D.W. KOZERA, INC.
PROFESSIONAL ENGINEERS & GEOLOGISTS

Geotechnical Engineering Study
Community Living Center
Hollidaysburg Veterans' Home
Hollidaysburg, Pennsylvania

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December 20, 2021, rev. 12/21/21

Hord Coplan Macht
1213 West Morehead St., Suite 507
Charlotte, NC 28208

Attn: Mr. Kurt Thompson, AIA
(kthompson@HCM2.com)

Subject: Geotechnical Engineering Study, Community Living Center, Hollidaysburg Veterans' Home, Duncansville, Pennsylvania (DWK Contract Number 20179.D)

Dear Mr. Thompson:

D.W. Kozera, Inc. is pleased to submit this report containing the results of the subsurface investigation and geotechnical engineering study of the proposed Community Living Center at the Hollidaysburg Veterans' Home. The scope of services referenced in this report was performed in accordance with our contract dated June 3, 2021.

We appreciate the opportunity to be of service to you. Please contact us if you have any questions related to this subsurface investigation report.

Very truly yours,
D.W. KOZERA, INC.

SHANA M. CARROLL
PE082509

Shana Carroll, PE
Commonwealth of Pennsylvania No. 82509
Expiration: 09/30/23

I hereby certify that this document was prepared or approved by me, and that I am a duly licensed Professional Engineer under the laws of the Commonwealth of Pennsylvania, License No. 82509, and Expiration Date: 09/30/23

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Geotechnical Engineering Study
Hollidaysburg Veterans' Home
Hollidaysburg, Pennsylvania
(DWK Contract Number 20179.D

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Existing Conditions

- The footprint of the proposed building lies in an area covered by grass, asphalt parking, and roads. The western part of site is relatively flat and ranges in elevation from approximately EL +1034± to EL +1031± while the eastern part of the site is sloping from west to east from approximately EL +1033± to EL +1012±.

Proposed Conditions

- The Eastern portion of the new building will have three supported levels above the ground level which is at EL +1034, and the Western portion of the new building will have two supported levels above the ground level at EL +1018.
- Fills of 0- to 6-ft. and cuts of up to 15-ft. are expected to achieve the proposed finished floor elevations

Soils Encountered

- The site is underlain by up to 10-ft. of Man-placed Fill underlain by natural Residual Soils, Disintegrated Rock, and Bedrock.

Groundwater

- Two observation wells were installed at this site. Over a period of two months, groundwater was observed in these wells at approximately EL +993 to +996 (i.e., 19- to 31-ft. below ground surface).

Risks Related to Karst Construction

- This site is underlain by limestone bedrock, which is susceptible to sinkhole formation.
- No surface depressions were observed during this investigation and the area is not mapped with active karst features. This indicates that active sinkholes related to the karst geology are not currently present at the site. However, the underlying bedrock is susceptible to sinkhole formation and measures should be taken during construction to minimize the potential for water to infiltrate and solution the bedrock, which could result in a sinkhole. Among other considerations, proper management of drainage and runoff throughout construction and the life of the building is very important on this site as the formation of sinkholes is known to be caused by water infiltration and sinkhole formation can lead to loss of support for the footings and/or slab-on-grade.

Foundations & Slab-on-Grade

- Spread footings can be supported on Disintegrated Rock (DI) or Bedrock (Stratum C or D) with an allowable design bearing pressure of 5,000 psf, assuming an allowable total settlement of 0.5-inches and differential settlement of 0.25- inches between columns.
- Where footings are not directly supported on Disintegrated Rock or Bedrock, they should be supported on Cement-treated Aggregate (CTA) Piers or Rigid Inclusions (RIs). Spread footings supported on CTA piers or RIs can be designed using a bearing pressure of 5,000 psf. The minimum length of a CTA Pier or RI should be 4-ft. In areas where DI or Bedrock are within 4-ft of the bottom of footing, excavation should be made to the bedrock and footings should be lowered to bear on bedrock.

- Once the CTA Pier or RI design is complete, additional investigation using air tracking is required for each RI location and any mudseams and/or voids found must be grouted prior to the installation of CTA Piers or RIs.

- The Specialty Contractor installing CTA Piers or RIs should anticipate difficult drilling through the Man-placed Fill present on-site.

- The slab-on-grade can be designed using a modulus of subgrade reaction (k) of 125 pci, provided the recommendations in this report are followed.

Earthwork

- Conventional earthmoving equipment is expected to be feasible for the cut and fill operations.

- The on-site soils are not expected to be suitable for reuse as compacted structural fill under structures or behind retaining walls. On-site soils may be used in non-structural site areas such as for landscaping.

- The on-site Lean Clay (CL) soils are prone to expanding upon wetting and care should be taken when exposing these soils during construction. These soils will become difficult to work with when they come into contact with moisture.

- Proofrolling should be performed prior to all fill placement. If the subgrade is found to be unstable, the soft soils should be removed and replaced with compacted structural fill or 21A.

- Where Man-placed Fill is present at slab subgrade elevation, 2-ft. of undercut and replacement should be performed prior to slab construction. This is expected to be the majority of the western portion of the building,

Support of Excavation

- Sloped excavation is expected to be feasible for the majority of the site excavations.

General Recommendations

The subsurface conditions below the site will vary across the site. The performance of the recommended foundation systems is dependent upon careful observations of these subgrades during construction. As the Geotechnical Engineer of Record, D. W. Kozera, Inc. is best suited to evaluate the foundation subgrades during construction, so that modifications in the design may be made as variations in the foundation conditions are encountered.

1.0 INTRODUCTION

This report contains the results of our geotechnical findings and recommendations for the proposed Hollidaysburg Veterans' Home located in Hollidaysburg, Pennsylvania. The report is based on the evaluation of test borings performed on the project site by our firm and available geologic data.

1.1 Scope

This report contains the results of our geotechnical investigation and analysis for the Hollidaysburg Veterans' Home located at 500 Municipal Drive in Hollidaysburg, Pennsylvania. The report is based on the evaluation of seventeen test borings performed on the project site and available geologic data. This study was conducted to characterize the subsurface conditions, and to establish engineering properties of the underlying materials in order to prepare recommendations for stormwater management, pavements, foundations, lower level retaining walls, earth work, and issues related to the construction of foundation and site work.

In accordance with our contract dated June 3, 2021, 9 test borings in the footprint of the proposed Community Living Center to depths of 25- to 35-ft. each and 8 test borings for Stormwater Management (SWM) each 10-ft. deep were performed. The subsurface investigation included:

a) Review of our test procedures, results of all testing conducted and available geotechnical and geological data from our previous studies.

b) Description of site geologic and groundwater conditions.

c) Presentation of subsurface soil stratigraphy with pertinent available physical properties.

d) Recommended geotechnical design parameters including soil strength, density, and compressibility, as applicable.

e) Recommendations for a shallow foundation system including allowable soil bearing pressure, anticipated settlements and embedment depth for frost.

f) Soil improvement techniques necessary for the support of foundations, floor slabs, and pavements. These may include lime/cement stabilization, rammed aggregate piers, geogrid reinforcement, and/or rigid inclusions.

g) Slab-on-grade design recommendations for the facility including the modulus of subgrade reaction, k, in pounds per square inch to be used to design the concrete slab-on-grade.

h) Recommendations for foundation drains and considerations and dewatering procedures, if applicable.

i) Lateral earth pressure diagrams for proposed basement retaining walls designed with restricted and unrestricted rotation at the top of the wall.

j) Determination as to whether on-site material will be suitable for use in control fills, and the extent to which acceptable on-site materials will be available and if off-site borrows will be required.

k) Site specific seismic classification per IBC 2015.

l) Stormwater management recommendations in accordance with the Pennsylvania Stormwater Best Management Practices Manual, 2006.

m) Recommendations on monitoring construction procedures including construction control measures, as well as recommended installation, monitoring of validation tests or instrumentation.

1.2 Existing Site Conditions

The site is located at 500 Municipal Drive in Duncansville, Pennsylvania. A site Vicinity Map is included as Figure 1.2-1. The site is bounded by a drive aisle to the North, existing parking lots to the West and South, and a grass covered slope to the East.

The footprint of the proposed building lies in an area currently covered by grass, asphalt parking, and roads. The western part of the site is relatively flat and ranges in elevation from approximately EL +1034± to EL +1031± while the eastern part of the site is sloping from west to east from approximately EL +1033± to EL +1012±.

According to historic aerial photographs, which are provided as Appendix A, a building was present on the western portion of the site as recent as 2017. Pennsylvania Department of General Services Public Works records show that demolition of the previous structures took place from June 2018 to January 2019. We understand that some of the basement and foundations of the previous building were removed. However, foundation walls, footings, and slabs should be expected to be encountered during earthwork.

The approximate location of the previous building and the proposed building is provided as Figure 1.2-2. Based on this investigation and the previous work on-site, the Contractor should expect to encounter Man-placed Fill including large debris in the area of the previous building.

1.3 Regional Geology and Karst Features

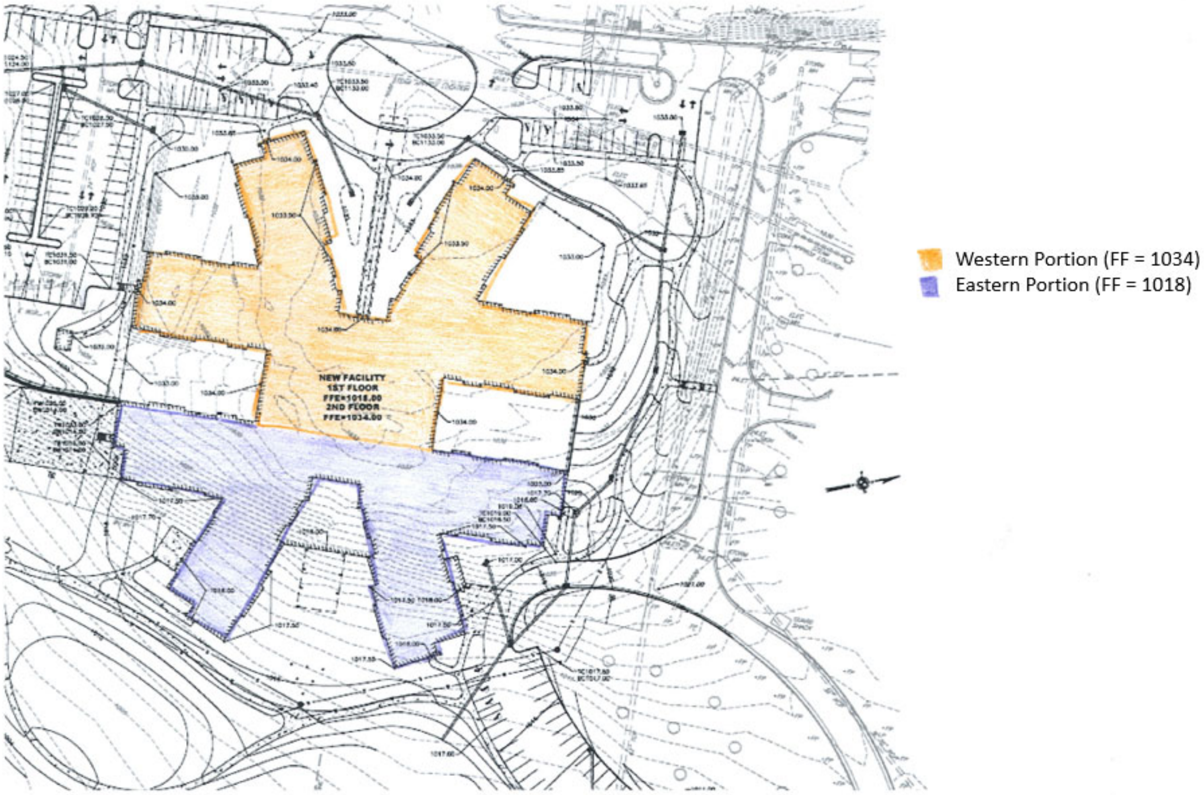
The site lies in the Appalachian Mountain Section of the Ridge and Valley Physiographic Province. According to the Geologic Map of the Juniata River Basin, Pennsylvania, prepared by Taylor, L.E., Werkheiser, W.H., DuPont, N.S., and Kriz, M.L., dated 1982, (Figure 1.3-1) the site geology is mapped as the Keyser and Tonoloway Formations. The United States Geological survey describes the Keyser Formation as consisting of medium-gray, crystalline to nodular, fossiliferous limestone, and the Tonoloway Formation consisting of medium-gray, laminated, mud-cracked limestone with some medium-dark- or olive-gray shale interbeds.

Based on a review of available data, the site does not appear to be in an area where active karst features are present (Figure 1.3-2). However, the presence of soluble rock that may dissolve with water flow present a risk of sinkhole formation on the site.

Karst describes a condition where water flowing through soluble rock can cause dissolutioning and the creation of mud-seams and voids in the rock. Limestone is known to be soluble and a sinkhole formation. While not observed in the area, sinkholes may occur on this site due to the underlying karst-prone limestone.

1.4 Proposed Construction

Based on the information provided to us, it is our understanding that a new Community Living Center is planned for the site with associated Stormwater Management (SWM) Features. The Eastern portion of the new building will have three supported levels above the ground level which is planned at EL +1034, and the Western portion of the new building will have two supported levels above the ground level which is planned at EL +1018. Fills of 0- to 6-ft. and cuts of up to 15-ft. are expected to achieve the proposed finished floor elevations. A plan showing the finished floor elevations in the different portions of the building is provided as Figure 1.4-1 for reference.



Base Plan: Grading Plan, Hollidaysburg Veterans' Home, Sheet C-5, prepared by Keller Engineers, Inc., dated 9/20/21.

Figure 1.4-1: Proposed Finished Floor Elevations

According to the structural engineer, maximum unfactored column loads are expected to be 250 kips and maximum wall loads are expected to be 5 kips per foot, or less. It is expected that the new structure will be able to tolerate 1-in. of total settlement and ½-in. differential settlement between adjacent columns.

2.0 SUBSURFACE INVESTIGATION

The subsurface investigation was performed on September 15 through 21, 2021. It included drilling a total of 17 test borings to depths of 10- to 35-ft. below ground surface (bgs) and performing infiltration testing at 8 locations at depths of 5-ft. bgs. A location plan showing the soil test borings is provided as Figure 2.0-1.

2.1 Soil Test Borings

Test borings were advanced using hollow-stem augers with an inside diameter (ID) of 3.25-in., as well as casing with an ID of 3-in. and a rollerbit. Soil samples were recovered from the borings at selected intervals by driving a 1-3/8-in. ID (2-in. outside diameter (OD) split-spoon sampler in general accordance with ASTM D-1586. Test borings remained open for a 24-hour period to obtain stabilized groundwater readings and were then backfilled with drill spoils.

Standard Penetration Tests (SPT) were conducted at changes in strata or at intervals not exceeding 5-ft. The sampler was first seated about 6-in. to penetrate through the loose cuttings and then driven an additional 1.5-ft. with blows of a 140-pound hammer falling 30-inches. The number of hammer blows required to drive the sampler the final foot is designated as the SPT (N) value and is recorded as Blows Per Foot (BPF).

Soils obtained from the sampling device were sealed in glass sample jars and transported to the soils testing laboratory. The recovered soil samples were inspected and classified by a Geotechnical Engineer using the ASTM Soil Classification System (ASTM D 2487). A description of the soils and conditions encountered at each test boring location are presented on the Boring Logs in Appendix B.

2.2 Groundwater Conditions

Groundwater levels were noted in each of the borings during drilling operations, immediately and after 24 hrs. of completion of drilling. Groundwater was not observed on the drill rods and in samples during drilling operations in all test borings. Groundwater readings at the end of drilling and after the HSA auger is pulled out were noted. All test borings were left open for the stabilized groundwater readings, except for test boring B-6, which was backfilled on completion for safety considerations. The groundwater depth and the corresponding groundwater reading time were recorded. These are included in the boring logs which are provided in Appendix B.

The groundwater elevations vary from EL +998 to EL +1014 (i.e., 13.3- to 31.0-ft. below the existing ground surface). It should be noted that groundwater level will fluctuate due to seasonal changes, precipitation, construction activities, etc. Note also that the highest groundwater observations are normally encountered in late winter and early spring.

Oservation wells were installed to evaluate stabilized groundwater elevations at B-5 and B- 7.

Water observations to date are summarized below.

Table 2.2-1: Groundwater Observations					
Test Boring	**Groundwater Observation (-ft. bgs/ EL)**				
	9-16-21	**9-17-21**	**9-20-21**	**9-21-21**	**11-22-21**
B-5	31-ft./ EL +998	31-ft./ EL +998	31-ft./ EL +998	31-ft./ EL +998	32.7-ft./ EL +996.3
B-7	NE	NE	19.9-ft./ EL +992.5	19.9-ft./ EL +	19.1-ft./ EL +993.3
NE = Not Encountered					

2.3 Soil Laboratory Testing

Soil samples recovered from the field explorations were transported to laboratory and selected soil samples were tested to determine additional engineering characteristics of the existing on-site soils.

Laboratory tests conducted on the selected soil samples include: Natural Moisture Content (ASTM D2216), Atterberg Limits (ASTM D4318), Sieve Analysis (ASTM D422), Moisture v. Density Relations (ASTM D698), and California Bearing Ratio (ASTM D1883). A bulk sample was retrieved from test boring IT-1 at a depth ranging from 2 to 5-ft. bgs and tested for Moisture v. Density Relations and California Bearing Ratio (CBR). The sample classified as a Sandy Lean CLAY (CL) with an optimum moisture content of 15.0% and maximum dry density of 112.0 pcf. The sample had a CBR value of 5.9% with a 0.4% maximum swell. A summary of the laboratory tests is included in Table 2-3.1 below and details are included in Appendix C.

Table 2.3-1: Soil Laboratory Test Results

Boring Location	Sample Number	Sample Depth (ft. bgs)	USCS Classification	Natural Moisture Content (%)	Plastic Limit	Liquid Limit
B-1	S-6	13-15	SILT (ML)	33.3	27	40
B-4	S-7	13-15	Sandy, Silty CLAY (CL-ML)	18.4	22	29
B-5	S-6	13-15	Lean CLAY (CL)	19.1	23	35
B-6	S-1	0-2	Lean CLAY with sand (CL)	16.7	24	37
IT-1	Bulk	2-5	Sandy Lean CLAY (CL)	7.1	19	33

3.0 SUBSURFACE INVESTIGATION

The Boring Logs contain details related to the subsurface conditions encountered at the test boring locations. Stratification lines shown on the Boring Logs and the Generalized Subsurface Profile provided as Figures 3.0-1 and 3.0-2 represent approximate transitions between material types. Strata changes can occur gradually or at different levels than those shown on the Boring Logs that depict conditions at the specific indicated locations and depths at the time of our subsurface exploration program. Groundwater levels are variable and are influenced by the existing soil conditions, seasonal and climatic changes. The test boring data, visual and laboratory classification of the sampled soils, and our knowledge of local geology was used to separate the soils into three strata: Topsoil, Man-placed Fill (Stratum A), Residual Soils (Stratum B), Disintegrated Rock (Stratum C), and Bedrock (Stratum D) which are described in the following sections.

3.1 Topsoil

Topsoil was encountered in all of the test borings. This stratum was found to range from 2- to 5-inches thick in each test boring location.

3.2 Stratum A: Man-Placed Fill

Man-Placed Fill was encountered in all of the test borings except test borings B-6, B-7, IT-5 and IT-6. The fill material was observed to consist of sand, clay, and rock and brick fragments. The fill appears to have been placed during past construction and grading activities at the site. The fill stratum extended from approximately 6-in. to 10-ft. below existing grade (i.e., elevations of EL 1002.9± to EL 1031.6±). The penetration resistance in the fill indicated a generally low density with SPT N-values ranging from 4 BPF to 57 BPF.

3.3 Stratum B: Residual Soils

Residual soils were encountered below the Man-placed Fill soils to the maximum depth explored. The residual soils generally consisted of Lean Clay (CL), Silty Clay (CL-ML), and Silt (ML) with varying amounts of sand and rock fragments. The density of these soils varied due to the degree of weathering within the profile, with SPT values of 3 to 46 BPF.

3.4 Stratum C: Disintegrated Rock

The disintegrated rock is defined as residual material with SPT values of greater than 60 blows per foot. This rock like material was encountered in test borings B-1 through B-9 to the refusal depths of the borings 8- to 35-feet. The disintegrated rock is interlayered within the soils of Stratum B in some of the soil borings.

3.5 Stratum D: Bedrock

The bedrock surface was defined as where the SPT blow count exceeded 100/2-inches and was encountered at depths of 15- to 33-feet below ground surface. Rock core samples were taken from test boring B-3. Based on the observation of the core samples and the regional geological maps, the underlying parent rock is identified as slightly to extremely fractured, moderately to severely weathered limestone. The recovery of the core samples ranged from 90- to 98-percent, and the RQD varied from 13- to 68-percent indicating good rock quality. Photographs of the rock cores are provided as Appendix D.

4.0 RISKS RELATED TO CONSTRUCTION IN KARST GEOLOGY

The proposed building is in a karst geologic formation. No surface depressions were observed within the proposed building footprint during this investigation and there does not appear to be any active sinkhole activity on the site or mapped nearby (Figure 1.3-2). However, the potential exists for sinkhole formation on this site due to the limestone bedrock. Sinkhole formation is problematic as loss of support for foundations and/or the slab-on-grade may occur if a sinkhole forms on the site. The risks of foundation design in karst are well explained in “Foundation Design in Karst Terrain,” prepared by Destephen and Wargo, dated 1992, provided as Appendix C. We encourage all project stakeholders to read this document to better understand the risks of construction in karst geography.

A reduction in the potential for a sinkhole to form on the site can be accomplished through design and construction measures. These are detailed further in Sections 4.1 and 4.2.

4.1 Engineering Measures to Reduce Sinkhole Potential

Manage drainage throughout the lifecycle of the project: In order to decrease the potential for a sinkhole to develop on the site in the long-term, on-site water must be continually managed.

The following measures should be taken to reduce the risk of sinkhole formation on the site during construction:

- Design grades should provide positive drainage throughout the life of the structure. Grades should be set to slope away from the building to encourage water to drain away from the building. This includes landscaping and subgrade grades such as for pavements and other site features like sidewalks, dumpster pads, etc.
- All roof drains should be connected to the on-site Stormwater Management (SWM) device and designed to be watertight.
- Parking areas should include curbs that direct runoff to on-site SWM devices to limit the potential for water to infiltrate into the underlying soils.
- Utilities should be installed with watertight seals and consideration should be given to installing utilities in a concrete duct bank to further limit the potential for infiltration of water due to a leak in the utility.
- Water bearing utilities should not be designed under, or adjacent to, spread footings.
- The SWM system should be designed in accordance with the “Pennsylvania Stormwater Best Management Practices Manual, dated 2006 which include design recommendations to minimize the potential for a sinkhole to form near the SWM device(s).
- A pavement maintenance program should be implemented for the service life of the project. This should, at a minimum, include crack and surface sealing and patching of deteriorated areas on a regular basis.

4.2 Construction Measures to Reduce Sinkhole Potential

Drainage must be managed throughout construction to reduce the potential for a sinkhole to form on the site. Steps to manage drainage should include, at a minimum:

- The potential for sinkhole development is particularly high during and after large precipitation events. We recommend that excavation be limited when wet weather is expected and that visual inspection of excavations, swales, drainage ditches, basins, etc. be performed by the Geotechnical Engineer of Record after rain events.
- Avoid the ponding of water. This is especially important during excavation, placement of compacted structural fill, and footing construction.
- All joints between asphalt paving and concrete curbing, or where the asphalt paving is in contact with concrete paving such as for dumpster pads, should be sealed.

The risk of sinkhole development is higher during excavation than at other times during the building construction because excavation allows water to infiltrate into soils that may otherwise not have been exposed to wetting. In order to limit risk during excavation we recommend:

- Limit the excavation required to the minimum extent required.
- Close observation should be made whenever excavating close to the rock surface because excavation closer to the rock surface has a higher potential for sinkhole development than excavation within the soil matrix.
- Excavations should be made during the drier months and backfilled as soon as practical. If an excavation is unable to be backfilled prior to a precipitation event, a mudmat should be used to reduce the potential for water to infiltrate into the soil or rock present at subgrade.
- Construction of earthen berms, dikes, and/or ditches around open excavations to reduce the potential for ponding (and subsequent infiltration) of water. Note that drainage channels, swales, and other water management features should be lined with impermeable liners to further reduce the potential for sinkhole development due to poor site drainage.
- Provide full time observation of subgrade during excavation and earthwork. If soft and/or wet soil is observed in the excavation, it may indicate a zone of solution activity. If encountered, these soils should be removed and replaced with structural compacted fill in accordance with the recommendations in this Report. In addition, if these types of unstable soils are encountered the Geotechnical Engineer of Record should evaluate the conditions and make recommendations for further remedial measures if needed.
- Blasting of bedrock for removal should be avoided as it can dramatically increase the likelihood of a sinkhole to form on the site.

Also, visual inspection during construction should be performed to observe indications of sinkhole formation and signs of soil instability including very soft or wet soils inconsistent with subsurface conditions encountered during this investigation. If these conditions are encountered the Geotechnical Engineer should be notified to develop remedial actions.

4.2.1 Sinkhole Repair

If a sinkhole develops on the site during, or after, construction, the Geotechnical Engineer of Record should be contacted as soon as possible to evaluate the type, size, and location of the sinkhole and provide repair recommendations. It is imperative that sinkhole repair be directed by a Geotechnical Engineer experienced in sinkhole repair techniques.

5.0 FOUNDATION AND SLAB-ON-GRADE RECOMMENDATIONS

The geotechnical analyses for foundation and floor slab design are based on the results of the test borings, laboratory tests, and our experience with similar geologic conditions.

As provided by the Structural Engineer, maximum un-factored column loads of 250 kips and wall loads of less than 5 kips per foot are expected.

The proposed building lower level has a Finished Floor (FF) of EL +1034 in the western portion and EL +1018 in the eastern portion. The subsurface investigation on this site revealed two challenges to construction with the proposed finished floor elevations and loads: the presence and depth of Man-placed Fill in the western portion and the depth of soil overlying the Disintegrated Rock (Stratum C). A plan providing the observed depth of Man-placed Fill requiring remediation in areas investigated and the depth of the soil overlying the Disintegrated Rock related to the proposed FF elevation is provided as Figure 5.0-1 and the concerns related to these two items are described below.

