Water Tower Site Evaluation

Preliminary Geotechnical Engineering and Concrete Testing Report

September 4, 2025 | Terracon Project No. N1255189

Revised: September 24, 2025

Prepared for:

Village of Indian Hill
6525 Drake Road
Cincinnati, Ohio 45243

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September 4, 2025
Revised: September 24, 2025

Village of Indian Hill
6525 Drake Road
Cincinnati, Ohio 45243

Attn: Ms. Cindy Klopfenstein, PE, CFM
P: (513) 561-6500 Ext. 6215
E: cklopfenstein@indianhill.gov

Re: Preliminary Geotechnical Engineering and Concrete Testing Report
Water Tower Site Evaluation
5355 Miami Road
Village of Indian Hill, Ohio
Terracon Project No. N1255189

Dear Ms. Klopfenstein:

We have completed the scope of Preliminary Geotechnical Engineering and Concrete Testing services for the above referenced project in general accordance with Professional Services Agreement dated June 17, 2025. This report presents the findings of the review of archive geotechnical information for the site, presents the results of the concrete testing services, and provides preliminary geotechnical recommendations concerning foundations for potential renovations to the existing elevated tank and a potential new elevated tank on the property. This report has been revised on September 24, 2025, based on updated information provided to us regarding the dimensions of the new tank option.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report or if we may be of further service, please contact us.

Sincerely,
Terracon

STATE OF OHIO
RICHARD BACH
86025
REGISTERED
PROFESSIONAL ENGINEER

09/24/2025

Richard L. Bach, PE
Senior Engineer

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Geotechnical Department Manager/Principal

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Note: This report was originally delivered in a web-based format. **Blue Bold** text in the report indicates a referenced section heading. The PDF version also includes hyperlinks which direct the reader to that section and clicking on the Terracon logo will bring you back to this page. For more interactive features, please view your project online at **client.terracon.com**.

Refer to each individual Attachment for a listing of contents.

Introduction

This report presents the results of our Preliminary Geotechnical Engineering and Concrete Testing services performed for the Water Tower Site Evaluation at 5355 Miami Road in the Village of Indian Hill, Ohio. The purpose of these services was to provide information and preliminary geotechnical engineering recommendations relative to:

- Subsurface soil (and rock) conditions
- Groundwater conditions
- Seismic site classification per IBC
- Compressive strength of the existing foundation concrete
- Foundation design and construction
- Construction considerations and challenges

The Scope of Services for this project included the review of historical information provided by IH, coring and testing of the existing foundation concrete, engineering analysis, and preparation of this report.

Drawings showing the site and historic boring locations by others are shown on the **Site Location** and **Exploration Plan**, respectively. The historic boring information by others was provided by the Village of Indian Hill (IH) for our review. Our interpretation of the subsurface conditions encountered by others is discussed in **Geotechnical Characterization** and shown graphically on the Subsurface Profiles included in **Figures**.

Project Description

Our initial understanding of the project was provided in our proposal and was discussed during project planning. Additional correspondence with IH during the report development process has resulted in the project information summarized in the following table.

Item	Description
Information Provided	An email request for proposal was provided by Ms. Cindy Klopfenstein on May 21, 2025. On May 30, 2025, Ms. Klopfenstein provided the following information: ■ 1934 Plans for the Indian Hill Tank and Supply Line prepared for the Board of County Commissioners for Hamiton County, Ohio by Charles F. Cellarius, Architect, and Fred W. Morrill, Structural Engineer. ■ 1963 Plans for two adjacent 1 million Gallon (MG) Underground Reservoirs prepared for the Village of Indian Hill, Ohio, Water Supply Improvements project by A.M. Kinney, Inc. ■ 1986 Plans for 1.2 MG Underground Reservoir prepared for the Village of Indian Hill by Woolpert Consultants. ■ 1986 Geotechnical Report for the 1.2 MG Underground Reservoir prepared by Soil & Material Engineers, Inc. On June 2, 2025, our Messrs. Steve Mary, PE and Richard Bach, PE, met with Ms. Klopfenstein and Mr. Ron Freson at the project site to review the site conditions and discuss the project. On June 9, 2025, Ms. Klopfenstein requested that the scope of services be expanded to include coring and testing of the foundation concrete. On September 16, 2025, Landmark Structures provided a preliminary concept drawing showing the elevation and section through the elevation of the proposed 1.0 MG elevated tank.

Item	Description
Project Description	The Village of Indian Hill desires to make improvements to the water supply system to include upgrading the capacity the existing 0.5 MG water tower to 1 MG and increasing the hydraulic grade line to approximately El. 967 feet by either replacing the existing tower with a new elevated composite tank or by rehabilitating/upgrading the existing tower. For scenarios involving replacement of the existing tower, it is desired that the existing tower remain in service through construction and the existing tower may remain in a decommissioned state after the construction of the new tower. Alternatively, consideration may be given to options that include decommissioning and razing the existing tower and/or a portion of one of the underground reservoirs, although a temporary standpipe tank may be needed to maintain the desired level of service for the community during construction. We understand that a new tower would consist of a composite elevated tank with a 54-foot sidewall depth designed to replicate the aesthetics of the existing tank. We understand that the overall height of the proposed structure is on the order of 124.5 feet relative to the bearing elevation. We have considered that shallow foundation options for the proposed tank would consist of a ring foundation or a mat foundation. At this time, loads for the existing, renovated existing, or proposed new tank options are not available. Location of the proposed feature(s) on the site are currently unknown and will be determined during detailed design development, but we understand that contractors for the installation of a new composite tank need on the order of 20 to 30 feet around the perimeter of the structure in order to construct the pedestal and elevated tank. As such, areas in the northwest corner and northeast corner of the property are being considered as potential locations for the new tower option when the existing tower remains.
Building Code	2024 Ohio Building Code

Terracon understands that the proposed construction indicated above is conceptual in nature and the project, if advanced from this preliminary planning stage, would undergo a detailed design development process. As such, our conclusions and recommendations contained in this report are preliminary and modifications to our recommendations may be necessary.

Site Conditions and Historic Plan Review

The following description of site conditions is derived from our site visit in association with the field services and our review of provided historic plans for the site.

Item	Description
Parcel Information	▪ The project is located at 5355 Miami Road in Village of Indian Hill, Ohio. ▪ Latitude/Longitude (approximate) 39.1643° N, 84.3676° W (See Exhibit D)
Existing Improvements	Existing water tower with underground storage reservoirs to the south and west of the tower. An asphalt access drive serves the tower from the east. We understand that a sewer and water line generally follow the northern edge of the driveway and are located under the driveway incised of the perimeter fence.
Existing Water Tower	The plans for existing water tower show that the water tower is supported by shallow foundations consisting of essentially two reinforced concrete rings connected by reinforced concrete struts between each interior column and pilaster along the perimeter wall. The foundation system is shown to bear at El. 880 feet with existing grade shown at El. 886 feet. The outer ring is shown to be 6 feet wide (extending 2.5 feet on each side of a 12-inch-wide perimeter wall) while the inner "ring" is 6.5 feet wide and flairs to essentially create a hexagon shape at each column location. Observations made at the exposed footings discussed in **Concrete Coring and Testing Procedures** are consistent with what is shown on the plans. The columns and perimeter wall extend up to the tank slab with the top of the tank slab at El. 936 and the top of the tank shown near El. 961.

Item	Description
Existing Underground Storage Reservoirs	Two 1 MG underground storage reservoirs are located to the south of the existing tower. The overall plan dimensions from the 1963 plans show 191 feet in the east-west direction and 96 feet in the north-south direction. The reservoirs are shown to share a common wall near the middle of the long side. The plans show that the top of the tank is roughly at El. 882.7 feet with the bearing elevation for the exterior wall foundations shown near El. 863.8 feet, the bottom of sump pits near El. 862.4 feet, and the bottom of an underdrain collection pit near El. 859.4 feet. The existing pump house is located on the northern wall line above the sump pits at the common wall between the reservoirs. A 1.2 MG underground reservoir is located to the southwest of the existing tower. The overall plan dimensions from the 1986 plans show 120 feet in the north-south direction and 88 feet in the east west direction. The plans show that the top of the tank is roughly at El. 882.7 feet with the bearing elevation for the exterior wall foundations shown near El. 863.8 feet, the bottom of sump pits near El. 862.4 feet, and the bottom of an underdrain collection pit near El. 859.4 feet. The pits are located in the northeast corner of the reservoir.
Historic Subsurface Information	The 1963 Plans contain graphic illustrations of boring logs on a profile for four borings (labeled B-1 through B-4) and three soundings (labeled S-1 through S-3) and the locations of the borings are shown on the site plan. No other information is available regarding these borings and soundings. The 1986 Geotechnical Report for the 1.2 MG reservoir included the results of eight borings (labeled 1 through 8) completed in December 1985 and January 1986. The locations of the historic test borings are shown on the **Exploration Plan** attached to this report. A discussion regarding the subsurface conditions interpreted from this historic information is included in **Geotechnical Characterization**.
Existing Topography	Based on topographic contours from CAGIS, the ground surface elevation ranges from EL. 886 ft. to El. 880 ft. across the property and generally drains away from the tower with low points near the northwest and northeast corners of the property and along the approximate midpoint of the access road. This generally aligns well with the proposed grading shown on the Site Plan in the 1986 Plans.

Geotechnical Characterization

We have developed a general characterization of the subsurface conditions based upon our review of the available historic subsurface information and our understanding of the project, geologic setting, and development history. Our interpretation of the subsurface conditions from the available historic information is shown graphically on the Subsurface Profiles included in the **Figures** attachment to this report. This characterization, termed GeoModel, included assigning soil and bedrock strata to layers. The GeoModel forms the basis of our geotechnical calculations and evaluation of the site. Conditions observed at each exploration point are indicated on the individual logs. For a more detailed view of the subsurface information, refer to the historic subsurface information which is included in the **Supporting Information** attachment.

Model Layer	Layer Name	General Description
1	**Surficial Materials**	Topsoil
2	**Existing Fill**	Lean clay with sand, medium stiff
3	**Native Fine-Grained Soil**	Predominantly lean clay with various amounts of sand and gravel with zones of silt, medium stiff to hard
4	**Weathered Bedrock**	Interbedded brown and gray shale, highly to moderately weathered, and gray limestone, slightly weathered to fresh
5	**Unweathered Bedrock**	Interbedded gray shale, slightly weathered to fresh, and gray limestone, slightly weathered to fresh

Two of the soundings from the 1963 plans (S-1 and S-2) were used in an attempt to find the top of the existing tower foundation. The sounding information indicates that the top of the footing was encountered around El. 881 feet which aligns well with the plans for the existing tower and the observations made of the exposed exterior portion of the foundation. These soundings are not included in the Subsurface Profiles as they provide no information regarding subsurface conditions.

Topsoil was noted in every historic boring across the site and is anticipated to be present at the ground surface. Existing fill was noted in only Boring 6 from the 1986 exploration to a depth of 2.5 feet below the ground surface. Based on a comparison of the proposed and existing grades shown in the 1986 Plans, up to 3.5 feet of fill was to be placed along the western side of the site. However, no records are currently available documenting the placement and compaction of this fill or any other earthwork activities at the site. As such, zones of deleterious materials or potential encumbrances (e.g., bury pits, debris, etc.) could exist below the surface.

Native fine-grained soils (predominately lean clays with some zones of silt) are present above the bedrock surface across the site. These materials generally consisted of a soft to medium stiff surficial zone in the upper 2.5 to 5 feet of the borings followed by a zone with consistencies ranging from stiff to hard. Based on the available information, it is assumed that the existing tower is bearing in the stiff to hard native fine-grained soils.

The top of bedrock noted in the historic borings ranged from near El. 877 feet near the southeast corner of the 1.2 MG reservoir to near El. 867 feet near the eastern edge of the property. Contours depicting the estimated bedrock surface across the site are included on the **Site Plan** attached to this report. It should be recognized that these contours are based on historical information and interpretations between historic boring locations. Furthermore, these contours do not take into account excavations that may have penetrated this surface such as for the underground reservoirs or for site utilities.

Bedrock in the Greater Cincinnati Area is typically categorized as weathered or unweathered, based on the degree of weathering of the shale component. The weathered zone is typically the uppermost zone, wherein the shale is brown to olive brown in color with occasional gray, exhibits high to moderate weathering characteristics, and is generally found to be extremely weak to very weak. In the unweathered zone, the shale is gray exhibits slight weathering characteristics or is fresh and is generally extremely weak to weak. Each zone is interbedded with limestone. It is not uncommon for the weathered bedrock zone to be absent due to differential weathering, erosion, or prior excavation. The Rock Classification Notes describes the varying degrees of weathering along with the rock strength descriptions.

Regarding the limestone, these layers are predominantly slightly weathered to fresh, and their strengths are estimated to range from medium strong to very strong. Occasionally, layers are encountered within the bedrock profile where groundwater seepage is concentrated, and weathering of the limestone layers is more advanced.

Groundwater was noted in the 1963 borings during drilling at depths ranging from approximately 1 to 4 feet below the ground surface, generally in or near zones of silt. It was not indicated if groundwater was observed at the end of drilling or after drilling in 1963. We interpret the observations from the 1963 borings to be perched groundwater in the surficial soils. Groundwater was not observed during the 1986 exploration. Based on the historical groundwater observations and our local experience, groundwater seepage is anticipated along the fill/native soil interface, along the overburden soil/bedrock interface, along limestone layers within the bedrock, and in the saturated zones of fill or surficial native soils that are within perched groundwater zones. Locally concentrated flow may occur along fractures in the bedrock. Additionally, groundwater levels, seepage amounts, and flow rates are expected to vary with time, location, season of the year, and amounts of precipitation.

Seismic Site Class

The seismic design requirements for buildings and other structures are based on Seismic Design Category. Site Classification is required to determine the Seismic Design Category for a structure. The Site Classification is based on the upper 100 feet of the site profile defined by a weighted average value of either shear wave velocity, standard penetration resistance, or undrained shear strength in accordance with Section 20.4 of ASCE 7 and the International Building Code (IBC). Based on the soil/bedrock properties observed at the site and as described on the exploration logs and results, our professional opinion is that a **Seismic Site Classification of C** be applied for the project but should be confirmed during final design with modern test boring(s) or geophysical testing. Historic Subsurface explorations at this site were extended to a maximum depth of 25.5 feet. The site properties below the boring depth to 100 feet were estimated based on our experience and knowledge of geologic conditions of the general area.

Concrete Compressive Strength Testing

Three concrete cores were obtained to evaluate the compressive strength of the footing concrete as part of our scope of services. The procedures used to obtain the cores and test the cores for compressive strength are documented in **Concrete Coring and Testing Procedures**. The results of the compressive strength testing varied from 7,110 pounds per square inch (psi) to 8,410 psi and are included in **Supporting Information**.

