EXHIBIT O – GEOTECHNICAL EVALUATION

Kumar & Associates, Inc.®
Geotechnical and Materials Engineers
and Environmental Scientists

An Employee Owned Company

6735 Kumar Heights
Colorado Springs, CO 80918
phone: (719) 632-7009
fax: (719) 632-1049
email: kacolospgs@kumarusa.com
www.kumarusa.com

Office Locations: Denver (HQ), Parker, Colorado Springs, Fort Collins, Glenwood Springs, and Summit County, Colorado

GEOTECHNICAL ENGINEERING STUDY
RAMAH WASTEWATER TREATMENT PLANT
IMPROVEMENT PROJECT
RAMAH, COLORADO

Prepared By:
Arben F. Kalaveshi, P.E.

COLORADO REGISTERED PROFESSIONAL ENGINEER
ARBEN F. KALAVESHI
45969
04-22-2022

Reviewed By:

Duane P. Craft, P.E.

Prepared for:

Nicholaus Marcotte, P.E.
Element Engineering, LLC
12687 West Cedar Drive, Suite 300
Lakewood, Colorado 80228

Project No. 22-2-102

April 22, 2022

TABLE OF CONTENTS

SUMMARY

1. This study was conducted in two areas. Borings 1 and 2, drilled southeast of The Town of Ramah encountered a layer of topsoil overlying sands and clays extending to the maximum drilled depth of 20 feet. Boring 3, drilled on the northwest end of town encountered topsoil overlying man placed fill extending to a depth of about 6 feet. Sands were found below the fill, and extended to the maximum drilled depth of 30 feet, but included a layer of sandy lean clay from about 27 to 29 feet.

2. Groundwater was encountered in Boring 3 both during drilling and when measured again six days later. The water depth at the time of our final reading was 15.3 feet below the ground surface. Although no groundwater was measured in the other two borings, perched surface water may occur within the sands above less permeable clays, particularly after precipitation events.

3. Borings 1 and 2 were drilled for new evaporative ponds to replace the existing system. The subsurface soil profile at this location included a clay zone from about 2½ feet to 9½ feet below the existing ground surface. The clays tested had a moderate to high swell potential upon wetting and are anticipated to have a relatively low permeability. While these soils will probably work well for use as evaporative ponds, the construction of shallow foundations here will be difficult due to the swell potential. If movement sensitive structures are constructed in this area, we recommend that they be constructed on deep foundations such as helical piers that extend to the underlying granular soils found below the clays.

4. Boring 3 was drilled for the construction of a new lift station. Because undocumented fill was encountered in this area, we recommend that it be removed and replaced with suitable materials where it is present below proposed shallow foundations. Alternatively, deep foundations may be considered. Based on the subsurface profile encountered at this location, foundations will need to extend to a depth of about 6 feet or greater to bear on native soils, but the depth and lateral extent of the existing fill was not determined beyond the boring location. Fill may extend to greater depths in the area of the proposed foundations.

5. We anticipate that gravel access drives may be constructed at each of the proposed sites. Based on their intended use, we have assumed an EDLA of 10 for these areas. Based on the subsurface conditions encountered and the relatively light estimated traffic volumes, we recommend the pavement section alternatives presented in the following table.

Pavement Section Thickness (in.)	
Area	Aggregate Base Course
Access Drives	8

PURPOSE AND SCOPE OF STUDY

This report presents the results of a geotechnical engineering study for the construction of various improvements to the existing wastewater treatment system in Ramah, Colorado. The project site is shown on Fig. 1. The study was conducted in accordance with our Proposal No. C22-104, dated January 10, 2022, to provide recommendations for foundations and gravel pavement section thickness.

This report has been prepared to summarize the data obtained during this study, and to present our conclusions and recommendations based on the proposed construction and the subsurface conditions encountered. Design parameters and a discussion of geotechnical engineering considerations related to the proposed construction are included in the report.

PROPOSED CONSTRUCTION

We understand the project will include a new lift station for a force main that will connect to a new evaporative pond system located about half a mile away. The new ponds will include three adjacent cells, and will replace the existing one located on the north end of town, which will be decommissioned. The lift station is anticipated to have a depth of about 12 feet, and the ponds will have sloped basins ranging in depth from about 2 to 8 feet. The approximate pond site layout is shown on the attached Fig. 1A.

We anticipate that bearing loads will be light for the proposed structures. Permanent grading will mostly consist of cuts, with depths up to about 8 feet for the proposed ponds. If the proposed construction is significantly different from that described above or depicted in this report, we should be notified to reevaluate the recommendations contained in this report.

