize the potential for excessive displacement. An allowable coefficient of friction between the base of the footing and the underlying soil of 0.2 (based on a factor of safety of 1.5) can be used in conjunction with the minimum downward load on the base of the footing.

5.4.2 Floor Slabs

Floor slabs can be supported on firm existing soils or on new compacted structural fill placed directly over firm existing soils. The slab subgrade should be prepared and inspected as described in Section 6.1 of this report, and any clearly unsuitable materials (i.e., fill that contains collapsible objects or degradable materials; concentrations of rubble and debris; remnants from previous structures or old utilities, such as sewers, cisterns, wells, etc.; softer or looser soils; soils containing organics; etc.) should be removed and replaced with engineered fill. Any higher-plasticity, clayey soils that are to be used within the upper 12 inches of the slab subgrade may require chemical modification prior to placement, as they may be prone to shrinking and swelling if not so treated.

It is recommended that the floor slab be supported on a 4-inch-thick (minimum) layer of granular material, such as sand and gravel or crushed stone. This is to help distribute concentrated loads and equalize moisture conditions beneath the slab. Provided that a minimum of 4 in. of granular material is placed below the slab, a modulus of subgrade reaction (k_{30}) of 125 lbs/cu.in. can be used for design of the floor slabs.

5.4.3 Proposed Parking Area - Pavement

The pavement subgrade soils should be prepared and inspected as described in Sections 6.1 and 6.2 of this report. Any softer, looser, or otherwise unsuitable materials (e.g., remnants from previous structures, rubble, debris, soils containing organics, degradable or collapsible objects, etc.) that are identified beneath the pavement subgrade level should be removed and replaced with well-compacted engineered fill material and compacted as described in Section 5.2 of this report.

Details regarding site grading in pavement areas are not available at this time; however, depending upon grading requirements and seasonal conditions, it is likely that the pavement subgrade in most areas of the site will be wet, soft, or yielding at the time of construction. Based on our experience with soils of the type underlying this site, the natural subgrade soils at this site may yield and become unstable under construction traffic, particularly if the construction will be done during seasons when heavy precipitation and cooler temperatures typically occur (such as late fall, winter, and spring). In general, yielding subgrade problems are more prominent in cut areas (where saturated or nearly saturated silty and clayey soils are exposed by the excavation) or where little or no fill is to be placed.

The extent to which yielding subgrades may be a problem is difficult to predict beforehand since it is dependent upon several factors, some of which are controllable and some of which are not, including seasonal conditions, precipitation, cut depths, sequencing and scheduling of earthwork, surface and subsurface drainage measures, the weight and traffic patterns of construction equipment, etc. In order to cope with constructability problems and to avoid schedule delays associated with these types of soil conditions, it would be prudent to develop a contingency plan for subgrade stabilization so that it can be implemented where deemed necessary by the geotechnical engineer at the time of construction based on the specific field conditions encountered.

If, at the time of construction, the subgrade is found to be excessively wet, soft, or yielding, it is recommended that the pavement subgrade soils be stabilized by discing, aerating, and recompacting. However, if it is not possible to improve the subgrade soils in this manner because of weather conditions, scheduling, or other conditions, it is recommended that the subgrade soils be improved or modified using either chemical stabilization (i.e., lime-kiln dust or cement), mechanical stabilization (i.e., a geogrid with additional crushed limestone placed over the subgrade), or removal of the unsuitable soils and replacement with crushed limestone and/or suitable fill soils determined to be appropriate by the geotechnical engineer.

The best method for stabilizing the pavement subgrade should be determined in the field at the time of construction based upon the actual field conditions in conjunction with the specific soil type(s) encountered at the locations requiring stabilization, the sizes of the areas requiring stabilization, and the construction schedule. Chemical stabilization (i.e., lime or cement stabilization), if implemented, should be performed by a specialty contractor, who has the necessary equipment and experience in the determination of the appropriate chemical stabilizer and in the application of chemical stabilization methods.

The pavement subgrade surface should be uniformly sloped to facilitate drainage through the granular base and to avoid ponding of water beneath the pavement. The storm water catch basins in pavement areas should be designed to allow water to drain from the aggregate base into the catch basins. At a minimum, subsurface trench drains should be included that extend out at least 20 ft from the catch basins in at least four directions. Subsurface perforated drainage pipes should also be included beneath the lowest lines of the pavement and between catch basins. If catch basins are not included within the pavement areas, subsurface drains should be included near the lower or outside edges of the pavement to prevent water from being trapped or dammed up within the aggregate base. Including subsurface perforated drainage pipes along the edges of the entrance roads (where a greater amount of concentrated truck traffic occurs) may also help enhance the long-term performance of the pavements in these areas.

Based on the results of classification tests and our experience with similar soils, a resilient modulus value of 4,500 lbs/sq.in. has been estimated for use in pavement design for the subgrade soils encountered at this site, provided the subgrade is prepared and evaluated as described herein and in Sections 6.1 and 6.2 of this report. The following report sections outline recommendations for asphalt pavements for automobile parking areas. It is important to note that the recommendations for the automobile parking areas are based on the assumption that these areas will not be subject to any heavy truck traffic. Therefore, in areas where truck traffic cannot be controlled (i.e., driveways), it is suggested that the thicker pavement section be utilized.

5.4.4 Asphalt Pavement

Based on a resilient modulus value of 4,500 lbs/sq.in., a design period of 15 years, and the conditions encountered at the site, the following asphalt pavement sections are recommended:

Automobile Parking Areas:	3 in. of asphaltic concrete over 6 in. of granular base
Driveway Areas and Truck Zones:	5 in. of asphaltic concrete over 9 in. of granular base

The aggregate base should consist of well-compacted crushed limestone that meets the requirements for coarse aggregate size No. 53 in accordance with Indiana Department of Transportation (INDOT) Standard Specifications. Aggregates that are locally referred to as “commercial grade” No. 53 crushed stone should not be used as pavement base material. The hot mix asphalt (HMA) pavement should be constructed in accordance with the 2022 INDOT Standard Specifications Section 400 – Asphalt Pavements, and the HMA mix design should be in accordance with INDOT Standard Specifications Section 402-Hot Mix Asphalt, HMA, Pavement.

It should be expected that normal maintenance compatible with asphalt pavement and the design period selected will be required during the life of the pavement. Furthermore, overlaying the pavement surface may be desirable at an intermediate time period to extend the life of the pavement and improve serviceability.

5.5 Site Grading and Drainage

Proper surface drainage should be provided at the site to minimize any increase in moisture content of the foundation soils. The exterior grade should be sloped away from the structure to prevent flow of surface water toward the building and to prevent ponding of water around the building. Any roof drains or downspouts should be channeled or piped to locations well away from the structure.

It is recommended that final cut and fill slopes be no steeper than 3 (horizontal) to 1 (vertical). Flatter cut slopes may be required in cases where there is ground water seepage or where the foundation soils are particularly poor. Where new fill is placed against existing slopes that are steeper than 6 (horizontal) to 1 (vertical), it will be necessary to “bench” the new fill into the existing slope in order to provide a good bond between the existing soil and the new fill and to prevent the development of a zone of weak soil at the interface.

The soils encountered in the test borings at this site consist of low permeability to virtually impermeable cohesive soils that are not conducive to disposal of storm water by infiltration methods. The cohesive soils at this site have an estimated infiltration rate of approximately 0.1 in./hr, or less.