First, the depth of Man-placed Fill required to be removed based on the FF of +1034 in the western portion of the building ranges from approximately 4- to 10-ft. The aerial extent of the Man-Placed Fill observed in the test borings appears to correspond to the location of the previous building on-site which we understand had a basement (see Figure 1.2-2). The presence of Man-placed Fill on this site limits the use of spread footings on natural soil for foundation support. Man-placed Fill has variable engineering characteristics and due to its non-homogeneous nature, it is generally not advisable to allow Man-placed Fill to remain in place below spread footings. Whenever Man-placed Fill is allowed to remain in place below spread footings, there is a risk of undesirable settlement.

Second, the thickness of overburden above the Disintegrated Rock varies from 0-ft. observed in Test Boring B-9 where proposed spread footings would be supported directly on the Disintegrated Rock to 24-ft. observed in Test Borings B-7 and B-4. We understand from the Structural Engineer that differential settlement should be limited to 0.5-in. between adjacent columns. Based on our analysis, footings supported on Residual Soils or compacted structural fill are expected to experience more than 0.5-in. of settlement. Meanwhile, footings supported directly on the Disintegrated Rock are not expected to have appreciable settlement as Disintegrated Rock, and the underlying parent bedrock are essentially incompressible when compared to the compressibility of the Residual Soil (Stratum B). This creates potential for unacceptable differential settlement between columns which could result in undesirable performance of the building.

The successful foundation support solution to these challenges must include remediation of the Man-placed Fill by excavating the Man-placed Fill completely and replacing it with compacted structural fill or reinforcing the Man-Placed Fill with a ground improvement system such as Cement-Treated Aggregate (CTA) Piers or Rigid Inclusions (RIs), as well as improving the stiffness of the overburden soils to reduce the settlement of spread footings supported on natural Residual Soils to accommodate less than 0.5-in. differential settlement between columns.

We do not recommend the use of traditional aggregate piers due to the potential for sinkholes to form in the karst geology. It is widely understood that the use of CTA Piers helps to lower the permeability of a traditional aggregate pier and mitigate water from infiltrating the bedrock where CTA Piers will terminate directly on Disintegrated Rock or Bedrock. In addition, we recommend that CTA Piers be installed with a vertical ramming technique that allows stress to be distributed laterally throughout the length of the pier to preclude punching of the pier at the bottom of the pier.

As an alternate to CTA Piers, Rigid Inclusion's (RIs) installed using drilled methods are feasible provided that additional investigation is performed to confirm that the RI is not terminated on a "rock shelf." It is possible in karst geology that a mudseam could exist beneath the rock. If a RI is allowed to terminate on a thin rock layer, the potential exists for the RI to "punch" through the rock as the rock may not provide enough bearing resistance to support the heavy load imposed by the RI. Therefore, if traditional RIs are used, we recommend that each RI location be air tracked and where mud seams exist, they should be grouted prior to installation of the RI in that location.

When selecting the installation method, the Specialty Contractor should carefully consider the groundwater elevation and the cave depths reported on the Test Boring Logs, and the presence of debris in the FILL soils.

These systems, discussed in detail below, are provided by Specialty Contractors that can provide pre-construction cost estimates, final design, and installation of these foundation support systems. We recommend that you contact them to review and analyze the subsurface data in this report, as well as the proposed structural geometry and loading for the project and provide you with a cost estimate during pre-construction planning of this project.

5.1 Spread Footings on Cement Treated RIs or Drilled RIs

Column and wall footings supported directly on Disintegrated Rock (Stratum C), or Bedrock (Stratum D) can be designed for an allowable bearing pressure of 5,000 psf and are expected to settle less than 0.5-in. For settlement compatibility, footings not supported directly on bedrock should be supported with CTA Piers or RIs. We recommend that the minimum depth of any CTA Pier or RI be 4-ft. In areas where bedrock is encountered within 4-ft. of the bottom of footing, we recommend the footing be lowered to bear on the Disintegrated Rock (Stratum C) or Bedrock (Stratum D).

For CTA, RI or Drilled RI supported column or wall footings, the total settlement, which includes settlement during construction of the building, should be limited to 0.5-in., unless otherwise allowed by the structural engineer. Differential settlement should be limited to 0.25-in. or less between adjacent columns, unless otherwise allowed by the structural engineer.

All spread footings should be designed for a minimum frost depth of thirty-six inches unless supported directly on Bedrock.

Spread footings should be poured as soon as possible following excavation to limit the potential for water to infiltrate the subgrade which may result in the formation of sinkholes. If it is not possible to construct the spread footing the same day as it is excavated, a mudmat should be used to limit infiltration.

5.2 Cement Treated Aggregate (CTA) Piers

CTA Piers are installed by constructing successive layers of densely compacted cement-treated aggregate in a pre-drilled or displaced shaft, typically measuring between 18 and 36-inches in diameter. The aggregate is densified using high-energy vertical ramming action. The ramming action compacts the aggregate and prestresses the surrounding matrix soils. Additional lifts of cement treated aggregate are then successively placed, creating a continuous shaft. The high-energy compaction process produces lateral prestraining and prestressing of the adjacent matrix soils that increase the lateral stress in the adjacent soils. The improved soils and the compacted aggregate shaft together increase the strength and stiffness of the supporting soil, allowing for the use of traditional shallow spread footing foundations.

CTA Piers are traditionally installed using replacement (drilled) methods. Considerations that could affect the design and installation of the CTA Piers include groundwater elevations above the CTA pier tip elevations, soft or loose soils that may collapse, and/or the potential for construction debris to be encountered in the existing FILL soils.

We recommend that each CTA Pier location be air tracked to investigate the presence of mudseams. Where mudseams are encountered, they must be grouted prior to installation of the CTA Pier to preclude punching of the CTA Pier through a thin rock “shelf.” The air track locations should be selected as part of the CTA Pier design and included the approved shop drawing. Specialty Contractors should consider the cost for this effort in their scope of work.

Because installation methods and design procedures vary by Specialty Contractor, the GER should be engaged to develop performance specification. The GER should also review CTA Pier design, testing, and quality control procedures prior to construction.

5.2.1 CTA Pier Design

The design of CTA Piers is not addressed by building codes. Rather, industry standards are used and, as a result, design methods vary between Specialty Contractors. We recommend that CTA Piers be designed for settlement using sustained gravity loads (e.g., dead and live load), and the Factor of Safety against a bearing failure be evaluated using the total load (e.g., dead, live, and transient).

An allowable soil bearing pressure of 5,000 psf is expected to be feasible with the use of CTA Piers and conventional spread footings can be designed using this value. Unlike a deep foundation system, RAPs transfer the applied load in skin friction. The stress distribution for a RAP supported footing is the same as for a footing bearing on unimproved soil. For purposes of the structural design, a 2:1 stress distribution should be assumed.

CTA Piers should extend through the Stratum A FILL soils and should terminate in natural soils. CTA Piers should be designed to satisfy footing bearing requirements and to limit total and differential settlement to the required tolerances.

A load transfer mechanism, typically consisting of gravel, should be placed between the top of the CTA Pier and the footing bottom to limit the stress on the conventional spread footing and to provide a shear break. The design of the CTA Pier and the load transfer mechanism should be part of the Specialty Contractor’s scope of work and compatible with the footing design. Due the karst geology, we recommend that, at a minimum, CTA Piers be 4-ft. long to provide a minimum of 2-ft. of cement treated aggregate above the bedrock (i.e, the upper 2-ft. may be non-cement treated aggregate to provide the required shear break, but the lower 2-ft. must be cement-treated aggregate).

5.2.2 CTA Pier Testing

A minimum of one load test should be performed as part of the installation process to verify soil strength assumptions used by the Specialty Contractor. This test should be performed to 150% of the maximum top of CTA Pier stress as indicated in the design and in accordance with ASTM D1143 Procedure A.

The Specialty Contractor should employ a QC program that is monitored full time. This QC program should ensure that production CTA Piers are constructed with consistent means and methods as the RAP tested in the load test. At a minimum, the QC program should include observation and testing of the load test RAP, and measurement of placement depths and material used in the test and production CTA Piers.

5.3 Rigid Inclusions (RIs)

Rigid inclusions are constructed with cement or grout to form a structural member that can transfer loads down to a competent soil or rock layer. RIs support allows the use of traditional shallow spread footing foundations, while greatly reducing the amount of stress imposed on adjacent structures.

Considerations that could affect the design and installation of the RIs include groundwater elevations above the RI tip elevations, soft or loose soils that may collapse during the excavation, and/or the likelihood of construction debris to be encountered in the existing fill soils.

Because installation methods and design procedures vary by Specialty Contractor, the GER should be engaged to develop performance specifications. The GER should also review the RI design, testing, and quality control procedures prior to construction.

In addition, because of the high stress concentration observed at the bottom of the RI, we recommend that each RI location be air tracked to investigate the presence of mudseams. Where mudseams are encountered, they must be grouted prior to installation of the RI to preclude punching of the RI through a thin rock "shelf." The air track locations should be selected as part of the CTA Pier design and include the approved shop drawing. Specialty Contractors should consider the cost for this effort in their scope of work.

5.3.1 RI Design

Similar to CTA Piers, the design of RIs is not addressed by building codes. Rather, industry standards are used which can vary significantly between Specialty Contractors. We recommend that RIs be designed for settlement using sustained gravity loads (e.g., dead and live load), and for strength conditions using total load (i.e., dead, live, and transient).

Preliminary spread footing design can be performed using an allowable bearing pressure of 5,000 psf with RI support. However, because RI design and installation is typically provided by a Specialty Contractor, and because RIs are significantly stiffer than CTA Piers, we recommend that you engage a Specialty Contractor during design to confirm that the footing design is compatible with their proposed system to limit overall project risk and minimize any foundation redesign cost. Due to the high stress concentrations above the RIs which do not exist in conventional spread footing design, we recommend that the structural engineer and the Specialty Designer confirm that the footing reinforcement and thickness are adequate for the proposed RI stiffness and configuration.

Depending on the top of pier stress, a load transfer mechanism, typically consisting of gravel, is placed between the top of the RI and the footing bottom to limit the stress on the conventional spread footing and provide a shear break. The design of the RI and the load transfer mechanism should be part of the Specialty Contractor's scope of work and compatible with the footing design. Similar to the CTA Piers, we recommend that the minimum RI length is 4-ft. and that at least 2-ft. above the bedrock be grouted to mitigate water migration into the bedrock.

RIs should extend through the Stratum A FILL soils and should terminate in dense natural soils. RIs should be designed to satisfy footing bearing requirements and to limit total and differential settlement to the required tolerances.

5.3.2 RI Testing

One on-site load test should be performed to confirm the amount of compression that an individual RI will experience at the maximum theoretical stress at the top of the RI. This test should be performed on a RI located in the weakest area of the site and loading of the test RI should be conducted up to 200% of the maximum theoretical stress to which the RI will be subjected as indicated in the design and in accordance with ASTM D1143 Procedure A. The RI settlement should not exceed the Davisson Criteria for any load increment.

The Specialty Contractor should employ a QC program that is monitored full time. This QC program should ensure that production RIs are constructed with consistent means and methods as the RI tested in the load test. At a minimum, the QC program should include observation and testing of the load test RI, measurement of placement depths, and material used in the test and production RIs. The initial compressive strength of the designed RI mix design should be tested to confirm it meets specifications, and cylinders of the cement aggregate mix should be tested throughout construction to confirm consistency.

5.4 Floor Slab Support

The floor slab is expected to be supported on natural soils at the lower level and compacted structural fill at the upper level.

Where Man-placed Fill is present at slab subgrade, which is expected to be most of the western portion of the building, we recommend that 2-ft. of the Man-placed Fill be undercut and replaced with compacted structural fill.

In all areas, the subgrade should be proofrolled prior to fill placement. If the subgrade is not found to be stable, additional undercut and subsequent proofrolling of the subgrade should occur as needed to achieve a stable subgrade. Soils undercut in the floor slab area should be replaced with compacted structural fill in accordance with recommendations in this report.

Proofrolling should be performed using the heaviest construction equipment available, for example, a loaded 20-ton dump truck or equivalent (at least a 3,000-lb. walk-behind roller), which can access the area and under the observation of a geotechnical engineer from our office. Any additional loose or unsuitable soils found to be excessively pumping or rutting during proofrolling should be removed and replaced with compacted fill.

Floor slabs on grade may be designed using a modulus of subgrade reaction, k equal to 125 pci.

On most projects, there exists a significant lag time between the initial grading and the placement of the floor slab. Environmental conditions and construction traffic often disturb the soil subgrade during this lag time. The contractor should make provisions in the construction specifications for the restoration of the subgrade to a stable condition prior to the placement of the floor slab at no additional cost to the owner.

6.0 SEISMIC CONSIDERATIONS AND SITE CLASS

This section presents the testing and analysis conducted to evaluate the liquefaction potential of the soils and the seismic site class for this project site, per the 2015 International Building Code (IBC).

6.1 Liquefaction Potential

Liquefaction typically occurs in loose, cohesionless sands located below the water table. As these conditions are not present at this site, no further liquefaction analysis is warranted.

Based on our investigation and engineering judgement, the building site is not susceptible to liquefaction under the design earthquake magnitude mandated by code.

6.2 IBC Seismic Site Class and Design Parameters

Seismic design parameters were determined in accordance with the 2015 International Building Code (IBC). The "U.S. Seismic Design Map Web Application" available through the USGS website provides hazard curves, uniform hazard response spectra, and design parameters for sites in the 50 states of the United States, Puerto Rico, and the U.S. Virgin Islands. These parameters were developed using two-percent probability of exceedance (PE) in 50 years. Following are the mapped spectral response acceleration values for the project site at Latitude 40.44394490, Longitude: -78.41523680.

Table 6-1: Mapped Spectral Response Acceleration Values

Description	Period (Sec)	Sa
Mapped Short Period Spectral Response Acceleration (S_S)	0.2	0.114 g
Mapped 1-Second Period Spectral Response Acceleration (S_1)	1.0	0.051 g

The Seismic Site Classification influences the determination of the Site Coefficients, the Design Spectral Response Acceleration values, and ultimately the Seismic Design Category. Note that the Seismic Site Classification is based on the characteristics of the upper 100-ft. of soils and rock below the site. The IBC requires the use of Standard Penetration Test Resistance (test borings), Shear Wave Velocity (geophysical methods), and/or Undrained Shear Strength (soil laboratory testing) to categorize the Seismic Site Classification.

The Seismic Site Classification was determined to be Site Class D based on the Standard Penetration Test results from the borings. For a Site Class D, with the above-indicated mapped spectral acceleration values and Risk Category II, the following are the calculated Site Coefficient values and the Maximum and Design Spectral Response Acceleration values, per IBC Section 1613.2.2.

Table 6-2: Site Class, Site Coefficients, and Design Spectral Response Acceleration

Site Class	D
Soil Profile	Stiff Soil
Site Coefficient (Fa)	1.6
Site Coefficient (Fv)	2.4
Short Period, Maximum Spectral Response Acceleration (SM_S)	0.182 g
1.0 Second Period, Maximum Spectral Response Acceleration (SM_1)	0.122 g
Short Period, Design Spectral Response Acceleration (SD_S)	0.121 g
1.0 Second Period, Design Spectral Response Acceleration (SD_1)	0.082 g

The Design Spectral Response Acceleration values are to be used with the Risk Category (ASCE 7-10) of the building or structure to determine the Seismic Design Category. Complete results of Spectral Acceleration with varying period are given in Appendix F.

7.0 EARTHWORK

Fills of 0- to 6-ft. and cuts of up to 15-ft. are expected to achieve the proposed finished floor elevations

Careful subgrade preparation, including stripping of organic layers or soft surface soils is required to prepare suitable fill and slab subgrades. Earthwork is recommended to take place in the warmer, drier months between May and October. The use of scarification and drying techniques, or additives such as quick lime, or Portland cement may also be useful in expediting fill operations in inclement weather.

It is important to maintain good site drainage practices throughout the construction of this project to limit the risk of sinkhole formation. This includes proofrolling as soon as is practical after excavation to subgrade, grading the site to promote runoff and using drainage, using drainage swales, ditches, etc. to facilitate drainage.

7.1 Excavation Characteristics

Excavation of this site is expected to be performed using conventional earthmoving equipment. Careful preparation of subgrades, proper placement and compaction of structural fill and backfill are both necessary to prepare a suitable site for the support of the proposed addition. Details of these requirements are included in the following sections.

A building was present on the western portion of the site as recent as 2017. We understand that some of the basement and foundations of the previous building were removed. However, large construction debris in the Man-placed Fill as well as historic subgrade walls, footings, and slabs should be expected to be encountered during earthwork.

7.2 Fill and Floor Slab Subgrade Preparation

In areas where Man-placed Fill is present at slab subgrade (i.e., the western portion of the building) the floor slab subgrade should be undercut a minimum of 2-ft. and replaced with structural compacted fill.

Based on the test borings performed, we recommend the suitability of the existing soil be evaluated by proofrolling prior to slab or pavement construction. All fill subgrades should be proofrolled prior to fill placement or slab construction. If the subgrade is found to be unstable, additional undercut should be performed under the direction of the Geotechnical Engineer of Record. Upon achieving stable subgrade, the contractor should replace undercut soils with compacted structural fill in compacted lifts in accordance with Section 6.3.

All subgrades should be “sealed” using a roller at the end of the day, especially before expected wet weather events to limit the potential for water to infiltrate the subgrade which increases the risk of a sinkhole forming on the site.

For budgeting purposes, the Contractor should assume 2-ft. of undercut and replace for the floor slab in the western portion of the building.

All vegetation and topsoil located below proposed structures should be removed from the subgrade prior to filling. Fill subgrades should be proofrolled to assure that all unsuitable, soft and loose soils have been removed from below the building. During proofrolling, the subgrades should be observed by the Geotechnical Engineer of Record. Any unsuitable soils that are observed to be excessively settling or to be pumping during proofrolling, should be removed down to firm soils and then replaced with satisfactory soil materials compacted in accordance with the project specifications.

Some of the on-site soils at the lower-level subgrade classify as Lean Clay (CL) and are likely to become unstable in wet weather and under construction traffic. Significant undercutting of fill subgrades should be expected if the subgrades are exposed to the above events. In addition, Project Specifications should require the contractor be responsible for protecting the subgrades from weather and equipment damage.

Demolition which requires excavation below foundations, including utility abandonment, must be replaced with compacted structural fill or flowable fill.

7.3 Compacted Structural Fill

Compacted structural fill and backfill for use below or behind structures and behind walls should consist of satisfactory soils classified as SM or better in accordance with the Unified Soil Classification System, ASTM D-2487. Soils meeting this requirement classify as SM, SP, SW, GM, GC, GP, and GW. Unsatisfactory soils are those classified as SC, ML, OL, OH, CH, CL, and MH.

Material excavated from this site is NOT expected to be suitable for use as compacted structural fill under structures or as backfill behind retaining walls. On-site soils may be used for non-structural site features such as for landscaping.

Soils used for compacted structural fill should be free of unsuitable materials, such as topsoil and other organics, rubble, and rock larger than 3-in. in diameter. The in-place moisture content of the satisfactory soils' material shall be adjusted by the contractor through wetting or drying, to within three percent of the optimum moisture content.

Compacted structural fill should be placed in approximately horizontal layers, each layer having a loose thickness of not more than 8-in. All structural fill should be compacted to 95 percent of the maximum dry density in accordance with ASTM D-698, Standard Proctor. The contractor should select appropriate compaction equipment to achieve the required compaction.

8.0 DRAINAGE

Grades should be designed to provide positive drainage away from the Addition throughout construction and be maintained throughout the life of the building. Allowing water to pond near the perimeter of the building during construction or throughout the life of the structure may result in greater settlement than discussed in this report.

Landscaped irrigation adjacent to the foundation systems should be minimized or eliminated. If landscaped areas are constructed within 10-ft. of the foundation systems, the areas should be designed to have positive drainage away from the foundation, and this drainage should not be hindered by landscape edging, grade variations or vegetation.

8.1 Floor Slab Subdrainage

Groundwater is estimated to be greater than 5-ft. below the proposed finished floor grade for the upper level and a special under-floor subdrainage system is not considered necessary. However, where moisture sensitive flooring is used, a true vapor barrier such as a 10-mil. Stego® Wrap should be placed between the compacted structural fill placed for floor slab subgrade and the concrete slab-on-grade.

The lower floor slab is expected to be at EL +1018 which is within 2-ft. of the groundwater table level observed in Test Boring B-9. It is possible that this groundwater exists in a perched condition and the need for a subdrainage system can be re-evaluated during construction. We recommend that a subdrainage system designed to collect groundwater around the perimeter walls and below the floor slab of the structure is required to maintain groundwater below the floor level. A typical subdrainage system sketch, intended to graphically depict our recommendations, is included as Figure 8.1-1. If groundwater is observed to be perched during construction through test pit observation, the Owner may consider deletion of the subdrainage system described herein. However, we recommend that a waterproofing membrane be used regardless of the installation of a subdrainage system.

The proposed underfloor subdrainage system is discussed below. A layer of plastic should be used above the subdrainage system, between the concrete floor slab and the gravel layer, so as to prevent concrete intrusion into the gravel. The subdrainage system should be placed shortly before slab construction to minimize damage to the piping from construction operations.

Because the proposed area is to be used as a habitable space, the use of both a waterproofing system and underfloor subdrainage system is recommended. The basement walls and floor need not be designed for hydrostatic water pressure when subdrainage is installed as detailed herein. However, walls below grade and slabs-on-grade must be waterproofed.

The system may consist of perforated, closed joint drain tiles located around the interior perimeter of the below-grade areas, as close as feasible to the exterior wall, below the finished floor level. A network of interior pipes is also needed. Since an earth retention system will likely be required for the construction, it is anticipated that “lot line” construction will be used. Weep holes, which convey drainage from behind the walls to the under slab subdrainage system, should be placed at a spacing of no greater than 8-feet on center, generally designed to align between the soldier piles of the earth retention system. The weep holes should be a minimum of 4-in. in diameter and should freely drain from the exterior drainage medium to be collected by the interior perimeter drain line just inside the base of the wall. The drain lines should be surrounded by 6-inches of gravel or clean sand material having a gradation compatible with the size of the opening utilized in the drain lines and the surrounding soils to be retained. We recommend that the perimeter and under-slab drain system for the proposed structure be designed to flow to at least one permanent sump at a location to be determined by the design team. In addition, the permanent sump shall be designed with a full duplex capability (i.e., 2 pumps per pit), with each individual pump rated at no less than 50 gpm. With this configuration, under emergency conditions, the individual sump would have the capacity to pump 100 gpm. Once the plans are further developed, we should be contacted to refine our preliminary pumping estimates.

Lateral drain lines under the floor slab should be placed at no more than 40-feet on center. Underslab drain lines should have a minimum diameter of four inches, and they should be slotted or appropriately perforated. Clean out access should be installed at all sharp bends and at approximately every 100-feet for straight runs. A grit collection chamber should be installed upstream of the sump to reduce the amount of granular materials reaching the pumps.

A layer of drainage fill, consisting of a minimum of four inches of washed gravel or open-graded crushed stone, should be placed below all floor slabs as a capillary break.

8.2 Drainage During Construction

The Contractor should provide for proper drainage of surface water away from any excavations, including for installation of the spread footings, utilities, elevator pit, etc. All excavations should be conducted during dry weather and grades should provide effective drainage away from the elevator pit during and after construction

On most projects, there exists a significant lag time between the initial grading and the placement of the floor slab. Environmental conditions and construction traffic often disturb the soil subgrade during this lag time. The contractor should make provisions in the construction specifications for the restoration of the subgrade to a stable condition prior to the placement of the floor slab at no additional cost to the owner.

Upon completion of the building construction, we recommend that verification of final grading be performed to document that positive drainage, as described above, has been achieved.

9.0 EXCAVATION SUPPORT, RETAINING WALLS AND LATERAL EARTH PRESSURES

Sloped excavation is expected to be used to allow excavation for construction of the proposed lower level of the building.

9.1 Temporary Sloped Excavation

Sloped excavation may be used where excavation depth is shallow, the extent of excavation is small, and ground movements as a result of excavation would not impact the performance of existing structures. Sloped excavations should follow the United States Department of Labor Occupational Safety and Health Administration (OSHA) Safety and Health Regulations for Construction Standards. Slopes are expected to be able to be designed as OSHA Type B soils. However, this should be confirmed by the Geotechnical Engineer or Record during construction. Sloped excavation below the GWT is not recommended, and a mechanical excavation support system should be used.

9.2 Braced and Cantilever Walls

The lower level retaining wall will be required to retain backfill. These walls must be designed to resist lateral earth pressures developed from the surrounding soils and any surcharge. Figure 9.1-1 provides recommended pressure diagrams for the design of retaining walls in cantilevered and braced conditions. These pressure diagrams include earth pressures developed from backfill soil placed behind the walls.

It is expected that the at-rest pressure can be used if backfill is compacted against walls that are braced at top and bottom, and the active condition can be used if the walls are designed to be cantilevered. The ponding of precipitation behind the walls should be avoided during construction as the pressure diagrams included do not include hydrostatic pressure. Conventional foundation subdrainage or weep holes should be used to prevent buildup of hydrostatic pressure behind walls. Any vertical surcharge load from temporary construction equipment should be added to the lateral earth pressure with a rectangular force diagram as indicated in the pressure diagrams. The surcharge load from temporary construction equipment should be a minimum of 250 psf. A Factor of Safety of at least 1.5 should be used for evaluation of overturning and sliding of the walls using the parameters indicated on the lateral earth pressure diagrams.

Specific material and compaction requirements for fill against walls below grade are included in Section 4. Compacted fill behind and in front of the walls should be free of organics and rocks larger than 3-in. in diameter and should consist of soils classifying SM or coarser. Compaction equipment exceeding 3,000 pounds in dead weight should not be used within 5-ft. of the walls in order to avoid overloading the walls. All building walls should be braced prior to backfillling unless they are designed to be cantilevered walls.