SITE CONDITIONS

The proposed evaporation pond site is located southeast of the Town of Ramah Cemetery, and is bordered by Ramah Road on the south and west sides, and open fields on the north and east. The area is relatively flat, with some small hills and draws, with a gentle slope down to the northeast. An ephemeral tributary to Big Sandy Creek is located about 1,000 feet east of the site, and was dry at the time of our study. This area appeared to be actively used as an agricultural field, and the vegetation had been tilled.

The site of the proposed lift station is located on the northwest part of town, just west of the intersection of Rock Island Avenue and Pikes Peak Avenue. The site is bordered to the north and east by Pikes Peak avenue, and to the south and west by private property. Houses and other small structures are located near the site. This area is relatively flat with a light downward slope to the north. Big Sandy Creek is located about 500 feet to the northwest, and the existing evaporation pond (lagoon) is located about 1,000 feet to the northeast. Vegetation in this area consisted of a grass lawn and several trees.

SUBSURFACE CONDITIONS

Information on subsurface conditions was obtained by conducting a site reconnaissance and drilling three exploratory borings at the approximate locations shown on Fig. 1. The boring logs and corresponding legend and notes are shown on Figs. 2 and 3. The results of swell-consolidation tests and gradation tests conducted on selected soils are presented on Figs. 4 through 6, and Figs. 7 and 8, respectively. A summary of the test results is presented on Table I. The laboratory testing was conducted in general accordance with applicable ASTM standards.

Borings 1 and 2 were drilled at the site of the proposed evaporation ponds. Below a layer of topsoil the subsurface soil profile at this location consisted of clayey sands extending to a depth of about 2½ feet underlain by a layer of lean clay with varied amounts of sand, followed by discontinuous layers of clayey sand and well graded sand with silt extending to the maximum depth explored of 20 feet. Based on vertical expansion ranging from about 3.4 to 5.7 percent upon wetting under a surcharge pressure of 1,000 psf, the clays in this area possess a moderate to high swell potential.

Boring 3 was drilled at the site of the proposed lift station. Below a layer of vegetated topsoil, the subsurface soil profile at this location consisted of man placed fill extending to a depth of about 6 feet, and underlain by clayey sand extending to a depth of 9½ feet. Well graded sand was found below the clayey sand, and extended to a depth of about 27 feet, where it was underlain by a layer of sandy lean clay. The clay layer was relatively thin, and was underlain by clayey sand from 29 feet to the maximum explored depth of 30 feet. The fill tested did not appear to possess a significant swell potential based on a vertical expansion of 0.2 percent upon wetting under a surcharge pressure of 1,000 psf.

Detailed descriptions of the soils and the depths at which they were encountered can be found on Figs. 2 and 3.

ENGINEERING CONSIDERATIONS

Existing fill was encountered to a depth of about 6 feet at the location of the proposed lift station. The lateral or vertical extents of the fill were not determined in the scope of this study, but we understand that the base of this structure will be about 12 feet below the ground surface, and if this is the case for all foundations, the existing fill is not likely to be a factor for the design of shallow foundations. If portions of the structure or ancillary structures will be constructed at shallower depths, fill may be present below the base of shallow foundations. In all cases, fill should be removed and replaced with suitable material where it is present below foundations. Alternatively, foundations extending to native soils or deep foundations may be considered. Recommendations for both footing/pad foundations and deep helical foundations have been presented in this report.

Groundwater was measured at a depth of about 15.3 feet in the boring drilled for the proposed lift station measured six days after drilling. This depth is near the elevation of the base of the lift station, and groundwater may be a construction consideration at this site. A detailed discussion is presented in the "Site Grading and Earthwork" Section.

The subsurface soil profile at the location of the proposed evaporation ponds included a clay zone from about 2½ feet to 9½ feet below the existing ground surface. The clays tested had a moderate to high swell potential upon wetting and are anticipated to have a relatively low permeability. While these soils will probably work well for use as evaporative ponds, the construction of shallow foundations here will be difficult due to the swell potential. If structures that are sensitive to heave related movement are constructed in this area, we recommend that they be constructed on deep foundations such as helical piers that extend to the underlying granular soils found below the clays.

The clay soils encountered in our study will have relatively low permeability, but are natural materials and will vary throughout the site area. An engineered liner system should be implemented at the basin of each pond if specific permeability limits are required for this project.