It is extremely important to understand that subsurface soil and ground water conditions can change through time, such as a rise in the ground water level and a decrease in the permeability of some subsurface soils (such as can happen in granular soils due to intrusion of fines transported by the storm water into the soils). Therefore, where storm water infiltration systems are implemented, it is essential that measures are included for cleaning and maintaining the performance of the infiltration elements. Furthermore, appropriate storage capacity and/or an alternate surface discharge outfall should be included for cases where variability in infiltration rates may be expected (where the soil conditions are erratic and inconsistent), for cases where the system performance will be diminished or impaired over time (e.g., sedimentation loading, vegetation growth, etc.), and particularly for cases when the ground water level is higher and the infiltration characteristics are much less reliable.

6. GENERAL CONSTRUCTION PROCEDURES AND RECOMMENDATIONS

Since this investigation identified actual subsurface conditions only at the test boring locations, it was necessary to extrapolate these conditions in order to characterize the entire project site. Even under the best of circumstances, the conditions encountered during construction can be expected to vary somewhat from the test boring results and may, in the extreme case, differ to the extent that modifications to the recommendations provided herein become necessary. Therefore, we recommend that Atlas be retained as geotechnical consultant through the earth-related phases of this project to correlate actual soil conditions with test boring data, identify variations, conduct additional tests that may be needed and recommend solutions to earth-related problems that may develop.

6.1 Site Preparation

All areas that will support footings, floor slabs, and pavements should be properly prepared. After clearing all vegetation, topsoil, and any other unsuitable materials, after rough grade has been established in cut areas, and prior to placement of fill in all fill areas, the exposed subgrade should be carefully observed by the geotechnical engineer, or a qualified soils technician working under the direction of the geotechnical engineer, by probing and testing as needed. Any clearly unsuitable fill materials (such as miscellaneous fill containing remnants from previous construction, concentrations of rubble or debris, degradable materials, etc.); soils containing organics; frozen, wet, or softer/looser soils; and other undesirable materials should be completely removed. The exposed subgrade should furthermore be evaluated by proof rolling with suitable equipment to check for pockets of soft or weak material hidden beneath a thin crust of better soil. Any unsuitable materials thus exposed should be removed and replaced with well-compacted engineered fill as outlined in Section 6.2.

Based on our experience, it appears likely that some of the subgrade soils at this site will be wet, soft, or yielding at the time of construction, especially in areas of cut or where little or no fill is to be placed or if the construction will be done during seasons when heavy precipitation and cooler temperatures typically occur (such as late fall, winter, and spring). It may be possible to stabilize the pavement subgrade soils in areas that are found to be excessively wet, soft, or yielding at the time of construction by discing, aerating, and recompacting. However, if it is not possible to improve the subgrade soils in this manner, stabilization or modification of the subgrade soils (such as by removal of unsuitable soils and replacement with compacted fill, chemical stabilization with lime-kiln dust or cement, mechanical stabilization, etc.) may be required in order to develop a firm working base upon which to construct the fill for the building pad or pavement section. It is suggested that the project include contingency plans for stabilization or modification, as described above, to be used as determined appropriate by the geotechnical engineer at the time of construction. Refer to Section 5.4.3 for further details and recommendations concerning subgrade stabilization and modification.

Care must be exercised during the grading operations at the site. Due to nature of the near-surface soils, the traffic of construction equipment will tend to create pumping and general deterioration of the shallower soils, especially if excess surface water is present. The grading, therefore, should be done during a dry season, if at all possible. Furthermore, it is important that positive surface drainage be established at the beginning of the earthwork operations and be maintained throughout the project. Surface water must not be allowed to pond. Compaction and sealing of the subgrade surface is important when precipitation is expected, and the site storm drainage elements (i.e., catch basins, pipes, manholes, etc.) should be installed as early as possible, which will aid in control of surface water and ground water.

6.2 Fill Compaction

All engineered fill beneath footings, floor slabs, and pavements should be compacted to a dry density of at least 98 percent of the standard Proctor maximum dry density (ASTM D698). The compaction should be accomplished by placing the fill in about 8-inch-thick (or less), loose lifts and mechanically compacting each lift to at least the specified minimum dry density. The moisture content of the fill materials should be within a range of approximately 2 percent below the optimum moisture content to 1 percent above the optimum moisture content. Field density tests should be performed on each lift as necessary to verify that adequate moisture conditioning and compaction are being achieved.

All soils encountered in the test borings made at this site are considered suitable as general fill material, with the exception of topsoil and any soils containing organics or marl that may be encountered. The need for some aeration or chemical modification of any higher-plasticity, clayey soils should be expected before they can be placed and compacted to the specified density. If any higher-plasticity, clayey soils are to be used within the upper 12 inches of the slab subgrade, the soils may require chemical modification prior to placement, as they may be prone to shrinking and swelling if not so treated.

Any off-site fill materials required for general site-filling purposes should consist of natural soil, sand and gravel, or crushed limestone with the following characteristics:

- Organic content less than 5 percent by dry weight of soil;
- Liquid Limit less than 50 and Plasticity Index less than 25 and greater than 7;
- Free of large rock fragments (particles larger than 3 inches in diameter), debris, rubble, wood, and any other deleterious materials;
- Amount retained on the ¾ inch sieve is less than 30 percent;
- Maximum dry density (ASTM D698) of at least 105 lbs/cu.ft;
- Not an essentially one-size material (e.g., “pea gravel”, etc.).

It is recommended that only well-graded granular material, such as “pit-run” sand and gravel, INDOT No. 53 crushed limestone, or lean concrete, should be used to fill undercut excavations beneath footings and other excavations of limited lateral dimensions where proper compaction of cohesive materials is difficult and compaction can only be accomplished with small vibratory equipment. Aggregates that are locally referred to as “commercial grade” No. 53 crushed stone should not be used as fill material, and only aggregates that meet the INDOT gradation requirements should be used.

6.3 Foundation Excavations

The soil at the base of each spread footing excavation should be carefully observed and evaluated by a geotechnical engineer, or a qualified geotechnical technician working under the direction of the geotechnical engineer. Any softer, looser, or wet soils; existing fill materials; and otherwise undesirable material, such as soils containing organics and/or marl, must be removed within the zone of influence of the foundations for the proposed public restroom building so that the foundations will bear on satisfactory material. Any softer, looser, or otherwise undesirable materials as described above that are encountered should be removed and replaced with engineered fill. At the time of such observation, it will be necessary to make hand auger borings or use a hand penetration device in the base of the foundation excavation to evaluate the soils below the base. The necessary depth of penetration shall be established by the geotechnical engineer or technician.

Where undercutting is required to remove unsuitable materials (i.e., miscellaneous or uncontrolled fill soils, soils containing marl and/or organic materials, and any softer or looser natural soils, etc.), the proposed footing elevation may be re-established by backfilling after all undesirable materials have been removed. The undercut excavation beneath each footing should extend to suitable bearing soils, and the dimensions of the excavation base should be determined by imaginary planes extending outward and downward on a 2 (vertical) to 1 (horizontal) slope from the base perimeter of the footing (see Figure 4 in the Appendix). The entire excavation should then be refilled with engineered fill, which should be limited to well-graded sand and gravel or crushed stone (e.g., INDOT coarse aggregate size No. 53 crushed stone) and compacted to the minimum dry density recommended in Section 6.2.

Alternatively, lean concrete may be used in lieu of engineered backfill to fill an undercut excavation beneath spread footings. In cases where lean concrete will be used to fill an undercut excavation (rather than enlarging the base of the undercut excavation as recommended above and placing compacted granular fill materials in 8-inch-thick lifts), the dimensions of the base of the undercut excavation can be made the same as the dimensions of the footings. Special care should be exercised to remove any sloughed, softer, or looser materials near the base of the excavation slopes. In addition, special care should be taken to "tie-in" the compacted fill, if used, with the excavation slopes with benches as necessary. This is to ensure that no pockets of loose or soft materials will be left in place along the excavation slopes below the foundation bearing level.