In 2022 while working for a Contractor for the Village of Indian Hill, Terracon obtained cores from the exterior wall, an interior column, and a pilaster and subjected the cores to compressive strength and carbonation testing. The results of this testing from 2022 are included in **Supporting Information**. The cores from the foundation concrete were not subjected to carbonation testing as the foundation concrete is buried and not exposed to atmospheric conditions, therefore reinforcing steel corrosion induced by carbonation is unlikely. Carbonation occurs from the exposed surface and moves inward over time. Depending on the concrete mix, we understand carbonation moves around 1mm per year. The testing performed in 2022 showed minimal carbonation on the exposed concrete surfaces.

Settlement Analyses

Settlement analyses were conducted using interpreted subsurface conditions based on the historical subsurface information. Based on the available plan information, we have assumed that the existing tower is bearing in the stiff to hard native fine-grained soils and engineering parameters used to develop a model of consolidation properties have been derived based on correlations to index properties from the 1986 exploration by others and our experience in the Greater Cincinnati Area. The consolidation parameters used are included in the following table.

Parameter	Value
Moist Unit Weight, γ	125 pcf
Effective Unit Weight, γ'[1]	67.6 pcf
Preconsolidation Pressure, P_c	6,000 psf
Compression Index, C_c	0.2
Recompression Index, C_r	0.05
Initial Void Ratio, e_o	0.7

1. Effective Unit Weight used below the water table assumed to be located two feet above the bedrock surface in the analyses.

Regarding loading conditions, bearing stresses for the existing tank or proposed tank were not available at the time of our preliminary services. In all analyses, it was assumed that the foundations are flexible, (i.e., the stress distribution on the ground immediately beneath the load is equal to the loading stress distribution). The existing tank was modeled as two independent ring foundations; although it is acknowledged that the two "rings" are connected with reinforced concrete struts that will limit differential movements of the foundations and that the foundations are reinforced such that there will be some redistribution of loading stress through the foundation. Analyses that consider these struts, the concrete reinforcement, and stress redistribution are beyond the scope of these preliminary services.

The bearing stress at the base of the existing tank foundations were estimated based on the existing tank plans and water volume. A detailed breakdown of the structural and architectural components was not performed and transient loads such as wind, snow, etc. were not considered. It was assumed that approximately 66 percent of the water and tank dead loads acted on the interior ring based on the tributary area while the remaining loads act on the outer ring. It was assumed that loading conditions for the scenario where the existing tank is improved from 0.5MG to 1MG or new proposed 1 MG tanks would at least double the load on the foundation system (i.e., the load would be at least twice that of the load on the existing foundation system). Additionally, the settlement analyses in this report have been updated from the version of this report dated September 4, 2025, based on the revised understanding that the proposed tower pedestal will be on the order of 54 feet in diameter.

The existing tank was modeled to bear at El. 880 feet while the proposed tank options were modeled to bear at El. 878 feet at locations northeast and northwest of the existing tank. The results of settlement analyses summarized in the table below.

Scenario (Bearing Stress)	Estimated Settlement Range[1] (inch)
Existing Tower - 0.5 MG - Full Tank	
Inner Ring (9.6 ksf[2])	0.9 – 2.0
Outer Ring (5 ksf)	0.6 – 1.1
Existing Tower - 0.5 MG – Water 2 Feet Below Overflow	
Inner Ring (9.1 ksf)	0.9 – 1.9
Outer Ring (4.9 ksf)	0.6 – 1.1
Existing Tower – 1.0 MG Upgrade	
Inner Ring (19.2 ksf)	2.0 – 3.6
Outer Ring (10 ksf)	0.8 – 2.3
Proposed Tower – 54’ Diameter Pedestal	
10’ Wide Ring Foundation (10 ksf)	1.2 – 4.3
10’ Wide Ring Foundation (13 ksf)	1.7 – 5.4
10’ Wide Ring Foundation (16 ksf)	2.2 – 6.3
70’ Dia. Mat Foundation (2 ksf)	0.5 – 1.3
70’ Dia. Mat Foundation (4 ksf)	0.9 – 2.0
70’ Dia. Mat Foundation (6 ksf)	1.1 – 3.1

1. Range accounts for variation in the bedrock surface as well as differences in stress accumulation at the edge of the ring versus near the middle of the ring.
2. ksf stands for kips per square foot. 1 kip is equivalent to 1,000 pounds.

In general, we anticipate that the predicted settlement from an increase in loading on the existing foundation system or the predicted settlement from loading on a new shallow foundation will not be tolerable for a soil-bearing shallow foundation system. As such, we recommend that preliminary planning efforts consider the following:

- Underpinning the existing foundations using elements extending through the compressible native fine-grained soils and into the underlying bedrock formation for scenarios involving upgrade of the existing tower.
  - Given the existing limitations on access/space inside of the existing tower, we anticipate that a system of micropiles may be required as other underpinning methods (excavation and mass concrete, drilled shafts, etc.) generally require more room to work while we do not anticipate that systems such as helical piles or push piers will provide sufficient capacity.
- For proposed new tank locations, the foundation system should consist of either:
  - A deep foundation system consisting of reinforced concrete drilled shafts and grade beams/structural slab;

- Extending the shallow ring foundation system to bear entirely on bedrock at deeper elevations; or
  - Based on the available information, this option will likely require temporary shoring to prevent excavations from either undermining existing infrastructure (e.g., underground utilities or existing foundations) or from extending beyond available construction limits (e.g., property lines).
- A shallow foundation system (ring or mat) supported by a ground improvement system.

During detailed design development, we recommend the following:

- Borings be obtained around the perimeter of the tank pedestal to better define the engineering properties of the encountered materials, potential groundwater conditions, the depth to bedrock.
  - Generally, boring layout should include at least 3 borings equally spaced around the perimeter of the pedestal and the exploration depths should be established to at least extend to the underlying bedrock. Sampling of the bedrock by overdriving with a split-spoon sampler or rock coring may be necessary depending on the potential options to evaluate.
- Samples of the native fine-grained materials be obtained subjected to consolidation testing to evaluate the consolidation parameters of the soils at the site.
- A structural engineer be retained to provide consulting on the structural elements of the existing tower or proposed tower including providing loads to use in updated settlement analyses.
- A civil engineer/surveyor be retained to develop the site plan including any required building setback limits and property lines.

Deep Foundations

The foundations for a proposed tank may be designed as a system of cast-in-place reinforced concrete drilled shafts and grade beams that transfer the structure loads to the underlying bedrock.

Drilled Shaft Design Parameters

We recommend that the axial compressive resistance of the drilled shafts be derived from end resistance only, and that side resistance be ignored. Soil design parameters are provided below in the **Drilled Shaft Design Summary** table for the design of drilled shaft foundations. The values presented for allowable side friction and end bearing include a factor of safety of 2.75 for skin friction and 3 for end bearing, respectively.

Drilled Shaft Design Summary [1]

Depth[2] (feet)	Stratigraphy [2]		Allowable Skin Friction (psf) [3]	Allowable End Bearing Pressure (psf) [4]
	No.	Material		
0 – 5	2/3	Existing Fill/Native Fine-Grained Soil	0	0
5 – TOR	3	Native Fine-Grained Soil	300	0
TOR - BOWR	4	Weathered Bedrock	600	8,000
BOWR	5	Unweathered Bedrock	600	20,000

1. Design capacities are dependent upon the method of installation and quality control parameters. The values provided are estimates and should be verified when installation protocols have been finalized.
2. Depth below proposed ground surface grades.
   TOR = Top of Rock Depth
   BOWR = Base of Weathered Rock
3. See Subsurface Profiles in **Figures** for more details on stratigraphy.
4. Applicable for compressive loading only. Reduce to 2/3 of values shown for uplift loading. The effective weight of the shaft can be added to uplift load resistance to the extent permitted by IBC.
5. Shafts should extend at least one diameter into the required bearing stratum.

Shafts should be reinforced as designed by the Structural Engineer for both tension and shear to sufficient depths. Buoyant unit weights of the soil and concrete should be used in the calculations below the highest anticipated groundwater elevation.

Drilled shaft should have a minimum (center-to-center) spacing of three diameters. Closer spacing may require a reduction in axial load capacity. Axial capacity reduction can be determined by comparing the allowable axial capacity determined from the sum of individual piles in a group versus the capacity calculated using the perimeter and base of the pile group acting as a unit. The lesser of the two capacities should be used in design.

Drilled shafts should bear at least 5 times the shaft diameter below the ground surface and grade beams and should extend into the bearing strata at least one shaft diameter for the allowable end-bearing pressures listed in the above table.

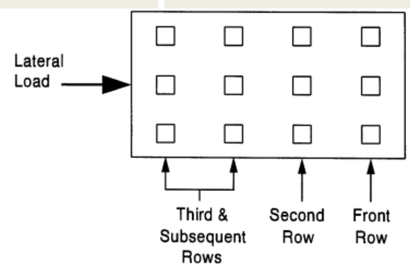
Drilled Shaft Lateral Loading

Table 1 in **Supporting Information** lists input values for use in LPILE analyses. Since deflection or a service limit criterion will most likely control lateral capacity design, no safety/resistance factor is included with the parameters.

When shafts are used in groups, the lateral capacities of the shafts in the second, third, and subsequent rows of the group should be reduced as compared to the capacity of a single, independent shaft. Guidance for applying p-multiplier factors to the p values in the p-y curves for each row of pile foundations within a pile group are as follows:

Center to Center Pile Spacing [1,2]	P-Multiplier, P_m [3]		
	Front Row	Second Row	Third and Subsequent Rows
3B	0.8	0.4	0.3
4B	0.9	0.65	0.5
5B	1.0	0.85	0.7
6B	1.0	1.0	1.0

1. Spacing in the direction of loading. B = pile diameter
2. For the case of a single row of piles supporting a laterally loaded grade beam, group action for lateral resistance of piles would need be considered when spacing is less than three pile diameters (measured center-to-center).
3. See adjacent figure for definition of front, second and third rows.



Spacing closer than 3D (where D is the diameter of the shaft) is not recommended without additional geotechnical consultation due to potential for the installation of a new shaft disturbing an adjacent installed shaft likely resulting in axial capacity reduction.

Pile caps and/or grade beams could be subject to uplift loading due to frost action; thus, pile caps and/or grade beams should extend at least 2.5 feet below the lowest adjacent finished grade for frost protection.

The load capacities provided herein are based on the stresses induced in the supporting soil strata. The structural capacity of the shafts should be checked to assure they can safely accommodate the combined stresses induced by axial and lateral forces. Lateral deflections of shafts/piles should be evaluated using an appropriate analysis method, and will depend upon the pile's diameter, length, configuration, stiffness and "fixed head" or "free head" condition. We can provide additional analyses and estimates of lateral deflections for specific loading conditions upon request. The load-carrying capacity of shafts may be improved by increasing the diameter and possibly the length.

Drilled Shaft Construction Considerations

The drilling contractor should be experienced in the subsurface conditions observed at the site, and the excavations should be performed with equipment capable of providing a clean bearing surface. The drilled straight-shaft foundation system should be installed in general accordance with the procedures presented in "Drilled Shafts: Construction Procedures and Design Methods," FHWA Publication No. FHWA-NHI 18-024.

The drilled shaft excavations should be made straight and plumb with level bottoms, using dry construction methods. Loose, soft, wet, or otherwise disturbed materials should be removed from the bearing surfaces to expose undisturbed bedrock before the reinforcing steel and concrete are placed. Concrete should not be placed through more than 3 inches of water in the bottom of any shaft, and the rate of inflow of groundwater should be less than 12 inches per hour, unless wet construction methods are implemented. Concrete should also be placed the same day that the bedrock sockets of the shafts are drilled to prevent softening/slaking of the soils and/or bedrock in the drilled shafts. We recommend that each drilled shaft excavation be reviewed by the Project Geotechnical Engineer, or a representative thereof, to confirm that the soil and bedrock conditions encountered within the drilled shaft are consistent with those encountered in the borings and with the design recommendations of this report.

Subsurface water levels are influenced by seasonal and climatic conditions, which result in fluctuations in subsurface water elevations. Additionally, it is common for water to be present after periods of significant rainfall. While not anticipated based on the available information, full-depth temporary casing from the ground surface to the top of bedrock may be needed to control groundwater and/or caving overburden soils. We recommend that the Contract Documents include a bid item for casing shafts as recommended by the Project Geotechnical Engineer, or the representative thereof, on a "cost per cased shaft" basis.

While withdrawing temporary casing, care should be exercised to maintain concrete inside the casing at a sufficient level to resist earth and hydrostatic pressures acting on the casing exterior. Arching of the concrete, loss of seal and other problems can occur during casing removal and result in contamination of the drilled shaft. These conditions should be considered during the design and construction phases. Placement of loose soil backfill should not be permitted around the casing prior to removal.

The drilled shaft installation process should be performed under the observation of the Geotechnical Engineer. The Geotechnical Engineer should document the shaft installation process including soil/rock and groundwater conditions observed, consistency with expected conditions, and details of the installed shaft.

Shallow Foundations on Bedrock

As indicated in **settlement analyses**, shallow foundations may be considered for two scenarios for a new tank. Either a ring foundation should be deepened to bear entirely on bedrock as discussed in this section or a ring can bear in the overburden if a **ground improvement** system is incorporated into the design.

Design Parameters – Compressive Loads

Item	Description
Maximum Net Allowable Bearing Pressure [1, 2]	8,000 psf - foundations bearing upon intact, weathered shale bedrock
Required Bearing Stratum [3]	GeoModel Layers 4 or 5
Ultimate Passive Resistance[4] **(equivalent fluid pressures)**	220 pcf (cohesive soils) 600 pcf (shale bedrock)
Sliding Resistance [5]	0.35 ultimate coefficient of friction
Minimum Embedment below Finished Grade [6]	30 inches

1. The maximum net allowable bearing pressure is the pressure in excess of the minimum surrounding overburden pressure at the footing base elevation.
2. Preliminary values provided are based on local experience. Final evaluation will be necessary based on recovered samples of the bedrock.
3. The bearing elevation should be deepened or the overburden soils undercut to the required bearing stratum and replaced with lean concrete to design bearing elevation.
4. Use of passive earth pressures require the sides of the excavation for the spread footing foundation to be nearly vertical and the concrete placed neat against these vertical faces. Passive earth pressure should not be accounted for above the minimum embedment below finished grade
5. Can be used to compute sliding resistance where foundations are placed on suitable intact weathered shale bedrock. Frictional resistance is dependent on the bearing pressure which may vary due to load combinations.
6. Embedment necessary to minimize the effects of frost and/or seasonal water content variations.

Design Parameters – Overturning

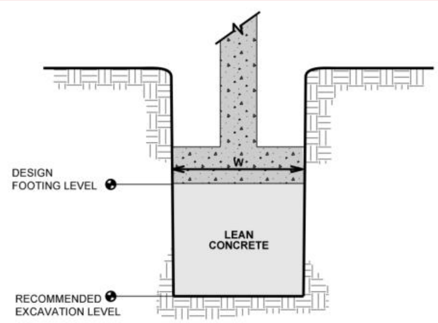
Shallow foundations subjected to overturning loads should be proportioned such that the resultant eccentricity is maintained in the center-third of the foundation (e.g., $e < b/6$, where b is the foundation width). This requirement is intended to keep the entire foundation area in compression during the extreme lateral/overturning load event. Foundation oversizing may be required to satisfy this condition.