FOUNDATIONS

Shallow Foundations: The design and construction criteria presented below should be observed for a shallow footing system. The construction details should be considered when preparing project documents.

1. The maximum net allowable bearing pressure for footings placed on native granular soils or suitable fill will be a function of the embedment depth of the foundation considered. Allowable pressures for the anticipated foundation depths have been presented in the following table. These values may be increased by a factor of 1/3 for transient loading.

Foundation Bury Depth (feet)	Allowable Bearing Pressure (psf)
3	2,500
12	4,500

   Mat foundations that are not considered rigid may use a design modulus of vertical subgrade reaction of 150 pci. This value is for a 1 ft. x 1 ft. square plate and should be corrected for the shape and size of the actual mat.

2. We estimate total settlement for shallow foundations designed and constructed as discussed in this section will not exceed approximately 1 inch.

3. Continuous footings should have a minimum width of 16 inches, and isolated pads should have a minimum width of 24 inches.

4. Exterior footings and footings beneath unheated areas should be provided with adequate soil cover above their bearing elevation for frost protection. Based on our experience with similar projects, we recommend the foundations be placed at least 36 inches below the existing grade.

5. The lateral resistance of a foundation placed on properly compacted fill material or bedrock will be a combination of the sliding resistance of the footing on the foundation materials and passive earth pressure against the side of the footing. Resistance to sliding at the bottoms of the footings may be calculated based on an allowable coefficient of friction of 0.35. Passive pressure against the sides of the footings may be calculated

using an allowable equivalent fluid unit weight of 190 pcf. These values are working values. The specifications for compaction of fill against the sides of foundations to resist lateral loads are presented under the "Site Grading and Earthwork" section of this report.

6. Earthwork recommendations for shallow foundations are presented in the "Site Grading and Earthwork" section of this report.

7. Existing fill, or areas of loose material encountered within the foundation excavation should be removed and the footings extended to adequate natural bearing material.

8. A representative of the geotechnical engineer should observe all footing excavations prior to fill and concrete placement.

Helical Pier Foundations: The axial design load of helical piers should be determined in general accordance with the current International Building Code (IBC), which states the allowable axial design load, P_a, should be determined as follows:

$P_a = 0.5\ P_u$, where P_u (the ultimate load) is the least value of:

1. Sum of the areas of the helical bearing plates times the ultimate bearing capacity of the soil or rock comprising the bearing stratum.

2. Ultimate capacity determined from well-documented correlations with installation torque.

3. Ultimate capacity determined from load tests.

4. Ultimate capacity of pile shaft.

5. Ultimate capacity of pile couplings.

6. Sum of the Ultimate axial capacity of helical bearing plates affixed to pile.

Items 1 through 3 are related to the geotechnical capacity of the piers; Items 4 through 6 are related to the structural capacity and should be evaluated by the structural engineer. The owner and structural designer should be aware that certain proprietary helical pier systems have been subjected to acceptance testing administered by the International Code Council (ICC), while other systems provided by specialty contractors may be fabricated according to designs by registered professional engineers. The certified systems have documentation that addresses many of the structural capacity issues, while the non-certified systems require structural design by an engineer. Many of the lighter-duty helical pile systems available, with working capacities on the order of 50 kips or less, are certified, which can simplify the design and submittal process. However, higher capacity systems, where single piers may have working capacities of 200 kips or more, sometimes referred to as screw piles, are often designed and fabricated and are not certified, manufactured systems.

Based on consideration of bearing capacity theory and published correlations of boring penetration resistance values with ultimate bearing capacity, we recommend an ultimate bearing capacity of 10 ksf for a helical pile embedded in the native sands. We anticipate it will be possible to achieve adequate capacities at nominal depths of about 15 feet by using the appropriate size and number of bearing plates. Nominal depths should be measured from the topmost bearing plate. A greater bearing capacity will be achievable if the piers extend to the underlying claystone bedrock.

Helical piers are typically very slender foundation elements with a low capacity for resisting lateral loads. Lateral restraint of a helical pile foundation system is normally provided through the use of passive pressure on pile caps or foundation walls, or through the use of battered piers. It is normally assumed that a battered pile can be designed for the same axial load as a vertical pile, with the lateral restraint being provided by the horizontal component of the battered pile. Helical piers are often assumed to have tension capacities similar to the axial compressive capacity, although that should be evaluated through load testing or otherwise addressed by the specialty contractor’s submittal.