For the purpose of project planning and cost estimating, based on the results of the test borings drilled for this project, it should be expected that undercutting of unsuitable soils will be required at most, if not all, spread footing locations to depths of approximately 3.5 ft below the existing ground surface. Occasionally undercutting to greater depths may be necessary at isolated locations.

Soils exposed in the bases of all satisfactory foundation excavations should be protected against any detrimental change in condition, such as from disturbance due to foot traffic and construction activities and deterioration caused by ground water seepage, rain, and/or freezing; otherwise stable soils can easily be disturbed and deteriorate due to construction activities and excess moisture; thus, exposure should be limited. Surface run-off water should be drained away from the excavation and not allowed to pond. If possible, all footing concrete should be placed the same day the excavation is made. If this is not practical, the footing excavations should be adequately protected, such as by construction of a concrete “mud mat” at the base of the excavations, which may also aid in the proper placement of reinforcing steel.

6.4 Drilled Pier Installation Observations

It is essential that the geotechnical consultant observe the entire drilling operations during the drilled pier installation process to determine the depth at which the shale is encountered and to document that the minimum drilled pier lengths and embedment into the shale is attained. The inspection of the drilled piers can be performed without entering the pier excavations by observing the drilling operations and auger-cuttings throughout the entire length of the drilled pier excavation to verify the depth at which the shale is encountered and to verify that the pier extends to the proper depth into the shale based upon the pier capacities prescribed in Section 5.3.

It is important that the drilled pier excavations and subsurface conditions be monitored until the concrete is placed to verify that the otherwise competent soils are not adversely affected by improper construction methods. As noted, temporary steel casing will be needed to prevent caving of the soil above the tip of the drilled shaft. It is important that the concrete be placed and the casing removed in such a fashion as to prevent "necking" of the drilled pier or other anomalies within the concrete. If unsuitable conditions are encountered at the base of a pier excavation, the pier excavation should be extended to the bottom of such undesirable materials and reinspected.

The drilled pier excavations will require temporary casing if they are to be entered (which is not recommended). If a pier excavation is to be entered, all local, state and federal safety regulations regarding confined space entry should be followed. No open flame should be permitted on the site near the drilled pier excavation and no personnel should be allowed to enter the excavation until proper safety precautions for confined space entry have been taken. Such precautions should include proper personal protective equipment and monitoring of the excavations for explosive vapors and oxygen deficiency. Additional safety measures may be needed depending upon the specific conditions at the foundation locations, the construction procedures employed and the applicable local, state and federal Occupational Safety and Health Administration (OSHA) Regulations.

The need for some dewatering should be anticipated. While groundwater was not encountered at the time of our investigation, it is possible that ground water will be encountered in the drilled pier excavations.

6.5 Construction Dewatering

At the time of the field investigation, the ground water level appeared to generally be below the anticipated footing excavation depths. However, depending on the seasonal conditions, some seepage of ground water into excavations may be experienced due to “perched” water that may be encountered in sand seams or that may be encountered within old miscellaneous fill materials, abandoned utilities, utility trenches, etc. It is anticipated that such seepage can be handled by conventional dewatering methods such as by pumping from sumps. However, in cases where a saturated sand layer is encountered in the base of the excavation, it will not be possible to pump water directly from the base of the excavation without causing deterioration of the foundation soil. In this case, it will be necessary to pump from a sump located adjacent to the excavation or to depress the ground water using wells or well points. The best dewatering system for each case must be determined at the time of construction based upon actual field conditions.

7. LIMITATIONS OF STUDY

An inherent limitation of any geotechnical engineering study is that conclusions must be drawn on the basis of data collected at a limited number of discrete locations. The recommendations provided in this report were developed from the information obtained from the test borings that depict subsurface conditions only at these specific locations and at the specific times designated on the test boring logs. Soil, bedrock and ground water conditions at other locations may differ from those conditions occurring at the specific test boring locations, and ground water conditions will also vary through time. The nature and extent of variations between the test borings may not become evident until the course of construction. If variations then appear evident, it will be necessary to re-evaluate the recommendations of this report after performing on-site observations during the excavation and construction period and noting the characteristics of any variations.

Any comments or recommendations made herein regarding construction related issues or temporary conditions are solely for the purpose of evaluating feasibility and constructability and planning the design of the proposed facilities. The scope of this investigation is not sufficient to identify all potential construction related issues, variations, anomalies, etc. or all factors that may affect construction means, methods and costs.

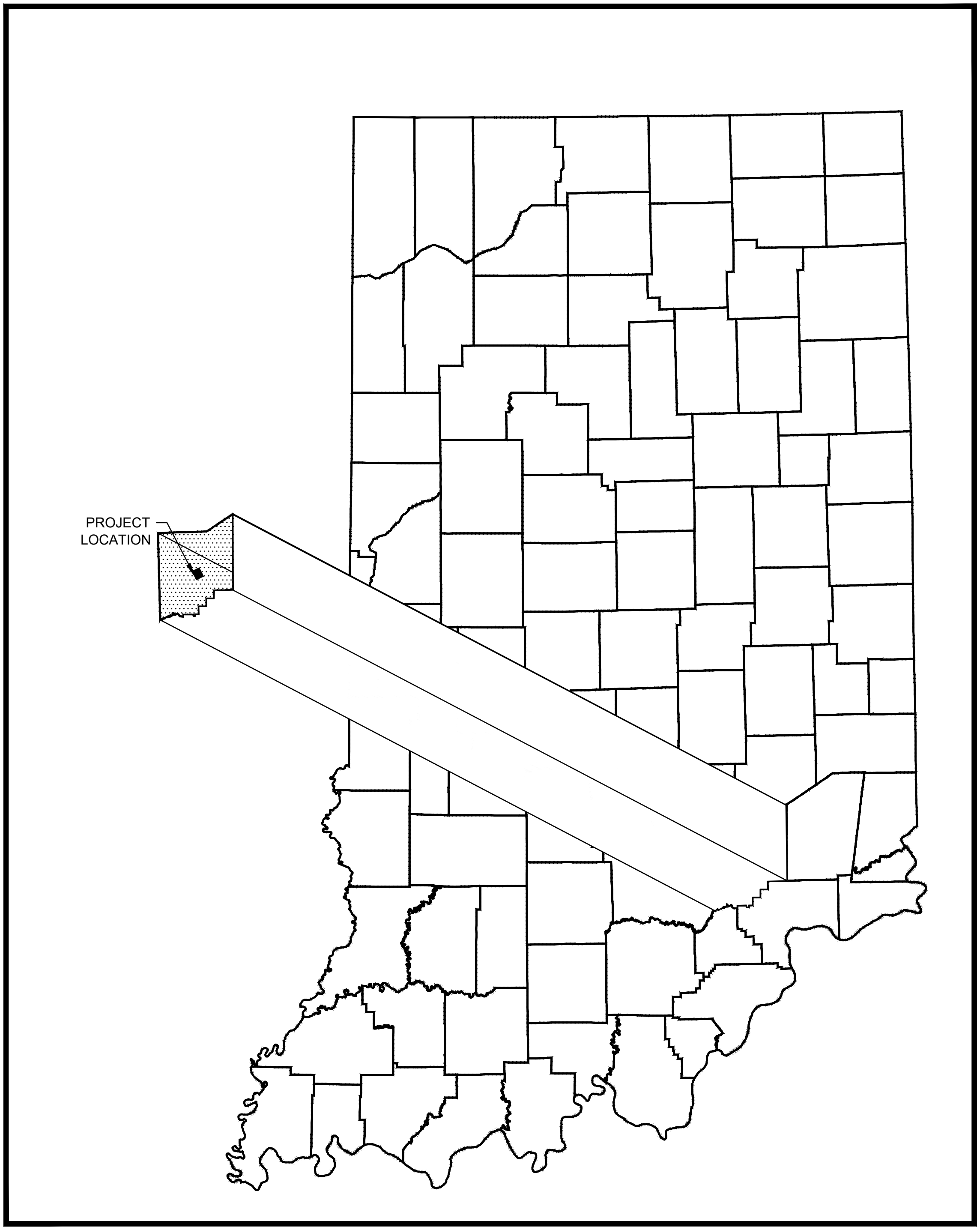
Our professional services have been performed, our findings obtained and our recommendations prepared in accordance with customary principles and practices in the field of geotechnical engineering at the time when the services were performed and at the location where the services were performed. This warranty is in lieu of all other warranties either express or implied. This company is not responsible for the independent conclusions, opinions or recommendations made by others based on the field exploration and laboratory test data presented in this report.