Foundation Construction Considerations

The footing excavations should be evaluated under the observation of the Geotechnical Engineer. The base of all foundation excavations should be free of water and loose soil/bedrock, prior to placing concrete. If a limestone layer is exposed in the bottom of the footing excavation, we recommend that the excavation be deepened to penetrate the limestone layer, unless it can be determined that there is no softening of the shale beneath the limestone. Additionally, disturbed or loosened beds of limestone should be removed from the bearing surface.

Concrete should be placed soon after excavating to reduce bearing soil disturbance. Care should be taken to prevent wetting or drying of the bearing materials during construction. Excessively wet or dry material or any loose/disturbed material in the bottom of the footing excavations should be removed/reconditioned before foundation concrete is placed.

As an alternative to lowering the footing bearing level to bear entirely on intact bedrock, the excavation could be extended from design bearing elevation to the top of bedrock, and the footings could bear directly on lean concrete backfill placed in the excavations. The lean concrete replacement zone is illustrated on the sketch below.



LEAN CONCRETE BACKFILL

NOTE: EXCAVATIONS ARE SHOWN VERTICAL; HOWEVER, THE SIDEWALLS SHOULD BE SLOPED AS NECESSARY FOR SAFETY

Regardless of whether the footing bearing elevation is lowered or if the lean concrete backfill option is utilized, final design will need to consider the relationship between nearby existing infrastructure to remain in service (e.g., existing foundations, underground reservoirs, underground utilities,

Ground Improvement

As an alternative to extending shallow foundation excavations to bedrock, the structures could be supported on spread footings or a mat foundation within existing native soils if ground improvement methods are utilized. Ground improvement methods are proprietary systems designed by licensed contractors who could provide further information regarding support options.

We anticipate compacted aggregate piers to be well-suited for the site. Aggregate piers systems are used beneath foundations to improve the bearing capacity of soils and to control settlement. The system is a proprietary design that depends upon the means of aggregate delivery, compaction method and whether the soil profile can maintain an open shaft. Typically, the specialty contractor is provided design spread footing loads and a settlement tolerance.

The aggregate piers are constructed by drilling a hole (typically 24 to 30 inches in diameter) to the specified design depth by auguring into the ground. The installation method selected for the aggregate piers should prevent the loss of bearing materials supporting existing structures (e.g., temporary casing may be necessary for the installation of aggregate piers). Thin lifts (6- to 12-inch-thick) of select aggregate (typically crushed stone or equivalent) are then compacted in the cavity with a high-impact or vibration densification system. The structure can then be supported on spread footings or mat foundation bearing on soil improved with the aggregate pier system. One demonstration pier and on-site load testing (modulus testing) is strongly recommended to confirm the performance of the aggregate pier system.

In addition to pre-construction and post-construction conditions reviews, the owner or contractor should also perform vibration monitoring during installation of the aggregate piers.

Spread Footings

From experience, spread footings bearing on soils improved with aggregate pier systems can be designed based on an estimated allowable bearing capacity of 5,000 psf or more. However, the design will be performed by aggregate pier specialty contractor and bearing capacities are dependent on subsurface conditions and aggregate pier element spacing and length. The aggregate pier depths should be determined by the specialty contractor’s engineer, based upon loads and tolerable settlement criteria set by the project structural engineer, but we anticipate that the elements would extend to bedrock at this site. On a preliminary basis, an ultimate friction coefficient of 0.5 can be used between the concrete footings and underlying aggregate pier-enhanced soil and should be applied to dead normal loads only. Detailed foundation design will be performed by the aggregate pier engineer/specialty contractor based on a performance basis.

Mat Foundations

Based on our understanding of the project, we anticipate that a mat foundation will be suitable for support of the proposed water tower if utilized with a ground improvement system. The planned mat has been assumed near approximate elevation 878, and our analysis has assumed compressible soil strata to the bedrock surface.

The design of mat foundations is typically performed in an iterative process between the Geotechnical Engineer and Structural Engineer. Use of finite element analysis is often performed when loads acting on the mat are asymmetric or non-uniform, but estimates of soil pressure and resulting settlement can converge quickly absent of sophisticated design procedures in cases where uniform loads are acting upon the mat. This procedure is outlined in ACI 336.2 “Suggested Analysis and Design Procedure for Combined Footings and Mats”.

Due to the project description, we anticipate that the mat will include areas of higher stresses despite provisions made to promote mat rigidity. Analysis of such complex conditions is often best conducted with the use of finite element modeling. This analysis is not included in our current scope of services. If conducted by others, an initial modulus of subgrade reaction of 25 pounds per cubic inch (pci) can be assumed for the subgrade across the mat, however, the aggregate pier engineer/specialty contractor should provide additional analysis and consultation following development of the mat stresses by the Structural Engineer to provide refined estimates of the crude initial estimate provided above. The Structural Engineer and the aggregate pier engineer/specialty contractor should work together to provide this analysis in an iterative manner. Alternatively, if the Structural Engineer can provide the loads acting above the mat, Terracon can provide a proposal to perform the analysis and design of the mat foundation.

General Construction Considerations

The following list of general considerations will apply to design and construction regardless of the selected foundation system or option to advance to final design.

- Expect the existing paved driveway to not hold up to construction loads. Replacement of access to the facility should be included in the design and construction.
- Delivery of equipment and materials to the site may require widening of the existing driveway and relocation of signage along Miami Road or may require temporary traffic control to facilitate access back the driveway depending on the size of the load that is delivered.
- We understand that the existing underground reservoirs were not designed to accommodate surcharge loads or construction loading. An appropriate buffer should be established during design and should be maintained during construction with visual aids to prevent damage to the existing infrastructure. For preliminary planning, assume buffer zone based on a 1H:1V slope up and away from the bearing elevation of the underground reservoirs plus 5 additional feet.

- The location of all existing underground utilities must be accurately determined during final design and markings maintained during construction. Potential loss of support for pressurized pipelines due to nearby excavations or overstressing the pipeline due to excessive movements/loading would result in a loss of service for IH until repairs could be made. Relocation of utilities may be necessary depending on final design.
- It is anticipated that a crane and material stockpile areas will be needed for construction. Construction planning should consider the potential swing radii and load limitations for potential equipment required by contractors. Construction easements may need to be considered.
- Existing overhead utilities (both at the tank and along Miami Road) may impact final design and construction. Consideration should be given to relocating existing overhead lines around the existing tank underground to reduce the potential constraint for construction equipment.
- Additionally, consideration may need to be given to temporarily relocating low wires crossing the driveway along Miami Road and trimming trees along the driveway depending on the size of loads/equipment to be delivered.

General Comments

Our analysis and opinions are based upon our understanding of the project, the geotechnical conditions in the area, and the data obtained from our site exploration. Variations will occur between exploration point locations or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. Terracon should be retained as the Geotechnical Engineer, where noted in this report, to provide observation and testing services during pertinent construction phases. If variations appear, we can provide further evaluation and supplemental recommendations. If variations are noted in the absence of our observation and testing services on-site, we should be immediately notified so that we can provide evaluation and supplemental recommendations.

Our Scope of Services does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

Our services and any correspondence are intended for the sole benefit and exclusive use of our client for specific application to the project discussed and are accomplished in accordance with generally accepted geotechnical engineering practices with no third-party beneficiaries intended. Any third-party access to services or correspondence is solely for information purposes to support the services provided by Terracon to our client. Reliance upon the services and any work product is limited to our client and is not intended for third parties. Any use or reliance of the provided information by third parties is done solely at their own risk. No warranties, either express or implied, are intended or made.

Site characteristics as provided are for design purposes and not to estimate excavation cost. Any use of our report in that regard is done at the sole risk of the excavating cost estimator as there may be variations on the site that are not apparent in the data that could significantly affect excavation cost. Any parties charged with estimating excavation costs should seek their own site characterization for specific purposes to obtain the specific level of detail necessary for costing. Site safety and cost estimating including excavation support and dewatering requirements/design are the responsibility of others. Construction and site development have the potential to affect adjacent properties. Such impacts can include damages due to vibration, modification of groundwater/surface water flow during construction, foundation movement due to undermining or subsidence from excavation, as well as noise or air quality concerns. Evaluation of these items on nearby properties are commonly associated with contractor means and methods and are not addressed in this report. The owner and contractor should consider a pre-construction and post-construction review of surrounding development. If changes in the nature, design, or location of the project are planned, our conclusions and recommendations shall not be considered valid unless we review the changes and either verify or modify our conclusions in writing.

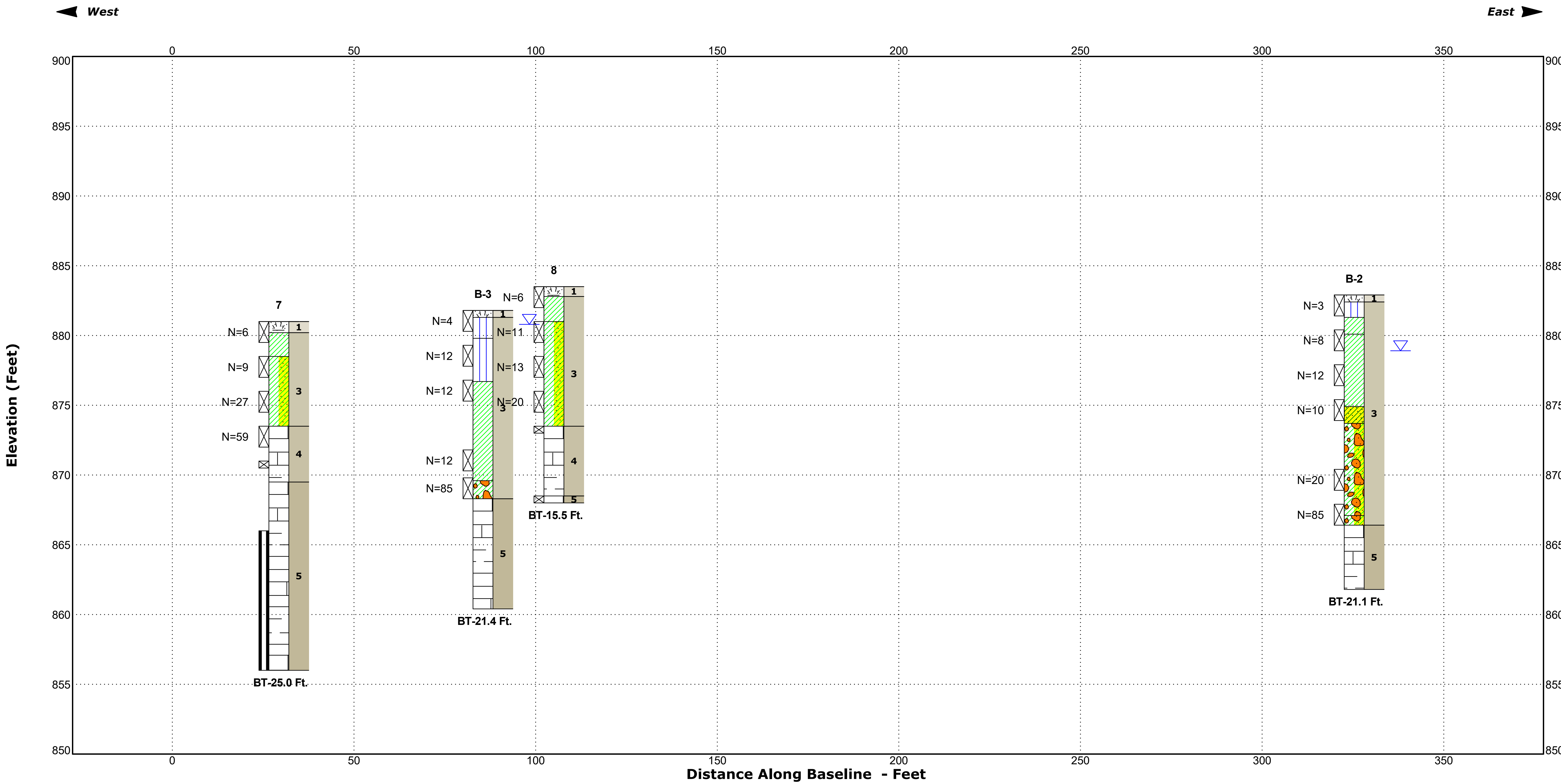
Figures

Contents:

Subsurface Profile

Section A-A

Terracon
611 Lunken Park Dr
Cincinnati, OH



Notes	Water Level Observations	Explanation
See Exploration Plan for orientation of soil profile. See General Notes in Supporting Information and GeoModelfor symbols and soil classifications. Soils profile provided for illustration purposes only. Soils between borings may differ AR - Auger Refusal BT - Boring Termination	Water Level Reading at time of drilling. Water Level Reading after drilling.	Sampling (See General Notes); Borehole Number; GeoModel Layer; Borehole Lithology; AR BT Borehole Termination Type