Suitable man-made drainage materials may be used in lieu of the granular backfill, adjacent to the below-grade walls. Examples of suitable materials include Enca Mat, Mira-drain, or Geotec drains. These materials should be covered with a filter fabric having an apparent open size (AOS) consistent with the size of the soil to be retained. The material should be placed in accordance with the manufacturer's recommendations. The lateral earth pressures indicated in Figure 9.1-1 are applicable for either granular backfill or the manufactured drainage medium. We recommend that all below-grade levels of the structure be waterproofed and include suitable water stops between the walls and floor slab at the foundation level.

10.0 STORMWATER MANAGEMNET RECOMMENDATIONS

10.1 Discussion

Keller Engineers, Inc. requested that infiltration tests be performed at eight locations, IT-1 through IT-8. The tests were requested at 5-ft. below ground surface (bgs).

Eight in-situ infiltration tests were performed in accordance with the Pennsylvania Stormwater Management Best Practices Manual, 2006 to evaluate the infiltration characteristics of the on-site soils. As allowed in the Manual, tests were performed as described in the Maryland Stormwater Manual Appendix D.1 using 5-inch diameter casing.

All casings were installed on September 16, 2021, and the test locations were pre-soaked the same day.

10.2 Stormwater Management Infiltration Recommendations

The Pennsylvania Department of Environmental Protection has set particular standards and specifications for the design and construction of stormwater infiltration devices. These regulations include parameters on soil textures, depth of limiting zone, and other considerations, which are described in the publication “Pennsylvania Stormwater Management Best Practices Manual, 2006.”

10.3 Depth to Limiting Zones

The aforementioned publication recommends that a two-foot distance be provided between the bottom of the infiltration system and any limiting zones. Limiting zones are defined as a seasonal high-water table or bedrock.

10.4 In-situ Infiltration Test Results and Summary

A minimum infiltration rate of 0.1-inches per hour can be used for design of infiltration SWM devices assuming an appropriate Factor of Safety is used in design. The infiltration measured after the results of the 24-hour pre-soak was less than 0.1-inches per hour in all test locations. Therefore, the four-hour test was not performed. Details of the in-situ infiltration test results are included in Appendix G. The following table summarizes results of testing and our observations.

Table 10-1: Infiltration Test Summary

Test Boring	Test Boring Depth (ft.)	Existing Grade (EL±)	Test Depth (ft.)	Groundwater Elev. (EL±)	Measured In-Situ Infiltration after 24-hour presoak (in/hr.)	Remarks
IT-1	10	1033.63	4.8	NE	0.02	1
IT-2	10	109.27	4.5	NE	0.03	1
IT-3	10	1009.18	4.6	NE	0.00	1
IT-4	10	1004.91	4.7	NE	0.01	1
IT-5	10	1004.14	4.5	NE	0.01	1
IT-6	10	999.99	4.6	NE	0.00	1
IT-7	10	1007.33	4.7	NE		1
IT-8	10	1032.57	5.0	NE	0.02	1

NE = Not Encountered

1- Infiltration practices not recommended due to low infiltration rates

10.5 Remarks

The low infiltration rates recorded at these locations indicate that infiltration-based stormwater management devices are not appropriate for these locations.

11.0 PAVEMENT DESIGN

The proposed parking lot for this project is currently planned to have flexible asphalt pavement vehicular drive aisles and parking spaces and a concrete pavement pad for dumpsters.

Recommendations for flexible and rigid pavements are provided in this section. Soil laboratory testing was performed for this site and a CBR value of 5 was assumed for pavement design.

These pavement designs assume that a continual maintenance program will be implemented during the service life of the project. This should, at a minimum, include crack and surface sealing and patching of deteriorated areas on a regular basis. This is of particular importance on this site as the underlying bedrock is Limestone and infiltration of water may facilitate solutioning and sinkhole development.

11.1 Pavement Subgrade Preparation

All subgrades should be proofrolled with a loaded 20-ton dump truck and any unsuitable soft or loose areas detected should be removed and replaced with compacted structural fill or stone base course. Where excavations made for utility abandonment or installation, the excavation shall be replaced with lean concrete or compacted structural fill.

Compacted fill placed for pavement support should be placed in accordance with recommendations made in this Report.

11.2 Flexible Pavement Design

Flexible pavement is anticipated for the entry and drive aisles on-site as well as the parking spaces. It is our understanding that driveways will be used to support automobiles and light delivery trucks. A pavement section with a maximum of 110,000 EASLs is recommended as these driveways and entrances may be used by heavier vehicles and tight turning radiuses are proposed.

The recommended flexible pavement section is provided in Table 9.2-1 below.

Table 11.2-1: Recommended Flexible Pavement Section

Layer	Thickness
Asphalt Surface Course:	2.0-in.
Asphalt Base Course:	3.0-in.
Stone Base Course:	7.0-in.
Subgrade	Proofrolled and approved by Geotechnical Engineer of Record

The asphalt surface and base course material should be selected by the civil engineer to provide a stable and relatively impervious pavement section. The stone base course should meet the specifications of MSHA GA Base and be compacted to at least 95 percent of the maximum dry density per AASHTO T180.

11.3 Rigid Pavement Design

A rigid concrete pavement should be used in areas of concentrated, repeated, heavy wheel loads such as in front of dumpsters, and in areas of tight turning radii and braking, where excessive wheel shearing forces could damage a flexible pavement. Traffic category A-1 with a maximum of one truck (vehicle with at least six wheels) per day was used for this analysis per American Concrete Institute Committee 330 for a 20-year design life.

The pavement section is provided in Table 11.3-1 below.

Table 11.3-1: Recommended Rigid Pavement Section

Layer	Thickness
Reinforced Portland Cement Concrete (RPCC)	5-in.
Dense Graded Aggregate	6-in.
Subgrade	Proofrolled and approved by Geotechnical Engineer of Record

The concrete should be 5,000 psi, air entrained. Construction and expansion joints should be based on the final site configuration but should not exceed 15-ft. in any horizontal direction.

11.4 Construction Considerations

The recommended pavement sections are not designed to accommodate construction traffic. It should be expected that damage will occur due to overloading of the pavement sections if they are subjected to construction traffic. This will be prevalent especially if water is allowed to collect on or in the pavement subgrades, and if only the base course is placed prior to the completion of the construction. Provisions should be made to minimize damage to the pavements during construction, including the use of subdrainage, temporary swales or berms, the limitation of construction traffic to certain areas, and/or an increased thickness of stone or base asphalt. An allowance should be reserved for the cost of repairs to the base paving prior to completion of the final surface-course.

12.0 CONSTRUCTION CONSIDERATIONS

Specific recommendations for foundation construction are given below:

12.1 Earthwork

Fill subgrades should be proofrolled under the observation of our representative. Any soft or unsuitable soils encountered should be removed and replaced with compacted fill. Abandoned underground utilities must be removed and replaced with compacted structural fill. Where excavations made for utility abandonment, demolition, or new utility installation trenches will intersect new footing subgrades, the excavation shall be replaced with lean concrete or compacted structural fill.

The contractor should expect to encounter debris in the Man-placed Fill and historic basements and walls from the previously demolished building on the western portion of the site.

12.2 Spread Footings on Disintegrated Rock

Care should be exercised during the excavation for all footings to minimize disturbance of the footing and fill subgrades. Footings should be excavated and concreted the same day in order to avoid ponding of surface runoff water in footing excavations and to avoid other disturbances such as freezing, extreme moisture variations (wetting or drying), etc. A mud mat consisting of a minimum of two inches of lean concrete may be placed to preserve the subgrades after the subgrade is approved by an engineer from our office. Hand cleaning of the disturbed soils left by the backhoe excavation will be required to produce a minimally disturbed subgrade. A flat-bladed excavation bucket will help to minimize the hand work.

12.3 Spread Footings on CTA Piers or RIs

Care should be exercised during the excavation for all CTA Pier or RI supported footings to minimize disturbance of the RIs. A flat-bladed excavation bucket should be used to excavate the footing and hand digging should be performed within six-inches of the top of the RI. If the top elevation of the RI is too high, or is not flat, the Specialty Contractor should be contacted for evaluation and repair methods. In no case should the top of a RI be “snapped off” creating an angled surface.

Footings should be excavated, the load transfer mechanism installed, and concrete placed the same day in order to avoid ponding of surface runoff water in footing excavations and to avoid other disturbances such as freezing, extreme moisture variations (wetting or drying), etc. A mud mat consisting of a minimum of two inches of lean concrete may be placed to preserve the subgrades after the subgrade is approved by an engineer from our office. Hand cleaning of the disturbed soils left by the backhoe excavation will be required to produce a minimally disturbed subgrade.

The Specialty Contractor should provide, as part of their design submittal, requirements, and installation details for the load transfer mechanism overlying the CTA Piers or RIs as well as any additional preparation requirements not listed in this report.

12.4 Compacted Structural Fill

Compacted structural fill should meet the requirements outlined in this report. All compacted structural fill and backfill below slabs and backfill behind foundation walls should be compacted to 95 percent of the maximum dry density per ASTM D698, Standard Proctor. Moisture conditioning, such as wetting or drying, should be expected to be required depending on the time of year construction occurs. However, it is recommended that earthwork be performed in the warmer, drier months between May and October. Soil additives such as lime or cement may be used to expedite compaction in soils above the optimum moisture for compaction.

Note also that adequate bracing of walls should be required during backfilling operations.

12.5 Review of Construction Documents

Any deviation to the project design subsequent to the date of this report, such as changes in floor grades, building loads and building location, should be brought to our attention to determine if our recommendations contained herein remain valid. We should be allowed to review the project drawings and specifications, as a follow-up to our design recommendations and as a precursor to our providing the geotechnical engineering services during construction.

12.6 Construction Observation and Testing

Regardless of the thoroughness of a geotechnical engineering exploration, there is always a possibility that conditions will vary from those encountered in the test borings, that conditions are not as anticipated by the designers, or that the construction process has altered the soil conditions. D.W. Kozera, Inc. considers construction observation and testing of the foundations and earthwork an integral part of the geotechnical design, and therefore these services should be provided by the geotechnical Engineer of Record. As the actual subsurface conditions are exposed and observed by us during construction, modifications to our report recommendations can be made promptly and efficiently as needed. Note that we cannot assume liability or responsibility for the adequacy of our foundation recommendations if we do not observe the foundation construction.

Observations and testing should at minimum include full-time observations of the excavation of footing, fill, and floor subgrades, and field density testing of compacted structural fill. Other services, including materials testing (concrete, reinforcing steel, bituminous concrete, masonry, etc.) can be provided upon request.

13.0 Limitations

This geotechnical study has been prepared in accordance with generally accepted geotechnical engineering practices. It is intended for the exclusive use of Hord Coplan Macht for the design and construction of the proposed building addition and site work as described herein. This report includes both factual and interpreted information. Factual information is defined as objective data based on direct observations, such as soil samples and laboratory testing results. Interpreted information or geotechnical engineering interpretation is based on the engineering judgment, correlation, or extrapolation from factual information.

This report is based on information for the proposed structure that was made available to us at the time of the writing of this report. No warranties, express or implied, are intended or should be assumed. D.W. Kozera, Inc. should be allowed to review the project drawings and specifications as a continuation of our design recommendations and as a precursor to our providing geotechnical engineering services during construction. In the event that any changes in the floor grades, building loads, or structure location as described in this report are planned, the conclusions and recommendations contained herein shall not be considered valid unless D.W. Kozera, Inc. reviews the changes, and either verifies or modifies the conclusions of this report in writing.

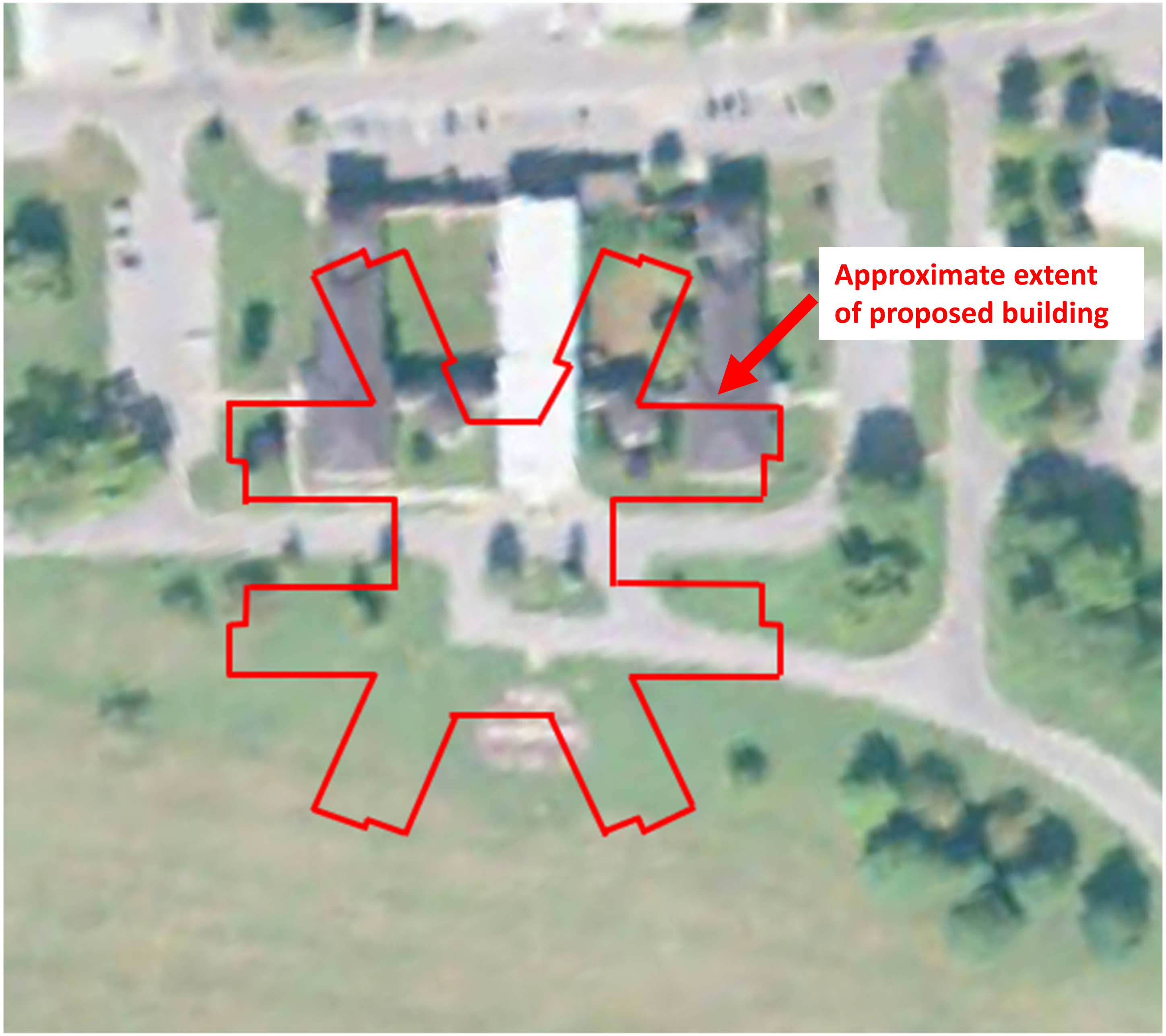
Information contained in this report is based on data obtained from limited subsurface exploration that represents the soil conditions only at the specific location and time investigated, and only to the depth penetrated. Subsurface conditions and groundwater levels at other locations or depths may differ from conditions occurring at the investigated locations. An attempt has been made to provide for normal contingencies, but the possibility remains that unexpected conditions may be encountered during construction.

D.W. Kozera, Inc. considers construction observations and testing of the foundations and earthwork an integral part of the geotechnical design, and therefore, these services should be provided by the Geotechnical Engineer of Record. This is necessary so that we may modify our assumptions and recommendations based on actual conditions that are exposed during construction and observed by us. We cannot assume responsibility or liability for the adequacy of our foundation recommendations if we do not observe the construction

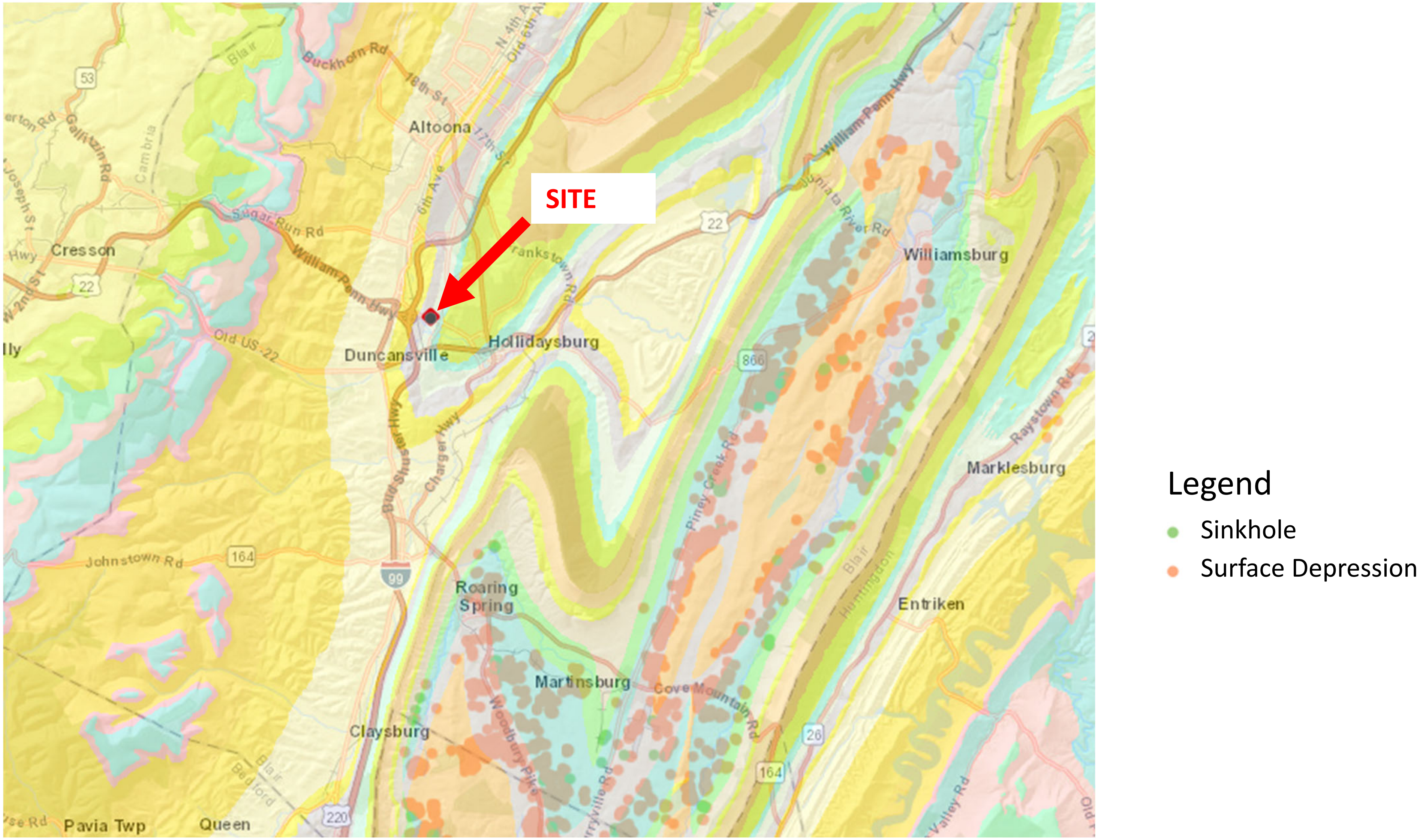
FIGURES

Project: Hollidaysburg Veterans' Home	Drawing: 1.2-1	Project Number:20179.D	Drawn: SC	D.W. Kozera, Inc 1408 Bare Hills Rd, St. 200 Baltimore, MD 21209
Title: Site Vicinity Map	Date: 8/31/21	Notes:		

Base photo: Historic Aerials 2013

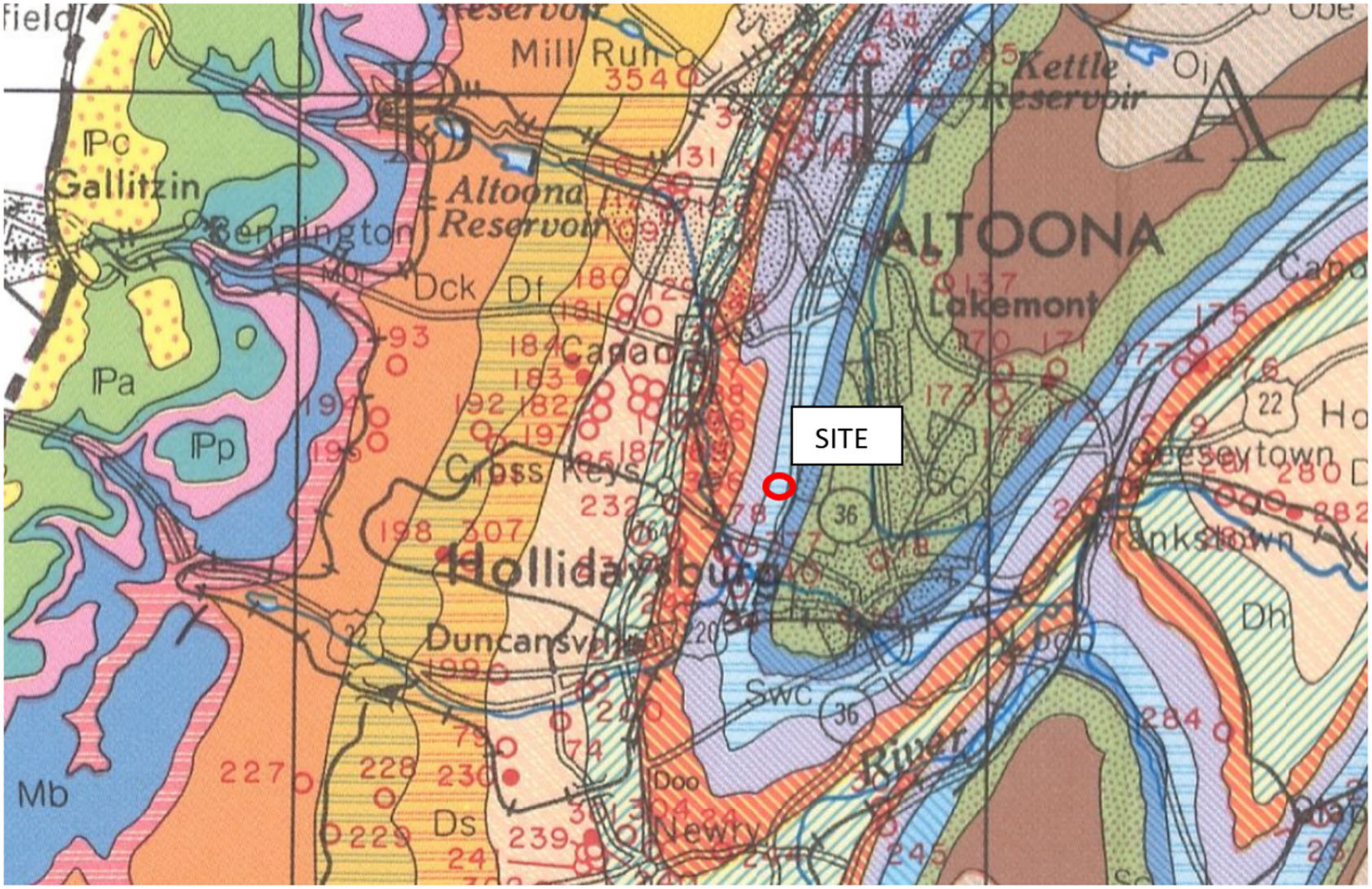


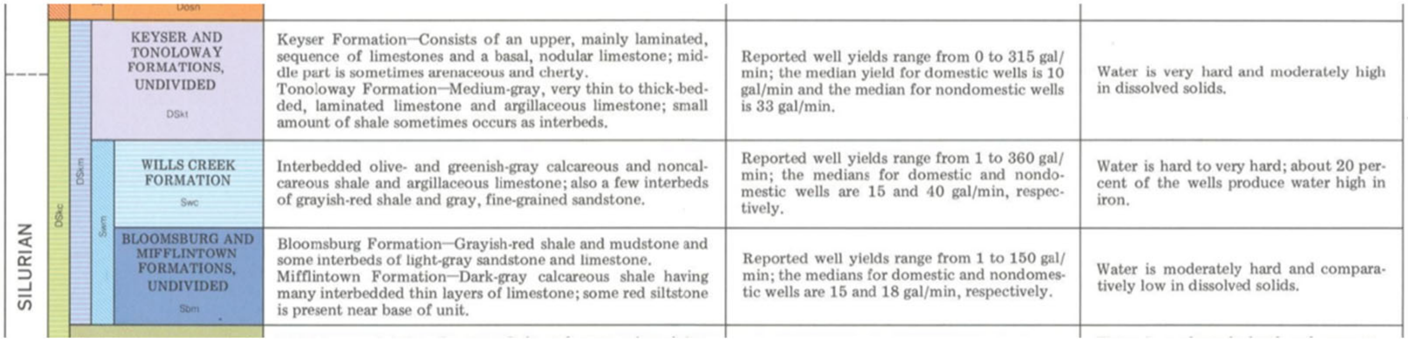
Project: Hollidaysburg Veterans' Home	Drawing: 1.2-2	Project Number:20179.D	Drawn: SC	D.W. Kozera, Inc 1408 Bare Hills Rd, St. 200 Baltimore, MD 21209
Title: Approximate Limits of Previous Development	Date: 8/31/21	Notes:		



From: PaGEODE – Pennsylvania GEOlogic Data Exploration Geological Survey, Pennsylvania Department of Conservation and Natural Resources, accessed 10/1221

Project: Hollidaysburg Veterans' Home	Drawing: 1.3-2	Project Number:20179.D	Drawn: SC	D.W. Kozera, Inc 1408 Bare Hills Rd, St. 200 Baltimore, MD 21209
Title: Map of Known Karst Features	Date: 10/12/21	Notes:		