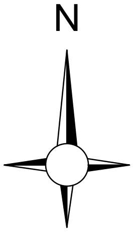
The scope of our services does not include any environmental assessment or investigation for the presence or absence of hazardous or toxic materials in the soil, ground water or surface water within or beyond the site studied. Any statements in this report or on the test boring logs regarding odors, staining of soils or other unusual conditions observed are strictly for the information of our client. Unless complete environmental information regarding the site is already available, an environmental assessment is recommended prior to the (purchase) (development) of this site.

These recommendations were developed from the information obtained from the test borings which depict subsurface conditions only at these specific locations and at the particular time designated on the logs. Soil conditions at other locations may differ from conditions occurring at these boring locations. It is emphasized that this is a preliminary study based on a relatively few widely spaced borings.

Atlas Technical Consultants LLC assumes no responsibility for any construction procedures, temporary excavations (including utility trenches), temporary dewatering or site safety during or after construction. Any recommendations, discussions or comments provided herein regarding temporary conditions during construction are solely for the use in planning and design of the project. Under no circumstances shall the information provided herein be interpreted to mean that Atlas Technical Consultants LLC is responsible for construction site safety or contractor means and methods, and no responsibility is implied or inferred. The contractor shall be solely responsible for all construction procedures, construction means and methods, construction sequencing and for all safety measures during construction as well as the protection of all existing facilities. All applicable federal, state and local laws and regulations regarding construction safety must be followed, including current Occupational Safety and Health Administration (OSHA) Regulations including OSHA 29 CFR Part 1926 “Safety and Health Regulations for Construction”, Subpart P “Excavations”, and/or successor regulations. The Contractor shall be solely responsible for designing and constructing stable, temporary excavations and should brace, shore, slope, or bench the sides of the excavations as necessary to maintain stability of the excavation sides and bottom and to protect the integrity of all existing facilities (i.e., existing foundations, floor slabs, structures, equipment, utilities, pavements, etc.).

APPENDIX I





PROJECT LOCATION MAP

VERNON HISTORIC TRAIL
CITY OF VERNON
JENNINGS COUNTY, INDIANA

Project Number: 170GC01588

Drn. By: MS

Date: 07/11/2023

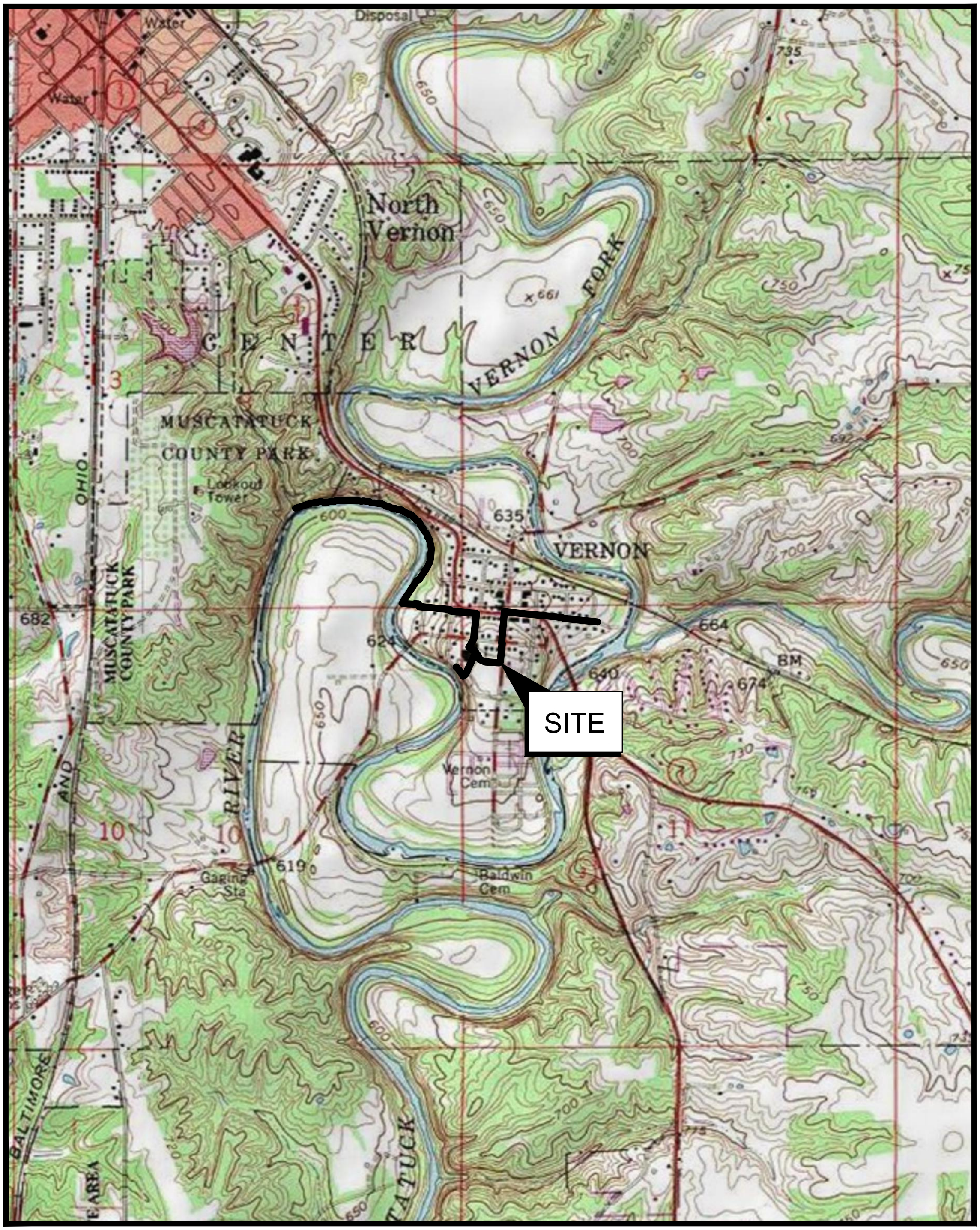
Scale: NOT TO SCALE

Ckd. By: LS

ATLAS

1

H:\20231 OTHER OFFICES\INDIANA\VERNON NLT TRAIL\170GC01588-BPLAN.DWG, FIG3



N

VICINITY MAP

VERNON HISTORIC TRAIL
CITY OF VERNON
JENNINGS COUNTY, INDIANA

Project Number:
170GC01588

Drn. By:
MS

Date:
07/11/2023

Scale:
1":2,000'

Ckd. By:
LS

ATLAS

2

H:\20231 OTHER OFFICES\INDIANA\VERNON NLT TRAIL\170GC01588-VIC.DWG, FIG1

BORING PLAN

VERNON HISTORIC TRAIL
CITY OF VERNON
JENNINGS COUNTY, INDIANA

Project Number: 170GC01588
Drn. By: MS
Date: 07/12/2023
Scale: AS SHOWN
Ckd. By: LS