Material Legend

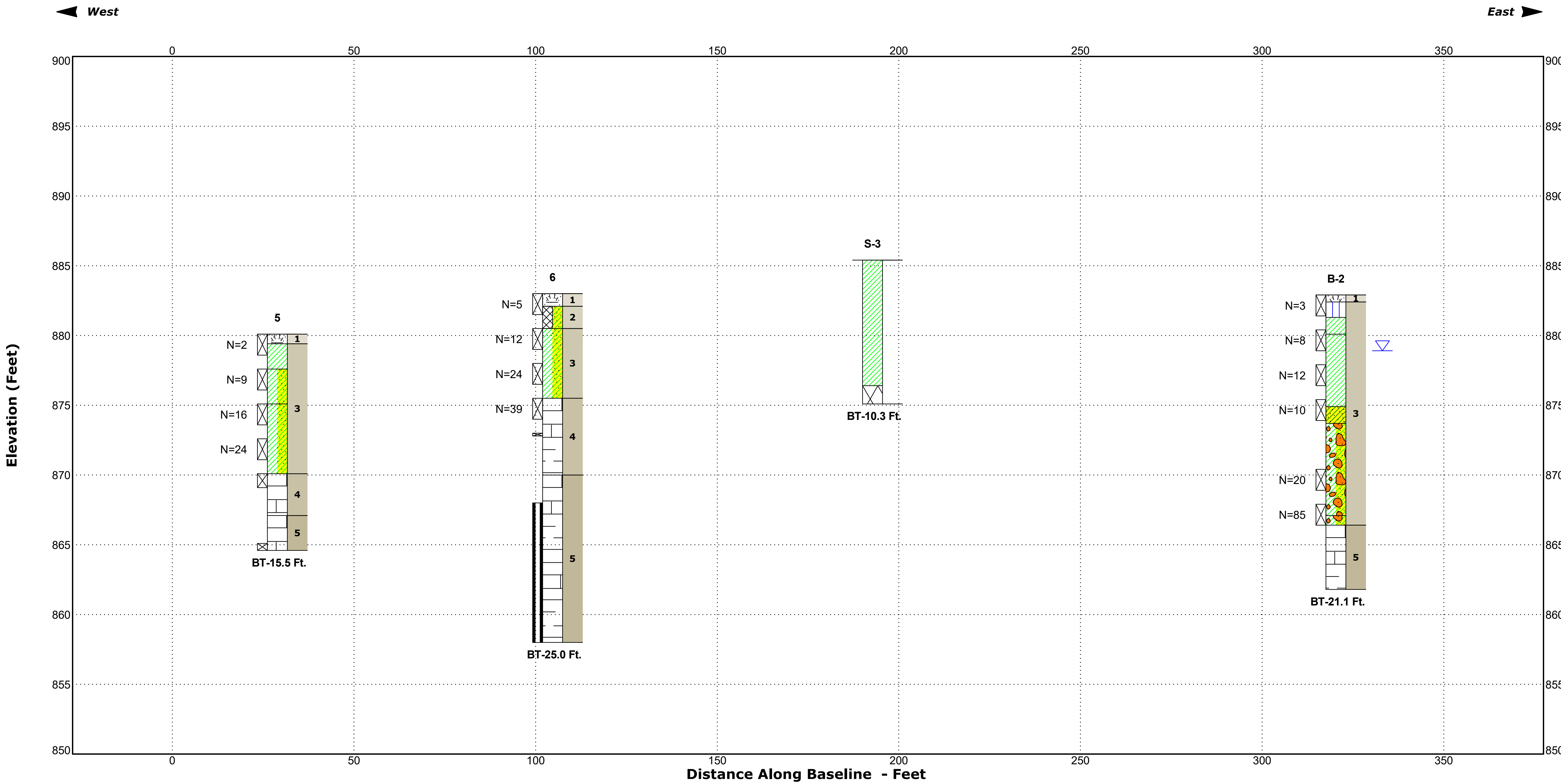
Topsoil, Lean Clay, Lean Clay with Sand, Interbedded Limestone and Shale, Silt, Sandy Lean Clay, Gravelly Lean Clay with Sand, Gravelly Lean Clay

Preliminary Water Tower Site Evaluation
5355 Miami Road | Cincinnati, OH
Terracon Project No. N1255189

Subsurface Profile

Section B-B

Terracon
611 Lunken Park Dr
Cincinnati, OH



Notes	Water Level Observations	Explanation
See Exploration Plan for orientation of soil profile. See General Notes in Supporting Information and GeoModelfor symbols and soil classifications. Soils profile provided for illustration purposes only. Soils between borings may differ AR - Auger Refusal BT - Boring Termination	Water Level Reading at time of drilling. Water Level Reading after drilling.	Borehole Number Sampling (See General Notes) GeoModel Layer Borehole Lithology AR BT Borehole Termination Type

Material Legend

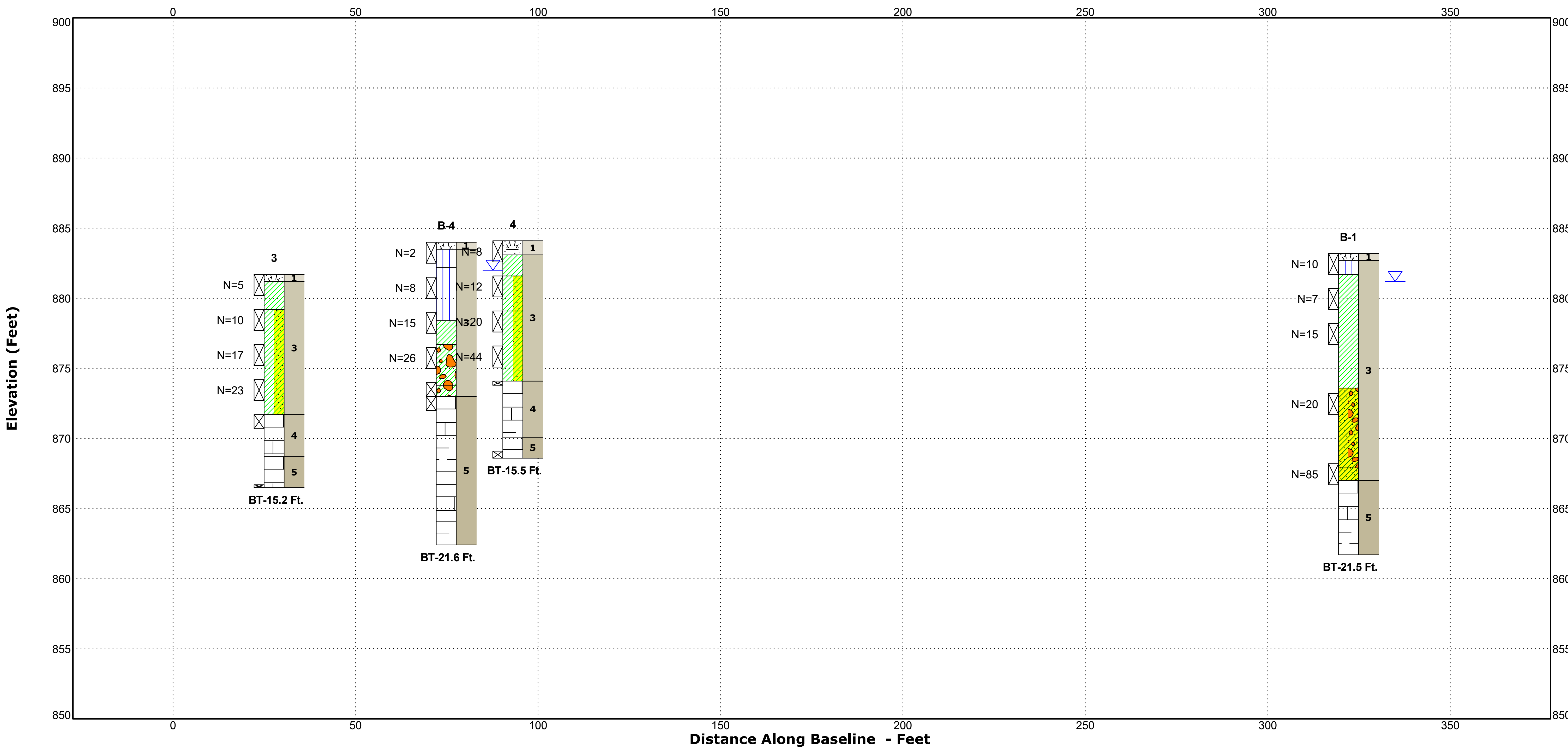
Topsoil | Lean Clay | Lean Clay with Sand | Interbedded Limestone and Shale | Silt | Sandy Lean Clay | Gravelly Lean Clay with Sand | Weathered Shale

Preliminary Water Tower Site Evaluation
5355 Miami Road | Cincinnati, OH
Terracon Project No. N1255189

Subsurface Profile

Section C-C

611 Lunken Park Dr
Cincinnati, OH



Notes	Water Level Observations	Explanation
See Exploration Plan for orientation of soil profile. See General Notes in Supporting Information and GeoModelfor symbols and soil classifications. Soils profile provided for illustration purposes only. Soils between borings may differ AR - Auger Refusal BT - Boring Termination	Water Level Reading at time of drilling. Water Level Reading after drilling.	Borehole Number Sampling (See General Notes) GeoModel Layer Borehole Lithology AR BT Borehole Termination Type

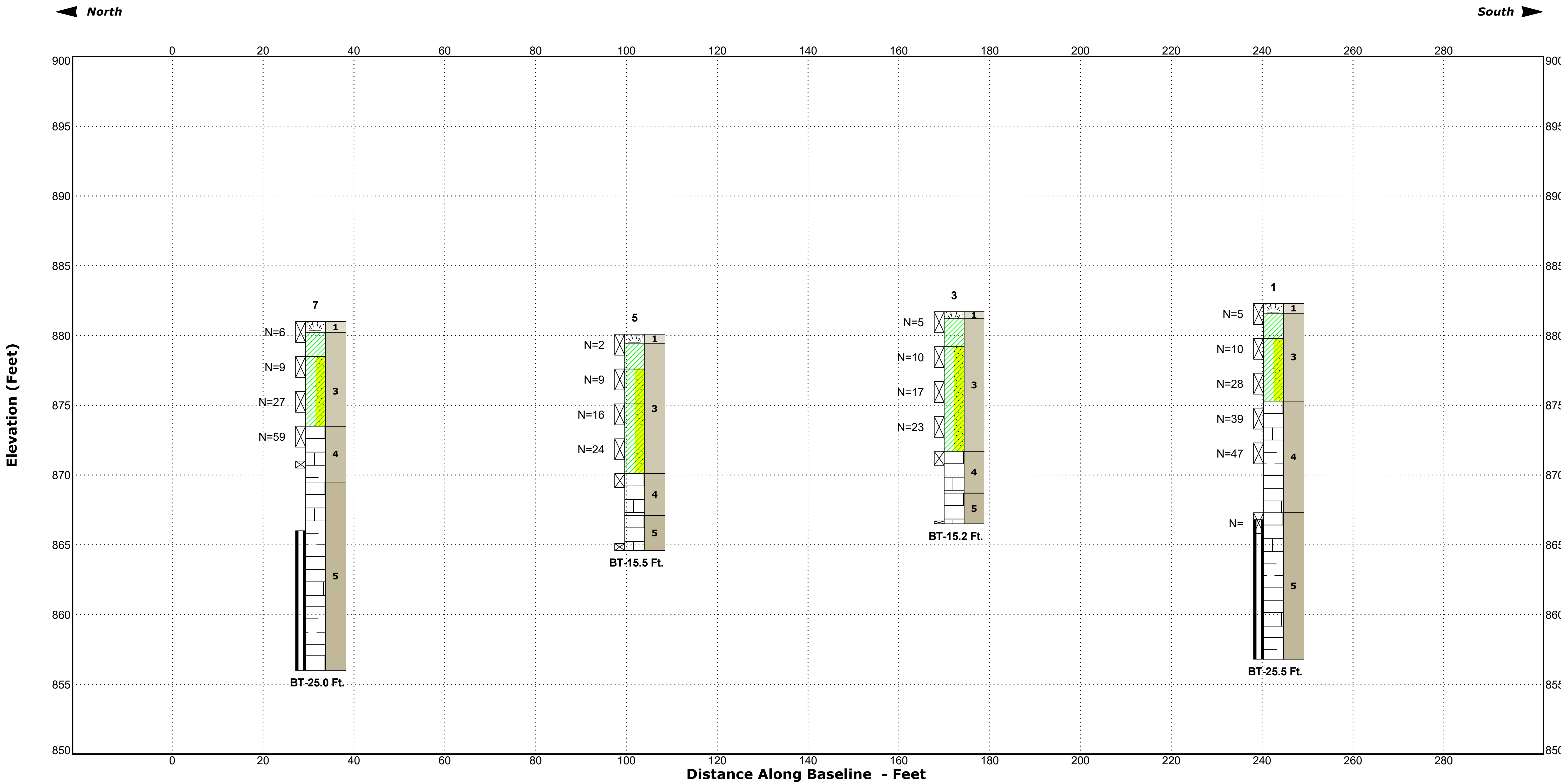
Material Legend

Topsoil; Lean Clay; Lean Clay with Sand; Interbedded Limestone and Shale; Silt; Sandy Lean Clay with Gravel; Sandy Lean Clay; Gravelly Lean Clay

Subsurface Profile

Section D-D

Terracon
611 Lunken Park Dr
Cincinnati, OH

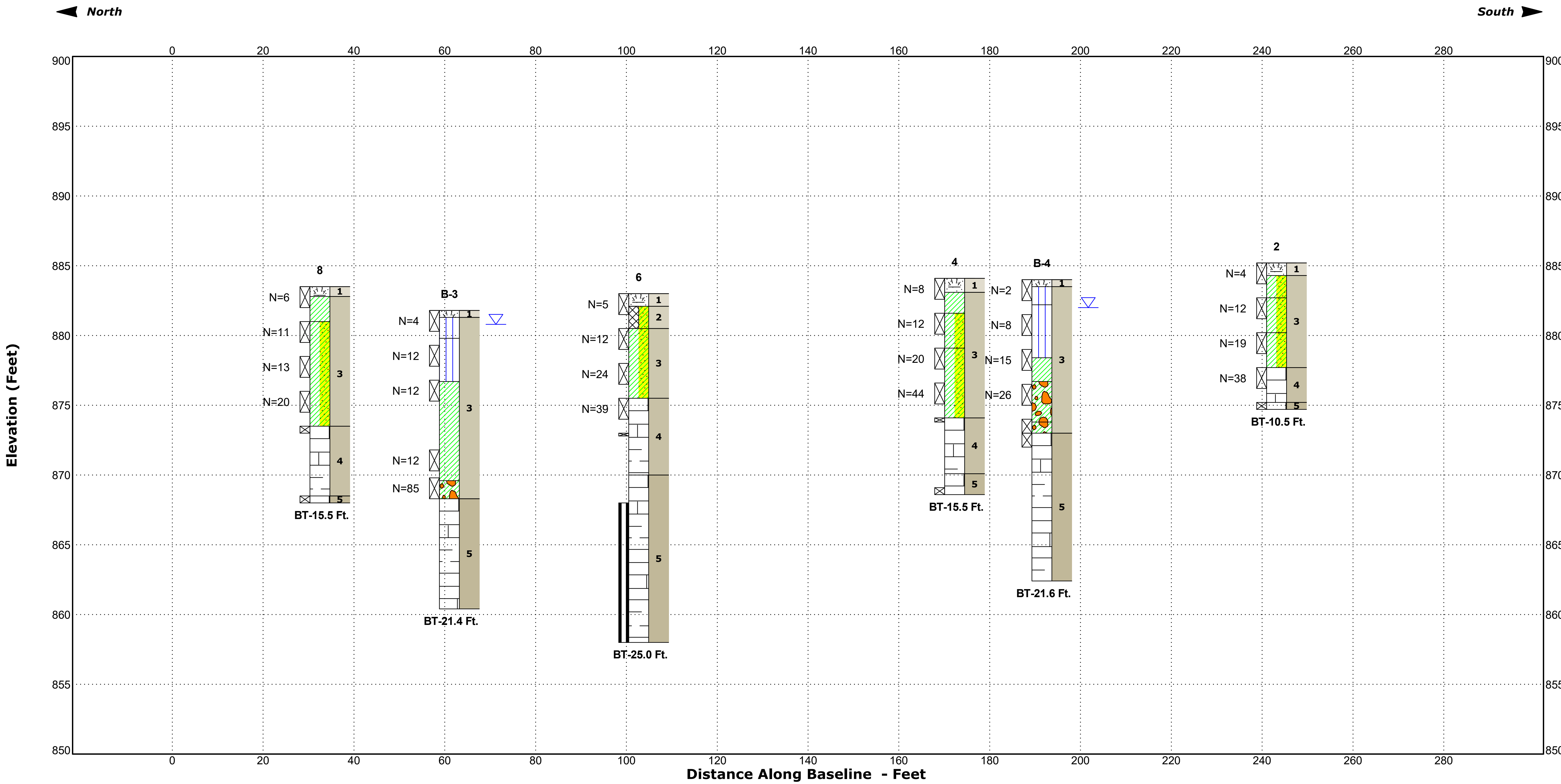


Notes	Water Level Observations	Explanation	Material Legend
See Exploration Plan for orientation of soil profile. See General Notes in Supporting Information and GeoModelfor symbols and soil classifications. Soils profile provided for illustration purposes only. Soils between borings may differ AR - Auger Refusal BT - Boring Termination	Water Level Reading at time of drilling. Water Level Reading after drilling.	1 — Borehole Number; Sampling (See General Notes); GeoModel Layer; Borehole Lithology; AR BT — Borehole Termination Type	Topsoil; Lean Clay; Lean Clay with Sand; Interbedded Limestone and Shale