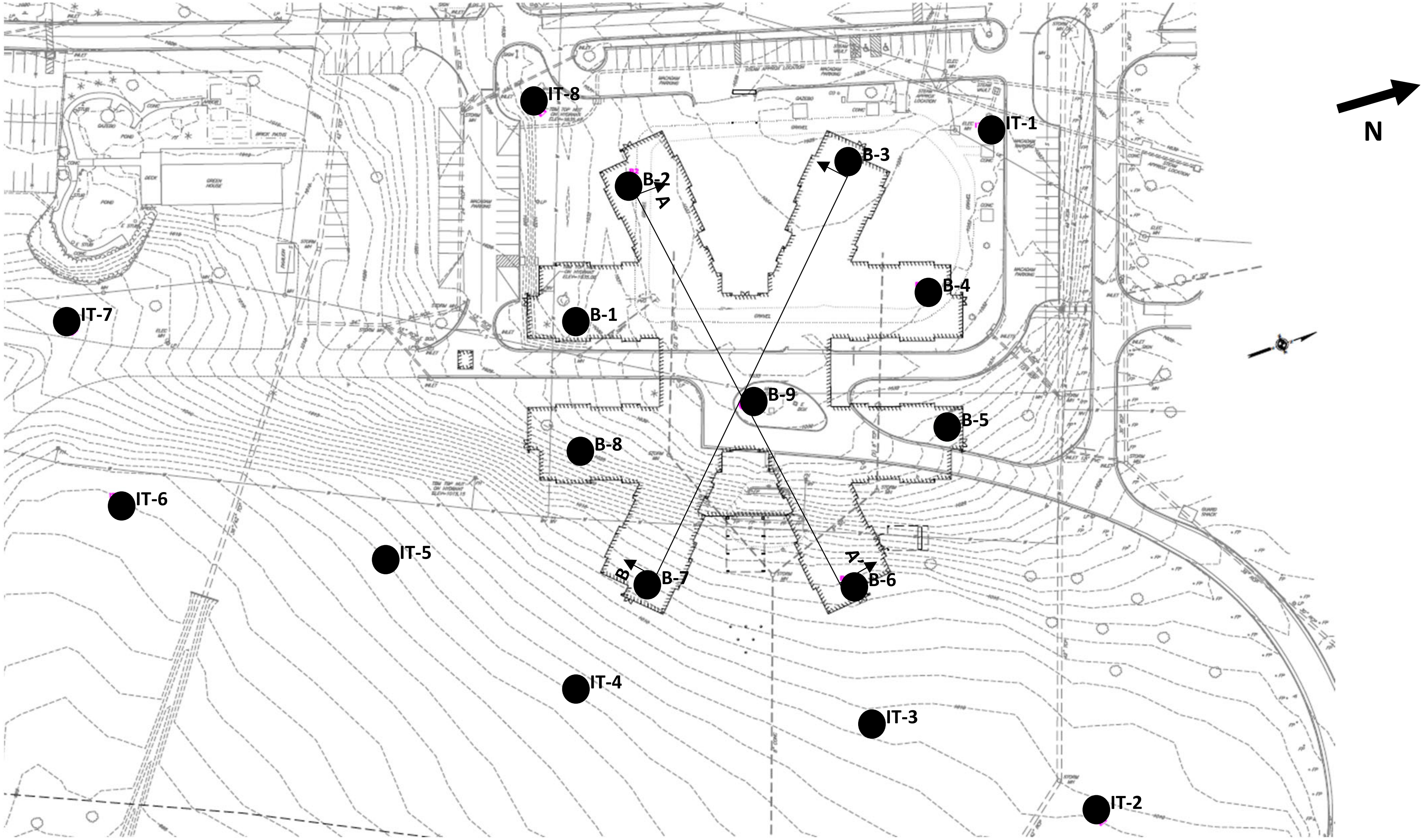


SILURIAN	DSkc	DSkm		KEYSER AND TONOLOWAY FORMATIONS, UNDIVIDED DSkt	Keyser Formation—Consists of an upper, mainly laminated, sequence of limestones and a basal, nodular limestone; middle part is sometimes arenaceous and cherty. Tonoloway Formation—Medium-gray, very thin to thick-bedded, laminated limestone and argillaceous limestone; small amount of shale sometimes occurs as interbeds.	Reported well yields range from 0 to 315 gal/min; the median yield for domestic wells is 10 gal/min and the median for nondomestic wells is 33 gal/min.	Water is very hard and moderately high in dissolved solids.
			Swm	WILLS CREEK FORMATION Swc	Interbedded olive- and greenish-gray calcareous and noncalcareous shale and argillaceous limestone; also a few interbeds of grayish-red shale and gray, fine-grained sandstone.	Reported well yields range from 1 to 360 gal/min; the medians for domestic and nondomestic wells are 15 and 40 gal/min, respectively.	Water is hard to very hard; about 20 percent of the wells produce water high in iron.
				BLOOMSBURG AND MIFFLINTOWN FORMATIONS, UNDIVIDED Sbm	Bloomsburg Formation—Grayish-red shale and mudstone and some interbeds of light-gray sandstone and limestone. Mifflintown Formation—Dark-gray calcareous shale having many interbedded thin layers of limestone; some red siltstone is present near base of unit.	Reported well yields range from 1 to 150 gal/min; the medians for domestic and nondomestic wells are 15 and 18 gal/min, respectively.	Water is moderately hard and comparatively low in dissolved solids.

From: Taylor, L.E., Werkheiser, W.H., duPont, N.S., and Kriz, M.L., 1982, Groundwater resources of the Juniata River basin, Pennsylvania: Pennsylvania Geological Survey, Water Resource Report 54, scale 1:250,000

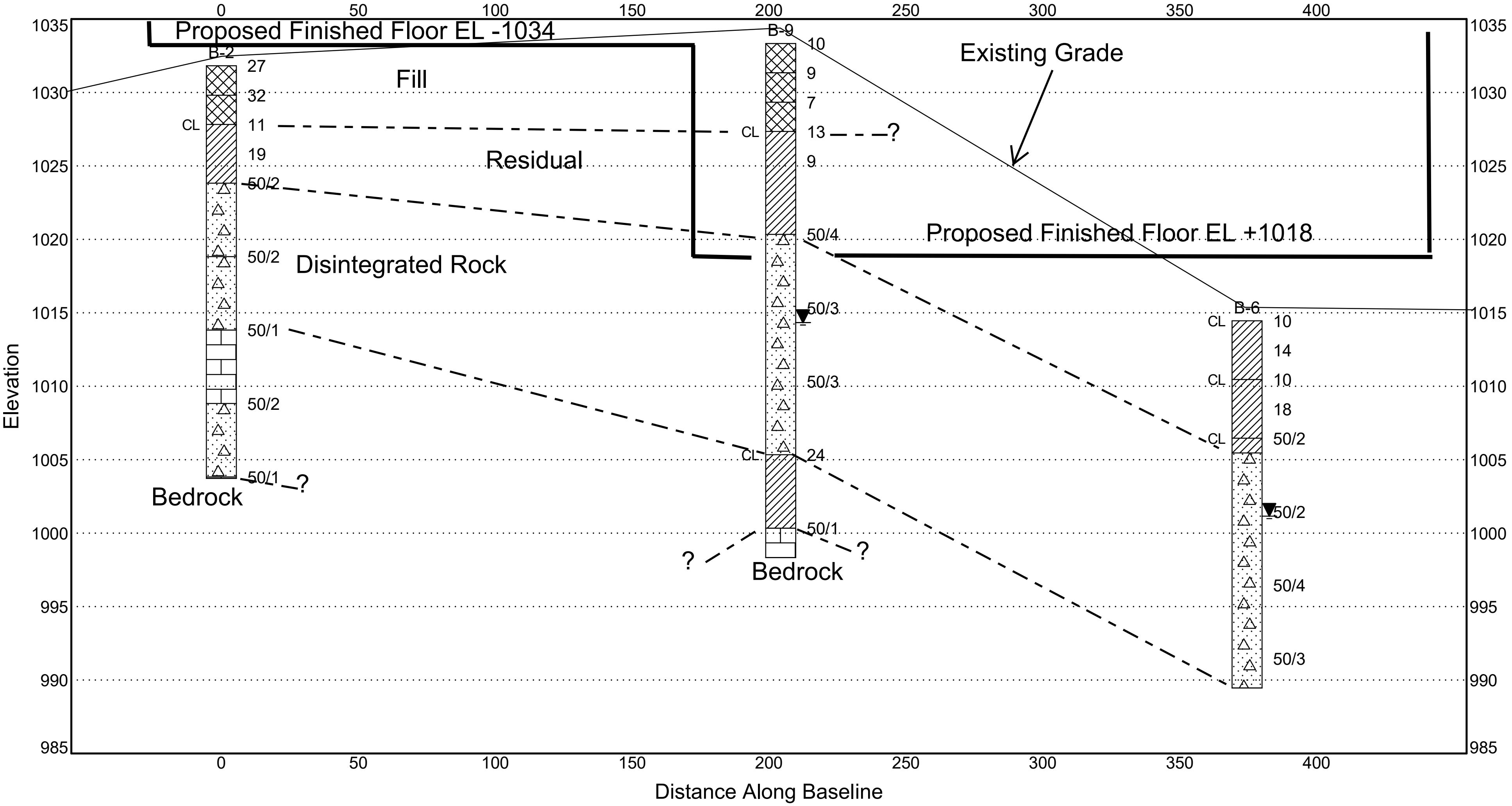
Plate 1: Geologic map of the Juniata River Basin, Pennsylvania, showing the locations of wells and springs — Image provided by Pennsylvania Geological Survey

Project: Hollidaysburg Veterans' Home	Drawing: 1.3-1	Project Number: 20179.D	Drawn: SC	D.W. Kozera, Inc 1408 Bare Hills Rd, St. 200 Baltimore, MD 21209
Title: Local Geology	Date: 10/12/21	Notes:		



Base Plan: Grading Plan, Hollidaysburg Veterans' Home, Sheet C-5, prepared by Keller Engineers, Inc., dated 9/20/21.

Project: HVH	Drawing:	Project Number:20179.D	Drawn: SC	D.W. Kozera, Inc 1408 Bare Hills Rd, St. 200 Baltimore, MD 21209
Title: Test Boring Location Plan	Date: 8/31/21	Notes:		



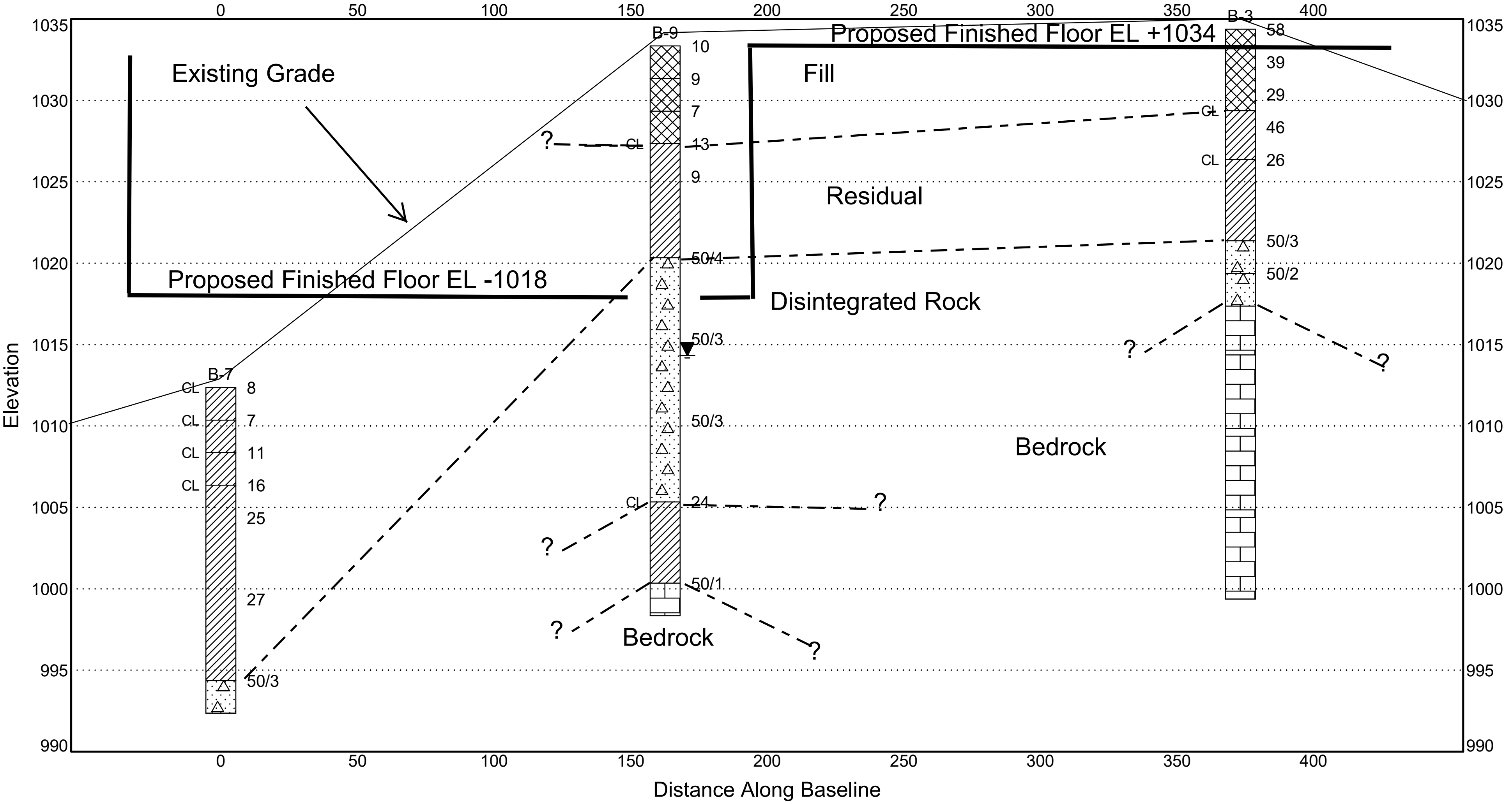
D. W. KOZERA, INC.
Baltimore, Maryland

PROFESSIONAL ENGINEERS & GEOLOGISTS

SUBSURFACE PROFILE
A-A'

Hollidaysburg Veterans Home
138 Veterans Blvd

Contract No.	Date	Drawing No.
20179.D	Nov 21	3.0-1



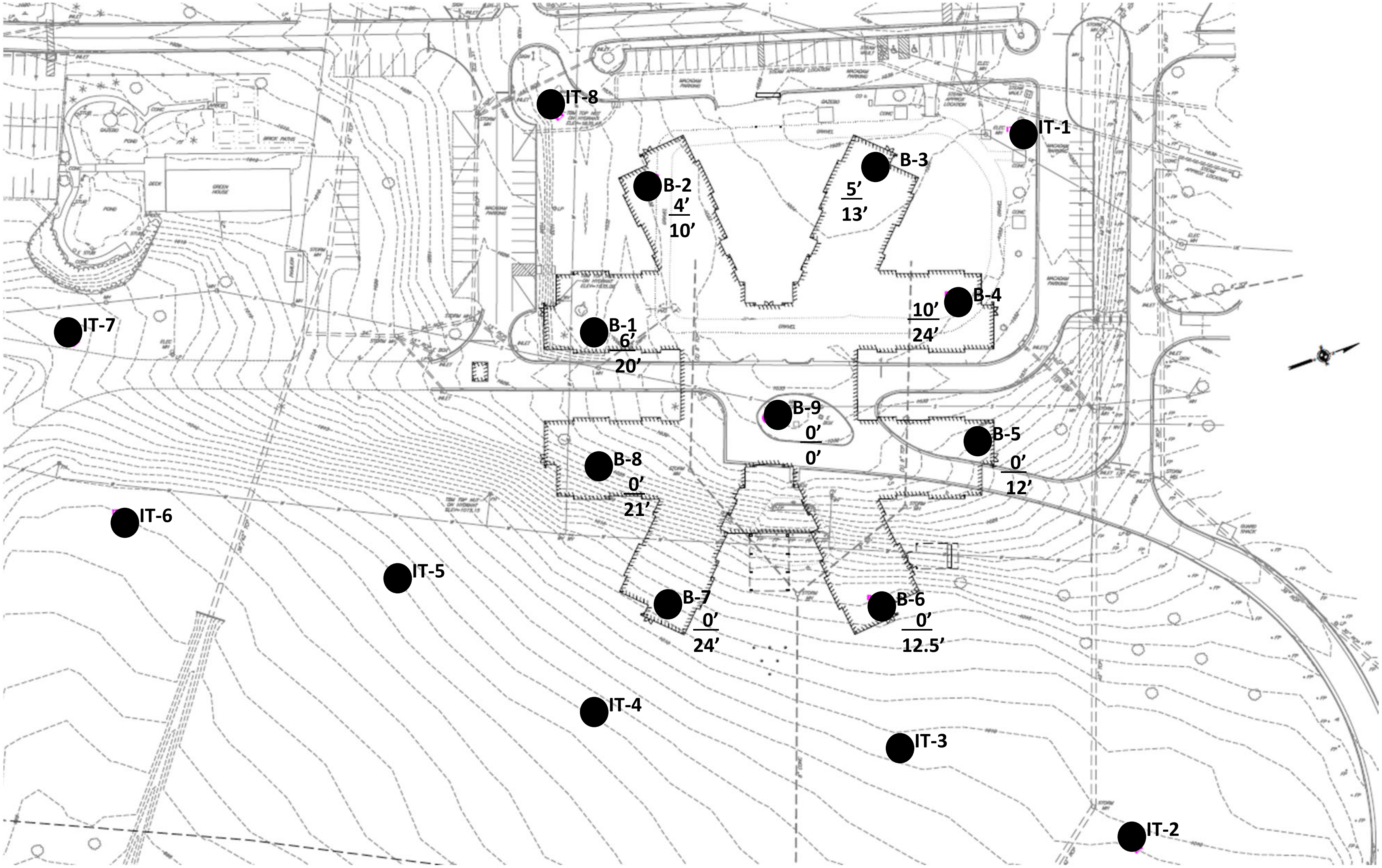
D. W. KOZERA, INC.
Baltimore, Maryland

PROFESSIONAL ENGINEERS & GEOLOGISTS

SUBSURFACE PROFILE
B-B'

Hollidaysburg Veterans Home
138 Veterans Blvd

Contract No.	Date	Drawing No.
20179.D	Nov 21	3.0-2

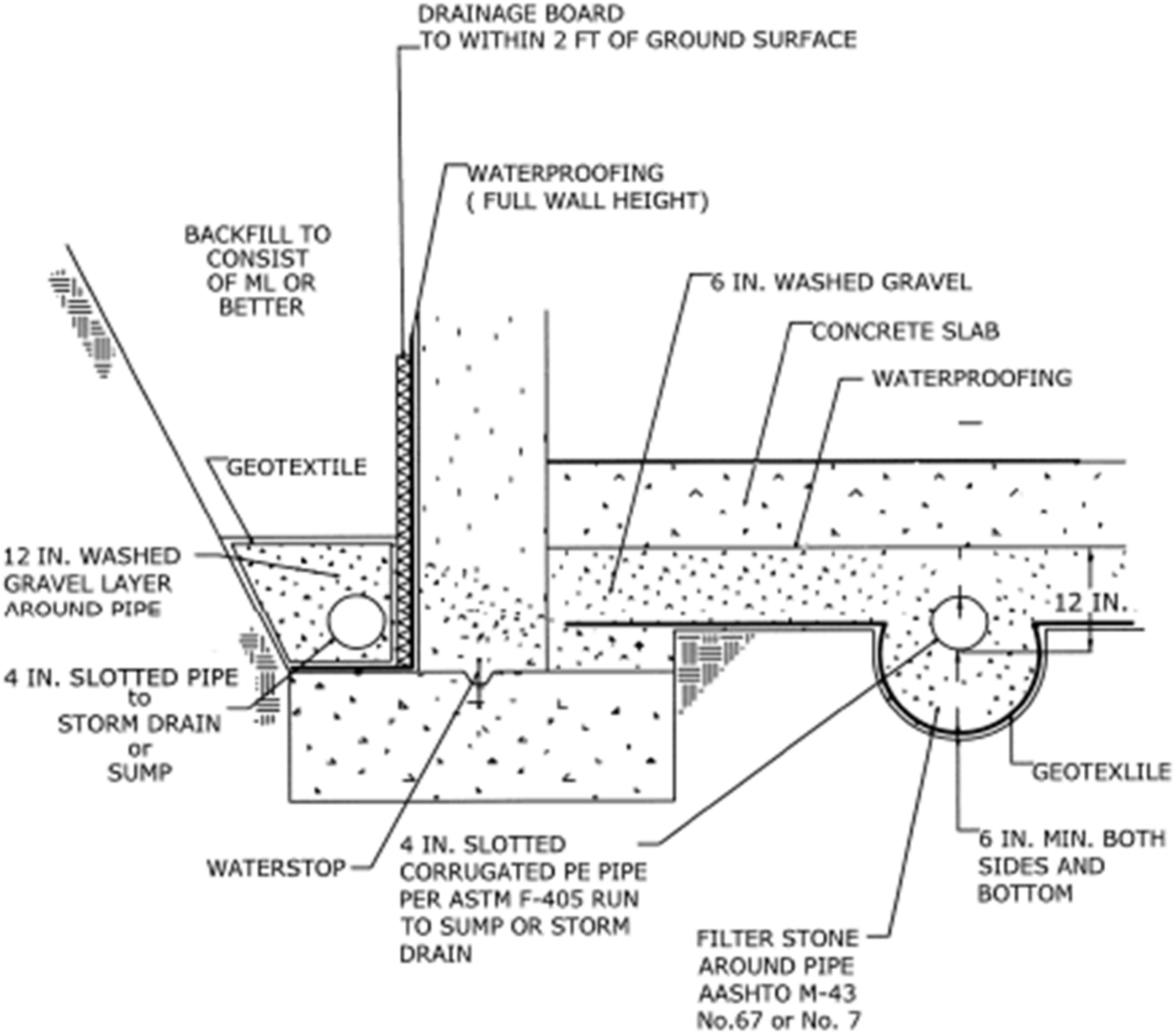


Base Plan: Grading Plan, Hollidaysburg Veterans' Home, Sheet C-5, prepared by Keller Engineers, Inc., dated 9/20/21.

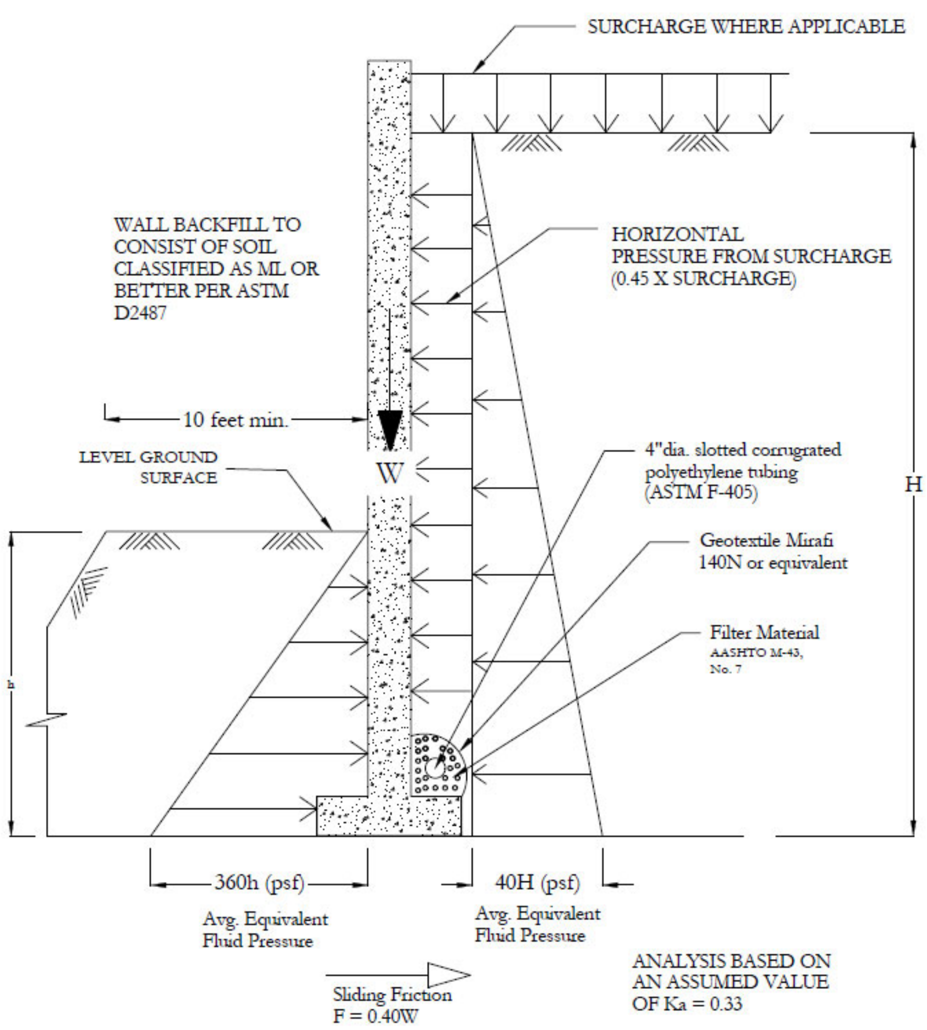
Legend

●	**Test Boring Location**
X'_	**Depth of Man-placed Fill Requiring Remediation**
X'	**Depth to Disintegrated Rock from Proposed Finished Floor Elevation (ft)**

Project: HVH	Drawing: 4.0-1	Project Number:20179.D	Drawn: SC	D.W. Kozera, Inc 1408 Bare Hills Rd, St. 200 Baltimore, MD 21209
Title: Fill and Overburden Depth	Date: 8/31/21	Notes:		