Acceptance of helical pile installation should be based on attaining a specified torque in the recommended bearing stratum determined in accordance with correlations of installation torque to capacity based on calibrated torque measurements and axial load test data. In our opinion, the ultimate bearing capacity recommended above may be exceeded if supported by adequate site-specific load test data. If site-specific load tests are not performed, the specialty helical pile contractor’s submittal should contain torque-to-capacity data for their pile system in similar soil conditions. If that information cannot be provided, site-specific load tests should be performed in accordance with ASTM D 1143.

We recommend that a qualified helical pile specialty contractor be retained to provide the required design submittal and to provide and install the helical piers. The project design should include a performance specification indicating required capacities, structural requirements, and submittal requirements. At a minimum, the submittal should be required to contain information supporting capacity determination, a description of equipment and installation procedures that will ensure penetration to the required depths, and acknowledgement that the helical bearing plates will be installed into the recommended bearing stratum, as well as all necessary information to satisfy the requirements of the project structural designer.

We should be retained to review the contractor's submittal, and to provide installation observation including monitoring depths and general conformance with the plans and specifications. Our observation and testing services will be intended to document that all of the helix bearing plates on the piers are installed into an adequate bearing stratum.

RETAINING STRUCTURES

Structures such as retaining or foundation walls should be designed for the lateral pressure generated by the backfill, which is a function of the degree of rigidity of the retaining structure and the type of backfill material used. Cantilevered retaining structures that can deflect sufficiently to mobilize the active earth pressure condition maybe designed using the active equivalent fluid pressure (EFP) presented in the following table. Retaining structures that are not expected to deflect should be designed using the at-rest EFP presented in the same table.

Condition	Soil Type	Equivalent Fluid Pressure (pcf)	
		Active	At-rest
Unsubmerged	Suitable On-Site Soil	50	70
Unsubmerged	CDOT Class 1 Structure Backfill	40	60
Submerged	Suitable On-Site Soil	88	99
Submerged	CDOT Class 1 Structure Backfill	83	94

All foundation and retaining structures should be designed for appropriate hydrostatic and surcharge pressures such as adjacent footings, traffic, construction materials and equipment. The unsubmerged pressures recommended above assume drained conditions behind the walls and a horizontal backfill surface. The buildup of water behind a wall or an upward sloping backfill surface will increase the lateral pressure imposed on a foundation wall or retaining structure. Retaining structures may be designed using the values presented for unsubmerged soils if adequate drainage is provided to prevent the buildup of hydrostatic pressure. This can be accomplished using an underdrain or weep holes. If such measures are not implemented, the structures should be designed using the submerged values presented.

WATER SOLUBLE SULFATES

The concentrations of water soluble sulfates measured in samples of the native clay and fill obtained from the exploratory borings ranged from 0.01 to 0.06 percent. These concentrations of water soluble sulfates represent a Class 0 severity of exposure to sulfate attack on concrete exposed to these materials. The degree of attack is based on a range of Class 0 to Class 3 severity of exposure as presented in ACI 201. Based on the laboratory data and our experience, special sulfate resistant cement will not be required for concrete exposed to the on-site soils.

SEISMIC DESIGN CRITERIA

Using estimated shear wave velocities for the subgrade materials encountered based on standard penetration testing, calculations indicate a design Site Class D per the International Building Code (IBC). Based on the subsurface profile and the anticipated ground conditions, liquefaction is not a design consideration.

SURFACE DRAINAGE

Providing proper surface drainage, both during construction and after the construction has been completed, is very important for acceptable performance of the development. The following recommendations should be used as guidelines and changes should be made only after consultation with the geotechnical engineer.

1. Excessive wetting or drying of the foundation and structure subgrades should be avoided during construction.

2. Care should be taken when compacting around the foundation walls to avoid damage to the structure.

3. The ground surface surrounding the exterior of the building should be sloped to drain away from the foundation in all directions. We recommend a minimum slope of 6 inches in the first 10 feet in unpaved areas. Site drainage beyond the 10-foot zone should be

designed to promote runoff and reduce water infiltration. A minimum slope of 3 inches in the first 10 feet is recommended in the paved areas. These slopes may be changed as required for handicap access points in accordance with the Americans with Disabilities Act.

4. Ponding of water should not be allowed on backfill material or within 10 feet of the foundation walls, whichever is greater.

5. Roof downspouts and drains should discharge well beyond the limits of all backfill.

SITE GRADING AND EARTHWORK

We recommend the following criteria be used when preparing the site grading plans.