Subsurface Profile

Section E-E

611 Lunken Park Dr
Cincinnati, OH



Notes	Water Level Observations	Explanation
See Exploration Plan for orientation of soil profile. See General Notes in Supporting Information and GeoModelfor symbols and soil classifications. Soils profile provided for illustration purposes only. Soils between borings may differ AR - Auger Refusal BT - Boring Termination	Water Level Reading at time of drilling. Water Level Reading after drilling.	Borehole Number; Sampling (See General Notes); GeoModel Layer; Borehole Lithology; AR BT Borehole Termination Type

Material Legend

Topsoil | Lean Clay with Sand | Interbedded Limestone and Shale | Lean Clay | Silt | Gravelly Lean Clay

Attachments

Concrete Coring and Testing Procedures

Field Services

Number of Locations	Location
3	Existing Tower Foundation

Core Location Layout: Terracon personnel reviewed public utility markings resulting from the request to OH811 and consulted with representatives from IH who provided access to the facility regarding locations of underground utilities and buried objects. Terracon personnel also performed private utility clearing of detectable underground utilities or obstructions around the perimeter of the existing water tower using non-invasive means, such as GPR and Electromagnetic (EM) methods. The detection of underground utilities is dependent upon the composition and construction of the utility line; some utilities are comprised of non-electrically conductive materials and may not be readily detected. Based on the results of the review of public utility markings, the consultation with representatives from IH, and the private utility clearance; three locations around the perimeter of the existing water tank were selected to expose the top of the concrete foundation. The approximate locations were recorded using field measurements from existing physical features and are shown on the **Exploration Plan**.

Coring Procedure: At each core location, the top of the concrete foundation was exposed by Terracon personnel using a mini-excavator and hand tools. After the top of the concrete foundation was exposed, ground penetrating radar was used to ensure that core locations were clear of reinforcing steel bars or other embedded items. A coring subcontractor cored and extracted an approximately 3.75-inch diameter core of the foundation concrete at each location and the core hole was patched by Terracon Personnel using fast setting bag-mix concrete.

Field measurements of the concrete foundations were also obtained before backfilling the excavations. In general, the depth to the top of the concrete foundation varied from approximately 4 to 4.5 feet below existing grade along the wall. The foundation was cast such that the surface of the footing slopes down and away from the wall for another foot vertically such that the top of the outer edge of the footing was roughly 5 to 5.5 feet below the ground surface. The outer edge of the footing was located between 2.5 and 2.7 feet from the outside face of the wall. After completion of the field measurements, the excavations were backfilled with soil in thin lifts by tamping with the bucket of the mini-excavator and the recovered cores were returned to our laboratory for compressive strength testing.

Laboratory Testing

The three concrete cores obtained as part of the services for this project were subjected to compressive strength testing in accordance with ASTM C42. The results of the compressive strength testing are included in the **Supporting Information**.

Site Location and Exploration Plans

Contents:

Note: All attachments are one page unless noted above.

Site Location Plan

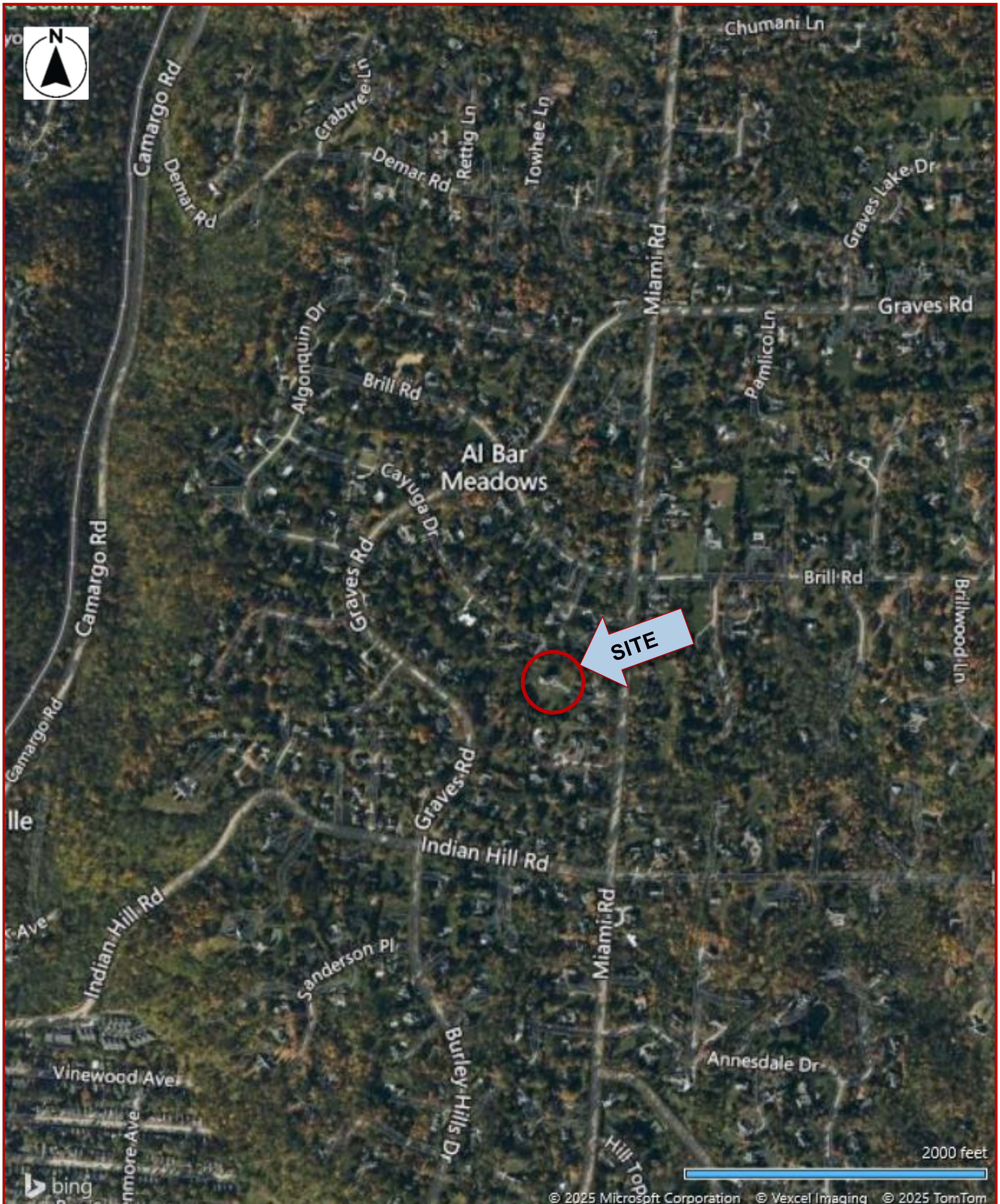


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES   MAP PROVIDED BY MICROSOFT BING MAPS

Exploration Plan

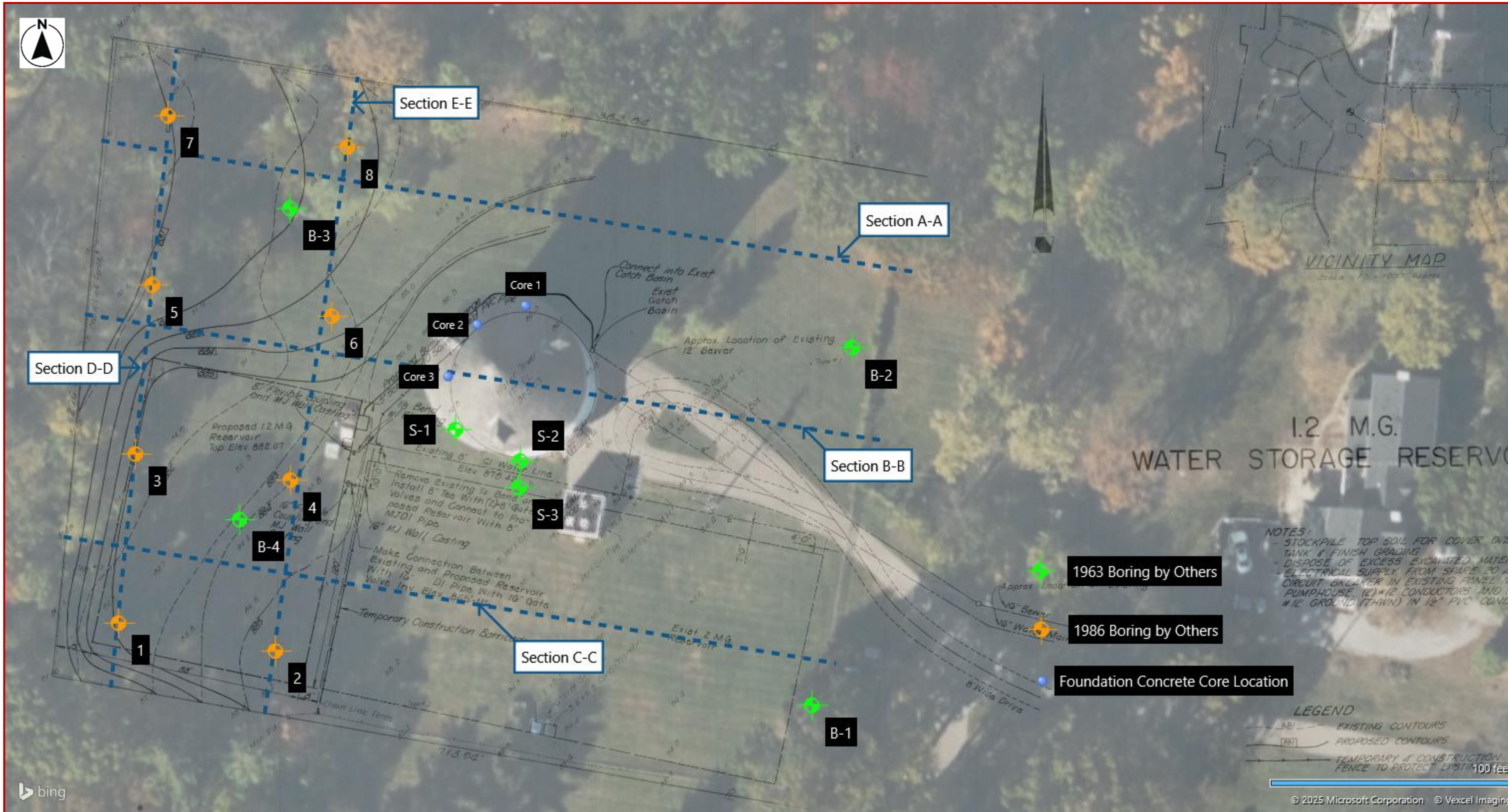


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS, SITE PLAN FROM 1986 PLANS FOR 1.2 MG WATER STORAGE RESERVOIR PROVIDED BY VILLAGE OF INDIAN HILL

Site Plan with Bedrock Contours

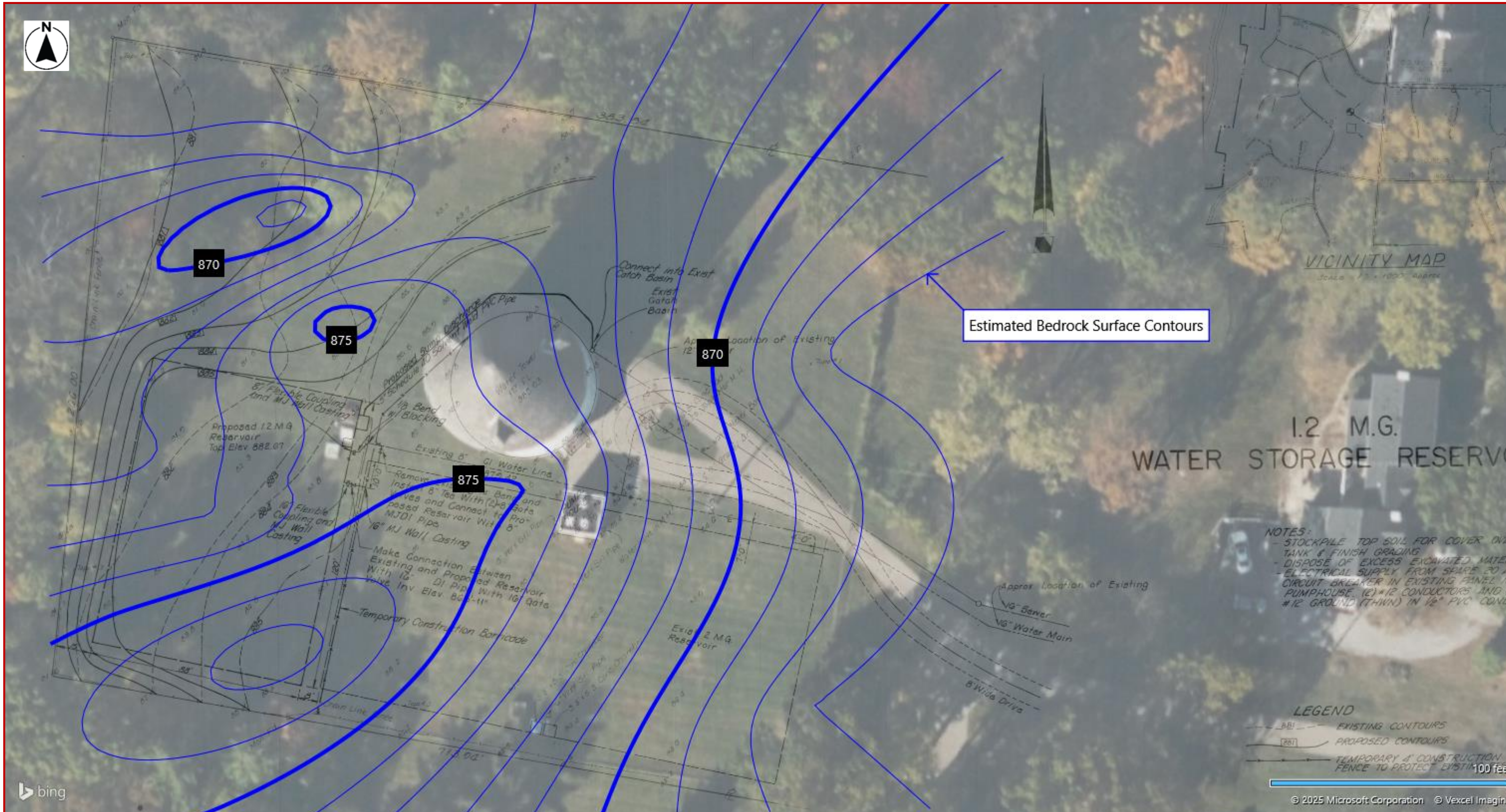


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS, SITE PLAN FROM 1986 PLANS FOR 1.2 MG WATER STORAGE RESERVOIR PROVIDED BY VILLAGE OF INDIAN HILL