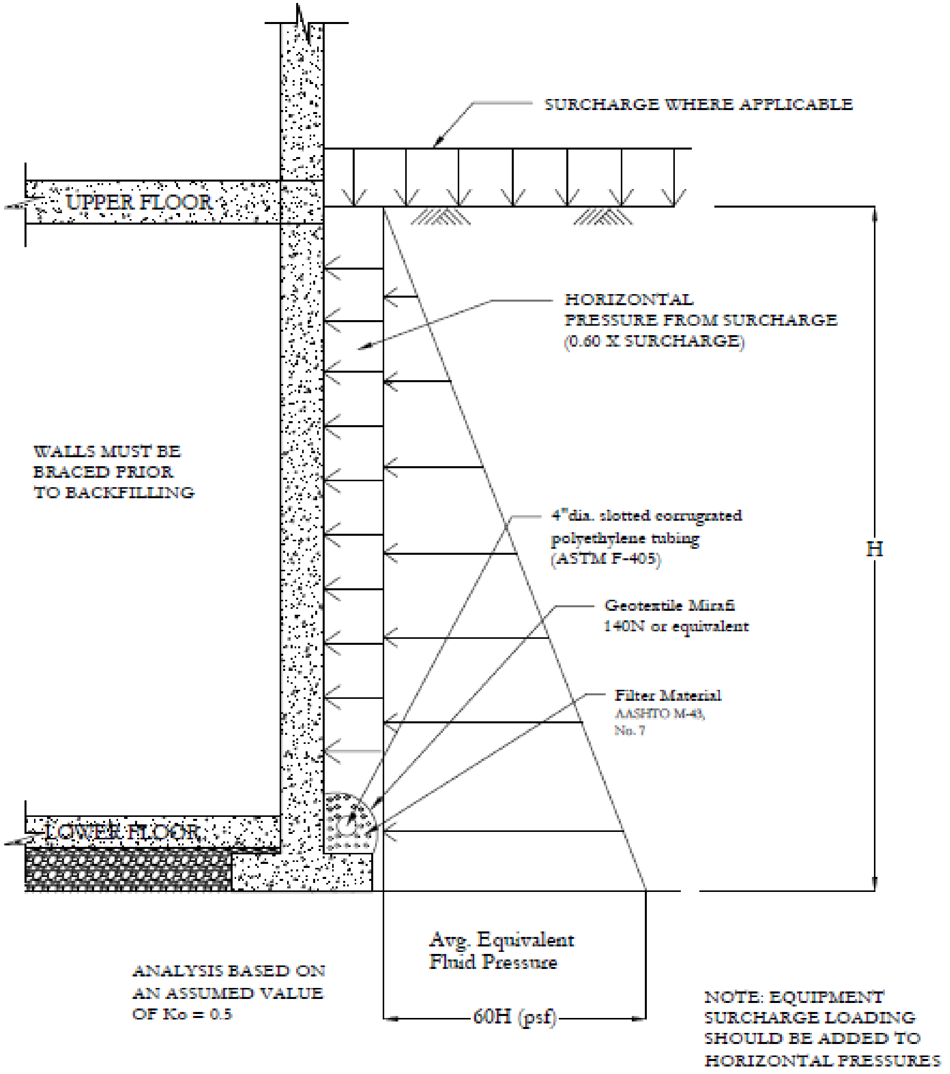


Project: Hollidaysburg Veterans' Home	Drawing: 8.1-1	Project Number:20179.D	Drawn: SC	D.W. Kozera, Inc 1408 Bare Hills Rd, St. 200 Baltimore, MD 21209
Title: Subdrainage Detail	Date: 10/12/21	Notes:		



CANTILEVERED WALLS

NOTE: LATERAL EARTH PRESURE DIAGRAMS DO NOT INCLUDE HYDROSTATIC PRESSURES



BRACED WALLS

(BACKFILLED AFTER CONSTRUCTION)

NOTE: LATERAL EARTH PRESURE DIAGRAMS DO NOT INCLUDE HYDROSTATIC PRESSURES

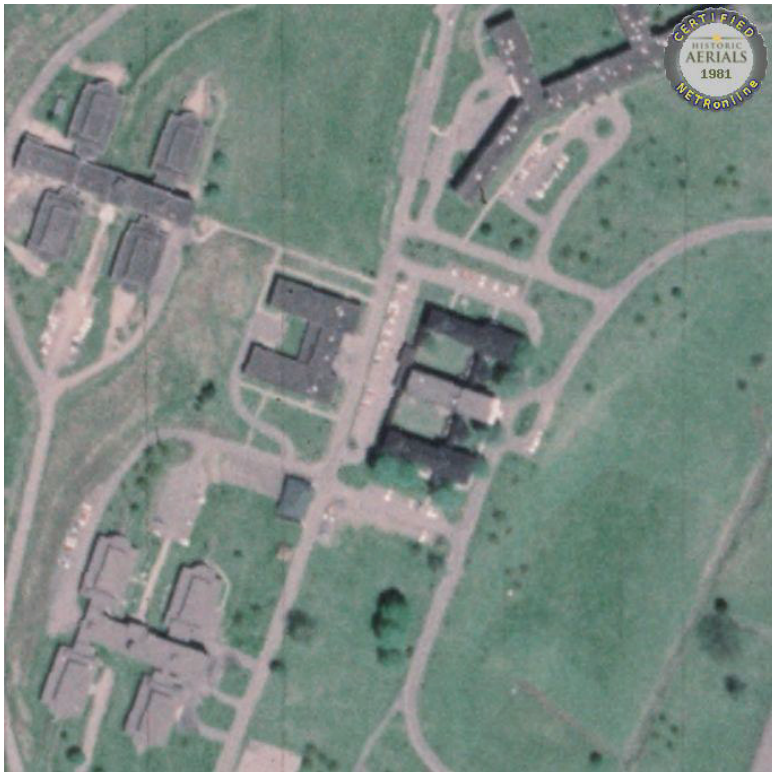
Project: Hollidaysburg Veterans' Home	Drawing: 9.1-1	Project Number:20179.D	Drawn: SC	D.W. Kozera, Inc 1408 Bare Hills Rd, St. 200 Baltimore, MD 21209
Title: Lateral Earth Pressures	Date: 12/20/21	Notes:		

APPENDIX A

Historic Aerial Photographs

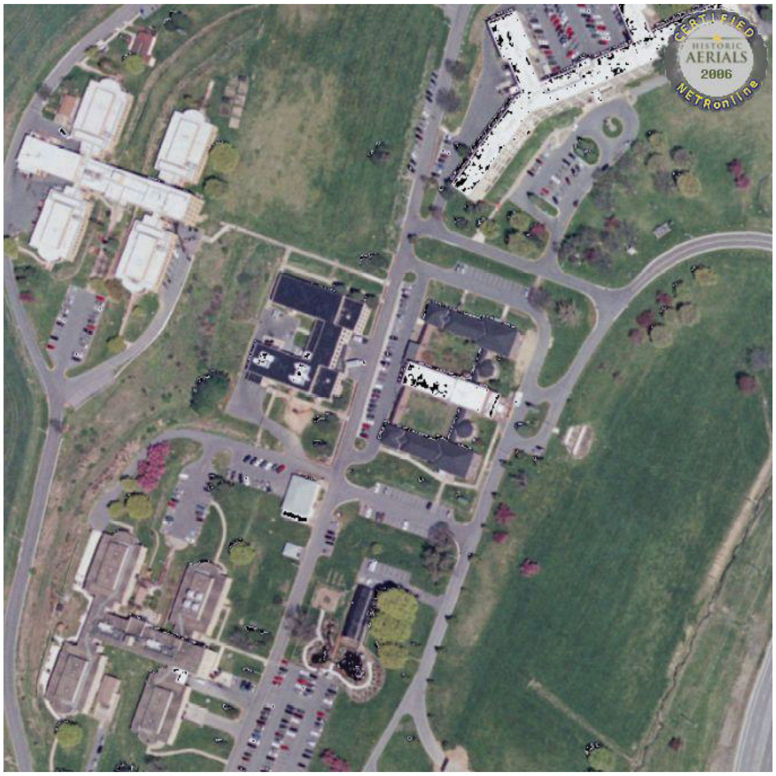

CERTIFIED
HISTORIC
AERIALS
1962
NETRonline

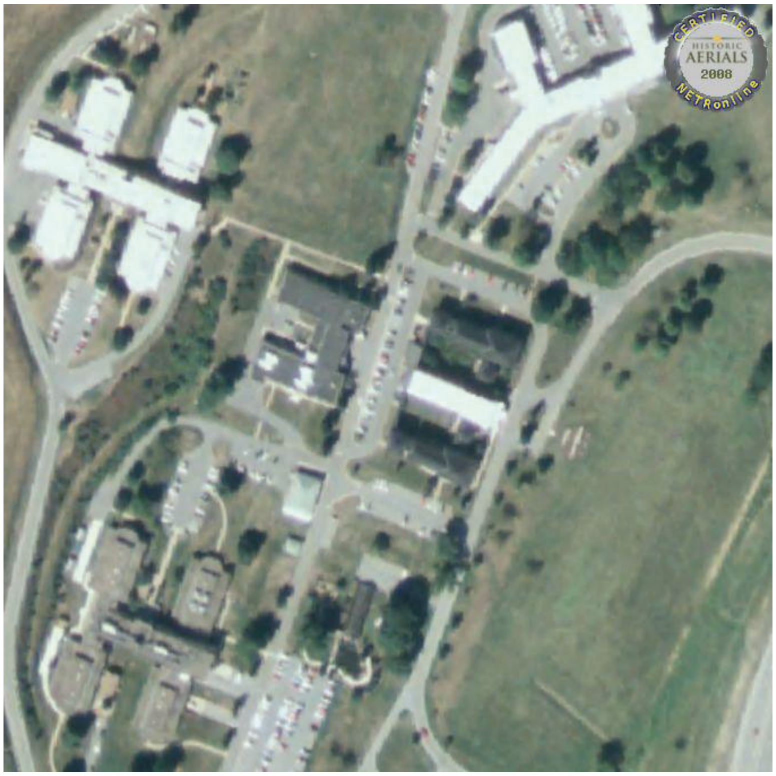

CERTIFIED
HISTORIC
AERIALS
1972
NETRonline


CERTIFIED
HISTORIC
AERIALS
1981
NETRonline


CERTIFIED
HISTORIC
AERIALS
1993
NETRonline


CERTIFIED
HISTORIC
AERIALS
2004
NETRonline


CERTIFIED
HISTORIC
AERIALS
2006
NETRonline


CERTIFIED
HISTORIC
AERIALS
2008
NETRonline


CERTIFIED
HISTORIC
AERIALS
2010
NETRonline


CERTIFIED
HISTORIC
AERIALS
2013
NETRonline


CERTIFIED
HISTORIC
AERIALS
2015
NETRonline


CERTIFIED
HISTORIC
AERIALS
2017
NETRonline

APPENDIX B

Subsurface Investigation

GENERAL NOTES FOR TEST BORINGS AND TEST PITS
Geotechnical Engineering Study, Hollidaysburg Veterans' Home
Hollidaysburg, Pennsylvania
(DWK Contract Number 20179.D)

1. Test Borings

Test borings are advanced by turning an auger with a center opening of 2-1/2 or 3-1/4 inches. Cuttings are brought to the surface by the auger flights. Sampling is performed through the center opening in the hollow stem auger by standard methods. No water was introduced into the borings using this procedure.

1.1. Standard Penetration Tests

Testing is performed by driving a two-inch O.D., 1-3/8-inch I.D. sampling spoon through three, six-inch intervals or as indicated, using a 140-pound hammer falling 30-inches according to ASTM D1586. The number given as the 'N' value is the sum of the blows required to drive the samples for the second and third intervals.

2. Test Pits

Test pits are logged to provide a record for geotechnical evaluation, construction inspection, or other specialized purpose such as building damage investigations, subgrade inspections, etc.

2.1. Test Procedures

PP, when indicated, denotes the results of tests performed with a Pocket Penetrometer. The numbers indicate the unconfined compressive strength of the undisturbed soils in tsf. DCP, when indicated, denotes the results of tests performed with a Dynamic Cone Penetrometer at an initial seating increment of two-inches, and 1-3/4-inch increments thereafter. The penetrometer is driven by a 15-pound hammer falling 20-inches, and the number of hammer blows per increment is recorded.

3. General

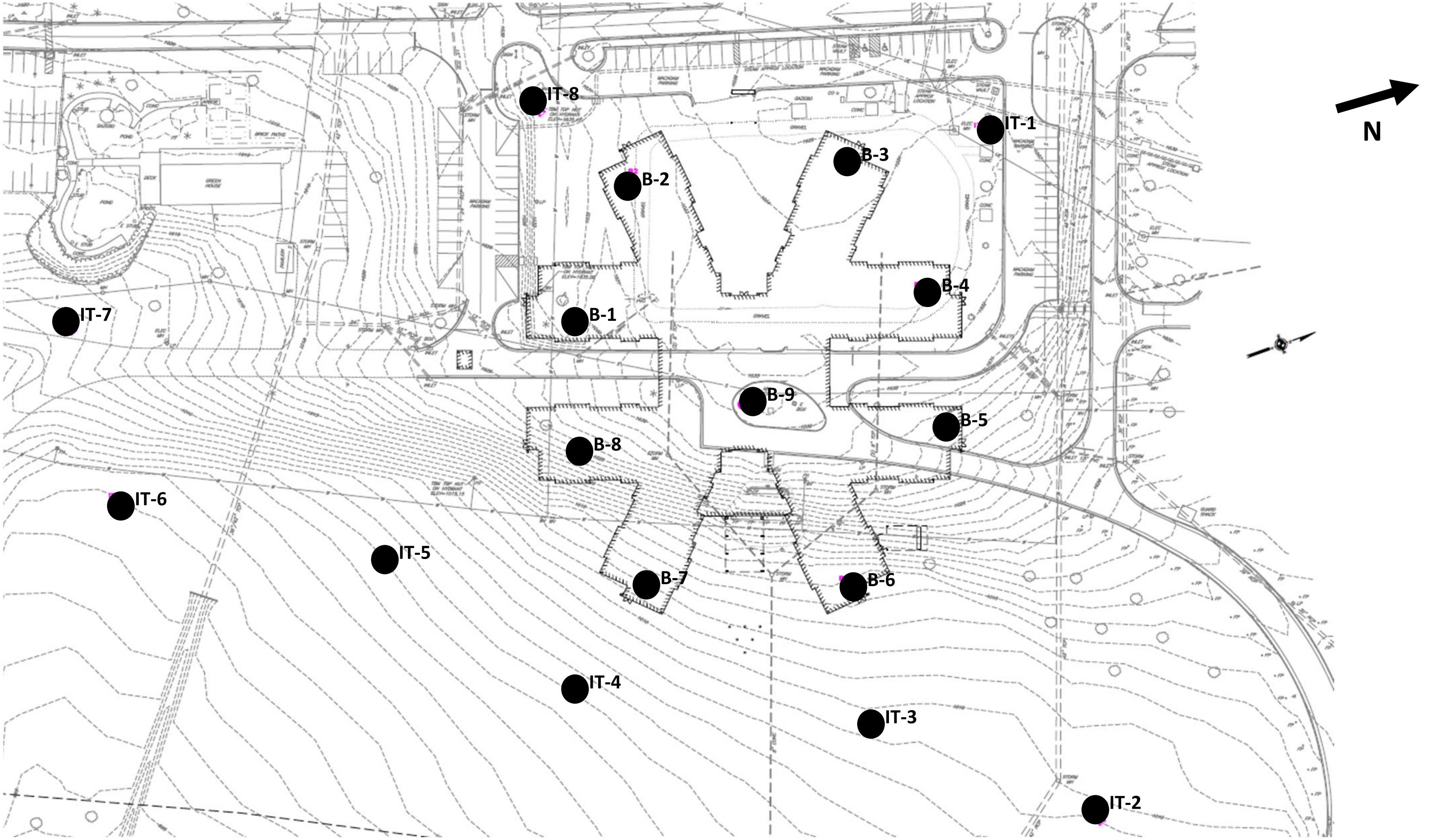
The test pits and test boring logs represent subsurface conditions only at the specified location and at the particular time excavated. The passage of time may result in changes in these conditions. Conditions at other locations on the site may differ from conditions occurring at the test pit or test boring location.

The stratification lines represent the approximate boundary line between soil and rock types as observed in the test pit and test boring. The soil profile, foundation dimensions, water level observations, and test results presented on the log have been made with reasonable care and accuracy but must be considered only an approximate representation of the subsurface conditions to be encountered at that particular location.

The observed water levels are considered a reliable indication of the groundwater table levels at the time indicated. The groundwater table may be completely dependent on the amount of precipitation at the site during a particular period of time. Fluctuations in the water table should be expected with variations in precipitation, surface run-off, evaporation, construction activity, etc.

4. Locations and Grades

The test borings were located in the field by Keller Engineers, Inc. who also provided ground surface elevations.



Base Plan: Grading Plan, Hollidaysburg Veterans' Home, Sheet C-5, prepared by Keller Engineers, Inc., dated 9/20/21.

Project: HVH	Drawing:	Project Number:20179.D	Drawn: SC	D.W. Kozera, Inc 1408 Bare Hills Rd, St. 200 Baltimore, MD 21209
Title: Test Boring Location Plan	Date: 8/31/21	Notes:		

D. W. KOZERA, INC.
Baltimore, Maryland
PROFESSIONAL ENGINEERS & GEOLOGISTS

TEST BORING LOG

Boring No.: B-1
Contract No.: 20179.D
Page: 1 of 1

Project: Hollidaysburg Veterans Home
Location: 138 Veterans Blvd
Ducansville, PA

Ground Surf. El. (±) : 1031.9
Date Started : 9-17-21
Date Completed : 9-17-21
Contractor : Echelberger
Driller : Ben Hurley
Rig : Diedrch D-50
Drill Method : 3" Casing/Rollerbit Auto Hammer
Inspector : E. Kussman

	GROUNDWATER OBSERVATIONS				
	Date	Time	Depth	Casing	Caved
Encountered	9-17	12:33	DRY	15.0	---
Completion	9-17	12:34	---	15.0	---
Casing Pulled	9-17	16:00	17.5	---	19.0
24-Hr Reading	9-20	08:15	DRY	---	5.0

Depth (ft)	Surf. Elev. 1031.9	Samples	Blow Counts	"N" Value	Water Level	Graphic	USCS	Description	Formation	Stratum	Remarks
0		1	3-5-7-2	12		▨		clay, brick and rock fragments, FILL, moist, red	Fill	A	Topsoil-3"
	1030	2	3-3-4-5	7		▨					No Recovery
5		3	5-3-5-6	8		▨		brick and rock fragments, FILL, moist, red			
	1025	4	6-7-10-12	17		▨	CL	SANDY LEAN CLAY, moist, light brown	Residual	B	
		5	10-13-13-7	26		▥	ML	SANDY SILT w/ rock fragments, moist, light brown			
10						▥					
	1020					▥					
		6	3-4-3-2	7		▥	ML	SILT w/ rock fragments, moist, light brown			w/c 33.3%
15						▥					
	1015					▥					
		7	50/2	50/2		△		DISINTEGRATED ROCK, moist, light brown	Disintegrated Rock	C	
20						△					
	1010					△					
		8	50/3	50/3		△		DISINTEGRATED ROCK, moist, grayish brown			
25						△					
		9	50/1	50/1				BEDROCK Bottom of Test Boring @ 28.1'	*	D	*Bedrock

D. W. KOZERA, INC.
Baltimore, Maryland

PROFESSIONAL ENGINEERS & GEOLOGISTS

TEST BORING LOG

Boring No.: B-2
Contract No.: 20179.D
Page: 1 of 1

Project: Hollidaysburg Veterans Home
Location: 138 Veterans Blvd
Ducansville, PA

Ground Surf. El. (±)	: 1031.8				
Date Started	: 9-17-21				
Date Completed	: 9-17-21				
Contractor	: Echelberger				
Driller	: Ben Hurley				
Rig	: Diedrch D-50				
Drill Method	: 3" Casing/Rollerbit Auto Hammer				
Inspector	: E. Kussman				

	GROUNDWATER OBSERVATIONS				
	Date	Time	Depth	Casing	Caved
Encountered	9-17	10:29	DRY	15.0	---
Completion	9-17	10:30	---	---	---
Casing Pulled	9-17	16:02	DRY	---	18.7
24-Hr Reading	9-20	08:17	DRY	---	18.7

Depth (ft)	Surf. Elev. 1031.8	Samples	Blow Counts	"N" Value	Water Level	Graphic	USCS	Description	Formation	Stratum	Remarks
0		1	3-14-13-10	27		▩		brick and rock fragment. FILL, moist, red	Fill	A	Topsoil-2"
	1030	2	10-17-15-9	32		▩		sand w/ brick and rock fragments, FILL, moist, red			
5		3	5-4-7-9	11		▨	CL	LEAN CLAY, moist, light brown	Residual	B	
	1025	4	8-9-10-11	19		▨					
		5	16-35-50/2	50/2		△		DISINTEGRATED ROCK, moist, light brown	Disintegrated Rock	C	
10	1020					△					
		6	18-50/2	50/2		△		SAA			
15	1015					△					
		7	50/1	50/1		▦		BEDROCK, moist, grayish brown			
20	1010					▦					
		8	50/2	50/2		△		SAA			
25	1005					△					
		9	50/1	50/1				BEDROCK Bottom of Test Boring @ 28.1'	*	D	*Bedrock
30											

D. W. KOZERA, INC.
Baltimore, Maryland
PROFESSIONAL ENGINEERS & GEOLOGISTS

TEST BORING LOG

Boring No.: B-3
Contract No.: 20179.D
Page: 1 of 1

Project: Hollidaysburg Veterans Home
Location: 138 Veterans Blvd
Ducansville, PA

Ground Surf. El. (±) : 1034.4
Date Started : 9-16-21
Date Completed : 9-17-21
Contractor : Echelberger
Driller : Ben Hurley
Rig : Diedrch D-50
Drill Method : 3 1/4" HSA Auto Hammer
Inspector : E. Kussman

	GROUNDWATER OBSERVATIONS				
	Date	Time	Depth	Casing	Caved
Encountered	9-16	17:00	DRY	15.0	---
Completion	9-16	17:02	DRY	15.0	---
Casing Pulled	9-17	07:30	DRY	---	121.0
24-Hr Reading	9-20	08:19	DRY	---	12.0

Depth (ft)	Surf. Elev. 1034.4	Samples	Blow Counts	"N" Value	Water Level	Graphic	USCS	Description	Formation	Stratum	Remarks
0		1	5-25-33-24	58		▩		brick and rock fragments, FILL, moist, red	Fill	A	Topsoil-2"
		2	8-21-18-18	39		▩					
5	1030	3	11-16-13-14	29		▨	CL	CLAY w/ rock fragments, moist, gray	Residual	B	
		4	19-22-24-22	46		▨					
	1025	5	19-16-10-8	26		▨	CL	SANDY CLAY w/ rock fragments, moist, orangish brown			
10						▨					
	1020	6	35-50/3	50/3		△		DISINTEGRATED ROCK, moist, gray	*	C	*Disintegrated Rock
15		7	50/2	50/2		△		DISINTEGRATED ROCK, moist, orangish brown			Auger Refusal @ 15'
		8	NQ			▦		LIMESTONE, brown, gray, soft, severly weathered, extremely fractured,reacts with 10% HCL when scratched Rec: 46" RQD: 13, 3%	Bedrock	D	Time = 11:03 min
20	1015	9	NQ			▦		LIMESTONE, gray to brown/gray, moderatley hard, moderatley weathered, moderatley fractured, reacts with 10% HCL when scratched Rec: 54" RQD: 40%			Time = 7:12 min
25	1010	10	NQ			▦		LIMESTONE, gray to blue gray, moderatley hard, moderatley weathered, slightly fractured, reacts with 10% HCL when scratched Rec: 58" RQD: 68.3%			Time = 8:42 min
30	1005	11	NQ			▦		LIMESTONE, gray to blue gray, moderatley hard, moderatley weathered, slightly fractured, reacts with 10% HCL when scratched Rec: 59" RQD: 58.3%			Time = 9:15 min
35	1000							Bottom of Test Boring @ 35'			

D. W. KOZERA, INC.
Baltimore, Maryland
PROFESSIONAL ENGINEERS & GEOLOGISTS

TEST BORING LOG

Boring No.: B-4
Contract No.: 20179.D
Page: 1 of 1

Project: Hollidaysburg Veterans Home
Location: 138 Veterans Blvd
Ducansville, PA

Ground Surf. El. (±) : 1032.9
Date Started : 9-17-21
Date Completed : 9-20-21
Contractor : Echelberger
Driller : Ben Hurley
Rig : Diedrch D-50
Drill Method : 3" Casing/Rollerbit Auto Hammer
Inspector : E. Kussman

	GROUNDWATER OBSERVATIONS				
	Date	Time	Depth	Casing	Caved
Encountered	9-20	10:00	DRY	18.0	---
Completion	9-20	10:09	DRY	18.0	---
Casing Pulled	9-20	16:01	DRY	---	19.0
24-Hr Reading	9-21	00:00	DRY	---	19.0

Depth (ft)	Surf. Elev. 1032.9	Samples	Blow Counts	"N" Value	Water Level	Graphic	USCS	Description	Formation	Stratum	Remarks
0								clay, brick and rock fragments, FILL, wet, red	Fill	A	Topsoil-3"
		1	3-16-41-29	57							
	1030	2	21-27-50/1	50/1							
5		3	8-27-50/2	50/2							
		4	13-17-11-7	28							
	1025										
		5	15-9-6-5	11							
10								LEAN CLAY w/ rock fragments, moist, light brown	Residual	B	
		6	8-8-11-8	29			CL				
	1020							SILTY CLAY WITH SAND w/ rock fragments, moist, brown			w/c 18.4%
		7	7-2-6-8	8							
15											
	1015						CL ML				
		8	6-16-19-14	35							
20											
	1010										
		9	22-20-50/2	50/2				DISINTEGRATED ROCK, moist, brown	*	C	*Disintegrated Rock
25								Bottom of Test Boring @ 25'			

D. W. KOZERA, INC.
Baltimore, Maryland
PROFESSIONAL ENGINEERS & GEOLOGISTS

TEST BORING LOG

Boring No.: B-5
Contract No.: 20179.D
Page: 1 of 1

Project: Hollidaysburg Veterans Home
Location: 138 Veterans Blvd
Ducansville, PA

Ground Surf. El. (±) : 1029.0
Date Started : 9-15-21
Date Completed : 9-15-21
Contractor : Echelberger
Driller : Ben Hurley
Rig : Diedrch D-50
Drill Method : 3 1/4" HSA Auto Hammer
Inspector : E. Kussman