ATLAS

3A

H:\202331 OTHER OFFICES\INDIANA\VERNON NLT TRAIL\170GC01588-BPLAN.DWG, FIG2A

BORING PLAN

VERNON HISTORIC TRAIL
CITY OF VERNON
JENNINGS COUNTY, INDIANA

Project Number: 170GC01588		Drn. By: MS
Date: 07/11/2023	Scale: AS SHOWN	Ckd. By: LS
ATLAS		3B

H:\20231 OTHER OFFICES\INDIANA\VERNON NLT TRAIL\170GC01588-BPLAN.DWG, FIG2B

BORING PLAN

VERNON HISTORIC TRAIL
CITY OF VERNON
JENNINGS COUNTY, INDIANA

Project Number:
170GC01588

Drn. By:
MS

Date:
07/11/2023

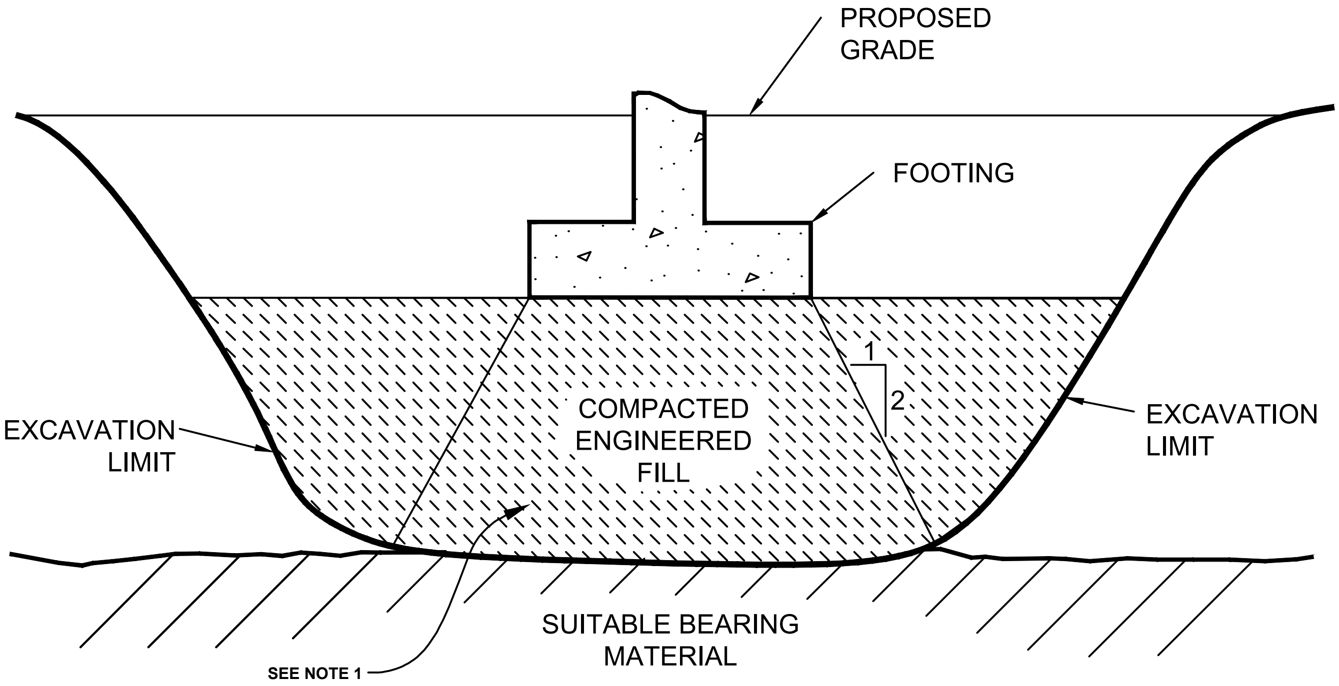
Scale:
AS SHOWN

Ckd. By:
LS

ATLAS

3C

H:\2023\1 OTHER OFFICES\INDIANA\VERNON NLT TRAIL\170GC01588-BPLAN.DWG, FIG2C



NOTE 1: Expanded (2V:1H) undercut zone not necessary where concrete/lean concrete is used as undercut backfill in lieu of engineered fill and where adequate bearing soils are exposed at the base of undercut. Refer to report Section 6.3.

DESIGN ILLUSTRATION FOOTINGS WITH UNDERCUTS

VERNON HISTORIC TRAIL
CITY OF VERNON
JENNINGS COUNTY, INDIANA

Project Number:
170GC01588

Drn. By:
MS

Date:
07/14/2023

Scale:
NOT TO SCALE

Ckd. By:
BH

4

APPENDIX II
CLASSIFICATION SYSTEM FOR SOIL EXPLORATION
TEST BORING LOGS

CLASSIFICATION SYSTEM FOR SOIL EXPLORATION

Particle Size Identification

(Based on INDOT Standard Specifications Section 903)

Boulders - 3 in. (75 mm) diameter or more
Gravel - 2.0 mm (No. 10 Sieve) to 3 in.
Sand (Coarse) - 0.425 mm to 2.0 mm
(No. 40 Sieve to No. 10 Sieve)
Sand (Fine) - 0.075 mm to 0.425 mm
(No. 200 Sieve to No. 40 sieve)
Silt - 0.002 mm to 0.075 mm (No. 200 Sieve)
Clay - Smaller than 0.002 mm

NON-COHESIVE SOILS

(Silt, Sand, Gravel and Combinations)

Density

Very Loose - 5 blows/ft or less
Loose - 6 to 10 blows/ft
Medium Dense-11 to 30 blows/ft
Dense - 31 to 50 blows/ft
Very Dense - 51 blows/ft or more

COHESIVE SOILS

(Clay, Silt and Combinations)

Consistency

Very Soft - 3 blows/ft or less
Soft - 4 to 5 blows/ft
Medium Stiff - 6 to 10 blows/ft
Stiff - 11 to 15 blows/ft
Very Stiff - 16 to 30 blows/ft
Hard - 31 blows/ft or more

Plasticity

Degree of Plasticity	Plasticity Index
None to slight	0 - 4
Slight	5 - 7
Medium	8 - 22
High to Very High	over 22

Classifications shown on the test boring logs are made by visual inspection of samples and confirmed / modified based on index property tests.

Standard Penetration Test (AASHTO T 206) — Driving a 2.0" O.D., 1-3/8" I.D. sampler a distance of 1.0 foot into undisturbed soil with a 140 pound hammer free falling a distance of 30 inches. It is customary for ATC to drive the sampler 6 inches to seat the sampler into undisturbed soil, then perform the test. The number of hammer blows for seating the sampler and making the test are recorded for each 6 inches of penetration on the drill log (Example — 6-8-9). The standard penetration test result can be obtained by adding the last two figures (i.e., 8 + 9 = 17 blows/ft).

Strata Changes — In the column "Soil Classification" on the test boring logs, the horizontal lines represent strata changes. A solid line (______) represents an actually observed change. A dashed line (_ _ _ _ _ _) represents an estimated change.

Ground Water observations were made at the times indicated. Porosity of soil strata, weather conditions, site topography, etc., may cause changes in the water levels indicated on the logs.