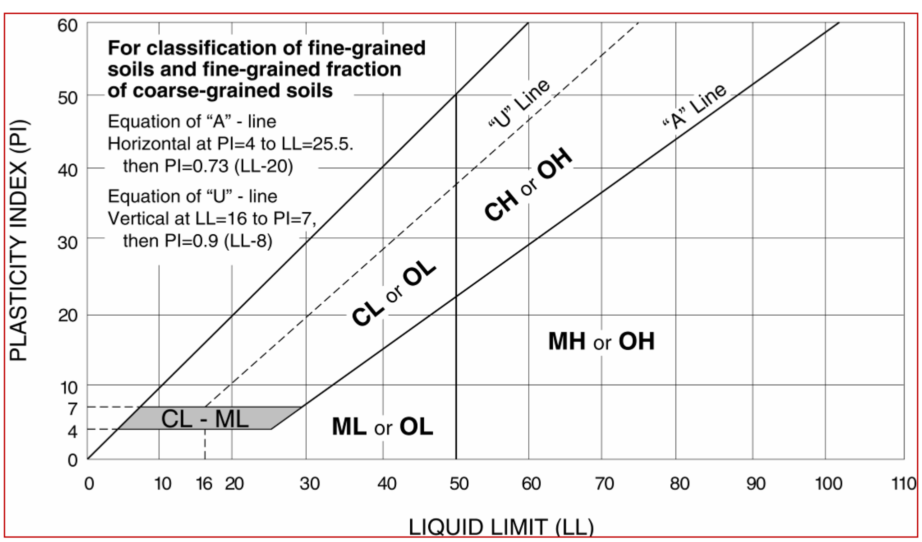
Supporting Information

Contents:

Unified Soil Classification System

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]				Soil Classification	
				Group Symbol	**Group Name [B]**
Coarse-Grained Soils: More than 50% retained on No. 200 sieve	**Gravels:** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels:** Less than 5% fines [C]	Cu≥4 and 1≤Cc≤3 [E]	GW	Well-graded gravel [F]
			Cu<4 and/or [Cc<1 or Cc>3.0] [E]	GP	Poorly graded gravel [F]
		Gravels with Fines: More than 12% fines [C]	Fines classify as ML or MH	GM	Silty gravel [F, G, H]
			Fines classify as CL or CH	GC	Clayey gravel [F, G, H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands:** Less than 5% fines [D]	Cu≥6 and 1≤Cc≤3 [E]	SW	Well-graded sand [I]
			Cu<6 and/or [Cc<1 or Cc>3.0] [E]	SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH	SM	Silty sand [G, H, I]
			Fines classify as CL or CH	SC	Clayey sand [G, H, I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	PI > 7 and plots above "A" line [J]	CL	Lean clay [K, L, M]
			PI < 4 or plots below "A" line [J]	ML	Silt [K, L, M]
		Organic:	$\frac{LL\ oven\ dried}{LL\ not\ dried} < 0.75$	OL	Organic clay [K, L, M, N]
					Organic silt [K, L, M, O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above "A" line	CH	Fat clay [K, L, M]
			PI plots below "A" line	MH	Elastic silt [K, L, M]
		Organic:	$\frac{LL\ oven\ dried}{LL\ not\ dried} < 0.75$	OH	Organic clay [K, L, M, P]
					Organic silt [K, L, M, Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-inch (75-mm) sieve.
[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.
[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay.
[E] $Cu = D_{60}/D_{10}$ $\quad Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$
[F] If soil contains ≥ 15% sand, add "with sand" to group name.
[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
[H] If fines are organic, add "with organic fines" to group name.
[I] If soil contains ≥ 15% gravel, add "with gravel" to group name.
[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.
[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.
[L] If soil contains ≥ 30% plus No. 200 predominantly sand, add "sandy" to group name.
[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.
[N] PI ≥ 4 and plots on or above "A" line.
[O] PI < 4 or plots below "A" line.
[P] PI plots on or above "A" line.
[Q] PI plots below "A" line.



Rock Classification Notes

WEATHERING	
Term	**Description**
Fresh	Mineral crystals appear bright; show no discoloration. Features show little or no staining on surfaces. Discoloration does not extend into intact rock.
Slightly weathered	Rock generally fresh except along fractures. Some fractures stained and discoloration may extend <0.5 inches into rock.
Moderately weathered	Significant portions of rock are dull and discolored. Rock may be significantly weaker than in fresh state near fractures. Soil zones of limited extent may occur along some fractures.
Highly weathered	Rock dull and discolored throughout. Majority of rock mass is significantly weaker and has decomposed and/or disintegrated; isolated zones of stronger rock and/or soil may occur throughout.

STRENGTH OR HARDNESS		
Description	**Field Identification**	**Uniaxial Compressive Strength, psi**
Extremely strong	Can only be chipped with geological hammer. Rock rings on hammer blows. Cannot be scratched with a sharp pick. Hand specimens require several hard hammer blows to break.	>36,000
Very strong	Several blows of a geological hammer to fracture. Cannot be scratched with a 20d common steel nail. Can be scratched with a geologist's pick only with difficulty.	15,000-36,000
Strong	More than one blow of a geological hammer needed to fracture. Can be scratched with a 20d nail or geologist's pick. Gouges or grooves to ¼ inch deep can be excavated by a hard blow of a geologist's pick. Hand specimens can be detached by a moderate blow.	7,500-15,000
Medium strong	One blow of geological hammer needed to fracture. Can be distinctly scratched with 20d nail. Can be grooved or gouged 1/16 in. deep by firm pressure with a geologist's pick point. Can be fractured with single firm blow of geological hammer. Can be excavated in small chips (about 1-in. maximum size) by hard blows of the point of a geologist's pick.	3,500-7,500
Weak	Shallow indent by firm blow with geological hammer point. Can be gouged or grooved readily with geologist's pick point. Can be excavated in pieces several inches in size by moderate blows of a pick point. Small thin pieces can be broken by finger pressure.	700-3,500
Very weak	Crumbles under firm blow with geological hammer point. Can be excavated readily with the point of a geologist's pick. Pieces 1 in. or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail.	150-700
Extremely Weak	Indented by thumbnail.	40-150

DISCONTINUITY DESCRIPTION			
Fracture Spacing (Joints, Faults, Other Fractures)		**Bedding Spacing (May Include Foliation or Banding)**	
Description	**Spacing**	**Description**	**Spacing**
Intensely fractured	< 2.5 inches	**Laminated**	< ½ inch
Highly fractured	2.5 to 8 inches	**Very thin**	½ to 2 inches
Moderately fractured	8 inches to 2 feet	**Thin**	2 inches to 1 foot
Slightly fractured	2 to 6.5 feet	**Medium**	1 to 3 feet
Very slightly fractured	> 6.5 feet	**Thick**	3 to 10 feet
		Massive	> 10 feet

ROCK QUALITY DESIGNATION (RQD) [1]	
Description	**RQD Value (%)**
Very Poor	0 - 25
Poor	25 – 50
Fair	50 – 75
Good	75 – 90
Excellent	90 - 100

1. The combined length of all sound and intact core segments equal to or greater than 4 inches in length, expressed as a percentage of the total core run length.

L-Pile Parameters

Table 1. L-Pile Design Parameters

Stratum[1,2]	GeoModel Layer	L-Pile Soil Model	γ' (pcf)	s_u (psf)	ε_{50}	q_u (psi)	E_m (psi)	RQD (%)	k_{rm}
Existing Fill/Native Fine-Grained Soils	2,3	Stiff Clay w/o Free Water	120	750	0.01	--	--	--	--
Native Fine-Grained Soils	3	Stiff Clay w/o Free Water	125/67.6[3]	1,500	0.007	--	--	--	--
Weathered Bedrock	4	Weak Rock (Reese)	140	--	--	100	1,000	50	0.0005
Unweathered Bedrock	5	Weak Rock (Reese)	140	--	--	200	2,000	50	0.0005

1. See Subsurface Profile in **Figures** for more details on Stratigraphy.
2. Lateral resistance should be ignored within 2.5 feet of the proposed ground surface.
3. Assume the groundwater table is located 2 feet above the top of rock for preliminary analyses.

Definition of Terms:

γ': Effective unit weight
s_u: Undrained shear strength
ε_{50}: Strain at 50% of unconfined compressive strength
q_u: Uniaxial compressive strength
E_m: initial modulus of rock mass
RQD: rock quality designation
k_{rm}: strain factor

Concrete Core Test Report

Report Number: N1255189.0001
Service Date: 07/08/25
Report Date: 08/13/25
Task: Foundation Concrete Cores and Testing->Compressive Strength Testing

Terracon
611 Lunken Park Dr
Cincinnati, OH 45226-1813
513-321-5816

Client

Village of Indian Hill
Attn: Dina Minneci
6525 Drake Rd
Cincinnati, OH 45243-2705

Project

Preliminary Water Tower Site Evaluation
5355 Miami Road
Cincinnati, OH 45243

Project Number: N1255189

Material Information

Specified Strength: Unknown

Specified Length:
Mix ID:
Nominal Maximum Size Aggregate:

Sample Information

Placement Date: Unknown
Date Tested: 07/14/25 **Time:** 1600
Sampled By: Nicolas Jamison
Drill Directions: Other - See Comments
Date Core Obtained: 07/08/25 **Time:** 0000
Date Ends Trimmed: 07/09/25 **Time:** 1200
Moisture Conditioning History: According to ASTM C-42

Laboratory Test Data

Core ID	Location	Cored Length (in)	Trim Length (in)	Capped Length (in)	Avg. Dia. (in)	Area (sq in)	Length / Diam. Ratio	Max Load (lbs)	Corr. Factor	Comp. Strength (psi)	Fracture Type	Density (pcf)
1	See attached	8.16	6.57	6.51	3.73	10.93	1.75	93738	0.980	8410	3	149.0
2	See attached	9.24	7.12	6.95	3.72	10.87	1.87	77259	1.000	7110	3	149.5
3	See attached	9.43	7.26	7.09	3.71	10.81	1.91	87315	1.000	8080	3	150.2

Comments:

Services: Secure cores from insitu concrete and test cores for compressive strength in accordance with ASTM C42.

Reported To:
Contractor: Terracon
Report Distribution:
(1) Village of Indian Hill, Cindy Klopfenstein, PE CFM

Test Methods: ASTM C42

GENERAL OBSERVATION REPORT

Report Number: N1221307.0001
Service Date: 06/16/22
Report Date: 06/24/22 Revision 1 - Add core results

611 Lunken Park Dr
Cincinnati, OH 45226-1813
513-321-5816

Client

Pittsburgh Tank & Tower
Attn: Jerry Smith
PO Box 517
Henderson, KY 42419-0025

Project

Pittsburgh Tank and Tower Group - Concrete Cores
5355 Miami Rd
Cincinnati, OH

Project Number: N1221307

GENERAL INFORMATION

Jerry Smith with Pittsburgh Tank and Tower requested Terracon to be on site for GPR core locations, concrete core extraction, core hole patching, and carbonation testing at the above referenced project. The results of observation(s) for today were reported to Jerry Smith.

SUMMARY OF SERVICES

Three cores were extracted from within the water tower. Cores were labeled as follows: C-1 (Column), P-1 (Pilaster), and W-1 (Wall). Prior to coring GPR was used to locate reinforcing steel. Cores were taken back to the Terracon laboratory for carbonation and compressive strength testing. Core holes were patched using fast set concrete.

SUMMARY OF TESTING

Carbonation testing results can be seen on the attached photo log. Carbonation testing showed minimal carbonation. Compressive strength results are shown on the following page of this report.

Services:

Terracon Rep.: Peter Lytle
Reported To: Jerry Smith
Contractor: N/A
Report Distribution:
(1) Pittsburgh Tank & Tower, Jerry Smith

Reviewed By:

Peter Lytle
Staff Engineer

Concrete Core Test Report

Report Number: N1221307.0002
Service Date: 06/16/22
Report Date: 06/24/22
Task:

611 Lunken Park Dr
Cincinnati, OH 45226-1813
513-321-5816

Client

Pittsburgh Tank & Tower
Attn: Jerry Smith
PO Box 517
Henderson, KY 42419-0025

Project

Pittsburgh Tank and Tower Group - Concrete Cores
5355 Miami Rd
Cincinnati, OH

Project Number: N1221307

Material Information

Specified Strength:

Specified Length:
Mix ID: Not Provided
Nominal Maximum Size Aggregate:

Sample Information

Placement Date: Unknown
Date Tested: 06/16/22 **Time:** 0000
Sampled By: Peter Lytle
Drill Directions: Horizontal
Date Core Obtained: 06/16/22 **Time:** 0000
Date Ends Trimmed: 06/16/22 **Time:** 0000
Moisture Conditioning History: According to ASTM C-42

Laboratory Test Data

Core ID	Location	Cored Length (in)	Trim Length (in)	Capped Length (in)	Diam. (in)	Area (sq in)	Length / Diam. Ratio	Max Load (lbs)	Corr. Factor	Comp. Strength (psi)	Fracture Type	Density (pcf)	Tested By
1	C-1 (Column)	8.25	7.46	7.72	3.74	10.99	2.06	81610	1.000	7430	3	148.6	BJM
2	P-1 (Pilaster)	8.25	7.46	7.63	3.74	10.99	2.04	66420	1.000	6050	3	148.6	BJM
3	W-1 (Wall)	12.38	7.44	7.68	3.74	10.99	2.05	73550	1.000	6690	3	147.9	BJM

Comments:

Services:
Terracon Rep.: William Meiser
Reported To:
Contractor:
Report Distribution:
(1) Pittsburgh Tank & Tower, Jerry Smith

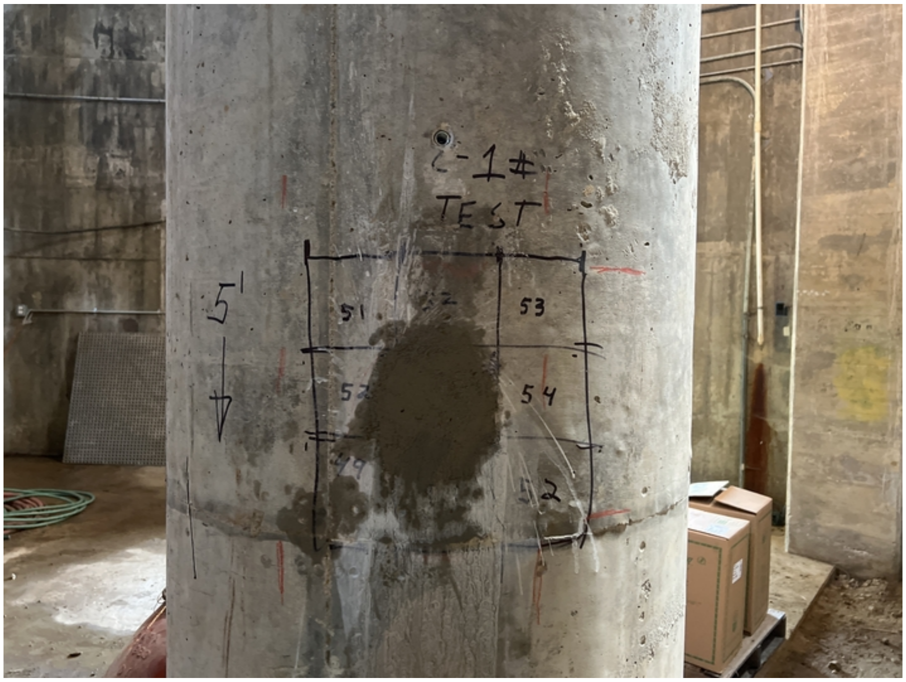
Reviewed By:
Peter Lytle
Staff Engineer

Test Methods:

The tests were performed in general accordance with applicable ASTM, AASHTO, or DOT test methods. This report is exclusively for the use of the client indicated above and shall not be reproduced except in full without the written consent of our company. Test results transmitted herein are only applicable to the actual samples tested at the location(s) referenced and are not necessarily indicative of the properties of other apparently similar or identical materials.