	GROUNDWATER OBSERVATIONS				
	Date	Time	Depth	Casing	Caved
Encountered	9-15	16:40	DRY	33.0	---
Completion	9-15	16:42	DRY	33.0	---
Casing Pulled	9-15	17:00	DRY	--	PIPE
24-Hr Reading	9-16	12:00	31.0	---	PIPE
	9-17	11:21	31.0	---	PIPE

Depth (ft)	Surf. Elev. 1029.0	Samples	Blow Counts	"N" Value	Water Level	Graphic	USCS	Description	Formation	Stratum	Remarks
0		1	2-2-5-4	7		▩		clay w/ rock fragments, FILL, moist, brown	Fill	A	Topsoil-4"
	1025	2	5-6-6-7	12		▨	CL	SANDY LEAN CLAY, moist, orangish brown	Residual	B	
5		3	2-3-3-4	6		▨					
		4	5-5-5-4	10		▨					
	1020	5	4-3-4-3	7		▨					
10						▨					
	1015	6	4-4-6-11	10		▨	CL	SANDY LEAN CLAY, moist, light brown			w/c 19.1%
15						▨					
	1010	7	4-3-3-2	6		▨	CL	LEAN CLAY, moist, orange/gray			
20						▨					
	1005	8	50/2	50/2		△		DISINTEGRATED ROCK, moist, light brown	*	C	*Disintegrated Rock
25						△					
	1000	9	8-10-8-12	18		▨	SC	CLAYEY SAND, moist, grayish brown	Residual	B	
30					▼	▨					
	995	10	8-7-6-9	13		▨	CL	LEAN CLAY, moist, light brown			
35								Bottom of Test Boring @ 35'			Installed piezometer upon completion

D. W. KOZERA, INC.
Baltimore, Maryland
PROFESSIONAL ENGINEERS & GEOLOGISTS

TEST BORING LOG

Boring No.: B-6
Contract No.: 20179.D
Page: 1 of 1

Project: Hollidaysburg Veterans Home
Location: 138 Veterans Blvd
Ducansville, PA

Ground Surf. El. (±) : 1014.5
Date Started : 9-20-21
Date Completed : 9-21-21
Contractor : Echelberger
Driller : Ben Hurley
Rig : Diedrch D-50
Drill Method : 3" Casing/Rollerbit Auto Hammer
Inspector : E. Kussman

	GROUNDWATER OBSERVATIONS				
	Date	Time	Depth	Casing	Caved
Encountered	9-21	08:31	DRY	13.0	---
Completion	9-21	09:40	--	---	---
Casing Pulled	9-21	09:50	13.3	---	20.0
24-Hr Reading					

Depth (ft)	Surf. Elev. 1014.5	Samples	Blow Counts	"N" Value	Water Level	Graphic	USCS	Description	Formation	Stratum	Remarks
0		1	3-4-6-5	10		▨	CL	LEAN CLAY, moist, brown	Residual	B	Topsoil-3"
		2	5-6-8-9	14		▨					w/c 16.7%
5	1010	3	3-3-7-7	10		▨	CL	LEAN CLAY w/ rock fragments, moist, brown			
		4	8-5-13-25	18		▨					
		5	21-24-50/2	50/2		▨	CL	SANDY LEAN CLAY w/ rock fragments, moist, brown	Disintegrated Rock	C	
	1005					△		DISINTEGRATED ROCK, moist, brown			
10						△					
		6	32-50/2	50/2	▼	△					
15	1000					△					
		7	15-50/4	50/4		△					
20	995					△					
		8	50/3	50/3		△					
25	990							Bottom of Test Boring @ 25'			

D. W. KOZERA, INC.
Baltimore, Maryland
PROFESSIONAL ENGINEERS & GEOLOGISTS

TEST BORING LOG

Boring No.: B-7
Contract No.: 20179.D
Page: 1 of 1

Project: Hollidaysburg Veterans Home
Location: 138 Veterans Blvd
Ducansville, PA

Ground Surf. El. (±) : 1012.4
Date Started : 9-15-21
Date Completed : 9-15-21
Contractor : Echelberger
Driller : Ben Hurley
Rig : Diedrch D-50
Drill Method : 3 1/4" HSA Auto Hammer
Inspector : E. Kussman

	GROUNDWATER OBSERVATIONS				
	Date	Time	Depth	Casing	Caved
Encountered	9-15	11:55	DRY	18.0	---
Completion	9-15	11:56	DRY	18.0	---
Casing Pulled	9-15	12:07	DRY	---	PIPE
24-Hr Reading	9-16	11:52	DRY	---	PIPE
	9-17	11:47	DRY	---	PIPE

Depth (ft)	Surf. Elev. 1012.4	Samples	Blow Counts	"N" Value	Water Level	Graphic	USCS	Description	Formation	Stratum	Remarks
0		1	3-3-5-5	8		▨	CL	SANDY LEAN CLAY, moist, brown	Residual	B	Topsoil-3"
	1010	2	7-8-9-9	7		▨	CL	SANDY LEAN CLAY, moist, organish brown			
5		3	4-6-5-6	11		▨	CL	LEAN CLAY, moist, orangish brown			
	1005	4	1-7-9-10	16		▨	CL	SANDY LEAN CLAY, moist, gray			
10		5	13-13-12-10	25		▨					
	1000					▨					
15		6	5-10-17-23	27		▨					
	995					▨					
		7	11-17-23-50/3	50/3		△		DISINTEGRATED ROCK, moist, gray	*	C	*Disintegrated Rock
20								Bottom of Test Boring @ 20'			

D. W. KOZERA, INC.
Baltimore, Maryland
PROFESSIONAL ENGINEERS & GEOLOGISTS

TEST BORING LOG

Boring No.: B-8
Contract No.: 20179.D
Page: 1 of 1

Project: Hollidaysburg Veterans Home
Location: 138 Veterans Blvd
Ducansville, PA

	GROUNDWATER OBSERVATIONS				
	Date	Time	Depth	Casing	Caved
Encountered	9-20	13:30	DRY	18.0	---
Completion	9-20	13:31	DRY	18.0	---
Casing Pulled	9-20	16:03	23.0	---	28.5
24-Hr Reading	9-21	09:06	DRY	---	28.5

Ground Surf. El. (±) : 1025.7
Date Started : 9-20-21
Date Completed : 9-20-21
Contractor : Echelberger
Driller : Ben Hurley
Rig : Diedrch D-50
Drill Method : 3" Casing/Rollerbit Auto Hammer
Inspector : E. Kussman

Depth (ft)	Surf. Elev. 1025.7	Samples	Blow Counts	"N" Value	Water Level	Graphic	USCS	Description	Formation	Stratum	Remarks
0	1025	1	4-5-4-6	9		▦		sandy clay, brick fragments, FILL, moist, brown	Fill	A	Topsoil-3"
		2	5-4-3-3	7		▦					
5	1020	3	2-3-1-2	4		▦		rock fragments, FILL, moist, brown			
		4	3-3-4-3	7		▨	CL	SANDY CLAY, moist, brown	Residual	B	
10	1015	5	3-7-9-11	16		▨	CL	CLAY, moist, orangish brown			
15	1010	6	8-8-9-8	17		▨	CL	SANDY CLAY w/ rock fragments, moist, brown			
20	1005	7	10-10-8-20	18		▨					
25	1000	8	16-17-18-17	35		▨	CL	CLAY, moist, grayish brown			
		9	10-13-24-50/2	50/2		▨					
30	995					△		DISINTEGRATED ROCK, moist, brown	Disintegrated Rock	C	
35		10	50/2	50/2		△					
								Bottom of Test Boring @ 35'			

D. W. KOZERA, INC.
Baltimore, Maryland
PROFESSIONAL ENGINEERS & GEOLOGISTS

TEST BORING LOG

Boring No.: B-9
Contract No.: 20179.D
Page: 1 of 1

Project: Hollidaysburg Veterans Home
Location: 138 Veterans Blvd
Ducansville, PA

Ground Surf. El. (±) : 1033.3
Date Started : 9-20-21
Date Completed : 9-20-01
Contractor : Echelberger
Driller : Ben Hurley
Rig : Diedrch D-50
Drill Method : 3" Casing/Rollerbit Auto Hammer
Inspector : E. Kussman

	GROUNDWATER OBSERVATIONS				
	Date	Time	Depth	Casing	Caved
Encountered	9-20	12:50	DRY	18.0	----
Completion	9-20	12:51	DRY	18.0	---
Casing Pulled	9-20	16:00	19.0	---	23.0
24-Hr Reading	9-21	09:04	19.0	---	23.0

Depth (ft)	Surf. Elev. 1033.3	Samples	Blow Counts	"N" Value	Water Level	Graphic	USCS	Description	Formation	Stratum	Remarks
0		1	7-5-5-5	10		▩		sandy clay, FILL, moist, brown	Fill	A	Topsoil-2"
	1030	2	6-5-4-4	9		▩		SAA			
5		3	3-3-4-3	7		▩		NO RECOVERY			
		4	4-5-8-9	13		▨		SANDY CLAY, moist, brown	Residual	B	
	1025	5	8-4-5-8	9		▨	CL				
10						▨					
	1020	6	50/4	50/4		△		DISINTEGRATED ROCK, moist, grayish brown	Disintegrated Rock	C	
15						△					
	1015	7	11-18-50/3	50/3	▼	△					
20						△					
	1010	8	50/3	50/3		△					
25						△					
	1005	9	18-16-8-41	24		▨	CL	SANDY CLAY w/ rock fragments, moist, brown	Residual	B	
30						▨					
	1000	10	50/1	50/1		▦		BEDROCK	*	D	*Bedrock
35								Bottom of Test Boring @ 35'			

D. W. KOZERA, INC.
Baltimore, Maryland
PROFESSIONAL ENGINEERS & GEOLOGISTS

TEST BORING LOG

Boring No.:	IT-1
Contract No.:	20179.D
Page:	1 of 1

Project: Hollidaysburg Veterans Home
Location: 138 Veterans Blvd
Ducansville, PA

Ground Surf. El. (±)	: 1033.6
Date Started	: 9-16-21
Date Completed	: 9-16-21
Contractor	: Echelberger
Driller	: Ben Hurley
Rig	: Diedrch D-50
Drill Method	: 3 1/4" HSA Auto Hammer
Inspector	: E. Kussman

	GROUNDWATER OBSERVATIONS				
	Date	Time	Depth	Casing	Caved
Encountered	9-16	14:06	DRY	5.0	---
Completion	9-16	14:07	DRY	5.0	---
Casing Pulled	9-16	14:10	DRY	---	3.0
24-Hr Reading	9-17	07:48	DRY	---	3.0

Depth (ft)	Surf. Elev. 1033.6	Samples	Blow Counts	"N" Value	Water Level	Graphic	USCS	Description	Formation	Stratum	Remarks
0		1	4-7-8-4	15		▦		clay w/ rock fragment, FILL, moist, gray/brown	Fill	A	Topsoil-4"
	1030	2	3-2-3-4	5		▨	CL	LEAN CLAY, moist, orange	Residual	B	Bulk Sample 2-5-ft. w/c 7.1%
5		3	2-3-6-5	9		▥	ML	SANDY SILT, moist, orangish brown			Infiltration Pipe Set @ 5'
		4	5-6-8-6	14		▨	CL	SANDY LEAN CLAY, moist, oranish brown			
	1025	5	5-6-7-8	13		▨	CL	LEAN CLAY, moist, grayish brown			
10								Bottom of Test Boring @ 10'			

D. W. KOZERA, INC.
Baltimore, Maryland
PROFESSIONAL ENGINEERS & GEOLOGISTS

TEST BORING LOG

Boring No.:	IT-2
Contract No.:	20179.D
Page:	1 of 1

Project: Hollidaysburg Veterans Home
Location: 138 Veterans Blvd
Ducansville, PA

Ground Surf. El. (±)	: 1009.3
Date Started	: 9-16-21
Date Completed	: 9-16-21
Contractor	: Echelberger
Driller	: Ben Hurley
Rig	: Diedrch D-50
Drill Method	: 3 1/4" HSA Auto Hammer
Inspector	: E. Kussman

	GROUNDWATER OBSERVATIONS				
	Date	Time	Depth	Casing	Caved
Encountered	9-16	08:24	DRY	8.0	---
Completion	9-16	08:28	DRY	8.0	---
Casing Pulled	9-16	08:35	DRY	---	3.0
24-Hr Reading	9-17	07:29	DRY	---	3.0

Depth (ft)	Surf. Elev. 1009.3	Samples	Blow Counts	"N" Value	Water Level	Graphic	USCS	Description	Formation	Stratum	Remarks
0		1	7-3-2-3	5		▦		sandy clay, FILL, moist, brown	Fill	A	Topsoil-4"
		2	6-6-7-6	14		▨	CL	LEAN CLAY, moist, brown	Residual	B	
5	1005	3	3-3-4-8	7		▨	CL	SANDY LEAN CLAY w/ rock fragments, moist, brown			Infiltration Pipe Set @ 5'
		4	16-15-12-13	27		▨	CL	LEAN CLAY, moist, grayish brown			
	1000	5	13-15-20-18	35		▨	CL	SANDY LEAN CLAY, moist, orangish brown			
10								Bottom of Test Borings @ 10'			

D. W. KOZERA, INC.
Baltimore, Maryland
PROFESSIONAL ENGINEERS & GEOLOGISTS

TEST BORING LOG

Boring No.: IT-3
Contract No.: 20179.D
Page: 1 of 1

Project: Hollidaysburg Veterans Home
Location: 138 Veterans Blvd
Ducansville, PA

Ground Surf. El. (±) : 1009.2
Date Started : 9-16-21
Date Completed : 9-16-21
Contractor : Echelberger
Driller : Ben Hurley
Rig : Diedrch D-50
Drill Method : 3 1/4" HSA Auto Hammer
Inspector : E. Kussman

	GROUNDWATER OBSERVATIONS				
	Date	Time	Depth	Casing	Caved
Encountered	9-16	09:00	DRY	8.0	---
Completion	9-16	09:39	DRY	8.0	---
Casing Pulled	9-16	09:45	DRY	---	4.0
24-Hr Reading	9-17	07:32	DRY	---	4.0

Depth (ft)	Surf. Elev. 1009.2	Samples	Blow Counts	"N" Value	Water Level	Graphic	USCS	Description	Formation	Stratum	Remarks
0						▩		clay, FILL, moist, brown	Fill	A	Topsoil-4"
		1	3-3-3-3	6		▩					
						▨		LEAN CLAY, moist, brown	Residual	B	
		2	6-7-8-13	15		▨	CL				
	1005					▨		SANDY LEAN CLAY, moist, orange			
5		3	6-11-11-10	22		▨					Infiltration Pipe Set @ 5'
						▨	CL				
		4	8-9-14-11	23		▨					
						▨		LEAN CLAY, moist, brown			
	1000	5	9-9-5-9	14		▨	CL				
10								Bottom of Test Boring @ 10'			

D. W. KOZERA, INC.
Baltimore, Maryland

PROFESSIONAL ENGINEERS & GEOLOGISTS

TEST BORING LOG

Boring No.: IT-4
Contract No.: 20179.D
Page: 1 of 1

Project: Hollidaysburg Veterans Home
Location: 138 Veterans Blvd
Ducansville, PA

Ground Surf. El. (±) : 1004.9
Date Started : 9-16-21
Date Completed : 9-16-21
Contractor : Echelberger
Driller : Ben Hurley
Rig : Diedrch D-50
Drill Method : 3 1/4" HSA Auto Hammer
Inspector : E. Kussman

	GROUNDWATER OBSERVATIONS				
	Date	Time	Depth	Casing	Caved
Encountered	9-16	10:09	DRY	8.0	---
Completion	9-17	07:35	DRY	---	4.0
Casing Pulled	9-19	10:10	DRY	8.0	---
24-Hr Reading	9-19	10:16	DRY	---	4.0

Depth (ft)	Surf. Elev. 1004.9	Samples	Blow Counts	"N" Value	Water Level	Graphic	USCS	Description	Formation	Stratum	Remarks
0						▩		clay, FILL, moist, brown			Topsoil-3"
		1	2-3-4-5	7		▩					
		2	8-9-10-10	19		▨	CL	LEAN CLAY, moist, brown			
5	1000	3	3-8-5-4	13		▨		LEAN CLAY, moist, orangish brown			Infiltration Pipe Set @ 5'
						▨	CL				No Recovery
		4	4-5-6-5	11		▨					
		5	9-14-14-21	28		◌	GC	CLAYEY GRAVEL w/ rock fragments, moist, gray			
10	995							Bottom of Test Boring @ 10'			

D. W. KOZERA, INC.
Baltimore, Maryland

PROFESSIONAL ENGINEERS & GEOLOGISTS

TEST BORING LOG

Boring No.: IT-5
Contract No.: 20179.D
Page: 1 of 1

Project: Hollidaysburg Veterans Home
Location: 138 Veterans Blvd
Ducansville, PA

Ground Surf. El. (±)	: 1004.1				
Date Started	: 9-16-21				
Date Completed	: 9-16-21				
Contractor	: Echelberger				
Driller	: Ben Hurley				
Rig	: Diedrch D-50				
Drill Method	: 3 1/4" HSA Auto Hammer				
Inspector	: E. Kussman				

	GROUNDWATER OBSERVATIONS				
	Date	Time	Depth	Casing	Caved
Encountered	9-16	10:39	DRY	8.0	---
Completion	9-16	10:40	DRY	8.0	---
Casing Pulled	9-16	10:48	DRY	---	3.0
24-Hr Reading	9-17	07:38	DRY	---	3.0

Depth (ft)	Surf. Elev. 1004.1	Samples	Blow Counts	"N" Value	Water Level	Graphic	USCS	Description	Formation	Stratum	Remarks
0		1	2-4-3-6	7		▨	CL	LEAN CLAY, moist, brown	Residual	B	Topsoil-4"
		2	8-15-13-11	28		▨	CL	LEAN CLAY w/ rock fragments, moist, brown			
5	1000	3	5-5-7-6	12		▨	CL	SANDY LEAN CLAY, moist, brown			Infiltration Pipe Set @ 5'
		4	7-7-7-8	14		▨	CL	SANDY LEAN CLAY, moist, orangish brown			
	995	5	7-11-8-12	19		▨					
10								Bottom of Test Boring @ 10'			

D. W. KOZERA, INC.
Baltimore, Maryland
PROFESSIONAL ENGINEERS & GEOLOGISTS

TEST BORING LOG

Boring No.: IT-6
Contract No.: 20179.D
Page: 1 of 1

Project: Hollidaysburg Veterans Home
Location: 138 Veterans Blvd
Ducansville, PA

Ground Surf. El. (±) : 1000.0
Date Started : 9-16-21
Date Completed : 9-16-21
Contractor : Echelberger
Driller : Ben Hurley
Rig : Diedrch D-50
Drill Method : 3 1/4" HSA Auto Hammer
Inspector : E. Kussman

	GROUNDWATER OBSERVATIONS				
	Date	Time	Depth	Casing	Caved
Encountered	9-16	12:15	DRY	8.0	---
Completion	9-16	12:16	DRY	8.0	---
Casing Pulled	9-16	13:10	DRY	---	3.5
24-Hr Reading	9-17	07:40	DRY	---	3.5

Depth (ft)	Surf. Elev. 1000.0	Samples	Blow Counts	"N" Value	Water Level	Graphic	USCS	Description	Formation	Stratum	Remarks
0						▨		LEAN CLAY, moist, brown		B	Topsoil-5"
		1	2-3-4-5	7		▨					
						▨	CL				
		2	8-9-9-8	18		▨					
						▨		SANDY LEAN CLAY, moist, brown			
5	995	3	3-4-10-11	23		▨	CL		Residual		Infiltration Pipe Set @ 5'
						▨		LEAN CLAY, moist, grayish brown			
		4	12-17-16-17	56		▨					
						▨	CL				
		5	20-21-17-12	38		▨					
10	990							Bottom of Test Boring @ 10'			

D. W. KOZERA, INC.
Baltimore, Maryland

PROFESSIONAL ENGINEERS & GEOLOGISTS

TEST BORING LOG

Boring No.: IT-7
Contract No.: 20179.D
Page: 1 of 1

Project: Hollidaysburg Veterans Home
Location: 138 Veterans Blvd
Ducansville, PA

Ground Surf. El. (±) : 1007.3
Date Started : 9-16-21
Date Completed : 9-16-21
Contractor : Echelberger
Driller : Ben Hurley
Rig : Diedrch D-50
Drill Method : 3 1/4" HSA Auto Hammer
Inspector : E. Kussman

	GROUNDWATER OBSERVATIONS				
	Date	Time	Depth	Casing	Caved
Encountered	9-16	13:40	DRY	8.0	---
Completion	9-16	13:41	DRY	8.0	---
Casing Pulled	9-16	13:45	DRY	---	4.5
24-Hr Reading	9-17	07:42	DRY	---	4.5

Depth (ft)	Surf. Elev. 1007.3	Samples	Blow Counts	"N" Value	Water Level	Graphic	USCS	Description	Formation	Stratum	Remarks
0						▩		sandy clay with brick, FILL, moist, brown		A	Topsoil-4"
		1	2-2-2-3	4		▩			Fill		
	1005					▨		SANDY CLAY, moist, brown		B	
		2	3-3-2-3	5		▨					
5		3	2-2-2-2	4		▨					
						▨	CL		Residual		
		4	2-1-2-2	3		▨					
	1000					▨					
		5	3-4-5-4	9		▨					
10								Bottom of Test Boring @ 10'			

D. W. KOZERA, INC.
Baltimore, Maryland

PROFESSIONAL ENGINEERS & GEOLOGISTS

TEST BORING LOG

Boring No.: IT-8
Contract No.: 20179.D
Page: 1 of 1

Project: Hollidaysburg Veterans Home
Location: 138 Veterans Blvd
Ducansville, PA

	GROUNDWATER OBSERVATIONS				
	Date	Time	Depth	Casing	Caved
Encountered	9-16	14:25	DRY	8.0	---
Completion	9-16	14:26	DRY	8.0	---
Casing Pulled	9-16	14:32	DRY	---	3.5
24-Hr Reading	9-17	07:45	DRY	---	3.5

Ground Surf. El. (±) : 1032.6
Date Started : 9-16-21
Date Completed : 9-16-21
Contractor : Echelberger
Driller : Ben Hurley
Rig : Diedrch D-50
Drill Method : 3 1/4" HSA Auto Hammer
Inspector : E. Kussman

Depth (ft)	Surf. Elev. 1032.6	Samples	Blow Counts	"N" Value	Water Level	Graphic	USCS	Description	Formation	Stratum	Remarks
0						▩		clay, FILL, moist, brown	Fill	A	Topsoil-3"
		1	5-4-4-5	8		▩					
	1030					▩					
		2	3-4-3-3	7		▩					
						▨		SANDY LEAN CLAY w/ rock fragments, moist, brown	Residual	B	
5		3	2-2-2-4	4		▨	CL				Infiltration Pipe Set @ 5'
						▨		LEAN CLAY, moist, orangish brown			
		4	4-4-5-4	9		▨					
	1025					▨	CL				
		5	6-7-9-8	16		▨					
10								Bottom of Test Boring @ 10'			

APPENDIX C

Soil Laboratory Test Results

Hollidaysburg Veterans Home

Project Number: 20179.D

Summary of Laboratory Testing

Jay Kay Testing, Inc.
(814) 404-9283
www.jaykaytesting.com

Location: Duncansville, PA
Sample Date:

Boring ID	Sample ID	Depth (ft)		WC %	OM %	Atterberg Limits			SG	% Fines	USCS
						LL %	PL %	PI %			
-	-	Top	Btm	D-2216	D-2974	D-4318	D-4318	D-4318	D-854	-	D-2487
B-1	S-6	13	15	33.3	-	40	27	13	-	98.1	ML
B-4	S-7	13	15	18.4	-	29	22	7	-	60.7	CL-ML
B-5	S-6	13	15	19.1	-	35	23	12	-	88.6	CL
B-6	S-1	0	2	16.7	-	37	24	13	-	81.9	CL
IT-1	Bulk	2	5	7.1	-	33	19	14	-	67.9	CL

Jay Kay Testing, Inc. is an AASHTO-Accredited laboratory

Boring ID	Sample ID	Top	Btm
IT-1	Bulk	2'	5'

Location: Duncansville, PA
Sample Date: -

Moisture-Density Relationship of Soils

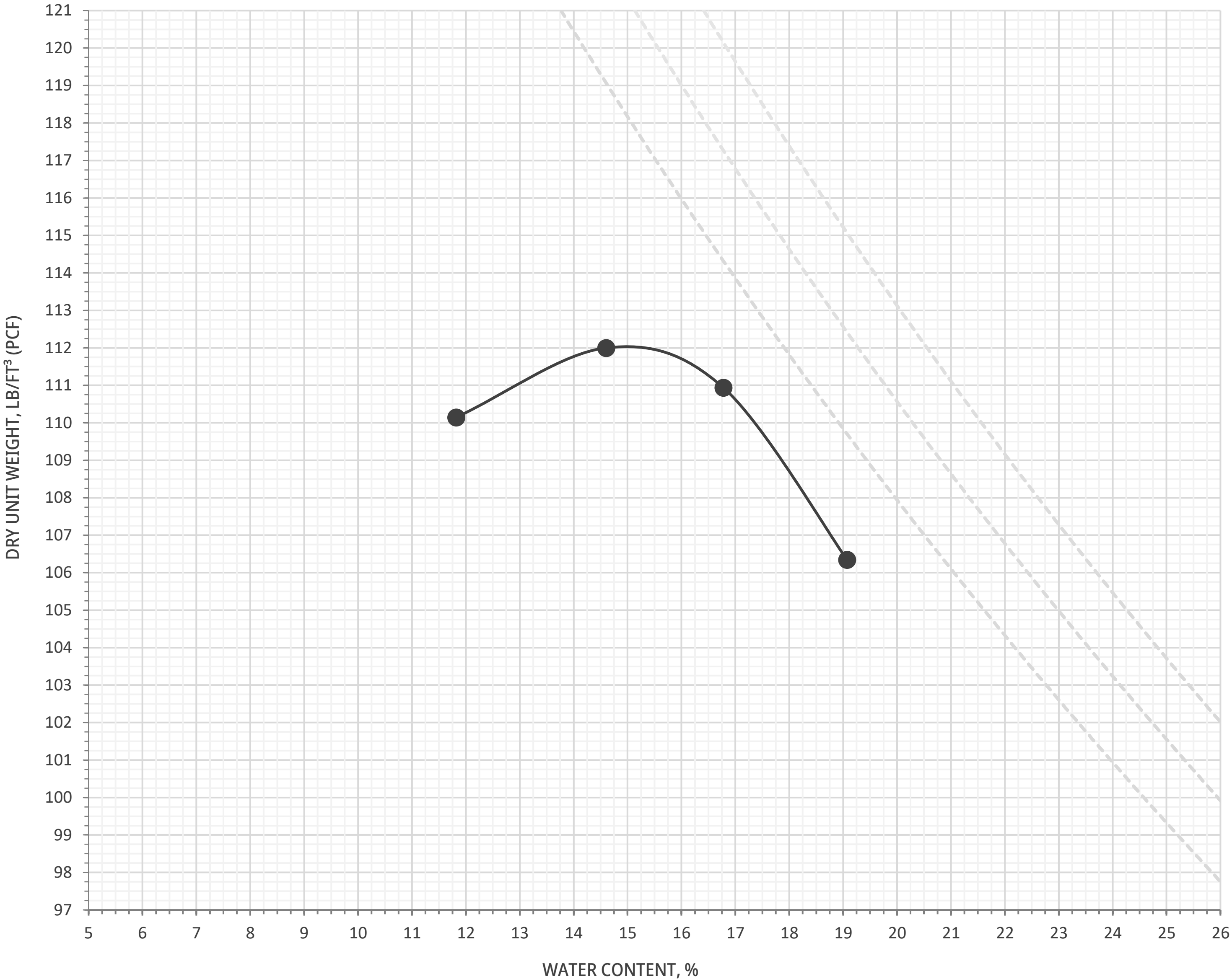
STANDARD PROCTOR
Test Method: ASTM D-698 (B)