ATLAS

BORING LOG

BORING NO.: **PR-B-201**
SHEET 1 OF 1
LATITUDE : 38.98459
LONGITUDE : -85.60545
DATE STARTED : 06-13-23
DATE COMPLETED : 06-13-23

GEOTECHNICAL CONSULTANT : Atlas Technical Consultants
DES NO. : --
STRUCTURE #: --
PROJECT TYPE: Multi Use Trail
LOCATION : Vernon Muscatatuck Trail; Town of Vernon, Indiana
COUNTY : Jennings
ATLAS PROJECT NO.: 170GC01588

ELEVATION : 630.5
STATION : 73+33
OFFSET : 75.0 ft Right
LINE : 'Trail'
DEPTH : 13.6 ft

BORING METHOD : HSA
RIG TYPE : D50 ATV (SN363)
CASING DIA. : --
CORE SIZE : --

HAMMER : Auto
DRILLER/INSP : M. Ladish/L. Shelton
TEMPERATURE : 75 °F
WEATHER : Partly Sunny

GROUNDWATER: ∇ Encountered at None ▼ At completion None ▼ -- After -- hours ▨ Caved in at 12.0 ft

ELEVATION	SAMPLE DEPTH	SOIL/MATERIAL DESCRIPTION	SAMPLE NUMBER	SPT per 6"	% RECOVERY	MOISTURE CONTENT	DRY DENSITY, pcf	POCKET PEN., tsf	UNCONF. COMP., tsf	ATTERBERG LIMITS LL	PL	PI	REMARKS
630.0		**Topsoil** 6 inches 0.5											0.0, Ground surface elevation estimated from survey provided by FPBH
			SS1	6-5-5	67	8.6		4.5+		24	14	10	
	2.5												
			SS2	3-3-4	33	11.6		4.0					
	5.0												
625.0													
		Sandy Loam A-2-4 (0), Reddish brown, slightly moist, medium stiff, (Lab No. 1)	SS3	4-4-5	67	13.7		0.25					
	7.5												
		-sand seam at 9ft	SS4	3-4-4	67								
	10.0												
620.0													
			SS5	3-3-4	33	14.0		<0.25					
	12.5												
		13.0											
		Silty Loam Brown, moist, hard, (visual) 13.6	SS6	50/0.1	83	38.1		0.25					13.6, Auger refusal at 13.6 ft
		Bottom of Boring at 13.6 ft											
	15.0												
615.0													
	17.5												
	20.0												
610.0													
	22.5												
	25.0												

ATLAS

BORING LOG

BORING NO.: **PR-B-202**
SHEET 1 OF 1
LATITUDE : 38.98443
LONGITUDE : -85.60539
DATE STARTED : 06-13-23
DATE COMPLETED : 06-13-23

GEOTECHNICAL CONSULTANT : Atlas Technical Consultants
DES NO. : --
STRUCTURE #: --
PROJECT TYPE: Multi Use Trail
LOCATION : Vernon Muscatatuck Trail; Town of Vernon, Indiana
COUNTY : Jennings
ATLAS PROJECT NO.: 170GC01588

ELEVATION : 623.5
STATION :
OFFSET :
LINE : 'Trail'
DEPTH : 8.6 ft

BORING METHOD : HSA
RIG TYPE : D50 ATV (SN363)
CASING DIA. : --
CORE SIZE : --

HAMMER : Auto
DRILLER/INSP : M. Ladish/L. Shelton
TEMPERATURE : 75 °F
WEATHER : Partly Sunny

GROUNDWATER: ∇ Encountered at None ▼ At completion None ▼ -- After -- hours ▨ Caved in at 7.7 ft

ELEVATION	SAMPLE DEPTH	SOIL/MATERIAL DESCRIPTION	SAMPLE NUMBER	SPT per 6"	% RECOVERY	MOISTURE CONTENT	DRY DENSITY, pcf	POCKET PEN., tsf	UNCONF. COMP., tsf	ATTERBERG LIMITS LL	PL	PI	REMARKS
	0.5	**Topsoil** 6 inches											0.0, Ground surface elevation estimated from survey provided by FPBH
	5.5	**Sandy Loam A-2-4**, Reddish brown, moist, medium stiff to soft, (Lab No. 1)	SS1	3-3-4	67	8.1		1.25					
			SS2	3-1-4	67	12.6		<0.25					
	8.6	**Interbedded Limestone and Shale** Tan, highly weathered, (visual)	SS3	10-12-15	67								
			SS4	50/0.1	83								8.6, Auger refusal at 8.6 ft
		Bottom of Boring at 8.6 ft											

BORING LOG

BORING NO.: **RB-103**
SHEET 1 OF 1
GEOTECHNICAL CONSULTANT : Atlas Technical Consultants
LATITUDE : 38.98340
DES NO. : --
STRUCTURE #: --
LONGITUDE : -85.60951
PROJECT TYPE: Multi Use Trail
DATE STARTED : 06-13-23
LOCATION : Vernon Muscatatuck Trail; Town of Vernon, Indiana
COUNTY : Jennings
ATLAS PROJECT NO.: 170GC01588
DATE COMPLETED : 06-13-23

ELEVATION : 692.5
STATION : 57+20
OFFSET : 23.0 ft Right
LINE : 'Trail'
DEPTH : 7.5 ft

BORING METHOD : HSA
RIG TYPE : D50 ATV (SN363)
CASING DIA. : --
CORE SIZE : --

HAMMER : Auto
DRILLER/INSP : M. Ladish/L. Shelton
TEMPERATURE : 75 °F
WEATHER : Partly Sunny

GROUNDWATER: ▽ Encountered at None ▼ At completion None ▼ -- After -- hours ▨ Caved in at 4.9 ft

ELEVATION	SAMPLE DEPTH	SOIL/MATERIAL DESCRIPTION	SAMPLE NUMBER	SPT per 6"	% RECOVERY	MOISTURE CONTENT	DRY DENSITY, pcf	POCKET PEN., tsf	UNCONF. COMP., tsf	ATTERBERG LIMITS LL	PL	PI	REMARKS
		Asphalt 5 inches 0.4											0.0, Ground surface elevation estimated from survey provided by FPBH
		Sand and Gravel 4 inches, Subbase 0.8											
690.0	2.5	**Clay A-7-6**, Reddish brown and gray, moist, stiff, (Lab No. 2) 3.0	SS1	2-4-8	56	21.0		3.75					
▨	5.0	**Silty Clay Loam A-7-6**, with shale fragments and thin sand seams, Reddish brown, moist, very stiff, (visual) 5.5	SS2	8-7-9	89	19.4		1.25		41	21	20	
685.0	7.5	**Shale** Dark brown, highly weathered, (visual) 7.5	SS3	16-26-29	56								7.5, Auger Refusal at 7.5 ft
		Bottom of Boring at 7.5 ft											
	10.0												
680.0	12.5												
	15.0												
675.0	17.5												
	20.0												
670.0	22.5												
	25.0												

ATLAS

BORING LOG

BORING NO.: **RO-B-1A**
SHEET 1 OF 1
LATITUDE : 38.98288
LONGITUDE : -85.61150
DATE STARTED : 06-13-23
DATE COMPLETED : 06-13-23

GEOTECHNICAL CONSULTANT : Atlas Technical Consultants
DES NO. : --
STRUCTURE #: --
PROJECT TYPE: Multi Use Trail
LOCATION : Vernon Muscatatuck Trail; Town of Vernon, Indiana
COUNTY : Jennings
ATLAS PROJECT NO.: 170GC01588

ELEVATION : 685.0
STATION : 4+57
OFFSET : 0.0 ft
LINE : 'Overlook'
DEPTH : 15.0 ft

BORING METHOD : HSA
RIG TYPE : D50 ATV (SN363)
CASING DIA. : --
CORE SIZE : --

HAMMER : Auto
DRILLER/INSP : M. Ladish/L. Shelton
TEMPERATURE : 75 °F
WEATHER : Partly Sunny

GROUNDWATER: ▽ Encountered at None ▼ At completion None ▼ -- After -- hours ▨ Caved in at 8.6 ft