CR0004, 11-16-12, Rev.5

611 Lunken Park Dr
Cincinnati, OH 45226-1813
513-321-5816



(P2) C-1 Patch

(P3) P-1 Patch

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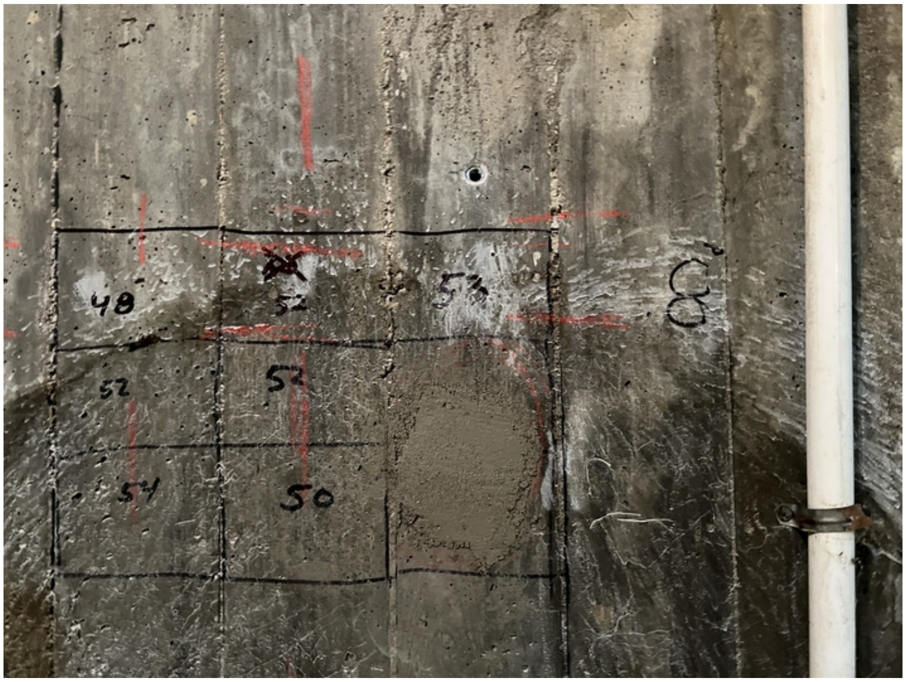

(P4) W-1 Patch

(P5) W-1 patch from outside

Terracon
611 Lunken Park Dr
Cincinnati, OH 45226-1813
513-321-5816

(P6) C-1 Carbonation

(P7) P-1 Carbonation

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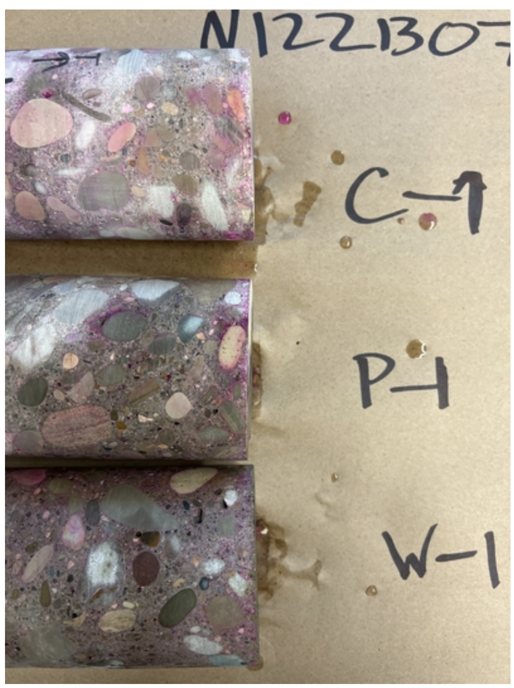
(P8) W-1 carbonation



(P10) Core edges interior side

611 Lunken Park Dr
Cincinnati, OH 45226-1813
513-321-5816

(P11) Core edges exterior side

RECORD OF BORING NO. 1

Client: Indian Hill Water Works
Project: Water Storage Reservoir
Project No.: 021-85-212

Elev. (ft.)	Depth (ft.)	Description	Sample No.	Sample Depth (ft.)	Sample Type	Blows per 6"	Recovery (in.)
882.3	0.0						
		Topsoil	1	0.0-1.5	SS	1-2-3	18
881.6	0.7						
		Light brown lean clay, moist - medium stiff					
879.8	2.5						
		Mottled light brown and brown lean clay with	2	2.5-4.0	SS	2-5-5	18
		fine sand moist - stiff to very stiff	3	5.0-6.5	SS	8-12-16	18
875.3	7.0						
		Light brown weathered shale with limestone layers moist	4	7.5-9.0	SS	12-17-22	18
872.3	10.0						
		Brown and gray weathered shale with limestone layers	5	10.0-11.5	SS	9-19-28	18
867.3	15.0						
		Gray shale and thinly interbedded fossiliferous limestone; (62% limestone)	6	15.0-15.5	SS	45/6"	6
864.1	18.2						
		Gray fossiliferous limestone;	R1	15.5-25.5	NXM		120
		1" shale layer @ 21' 2" shale layer @ 22.2' 3" shale layer @ 24' (85% limestone)					
856.8	25.5						
		Boring terminated at 25.5'					

Dates Drilled: 1-2-86

Driller: F. Moore

Water Depth: Initial: None

______ **Days after Completion:** ______

______ **Days after Completion:** ______

Drilling Method: 3.25" I.D. H.S.A.

SOIL & MATERIAL ENGINEERS, INC.
Cincinnati, Ohio

RECORD OF BORING NO. 2

Client: Indian Hill Water Works
Project: Water Storage Reservoir
Project No.: 021-85-212

Page 1 **of** 1

Elev. (ft.)	Depth (ft.)	Description	Sample No.	Sample Depth (ft.)	Sample Type	Blows per 6"	Recovery (in.)
885.2	0.0	Topsoil	1	0.0-1.5	SS	1-1-3	18
884.3	0.9	Light brown lean clay with fine sand very moist - soft					
882.7	2.5	Mottled brown and light brown lean clay with fine sand and concretions, moist - stiff	2	2.5-4.0	SS	3-5-7	18
880.2	5.0	Brown lean clay with fine sand and concretions and rock fragments, moist - very stiff	3	5.0-6.5	SS	5-7-12	18
877.7	7.5	Brown and gray weathered shale with thin limestone layers, moist	4	7.5-9.0	SS	10-17-21	18
875.2	10.0	Gray shale with thin limestone layers, moist	5	10.0-10.5	SS	75/6"	6
874.7	10.5	Refusal at 10.5 ft.					

Dates Drilled: 1-6-86

Driller: F. Moore

Water Depth: Initial: None

U.C. **Days after Completion:** None

Days after Completion:

Drilling Method: 3.25" I.D. H.S.A.

SOIL & MATERIAL ENGINEERS, INC.
Cincinnati, Ohio

RECORD OF BORING NO. 3

Client: Indian Hill Water Works
Project: Water Storage Reservoir
Project No.: 021-85-212

Elev. (ft.)	Depth (ft.)	Description	Sample No.	Sample Depth (ft.)	Sample Type	Blows per 6"	Recovery (in.)
881.7	0.0	Topsoil	1	0.0-1.5	SS	1-2-3	18
881.2	0.5	Light brown lean clay, very moist - medium stiff					
879.2	2.5	Light brown lean clay with fine sand and concretions, moist - stiff	2	2.5-4.0	SS	4-4-6	18
			3	5.0-6.5	SS	7-7-10	18
			4	7.5-9.0	SS	6-10-13	18
871.7	10.0	Brown weathered shale with thin limestone layers, moist	5	10.0-11.5	SS	21-37/6"	12
868.7	13.0	Gray shale with thin limestone layers, moist	6	15.-15.2	SS	60/2"	2
866.5	15.2	Refusal at 15.2'					

Dates Drilled: 1-6-86

Driller: F. Moore

Water Depth: Initial: None

U.C. **Days after Completion:** None

Days after Completion:

Drilling Method: 3.25" I.D. H.S.A.

SOIL & MATERIAL ENGINEERS, INC.
Cincinnati, Ohio

RECORD OF BORING NO. 4

Client: Indian Hill Water Works

Project: Water Storage Reservoir

Project No.: 021-85-212

Elev. (ft.)	Depth (ft.)	Description	Sample No.	Sample Depth (ft.)	Sample Type	Blows per 6"	Recovery (in.)
884.1	0.0	Topsoil	1	0.0-1.5	SS	2-3-5	18
883.1	1.0	Brown lean clay, moist - medium stiff					
881.6	2.5	Mottled light brown and gray lean clay with fine sand moist - stiff	2	2.5-4.0	SS	4-5-7	18
879.1	5.0	Brown lean clay with fine sand and concretions moist - very stiff	3	5.0-6.5	SS	5-8-12	18
			4	7.5-9.0	SS	16-19-25	18
874.1	10.0	Brown weathered shale with thin limestone layers, moist	5	10.0-11.5	SS	25/3"	3
870.1	14.0	Gray shale with thin limestone layers, moist	6	15.0-16.5	SS	50/6"	2
868.6	15.5	Refusal at 15.5 ft.					

Dates Drilled: 12-31-85

Driller: F. Moore

Water Depth: Initial: None

U.C. **Days after Completion:** None

Days after Completion:

Drilling Method: 3.25" I.D. H.S.A.

SOIL & MATERIAL ENGINEERS, INC.
Cincinnati, Ohio

RECORD OF BORING NO. 5

Client: Indian Hill Water Works
Project: Water Storage Reservoir
Project No.: 021-85-212

Page 1 **of** 1

Elev. (ft.)	Depth (ft.)	Description	Sample No.	Sample Depth (ft.)	Sample Type	Blows per 6"	Recovery (in.)
880.1	0.0	Topsoil	1	0.0-1.5	SS	1-1-1	18
879.4	0.7	Light brown lean clay, very moist - soft					
877.6	2.5	Brown lean clay with fine sand, moist - medium stiff	2	2.5-4.0	SS	3-4-5	18
875.1	5.0	Brown and gray lean clay with fine sand and concretions, moist - stiff	3	5.0-6.5	SS	5-7-9	18
			4	7.5-9.0	SS	6-10-14	18
870.1	10.0	Brown and gray weathered shale with thin limestone layers, moist	5	10.0-11.5	SS	17-33/6"	12
867.1	13.0	Gray shale with thin limestone layers, moist	6	15.0-15.5	SS	80/6"	6
864.6	15.5	Refusal at 15.5 ft.					

Dates Drilled: 1-6-86

Driller: F. Moore

Water Depth: Initial: None

U.C. **Days after Completion:** None

Days after Completion:

Drilling Method: 3.25" I.D. H.S.A.

SOIL & MATERIAL ENGINEERS, INC.
Cincinnati, Ohio

RECORD OF BORING NO. 6

Client: Indian Hill Water Works

Project: Water Storage Reservoir

Project No.: 021-85-212

Page 1 **of** 1

Elev. (ft.)	Depth (ft.)	Description	Sample No.	Sample Depth (ft.)	Sample Type	Blows per 6"	Recovery (in.)
883.0	0.0						
		Topsoil	1	0.0-1.5	SS	1-2-3	18
882.1	0.9						
		Brown lean clay with sand, very moist - medium stiff (Fill)					
880.5	2.5						
		Brown lean clay with fine sand and concretions, moist - stiff to very stiff	2	2.5-4.0	SS	3-5-7	18
			3	5.0-6.5	SS	10-11-13	18
875.5	7.5						
		Brown and gray weathered shale with thin limestone layers, moist	4	7.5-9.0	SS	12-17-22	18
			5	10.0-11.5	SS	50/2"	2
870.0	13.0						
		Gray shale and thinly interbedded fossiliferous limestone (45% limestone)					
865.0	18.0						
		Gray fossiliferous limestone; 5" shale layer @ 20', (78% limestone)	R1	15.0-25.0	NXM		120
858.0	25.0						
		Boring terminated @ 25.0'					

Dates Drilled: 1-3-86

Driller: F. Moore

Water Depth: Initial: None

_______ **Days after Completion:** _______

_______ **Days after Completion:** _______

Drilling Method: 3.25" I.D. H.S.A.