Percent oversize particles: 4.9%
Oversized particles sieve: 3/8-in.
Threshold for correction: > 5.0%

	Uncorrected	*Corrected**
Maximum dry unit weight, lb/ft^3	112.0	-
Optimum water content	15.0%	-

*Threshold not met for oversized particle correction

Maximum Dry Unit Weight	Optimum Water Content
112.0	15.0%
lb/ft^3 (PCF)	



Zero Air Voids (100% Saturation)
Zero air voids curves 2.65, 2.75, 2.85

WC	LL	PL	PI	% Fines	USCS	AASHTO	Soil Description (D-2487)
7.1%	33%	19%	14%	67.9	CL	-	-

Boring ID	Sample ID	Top	Btm
IT-1	Bulk	2'	5'

Location: Duncansville, PA
Sample Date: -

California Bearing Ratio of Laboratory-Compacted Soils (CBR)

Test Method: ASTM D-1883, Compaction Method: ASTM D-698 (B)

	Uncorrected	*Corrected*
Soaked (96 hours) CBR at 0.1"	5.9%	-
Soaked (96 hours) CBR at 0.2"	5.6%	-

Surcharge, lb/ft^2	62.5
Target MDD, lb/ft^3	112.0
Target OMC	15.0%

CBR at 0.1"	CBR at 0.2"
5.9%	5.6%
Specimen Swell	**0.40%**

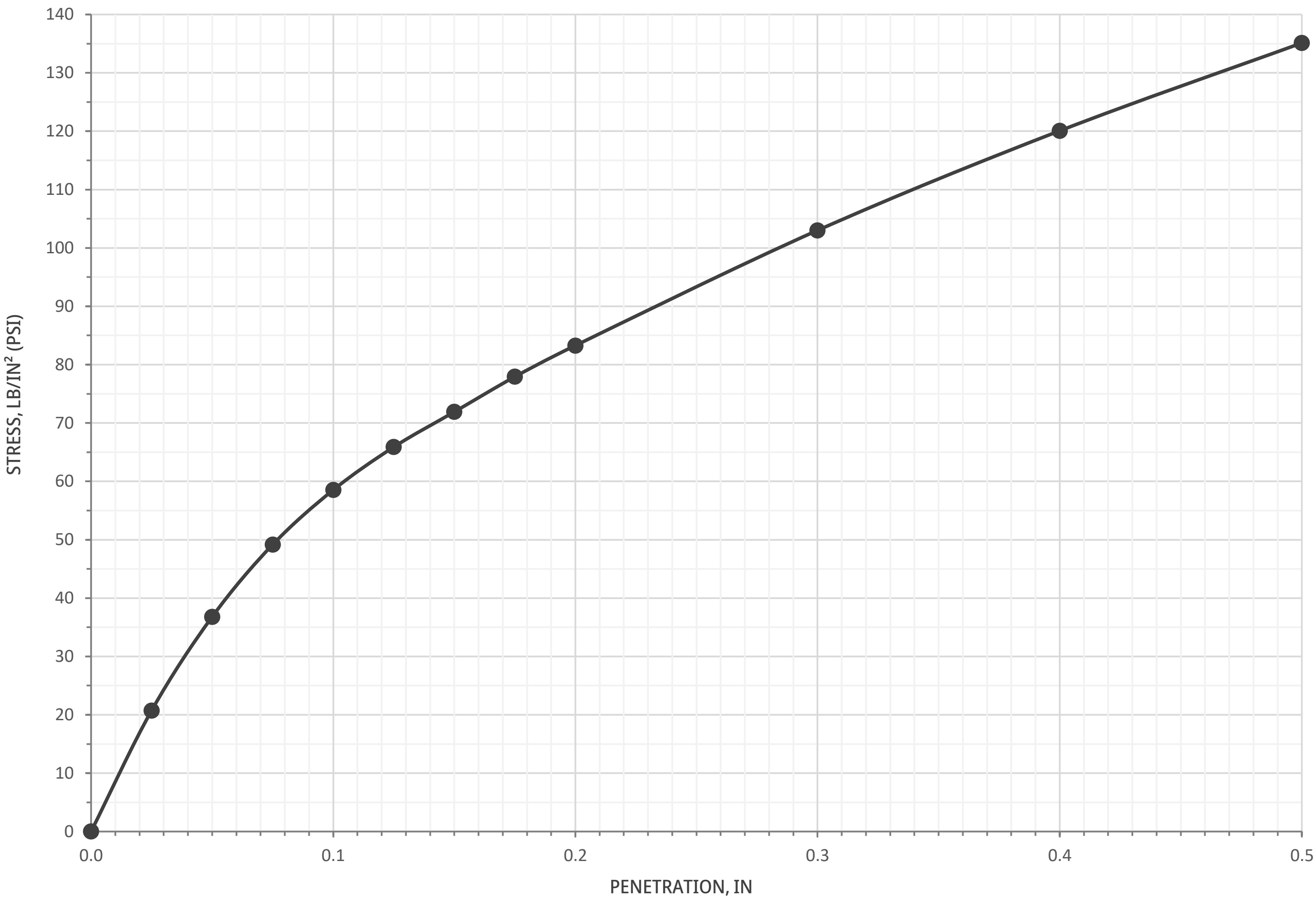
Specimen Data

AS-MOLDED

Dry unit weight, lb/ft^3	109.7	Blows per layer, #	25
Water content	15.4%	Achieved compaction	97.9%

AFTER-SOAK

Water content of top 1" layer -



WC	LL	PL	PI	% Fines	USCS	AASHTO	Soil Description (D-2487)
7.1%	33%	19%	14%	67.9	CL	-	-

10/05/21 Tested by: ST/JT Reviewed by: SC

APPENDIX D

Rock Core Photographs

Run#	Depth	% Rec	% RQD	Time
1	15-20'	76.7	13.3	11:03
2	20-25'	90	40	7:12
3	25-30'	96.7	68.3	8:42

Run #	Depth	% Rec	% RQD	Time
4	30-35'	98.3	58.3	9:15

APPENDIX E

Reference Paper: Foundation Design in Karst Terrain

Bulletin of the
Association of Engineering Geologists
Vol. XXIX, No. 2, 1992
pp. 165-173

Foundation Design in Karst Terrain

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West Chester, PA 19382

ABSTRACT

Karst terrain presents several risks related to development of structures, landfills, and dams. These include a) highly variable rock surfaces with pinnacles and slots, b) "enhanced weathered zones" and c) solution voids and cavities. The first can cause differential settlement and unanticipated construction costs and delays. The latter two can result in either sudden or ongoing subsidence. Although the risk that the project will be critically affected by these factors can be minimized by a thorough and well-planned subsurface exploration program, it is not possible to disclose all of these features on a site no matter what amount of money was spent on subsurface explorations. Indeed, they may not be revealed even during construction. Therefore, when designing foundations in karst, the risks of potential future subsidence should be defined for the owner so that he can make a rational decision about the amount of risk he is willing to accept. This can be accomplished by assessing both geologic risk factors and site development risk factors, providing a qualitative risk assessment, and presenting design alternatives.

INTRODUCTION

Karst terrains often occur over large areas extending throughout several counties or large portions of a state. Therefore, avoidance of karst for some areas is not a viable foundation design strategy. Many commercial and institutional facilities needed in a community, such as shopping centers, industrial parks, schools, churches, fire stations, and like structures, are lightly loaded and are normally supported on shallow spread footings. Because there will be uncertainties to varying degrees in spite of the most well-planned subsurface explorations and analyses, the degree of risk associated with a particular site must be evaluated. Traditional engineering approaches for settlement and bearing capacity must be considered in design, but cannot account for future sinkhole development. It is incumbent upon the design professional to assess the uncertainty of future subsidence (at least qualitatively) and communicate this risk to the owner, who must assume responsibility for the risk.

The design professional must provide the owner with foundation alternatives, and these alternatives must also define the degrees of risk and costs associated with them. As a minimum, a qualitative risk assessment should be provided as part of the site evaluation on every project located in karst terrain. The risk assessment must consider both geologic factors and project development factors.

GEOLOGIC FACTORS

Karst terrains are the result of dissolution of calcium and magnesium carbonate rocks. The focus of this paper is on the Cambro-Ordovician-age limestones associated with the Appalachian Ridge and Valley located in the Eastern United States as shown on Figure 1. Karst in these areas is characterized by shallow to moderate overburden soils of 10 to 100 ft (3 to 30 m) typically consisting of fine-grained residual soils. These soils generally classify as elastic silts (MH), fat clays (CH) or lean clays (CL). It is these fine-grained and cohesive, overburden soils that play a role in how the carbonate solution features manifest themselves and how they are distinguished from karst development in other parts of the country such as Florida.

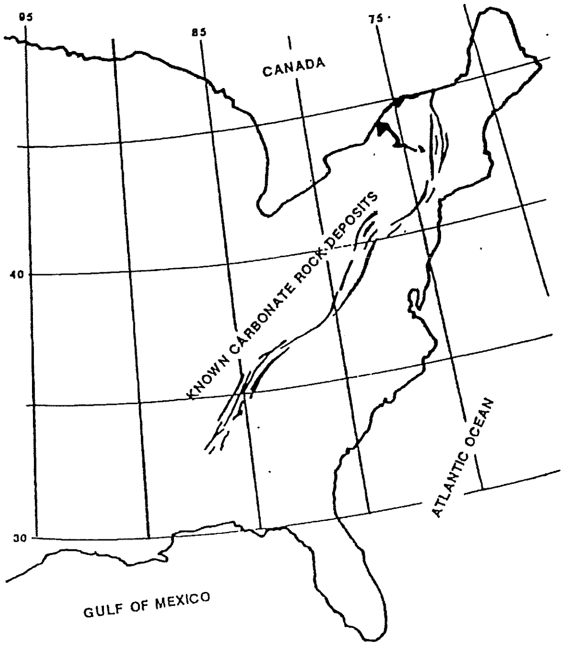

Figure 1. Cambro-Ordovician carbonates of the Appalachians (after Fischer and Fischer, 1989).

The overburden soils represent the insoluble material (alumina silicates) left behind after solutioning has taken place. Soils left behind as weathering products of the solutioning process are typically of medium stiff to hard consistency due primarily to desiccation (Belgeri and Shin, 1989).

The nature of the solutioning process creates a volume reduction, which is a function of the insoluble materials in the parent rock. Thus, because less volume is taken up by the remaining residual soil, it originates as a soft material increasing in stiffness with age due either to consolidation by overburden pressures from subsequent deposits, or by desiccation. This is an entirely different weathering process than for non-carbonate rocks, which gradually break down due to physical and chemical weathering, with the residual soils becoming more dense and "rock-like" with depth. As such, carbonate rock residual soils often develop very soft zones, referred to herein as "enhanced weathered zones," most often occurring where solutioning has formed a trough in the rock surface. Within these zones the process of consolidation is less likely to occur due to soil arching, wherein overburden stresses are shed to adjacent stiffer soil zones or rock pinnacles. Likewise, in these zones desiccation is inhibited because moisture is assured to these low areas. These zones are typically normally consolidated to only slightly preconsolidated and occur most often just above the rock contact. They present one of the major difficulties in designing foundations in karst. Although settlements attributable to these zones can be evaluated based on classical consolidation theory, the occurrence of these zones is as unpredictable as solution voids and cavities.

SINKHOLE CHARACTERISTICS

Sinkholes, otherwise known as dolines, are localized land surface subsidences or collapses due to karst processes. They are characteristically enclosed depressions resulting from solutioning of underlying rock. Three types of sinkholes have been defined (Beck, 1984): solution sinkholes, collapse sinkholes, and subsidence sinkholes.

Solution sinkholes are those caused purely by bedrock solutioning. These generally form due to pronounced solutioning at a preferential point such as an intersection of rock joints. This type of classic sinkhole formation is a very slow, gradual process, and one that generally does not present foundation problems. A solution sinkhole is illustrated in Figure 2A.

Collapse sinkholes refer to those formed by the actual collapse of the roof of a bedrock cavern as shown in Figure 2B. Although these types of sinkholes are sudden, and thus capable of causing foundation problems, their occurrence is rare. Thus, the probability of roof collapse of a cavern or void in the rock located below a building is very unlikely.

Subsidence sinkholes are by far the most common sinkholes, occurring with increasing regularity as man disturbs the environment. These sinkholes are generally formed by erosion of soil into voids and solution openings in the underlying bedrock as shown in Figure 2B. This process can be exacerbated by man's activities. These include activities such as

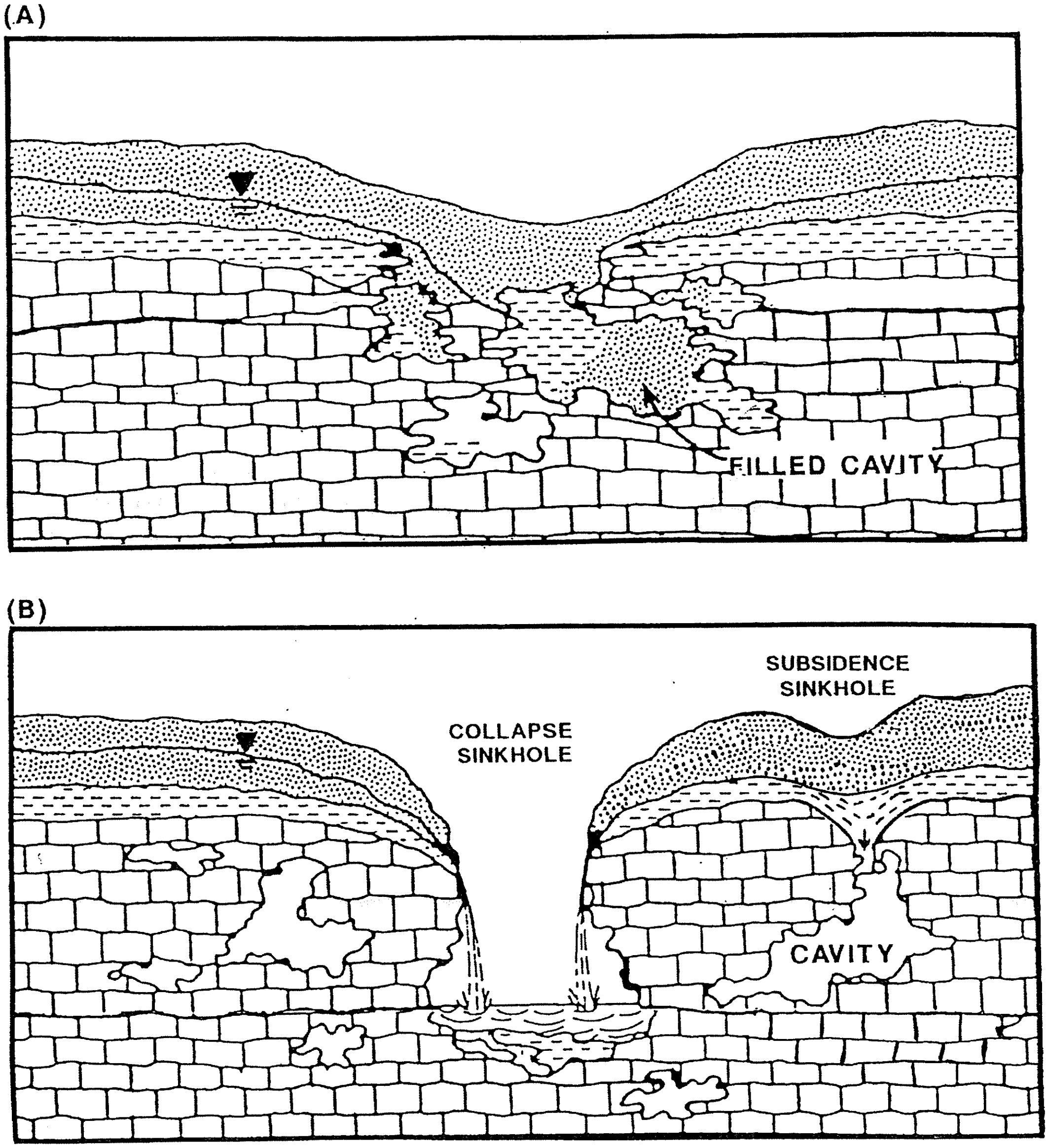

Figure 2. A) Solution sinkhole, and B) collapse and subsidence sinkholes (modified from Gass, 1981).

ground-water withdrawals, increased surface water infiltration, increased loadings, and vibrations.

ASSESSING GEOLOGIC RISK FACTORS

The likelihood of karst features affecting new development must be evaluated. In order to define the level of risk to the site, various geologic risk factors must be identified such as frequency of sinkholes, depths of overburden, fault zones, and ground-water conditions. Also to be considered are the impacts of project development, both positive and negative, imposed on the geologic conditions at the site. Geologic risk factors should be defined at each site through a) preliminary site evaluation, b) site reconnaissance and c) a subsurface exploration program. Each of these study phases is essential to proper evaluation of the geologic risk associated with the site.

Preliminary Site Evaluation

The preliminary site evaluation should include a thorough data review to obtain as much information as possible, including topography maps, geology maps, air photos, sinkhole maps, hydrogeologic reports, water well records, and previous test boring information. These sources should be used to provide an indication of existence of caves, sinkholes and disappearing streams, faulting, rock quality, depth of overburden, and well yields which might foretell the degree of solutioning or fracturing in the rock.

Topographic maps should be scrutinized for disappearing streams and closed-end depressions representing sinkholes. Drainage patterns should be noted. A trellis pattern, (stream tributaries at right angles), is common in karst terrain, where drainage is highly influenced by solutioning along joint sets, and controlled by the structure of the rock formation as it folds or dips in the direction of strike.

Geology maps are, of course, an essential part of the preliminary evaluation, and are used to confirm that the site lies within karst terrain. More importantly, however, it should be used to form a first opinion as to the probability of developing karst associated problems at the site. Geologic maps can help determine whether or not a particular geologic formation is notorious for sinkhole activity. This will be important to development of an adequate subsurface exploration program, as well as qualifying the overall risk level at the site.

Sinkhole maps are oftentimes prepared by State agencies for areas of known karst. These maps indicate individual sinkholes, thus revealing not only their location, but the frequency of occurrence for a given geologic formation or local area. Because these maps are prepared using aerial photography, small sinkholes typically less than about 30 ft (10 m) are often not shown. Their usefulness also depends in part on the scale to which they have been produced. Published sinkhole maps can be complemented by the use of aerial photographs to further locate sinkholes in the area of interest. Often a series of photographs can be obtained over a period of several decades, which might reveal the occurrence of sinkhole development in relation to man's activities, including mining, quarrying, well development, water mains, and earthwork projects.

The sinkhole frequency for a specific area may be obtained by combining the data from topographic and geologic maps and air photos. These data may be expressed as sinkholes per square mile. More sophisticated statistical approaches, developing probability distribution functions for sinkhole potential at a site can be evaluated (Raghu and Tiedman, 1984). However, these types of analyses are generally cost prohibitive for most building development projects.

Air photos and infrared photography can also be used to provide a fracture trace analysis to evaluate sinkhole probability since sinkholes are typically associated with lineaments, (linear surface features that reflect underlying fractures in the rock). Sinkholes are more likely to occur along lines of fracture, jointing or solution channels; frequently developing at the intersection of two or more lineaments (Gass, 1981).

Site Reconnaissance

Site reconnaissance should consist of thorough observation of site surface features by experienced personnel. It should also include interviews with persons familiar with a site. The site reconnaissance should be aimed at verifying previously identified features from air photos or maps as well as identification of other karst features not previously identified such as sinkholes, disappearing streams, and springs. Subtle features should be noted such as wide bowl-like depressions, small swales, changes in vegetation, unplowed or wooded areas in otherwise normally farmed lands, changes in soil moisture, and ponding of water (Fischer and Fischer, 1989). In rural areas, household wastes have often been conveniently disposed of in low areas or depressions, making these "open dumps" likely sinkhole candidates.

Subsurface Exploration Program

It is not possible to reveal every solution feature or enhanced weathered zone at a project site. This is illustrated by Figures 3A and 3B, indicating two subsurface profiles developed from the same three borings drilled across a building site. Figure 3A illustrates subsurface conditions which might be inferred from the data assuming non-karst conditions. This can be a dangerous viewpoint, as the actual conditions could well be as depicted in Figure 3B. With potential conditions such as Figure 3B firmly implanted in our thinking, the goal of the subsurface program should be to maximize the data obtained with a reasonable effort involving various exploration techniques. These techniques might be a combination of test borings, test pits, and air track probes, and in some instances, geophysical surveys.

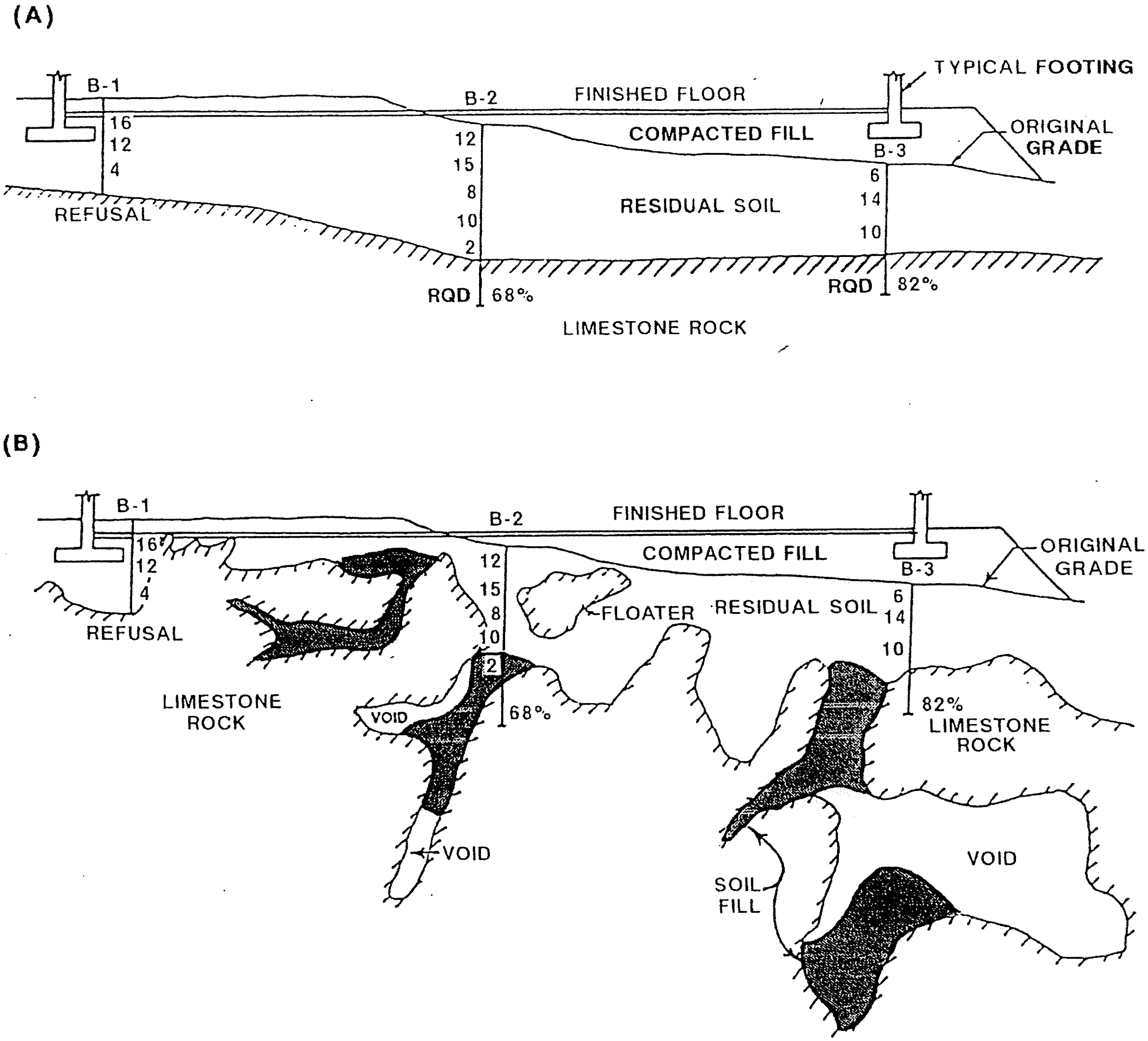

Figure 3. A) Interpretation of subsurface conditions assuming nonkarstic conditions. B) posssible subsurface conditions considering same test boring data as in Figure 3A.