ELEVATION	SAMPLE DEPTH	SOIL/MATERIAL DESCRIPTION	SAMPLE NUMBER	SPT per 6"	% RECOVERY	MOISTURE CONTENT	DRY DENSITY, pcf	POCKET PEN., tsf	UNCONF. COMP., tsf	ATTERBERG LIMITS LL	ATTERBERG LIMITS PL	ATTERBERG LIMITS PI	REMARKS
		Asphalt 4 inches 0.3											0.0, Ground surface elevation estimated from survey provided by FPBH
		Clay A-7-6, Brown and tan, moist, stiff, with highly weathered limestone fragments, (Lab No. 2)	SS1	1-5-8	56	15.2		3.25					
	2.5	3.0											
		Clay A-7-6, Reddish brown and gray, moist, medium stiff to stiff, (Lab No. 2)	SS2	3-4-7	67	20.1		3.0					
680.0	5.0												
			SS3	3-4-6	56	20.9		3.25					
	7.5												
▨			SS4	5-5-9	67	20.8		1.5					
675.0	10.0	10.5											
		Shale Tan and gray, completely weathered, (visual)	SS5	6-14-8	67	22.9		3.25					
	12.5	13.0											
		Sandy Clay Loam A-6, with shale fragments, Brown and tan, moist, stiff, (visual)	SS6	4-6-7	78	19.8		0.5		39	17	22	
670.0	15.0	15.0 Bottom of Boring at 15.0 ft											
	17.5												
665.0	20.0												
	22.5												
660.0	25.0												

BORING LOG

BORING NO.: **RO-B-2A**
SHEET 1 OF 1
LATITUDE : 38.98274
LONGITUDE : -85.61134
DATE STARTED : 06-13-23
DATE COMPLETED : 06-13-23

GEOTECHNICAL CONSULTANT : Atlas Technical Consultants
DES NO. : --
STRUCTURE #: --
PROJECT TYPE: Multi Use Trail
LOCATION : Vernon Muscatatuck Trail; Town of Vernon, Indiana
COUNTY : Jennings
ATLAS PROJECT NO.: 170GC01588

ELEVATION : 684.0
STATION : 3+65
OFFSET : 0.0 ft
LINE : 'Overlook'
DEPTH : 15.0 ft

BORING METHOD : HSA
RIG TYPE : D50 ATV (SN363)
CASING DIA. : --
CORE SIZE : --

HAMMER : Auto
DRILLER/INSP : M. Ladish/L. Shelton
TEMPERATURE : 75 °F
WEATHER : Partly Sunny

GROUNDWATER: ▽ Encountered at None ▼ At completion None ▼ -- After -- hours ▨ Caved in at 8.4 ft

SOIL/MATERIAL DESCRIPTION	Depth	SAMPLE NUMBER	SPT per 6"	% RECOVERY	MOISTURE CONTENT	DRY DENSITY, pcf	POCKET PEN., tsf	UNCONF. COMP., tsf	ATTERBERG LIMITS LL	PL	PI	REMARKS
Asphalt 5 inches	0.4											0.0, Ground surface elevation estimated from survey provided by FPBH
Clay A-7-6, Tan and gray, moist, medium stiff to stiff, (Lab No. 2)		SS1	3-3-5	44	16.7		2.0		66	18	48	
		SS2	3-5-7	56	18.7		2.25					
		SS3	5-4-5	67	20.7		2.0					
	10.5	SS4	6-7-8	22	18.2		2.0					
Clay A-6, with shale fragments, Tan, moist, (visual)	12.5	SS5	5-6-6	78	17.3		3.0		37	17	20	
Shale Tan, completely weathered, (visual) - thin sand seams near 14 feet	15.0	SS6	2-3-5	67	18.1		2.0					
Bottom of Boring at 15.0 ft												

APPENDIX III

SUMMARY OF CLASSIFICATION TEST RESULTS
GRAIN SIZE DISTRIBUTION REPORTS
ATTERBERG LIMITS RESULTS
SUMMARY OF SPECIAL LABORATORY TEST RESULTS

Sheet 1 of 1

Boring	Sample	Depth	Lab #	Soil Classification	Gravel %	Sand %	Silt %	Clay %	% Fines *(Passing No. 200)*	LL	PL	PI	Moisture %	LOI %	Ca/Mg %	Soluble Sulfate (ppm)	pH
PR-B-201	SS1	1	1	A-2-4 (0) SANDY LOAM	1.1	68.5	20.0	10.5	30.4	24	14	10	8.6				
RO-B-2A	SS1	1	2	A-7-6 (30) CLAY	8.2	25.9	35.5	30.5	66.0	66	18	48	16.7				

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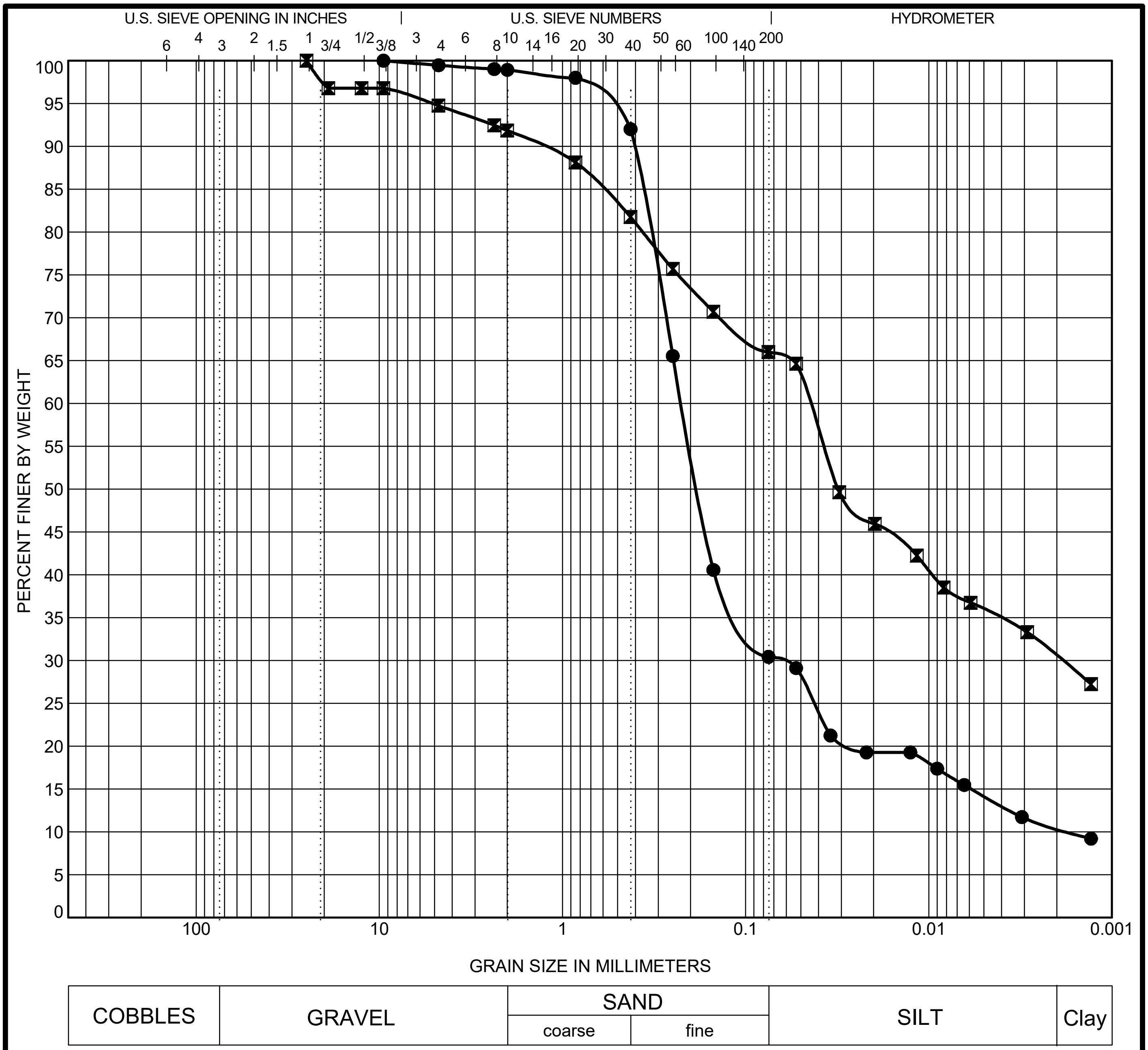
Summary of Classification Tests