SOIL & MATERIAL ENGINEERS, INC.
Cincinnati, Ohio

RECORD OF BORING NO. 7

Client: Indian Hill Water Works

Project: Water Storage Reservoir

Project No.: 021-85-212

Elev. (ft.)	Depth (ft.)	Description	Sample No.	Sample Depth (ft.)	Sample Type	Blows per 6"	Recovery (in.)
881.0	0.0						
		Topsoil	1	0.0-1.5	SS	1-3-3	18
880.2	0.8						
		Brown lean clay, very moist - medium stiff					
878.5	2.5						
		Light brown lean clay with fine sand and concretions, moist - stiff	2	2.5-4.0	SS	3-4-5	18
			3	5.0-6.5	SS	8-12-15	18
873.5	7.5						
		Brown weathered shale with limestone layers, moist	4	7.5-9.0	SS	17-24-35	18
			5	10.0-11.5	SS	55/6"	6
869.5	11.5						
		Gray shale and thinly interbedded fossiliferous limestone (65% limestone)	R1	15.0-25.0	NXM		108
856.0	25.0						
		Boring terminated at 25.0'					

Dates Drilled: 1-3-86

Driller: F. Moore

Water Depth: Initial: None

_______ Days after Completion: _______

_______ Days after Completion: _______

Drilling Method: 3.25" I.D. H.S.A.

SOIL & MATERIAL ENGINEERS, INC.
Cincinnati, Ohio

RECORD OF BORING NO. 8

Client: Indian Hill Water Works

Project: Water Storage Reservoir

Project No.: 021-85-212

Page 1 of 1

Elev. (ft.)	Depth (ft.)	Description	Sample No.	Sample Depth (ft.)	Sample Type	Blows per 6"	Recovery (in.)
883.5	0.0	Topsoil	1	0.0-1.5	SS	1-3-3	18
882.8	0.7	Dark brown lean clay, very moist - medium stiff					
881.0	2.5	Brown lean clay with fine sand and concretions, moist - stiff	2	2.5-4.0	SS	4-5-6	18
			3	5.0-6.5	SS	6-6-7	18
			4	7.5-9.0	SS	9-10-10	18
873.5	10.0	Brown weathered shale with thin limestone layers, moist	5	10.0-11.5	SS	40/6"	6
868.5	15.0	Gray weathered shale with thin limestone layers, moist	6	15.0-15.5	SS	60/6"	6
868.0	15.5	Refusal at 15.5 ft.					

Dates Drilled: 1-6-86

Driller: F. Moore

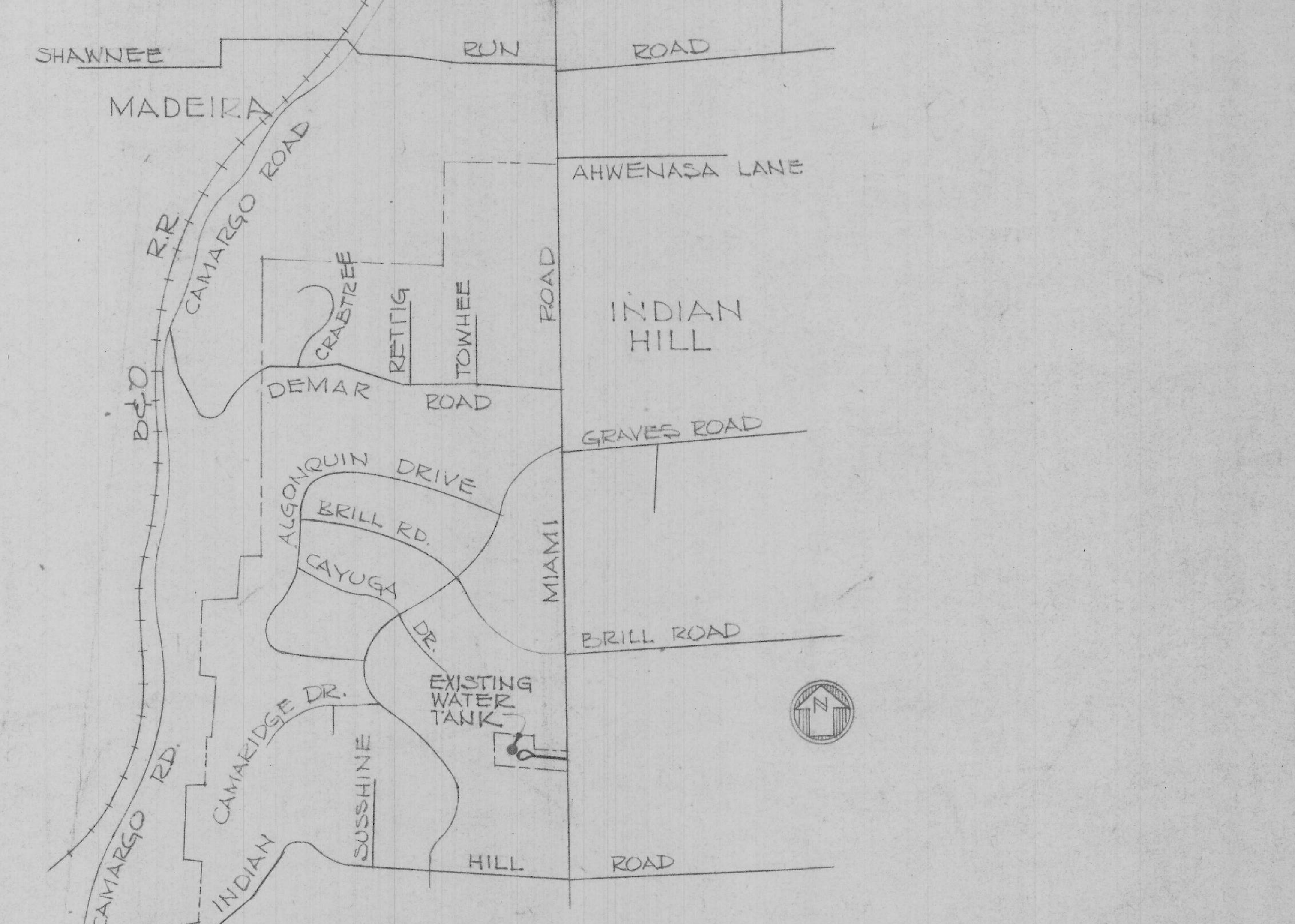
Water Depth: Initial: None

U.C. **Days after Completion:** None

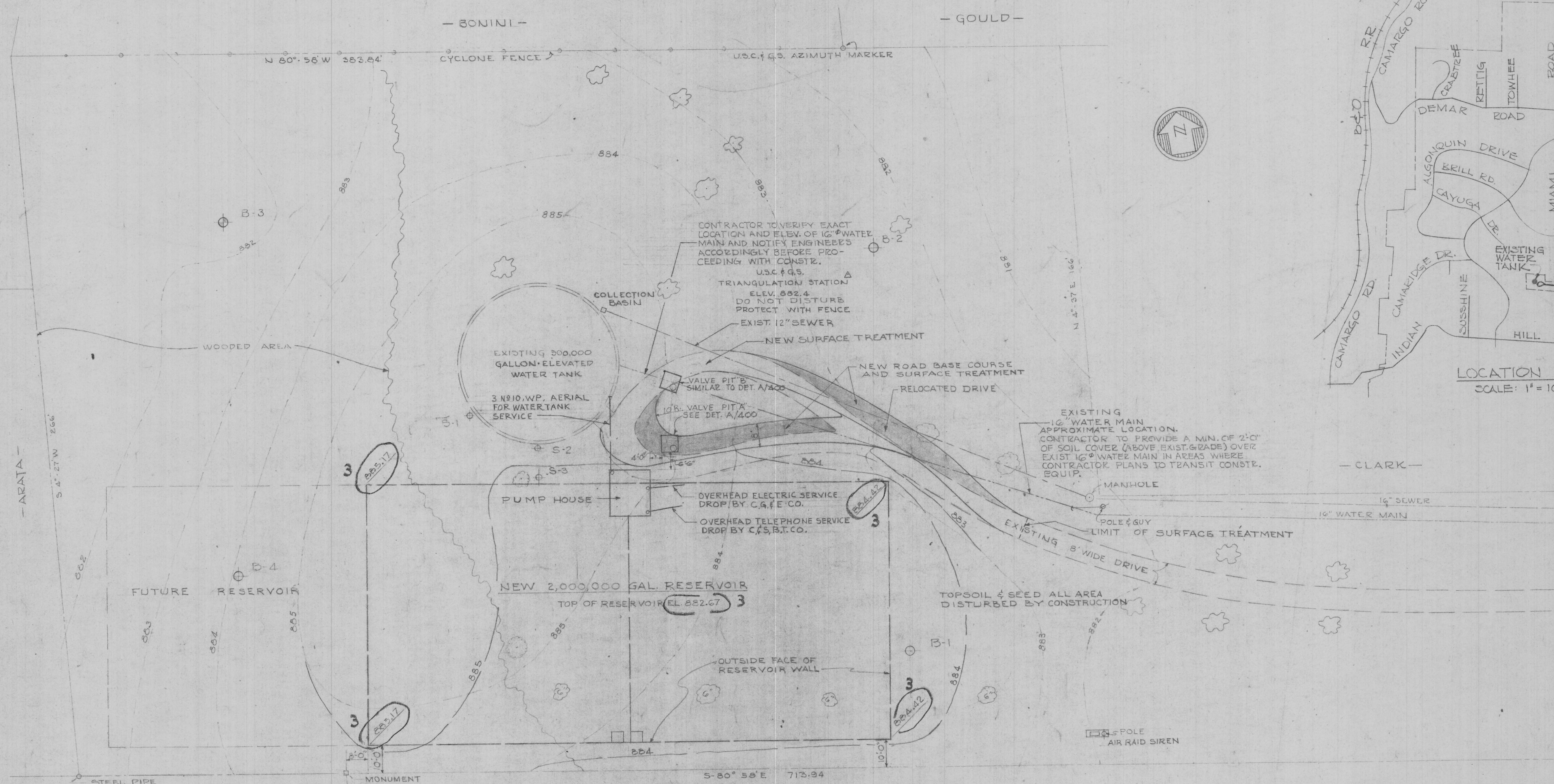
Days after Completion:

Drilling Method: 3.25" I.D. H.S.A.

SOIL & MATERIAL ENGINEERS, INC.
Cincinnati, Ohio



LOCATION MAP
SCALE: 1" = 1000'



SITE PLAN
SCALE 1" = 20'

NOTE: AREA OVER RESERVOIR TO BE GRADED IN SUCH A WAY AS TO PROVIDE A SMOOTH TRANSITION FROM PRESENT TO PROPOSED GRADES.

LEGEND

EXPLORATORY SOIL BORINGS
ROD SOUNDINGS "A" ROD

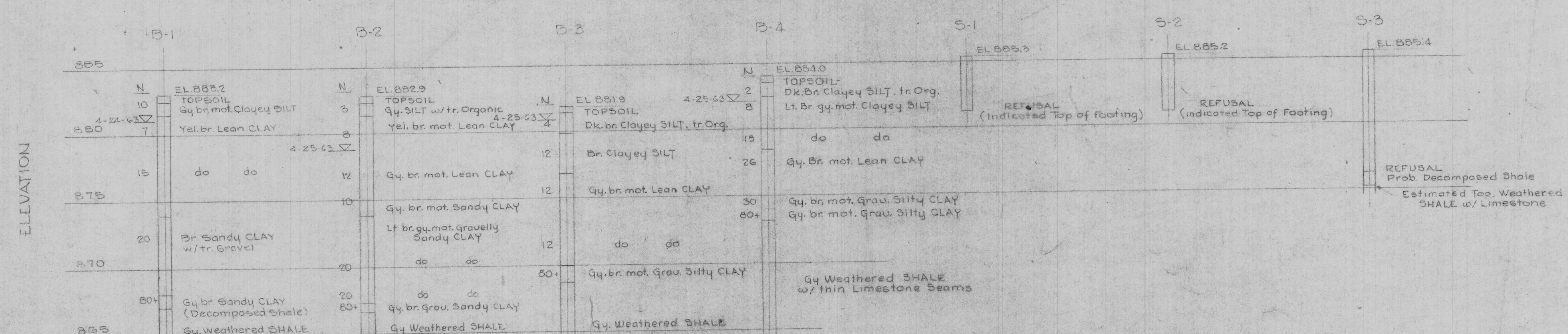
N INDICATES THE NUMBER OF BLOWS ON A 2-INCH O.D. SAMPLER NECESSARY TO ADVANCE THE SAMPLER 1 FOOT, USING A 140 LB. HAMMER FALLING 30 INCHES.

GROUND WATER TABLE AS OBSERVED DURING DRILLING, AND DATE OF OBSERVATION.

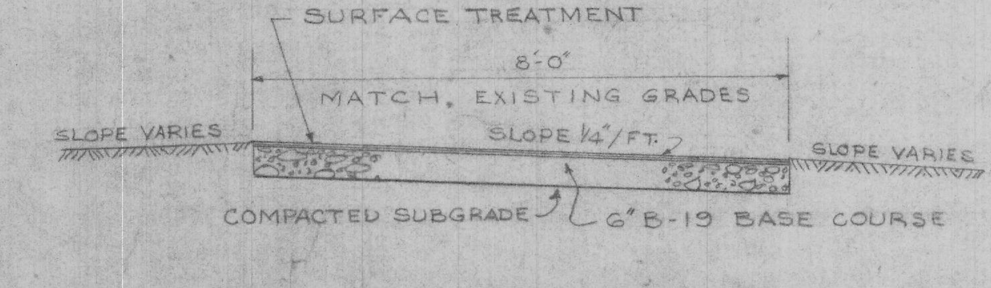
EXIST. CONTOUR
PROPOSED CONTOUR

NOTES

THE SUBSURFACE INFORMATION AND DATA FURNISHED HEREIN ARE NOT INTENDED AS REPRESENTATIONS OR WARRANTIES BUT ARE FURNISHED FOR INFORMATION ONLY. IT IS EXPRESSLY UNDERSTOOD THAT THE ENGINEERS WILL NOT BE RESPONSIBLE FOR THE ACCURACY THEREOF OR FOR ANY DEDUCTION, INTERPRETATION OR CONCLUSION DRAWN THEREFROM BY THE CONTRACTOR. THE INFORMATION IS MADE AVAILABLE IN ORDER THAT THE CONTRACTOR MAY HAVE READY ACCESS TO THE SAME INFORMATION AVAILABLE TO THE ENGINEERS AND IS NOT A PART OF THIS CONTRACT.



BORING LOGS



ROADWAY TYPICAL SECTION
SCALE 1/2" = 1'-0"

NO.	DATE	REVISIONS	BY
1	7-9-63	REVISED GRADING & ELEVATIONS	J.P.H.
2		GEN. REV.	J.P.H.
3	9-27-63	RESERVOIR RAISED 5"	WBS

APPROVED

STATE OF OHIO PROFESSIONAL ENGINEER LICENSE NO. 12609
JOHN W. WISEMAN

DRAWN G.C.W.
TRACED H.W.C.
CHECKED D.B. W.S.S.

SITE PLAN

VILLAGE OF INDIAN HILL, OHIO
WATER SUPPLY IMPROVEMENTS

A. M. KINNEY, INC.
CONSULTING ENGINEERS
CINCINNATI, OHIO NEW YORK, N.Y.

SCALE AS NOTED
DATE JUNE 18 1963
JOB NO. 1477
SHEET NO. 01
REVISION 3