Test borings should be drilled by experienced drillers and personnel. It is often advantageous to use rotary wash borings using water as a drilling fluid. With this type of drilling, the boring cannot only be advanced through boulders, but water losses can also be recorded throughout the drilling process. All pertinent information should be recorded on the boring log, including water loss, free fall of rods, stained joints, clay seams, rock coring times, and deflection of sampler. Ground-water levels in relation to top of rock will be important in assessing potential for sinkhole development. When the sampler deviates at an angle without attaining refusal, a sloping rock surface, representing either a boulder or pinnacle, can usually be inferred. Consideration should be given to grouting borings to above the rock surface in karst terrain to prevent development of avenues for subsurface erosion or contaminants to enter solution openings.

Air-track drilling (air-percussion drilling) is a cost effective way to obtain a number of data points throughout the site at reduced cost compared to wash borings. This method of drilling is quick and there is great mobility of the rig. These data can be used to supplement test borings and may prove invaluable in evaluating the variation of the rock surface and the occurrence of solution voids within the rock.

Geophysical methods such as seismic refraction, electromagnetic surveys (terrain conductivity), resistivity, and ground penetrating radar, have limitations and resolution problems when applied in karst terrains and therefore should not be used as the primary exploration tool. Seismic refraction surveys are compromised by pinnacled rock, boulders, voids, and lack of a coherent water table (Fischer and Fischer, 1989). Conductivity, resistivity and magnetic studies can be useful in evaluating variations in rock depth, although the data should be analyzed by someone knowledgeable in geophysical interpretation in order to be meaningful. Cavities and solution voids are unlikely to be evident in the data unless these features are very large (tens of feet) and thus terrain conductivity and ground magnetics should not be considered as sinkhole-finding tools. To the extent that geophysical methods can detect differences between deep saturated overburden soils (crevices and troughs), and shallower rock, they can give valuable clues as to the likelihood of solutioning in relative areas of the site. It is important to note that when applying geophysical methods, the interval between data points should be commensurate with the size of the feature being investigated.

Ground penetrating radar (GPR) has been successful in interpreting solution features in areas of shallow rock or sandy soils. However, within the typically clayey high moisture content overburden soils present within the Appalachian Ridge and Valley, GPR is not effective at depths greater than about five feet (1.5 m). Acoustic emissions has also been used, together with other techniques, to locate ground-water conduits (Stokowski, 1987)

ASSESSING DEVELOPMENTAL RISK FACTORS

Man-made changes to the geologic setting can either be positive or negative. These impacts involve both stress changes and changes to surface and ground-water conditions. Negative developmental risk factors include such things as a) ground-water pumping, b) reduction of overburden, c) concentration of surface water into unpaved ditches or improperly sealed utilities, d) creation of unlined water bodies and e) blasting.

Because the vast majority of sinkholes are subsidence sinkholes, formed by subsurface erosion of soil into pre-existing voids, most have been induced by allowing increased infiltration of surface water. Most notably these occur along leaking utility lines, at roof drain outlets, within unlined storm water detention and retention ponds, and unpaved ditches. Similarly, ground-water pumping can result in a downward migration of overburden soil by way of the following mechanisms:

1. Loss of buoyant support to residual soils arching above rock openings;
2. Increase in the velocity and seepage forces associated within ground-water movement;
3. Increase in the amplitude of water level fluctuations;
4. Movement of water to bedrock openings where recharge had previously been largely rejected (Newton, 1984).

For this reason, project design data obtained on karst sites should include not only structural loads and floor grades, but information regarding planned ponds, basins, and ground-water wells. Another important factor to note is that future pumping from adjacent sites might occur which cannot be controlled.

Vibrations due to blasting or heavy equipment have also been known to trigger subsidences. Vibrations are likely to contribute to cavity roof collapse in bedrock or soil. However, sinkholes induced by vibrations are far less frequent than those caused by water related mechanisms.

Excavation and grading operations resulting in a reduction in the thickness of overburden soils is an important developmental risk factor. Thinner overburden soils where the ground water is below rock make it much more likely that piping will occur or that foundation loads or construction vibrations will impose stresses exceeding those of the cavity roof soils. (Williams and Vineyard, 1976) noted that surface failure usually does not occur, even under wetted conditions, unless roof thickness is less than about 6 ft (2 m). It should be assumed that where overburden is less than about 10 to 15 ft (3 to 5 m), the risk of sinkhole development increases significantly. Where the water table is above the bedrock and overburden soils are thicker, the potential for soil piping into the voids is lessened (Newton, 1987).

QUALITATIVE RISK ASSESSMENT AND FOUNDATION DESIGN

Once the subsurface exploration has been completed and the earthwork and design parameters (floor grades, foundation loads, retention ponds, etc.) determined, a risk assessment of the site should be made as an integral part of the foundation design. This should consist of an overall qualitative assessment of whether there is low, moderate, or high probability of future sinkhole occurrence. This overall assessment is based on a weighted evaluation of the geologic and developmental risk factors discussed above.

The overall risk potential should be reflected in the design professional's report, and the foundation alternatives presented should be based on the assessment. The design report should not leave the owner with the impression that the consultant is insuring his building against future subsidence merely because he has calculated an acceptable total settlement within the overburden soils based on laboratory or *in situ* test data.

Having informed the client that there is indeed some risk of future subsidence, the design professional must be prepared to provide foundation alternatives which allow the owner to reduce or eliminate his risk. These alternatives can be generalized as regular shallow spread footings with or without soil improvement, rigid mats and grade beams, and deep foundations. Shallow spread footings are often combined with methods to reduce water infiltration as part of the overall design. Shallow foundations are used when it is calculated that the subsidence risk is low. Rigid foundations such as mats, post-tensioned slabs, and grade beams are used when it is concluded that the subsidence risk potential is low to moderate. Deep foundations are used when the subsidence risk is moderate to high.

Shallow foundations consist of spread footings on natural soils, or on either compacted soil fill or crushed stone fill following soil removal. The latter method is used to control settlements or as a remediation for enhanced weathered zones. This is often done within individual footing excavations, but can also be provided over the entire building area in conjunction with geogrid layers to form a stiffened "soil" layer for support of lightly loaded structures. Geosynthetics are also sometimes used as a precaution to prevent raveling of stone should a soil collapse occur with depth. Open graded stone (usually 1/2 to 3/4 in. size) can be used as long as concentrated water infiltration is prevented.

Grade beams are sometimes used in an attempt to bridge any small subsidences that could occur with time. These are often extended beyond building corners to prevent settlement from a building corner subsidence.

Deep foundations most often consist of drilled shafts (caissons). Pile foundations (both driven and auger cast) should be discouraged as it will not be possible to discern the actual bearing capacity of individual piles which dogleg on pinnacles, or deviate on sloped rock as shown in Figure 4.

Although drilled shaft foundations can be designed with virtually no risk of subsidence problems, they are generally very expensive. Some of the expense will be for grade beams if the floor slab is structurally supported, a likely scenario when subsidence potential is considered high. The most cost effective drilled shaft is the belled drilled shaft in which the load can be carried by an expanded bearing area formed by a belling tool. This minimizes the shaft diameter, reducing concrete volumes. Unfortunately drilled shafts typically cannot be successfully belled in karst areas due to perched ground water, soft overburden soil prone to caving, and hard sloping rock.

Although drilled shafts are often the only viable deep foundation alternative, they represent a large potential for extra costs during construction. Extra costs generally occur due to a) inability to dewater, b) additional depth to rock where troughs occur and c) rock excavation due to unsuitable bearing surfaces such as sloping rock, mudseams, voids, and

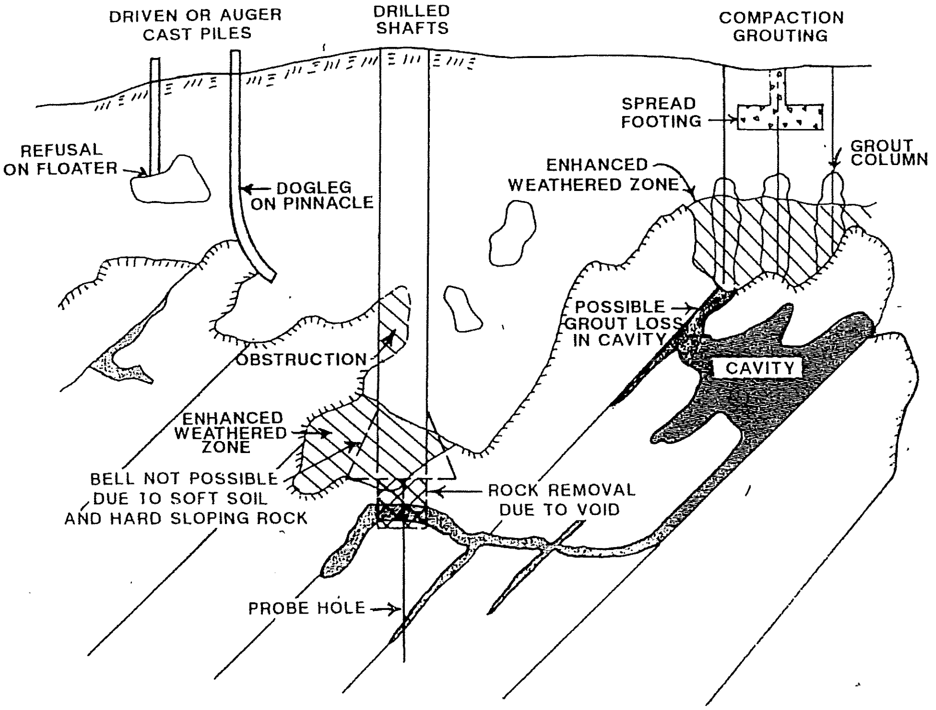

Figure 4. Foundation difficulties in karst.

weathered zones (Figure 4). Additional probe holes and observation time are also generally necessary to approve drilled shafts in carbonate rock.

For some situations, such as structures with few foundation elements, or where enhanced weathered zones must be improved, ground modification techniques can be considered. Compaction grouting, using a low slump grout under pressure to displace and consolidate soft zones, has some merit when the ground-water table is not above rock. Where ground water occurs in the soft soil zone, the grouting may not effectively consolidate the soil, but only build up pore pressure in the saturated material. Because grout columns are used to improve soil conditions generally occurring at or near the soil/rock contact, it is not always necessary to carry them to ground surface or even to foundation grade (Figure 4). Pressure grouting with a fluid grout in an attempt to "fill all the voids" is not recommended as it quickly becomes cost prohibitive. Furthermore, there is little control on where the grout goes. Pumping large quantities of grout to fill a rock cavity or utility is counterproductive.

Deep dynamic compaction (a method of densifying soil by dropping a large weight from heights of over 50 ft [15 m]) has been used to reduce the effects of karst solutioning. However, this method should not be counted on for anything but providing a moderate improvement of enhanced weathered zones. It does not eliminate the potential for soil piping into solution voids. And although its proponents can rightly claim this method can collapse soil cavity roofs, it can be argued that if it fails to do so, that the potential for future collapse may have been increased.

REDUCING RISK

There are a multitude of design considerations that may affect the long term performance of a development in karst terrain. The following list presents some general means of minimizing changes in stress and ground-water conditions which, in turn, will impact risks of future sinkhole occurrence. This list may not be all-inclusive and the impact of any particular item will depend on the existing geologic setting.

1. Incorporate designs that will tend to maintain ground-water levels consistent with those prior to development. This is generally difficult to do, although placement of water supply wells in the vicinity of buildings should be avoided.
2. Grading plans should reflect positive surface drainage away from buildings.
3. Do not plan utilities adjacent to or beneath shallow foundations.
4. Where significant utilities are planned beneath slabs-on-grade, it may be prudent to place them in a concrete duct bank.
5. Provide water-tight storm drains.
6. Tie roof drains directly into the storm drainage system.
7. Seal pavement curbs and catch basins. Do not allow concentrated flows in unpaved or unlined ditches or swales.
8. Minimize landscaped areas and sprinkler systems adjacent to buildings.
9. Use lined retention basins; keep away from building areas where possible.
10. Provide professional observation of the earthwork and foundation construction, as this provides a unique opportunity to see a significant area of the subsurface materials, further investigate critical areas, and recommend field changes aimed at reducing risk.

CONCLUSIONS

Foundation design in karst terrain involves uncertainties which cannot be predicted by applying traditional settlement and bearing capacity analyses. Although such analyses are needed to assess the support capability of a particular site, it is incumbent upon the design professional to provide an assessment of the risks of future subsidence to the structure due to undetected solution features or enhanced weathered zones. Alternative designs aimed at reducing risk should be presented.

ACKNOWLEDGMENTS

The writers would like to thank Dr. Ray E. Martin for assistance in reviewing the manuscript. We also gratefully acknowledge Schnabel Engineering Associates, Inc., for support of this effort through their inhouse research and development program.

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APPENDIX F

Spectral Acceleration Response

500 Municipal Dr, Duncansville, PA 16635, USA

Latitude, Longitude: 40.4439449, -78.4152368

Date	11/15/2021, 11:18:06 AM
Design Code Reference Document	ASCE7-10
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	0.114	MCE_R ground motion. (for 0.2 second period)
S_1	0.051	MCE_R ground motion. (for 1.0s period)
S_{MS}	0.182	Site-modified spectral acceleration value
S_{M1}	0.122	Site-modified spectral acceleration value
S_{DS}	0.121	Numeric seismic design value at 0.2 second SA
S_{D1}	0.082	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	B	Seismic design category
F_a	1.6	Site amplification factor at 0.2 second
F_v	2.4	Site amplification factor at 1.0 second
PGA	0.053	MCE_G peak ground acceleration
F_{PGA}	1.6	Site amplification factor at PGA
PGA_M	0.084	Site modified peak ground acceleration
T_L	12	Long-period transition period in seconds
SsRT	0.114	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	0.124	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.051	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.055	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.913	Mapped value of the risk coefficient at short periods

Type	Value	Description
C_{R1}	0.921	Mapped value of the risk coefficient at a period of 1 s

DISCLAIMER

APPENDIX G

Infiltration Test Results

D.W. KOZERA, INC.
BALTIMORE, MARYLAND
PROFESSIONAL ENGINEERS & GEOLOGISTS

In-Situ Infiltration Test

Project Name:	Hollidaysburg Veterans Home	Test Date:	
Contract No.:	20179.D	Test Location:	IT-1
Boring No.:	IT-1	Ground Surface Elev. (ft.):	+1033.63
Infiltration Pipe Length:	4.9		

Infiltration pipe bottom set at _4.8_below the existing grade

Top of infiltration pipe at ___0.1___ above the existing grade

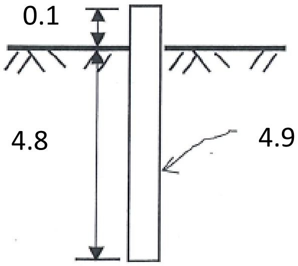
	Depth	Time	Date
Water level reading from the top of pipe after fill 2 feet of water (Pre-soaking)	2.91	3:20	9/16
Water level reading from the top of the pipe after 24 hrs. from filling 2 feet of water	3.29	7:48	9/17
Water level reading from the top of the pipe after re-filling (2 feet of water)			

			Water Level Reading			
Time of Measurement	Time Difference (hr.)	Cumulative Time (hr.)	Measurement from Top of Pipe (ft.)	Refilled water to depth of (ft.) (if required)	Difference (ft.)	Infiltration (in./hr.)

Infiltration Rate = in./hr.

*** Final field infiltration rate may be either the average of four observations, or the value of the last observation (MDE Stormwater Manual)**

Comments:



Test By: E. Kussman

D.W. KOZERA, INC.
BALTIMORE, MARYLAND
PROFESSIONAL ENGINEERS & GEOLOGISTS

In-Situ Infiltration Test

Project Name:	Hollidaysburg Veterans Home	Test Date:	
Contract No.:	20179.D	Test Location:	IT-2
Boring No.:	IT-2	Ground Surface Elev. (ft.):	+1009.27
Infiltration Pipe Length:	4.7’		

Infiltration pipe bottom set at __4.5____below the existing grade

Top of infiltration pipe at ___0.2___ above the existing grade

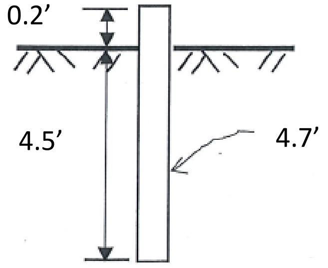
	Depth	Time	Date
Water level reading from the top of pipe after fill 2 feet of water (Pre-soaking)	2.24	8:52	9/16
Water level reading from the top of the pipe after 24 hrs. from filling 2 feet of water	2.94	7:29	9/17
Water level reading from the top of the pipe after re-filling (2 feet of water)			

			Water Level Reading			
Time of Measurement	Time Difference (hr.)	Cumulative Time (hr.)	Measurement from Top of Pipe (ft.)	Refilled water to depth of (ft.) (if required)	Difference (ft.)	Infiltration (in./hr.)

Infiltration Rate = in./hr.

*** Final field infiltration rate may be either the average of four observations, or the value of the last observation (MDE Stormwater Manual)**

Comments:



Test By: E. Kussman

D.W. KOZERA, INC.
BALTIMORE, MARYLAND
PROFESSIONAL ENGINEERS & GEOLOGISTS

In-Situ Infiltration Test

Project Name: Hollidaysburg Veterans Home

Test Date:

Contract No.: 20179.D

Test Location: IT-3

Boring No.: IT-3

Ground Surface Elev. (ft.): +1009.18

Infiltration Pipe Length: 4.7

Infiltration pipe bottom set at __4.6____below the existing grade

Top of infiltration pipe at ___0.1___ above the existing grade

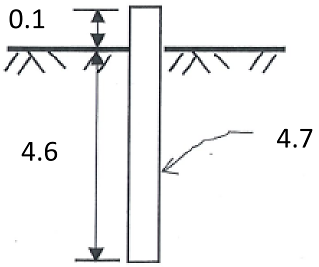
	Depth	Time	Date
Water level reading from the top of pipe after fill 2 feet of water (Pre-soaking)	2.65	9:48	9/16
Water level reading from the top of the pipe after 24 hrs. from filling 2 feet of water	2.72	7:32	9/17
Water level reading from the top of the pipe after re-filling (2 feet of water)			

			Water Level Reading			
Time of Measurement	Time Difference (hr.)	Cumulative Time (hr.)	Measurement from Top of Pipe (ft.)	Refilled water to depth of (ft.) (if required)	Difference (ft.)	Infiltration (in./hr.)

Infiltration Rate = in./hr.

*** Final field infiltration rate may be either the average of four observations, or the value of the last observation (MDE Stormwater Manual)**

Comments:



Test By: E. Kussman

D.W. KOZERA, INC.
BALTIMORE, MARYLAND
PROFESSIONAL ENGINEERS & GEOLOGISTS

In-Situ Infiltration Test

Project Name:	Hollidaysburg Veterans Home	Test Date:	
Contract No.:	20179.D	Test Location:	IT-4
Boring No.:	IT-4	Ground Surface Elev. (ft.):	+1004.91

Infiltration Pipe Length: 4.8

Infiltration pipe bottom set at ___4.7___below the existing grade

Top of infiltration pipe at ___0.1___ above the existing grade

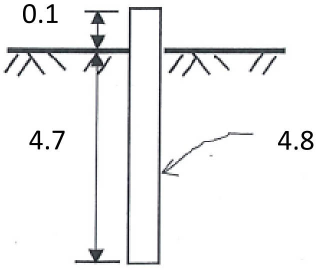
	Depth	Time	Date
Water level reading from the top of pipe after fill 2 feet of water (Pre-soaking)	2.77	11:34	9/16
Water level reading from the top of the pipe after 24 hrs. from filling 2 feet of water	3.02	7:35	9/17
Water level reading from the top of the pipe after re-filling (2 feet of water)			

			Water Level Reading			
Time of Measurement	Time Difference (hr.)	Cumulative Time (hr.)	Measurement from Top of Pipe (ft.)	Refilled water to depth of (ft.) (if required)	Difference (ft.)	Infiltration (in./hr.)

Infiltration Rate = in./hr.

*** Final field infiltration rate may be either the average of four observations, or the value of the last observation (MDE Stormwater Manual)**

Comments:



Test By: E. Kussman

D.W. KOZERA, INC.
BALTIMORE, MARYLAND
PROFESSIONAL ENGINEERS & GEOLOGISTS

In-Situ Infiltration Test

Project Name: Hollidaysburg Veterans Home

Test Date:

Contract No.: 20179.D

Test Location: IT-5

Boring No.: IT-5

Ground Surface Elev. (ft.): +1004.14

Infiltration Pipe Length: 4.8

Infiltration pipe bottom set at ___4.5___below the existing grade

Top of infiltration pipe at ___0.3___ above the existing grade

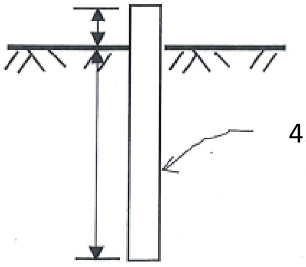
	Depth	Time	Date
Water level reading from the top of pipe after fill 2 feet of water (Pre-soaking)	2.80	11:06	9/16
Water level reading from the top of the pipe after 24 hrs. from filling 2 feet of water	3.05	7:38	9/17
Water level reading from the top of the pipe after re-filling (2 feet of water)			

			Water Level Reading			
Time of Measurement	Time Difference (hr.)	Cumulative Time (hr.)	Measurement from Top of Pipe (ft.)	Refilled water to depth of (ft.) (if required)	Difference (ft.)	Infiltration (in./hr.)

Infiltration Rate = in./hr.

*** Final field infiltration rate may be either the average of four observations, or the value of the last observation (MDE Stormwater Manual)**

Comments:



4.81

Test By: E. Kussman

D.W. KOZERA, INC.
BALTIMORE, MARYLAND
PROFESSIONAL ENGINEERS & GEOLOGISTS

In-Situ Infiltration Test

Project Name: Hollidaysburg Veterans Home

Test Date:

Contract No.: 20179.D

Test Location: IT-6

Boring No.: IT-6

Ground Surface Elev. (ft.): +999.99

Infiltration Pipe Length: 4.7

Infiltration pipe bottom set at ___4.6___below the existing grade

Top of infiltration pipe at ___0.1___ above the existing grade

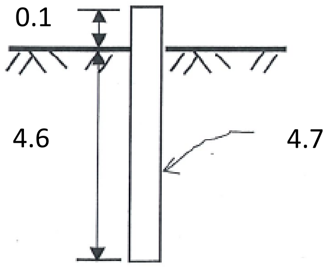
	Depth	Time	Date
Water level reading from the top of pipe after fill 2 feet of water (Pre-soaking)	2.70	12:17	9/16
Water level reading from the top of the pipe after 24 hrs. from filling 2 feet of water	2.73	7:40	9/17
Water level reading from the top of the pipe after re-filling (2 feet of water)			

			Water Level Reading			
Time of Measurement	Time Difference (hr.)	Cumulative Time (hr.)	Measurement from Top of Pipe (ft.)	Refilled water to depth of (ft.) (if required)	Difference (ft.)	Infiltration (in./hr.)

Infiltration Rate = in./hr.

*** Final field infiltration rate may be either the average of four observations, or the value of the last observation (MDE Stormwater Manual)**

Comments:



Test By: E. Kussman

D.W. KOZERA, INC.
BALTIMORE, MARYLAND
PROFESSIONAL ENGINEERS & GEOLOGISTS

In-Situ Infiltration Test

Project Name: Hollidaysburg Veterans Home

Test Date:

Contract No.: 20179.D

Test Location: IT-7

Boring No.: IT-7

Ground Surface Elev. (ft.): +1007.33

Infiltration Pipe Length: 4.7

Infiltration pipe bottom set at ___4.7___below the existing grade

Top of infiltration pipe at __0____ above the existing grade

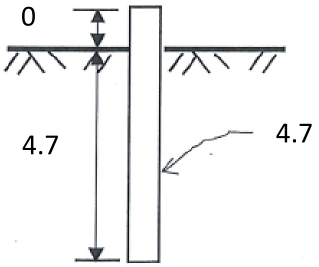
	Depth	Time	Date
Water level reading from the top of pipe after fill 2 feet of water (Pre-soaking)	2.70	1:55	9/16
Water level reading from the top of the pipe after 24 hrs. from filling 2 feet of water			9/17
Water level reading from the top of the pipe after re-filling (2 feet of water)			

			Water Level Reading			
Time of Measurement	Time Difference (hr.)	Cumulative Time (hr.)	Measurement from Top of Pipe (ft.)	Refilled water to depth of (ft.) (if required)	Difference (ft.)	Infiltration (in./hr.)

Infiltration Rate = in./hr.

*** Final field infiltration rate may be either the average of four observations, or the value of the last observation (MDE Stormwater Manual)**

Comments:



Test By: E. Kussman

D.W. KOZERA, INC.
BALTIMORE, MARYLAND
PROFESSIONAL ENGINEERS & GEOLOGISTS

In-Situ Infiltration Test

Project Name: Hollidaysburg Veterans Home
Test Date:
Contract No.: 20179.D
Test Location: IT-8
Boring No.: IT-8
Ground Surface Elev. (ft.): +1032.57
Infiltration Pipe Length: 5.4
Infiltration pipe bottom set at ___5.0___below the existing grade
Top of infiltration pipe at ___0.4___ above the existing grade

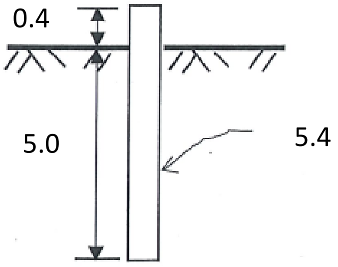
	Depth	Time	Date
Water level reading from the top of pipe after fill 2 feet of water (Pre-soaking)	3.33	2:35	9/16
Water level reading from the top of the pipe after 24 hrs. from filling 2 feet of water	3.74	7:45	9/17
Water level reading from the top of the pipe after re-filling (2 feet of water)			

			Water Level Reading			
Time of Measurement	Time Difference (hr.)	Cumulative Time (hr.)	Measurement from Top of Pipe (ft.)	Refilled water to depth of (ft.) (if required)	Difference (ft.)	Infiltration (in./hr.)

Infiltration Rate = in./hr.

*** Final field infiltration rate may be either the average of four observations, or the value of the last observation (MDE Stormwater Manual)**

Comments:



Test By: E. Kussman