DES # : --
County : Jennings
Route # : Vernon Muscatatuck Trail
Project # : 170GC01588
Project Type : Multi Use Trail
Location : Town of Vernon, Indiana



	Specimen Identification			Lab #	Textural Classification	LL	PL	PI	Cc	Cu
●	**PR-B-201**	**SS1**	**1.0**	**1**	**A-2-4 (0) SANDY LOAM**	**24**	**14**	**10**	**11.80**	**130.82**
☒	**RO-B-2A**	**SS1**	**1.0**	**2**	**A-7-6 (30) CLAY**	**66**	**18**	**48**		

	Specimen Identification			D60	D30	D10	LOI	Ca/Mg	%Gravel	%Sand	%Silt	%Clay	%Colloid
●	**PR-B-201**	**SS1**	**1.0**	**0.223**	**0.067**	**0.002**			**1.1**	**68.5**	**20.0**	**10.5**	
☒	**RO-B-2A**	**SS1**	**1.0**	**0.045**	**0.002**				**8.2**	**25.9**	**35.5**	**30.5**	

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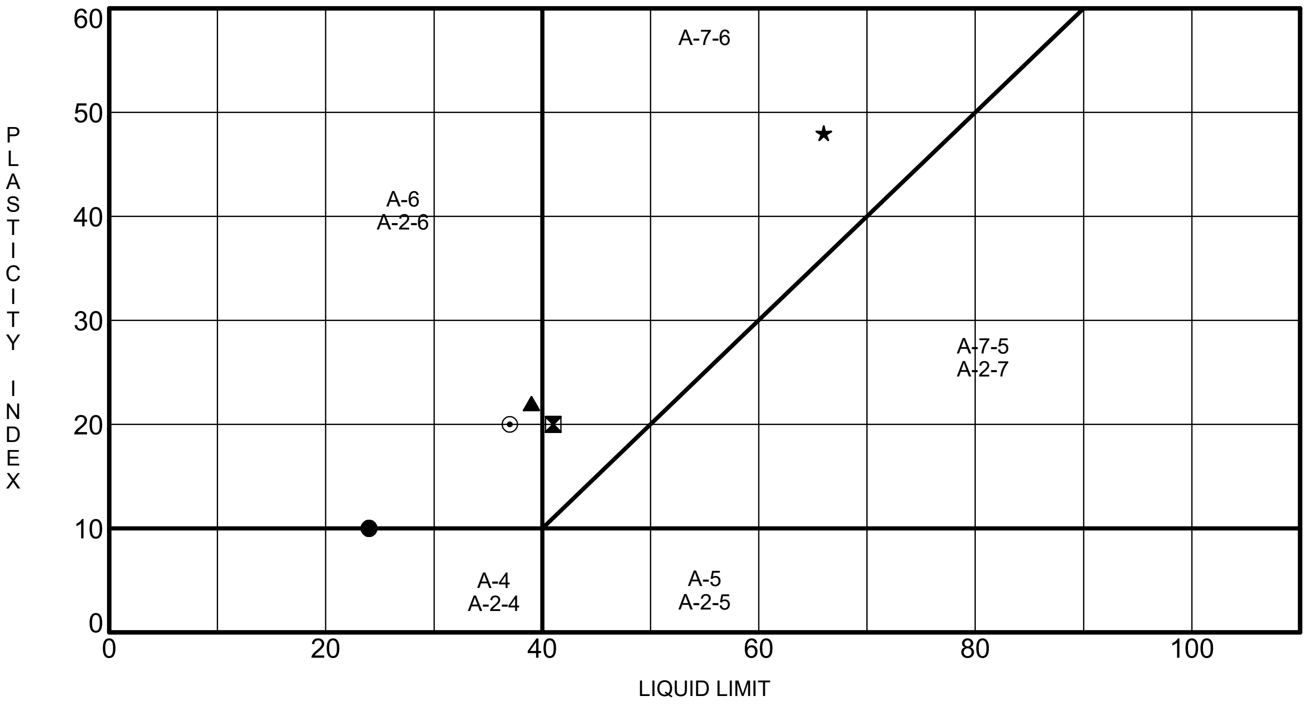
GRAIN SIZE DISTRIBUTION TEST REPORT

DES #: -- Structure #: --
Project #: 170GC01588
County: Jennings Route: Vernon Muscatatuck Trail
Location: Town of Vernon, Indiana



Specimen Identification			LL	PL	PI	Fines	Classification
●	**PR-B-201**	**1.00**	**24**	**14**	**10**	**30**	**A-2-4 (0) SANDY LOAM**
⧗	**RB-103**	**3.50**	**41**	**21**	**20**		**A-7-6 CLAY**
▲	**RO-B-1A**	**13.50**	**39**	**17**	**22**		**A-6 SANDY CLAY LOAM**
★	**RO-B-2A**	**1.00**	**66**	**18**	**48**	**66**	**A-7-6 (30) CLAY**
⊙	**RO-B-2A**	**11.00**	**37**	**17**	**20**		**A-6 CLAY**

ATLAS

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ATTERBERG LIMITS RESULTS

DES #: -- Structure #: --
Project #: 170GC01588
County: Jennings Route: Vernon Muscatatuck Trail
Location: Town of Vernon, Indiana

Sheet 1 of 1

Boring	Sample	Depth	Specific Gravity	Dry Density (pcf)	Qu (tsf)	c (tsf)	ϕ (deg)	Moisture %	Max Dry Density (pcf)	Opt. Moisture %	Resilient Modulus Remolded (psi)	Resilient Modulus In Situ Condition (psi)	Void Ratio	pH	Sulfate (ppm)	LOI (%)	Ca/Mg CO_3 (%)
PR-B-201	SS1	1 - 2.5						8.6									
PR-B-201	SS2	3.5 - 5						11.6									
PR-B-201	SS3	6 - 7.5						13.7									
PR-B-201	SS5	11 - 12.5						14.0									
PR-B-201	SS6	13.5 - 13.6						38.1									
PR-B-202	SS1	1 - 2.5						8.1									
PR-B-202	SS2	3.5 - 5						12.6									
RB-103	SS1	1 - 2.5						21.0									
RB-103	SS2	3.5 - 5						19.4									
RO-B-1A	SS1	1 - 2.5						15.2									
RO-B-1A	SS2	3.5 - 5						20.1									
RO-B-1A	SS3	6 - 7.5						20.9									
RO-B-1A	SS4	8.5 - 10						20.8									
RO-B-1A	SS5	11 - 12.5						22.9									
RO-B-1A	SS6	13.5 - 15						19.8									
RO-B-2A	SS1	1 - 2.5						16.7									
RO-B-2A	SS2	3.5 - 5						18.7									
RO-B-2A	SS3	6 - 7.5						20.7									
RO-B-2A	SS4	8.5 - 10						18.2									
RO-B-2A	SS5	11 - 12.5						17.3									
RO-B-2A	SS6	13.5 - 15						18.1									

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Summary of Special Lab Tests

DES # : --
County : Jennings
Route # : Vernon Muscatatuck Trail
Project # : 170GC01588
Project Type : Multi Use Trail
Location : Town of Vernon, Indiana

ATLAS SUMMARY SPECIAL TEST SULFATE GC01588.GPJ ATC INDOT GEOTECH.GDT 7/14/23