Fill Material Specifications: The following material specifications are presented for fills on the project site.

1. *Fill Beneath and Beside Foundations*: The on-site granular soils with the exception of any deleterious materials and rock larger than 4 inches in diameter will be suitable for reuse as structural fill. Import fill, if required, should consist of a minus 2-inch non-expansive soil having a maximum 35% passing the No. 200 sieve and a maximum plasticity index of 15. New fill should extend down from the edges of the foundations at a minimum 1:1 horizontal to vertical projection.

2. Gravel Pavement Subgrade Areas: Same as #1 above.

3. *Pipe Bedding Material*: Pipe bedding material should be a free draining, coarse-grained sand and/or fine gravel having a maximum size of 1 inch. We do not anticipate that the near surface on-site natural soils will be suitable for bedding due to the presence of larger particles.

4. *Utility Trench Backfill*: Materials excavated from the utility trenches may be used for trench backfill above the pipe zone fill provided they do not contain unsuitable material or particles larger than 4 inches.

5. *Material Suitability*: All fill material should be free of vegetation, brush, sod and other deleterious substances. The geotechnical engineer should evaluate the suitability of all proposed fill materials prior to placement.

Subgrade Preparation: The ground surface shall be stripped of vegetation/organics prior to foundation or fill placement. Loose, unstable or otherwise unsuitable soils shall be removed, where present, in order to provide a stable platform prior to placement of fill. The existing soils should be scarified, moisture conditioned, and recompacted to a depth of 12 inches prior to the placement of newl fill or structures

Compaction Requirements: A representative of the geotechnical engineer should observe fill placement operations on a full-time basis. We recommend the following minimum compaction criteria be used on the project.

Area	Percentage of Proctor Maximum Dry Density	
	Standard Proctor (ASTM D698)	Modified Proctor (ASTM D1557)
Fill beneath foundations	98%	--
Foundation wall backfill	95%	--
Slab Subgrade	95%	--
Beneath pavement Areas/ Flatwork/Utility Trenches	95%	--
Aggregate Pavements	--	95%
Landscape and Other Misc. Overlot Fill Areas	95%	--
For compaction of suitable granular soils, a moisture content within 2 percent of optimum should be maintained. For the compaction of cohesive soils, a moisture content within 0 to 4 percent above optimum should be maintained. A moisture content sufficient to achieve adequate compaction may be used for materials with few fines, such as the aggregate base course used for aggregate pavements.		

PAVEMENT DESIGN

A pavement section is a layered system designed to distribute concentrated traffic loads to the subgrade. Performance of the pavement structure is directly related to the physical properties of the subgrade soils, pavement section, and traffic loadings. We anticipate that gravel surfaced pavements will be used for access drives at both these sites.

Subgrade Materials: Based on the American Association of State Highway Transportation Officials (AASHTO) classification system the soils tested near the proposed subgrade elevation consisted of A-6 soils with a group index of 8. These soils are rated as poor for use as subgrade material.

Design Traffic: We have assumed that after construction, the roads will only receive occasional truck traffic. For our pavement thickness design calculations, we assumed an equivalent 18-kip daily load application (EDLA) of 10 . If it is determined that actual traffic is significantly different from that estimated, we should be contacted to reevaluate the pavement thickness design.

Pavement Sections: The pavement section presented in the following table is recommended for gravel drives constructed for this project.

Pavement Section Thickness (in.)	
Area	Aggregate Base Course
Access Drives	8

Subgrade Preparation: For general subgrade preparation, we recommend the pavement subgrade be thoroughly scarified and well-mixed to a minimum depth of 12 inches, moisture conditioned, and compacted to the specifications presented in the “Site Grading and Earthwork” Section.

Proof Roll: Before paving, the subgrade should be proof rolled with a heavily loaded, pneumatic-tired vehicle. The vehicle should have a gross weight of at least 50,000 pounds, with a single loaded axle weight of 18,000 pounds, and a tire pressure of 100 psi. Areas that deform excessively under heavy wheel loads are not stable and should be removed and replaced with suitable material to achieve a stable subgrade prior to paving.

Subgrade Stabilization: Although not anticipated, areas of unstable subgrade soils may be encountered during subgrade preparation for construction of the new pavement. Unstable foundation soils may be stabilized by overexcavation and replacement of the subgrade with suitable, imported, angular, well-graded materials. Other alternatives include the use of Type 2 biaxial geogrid reinforcement in combination with a layer of Class 6 aggregate base course. It has been our experience that the use of a crushed concrete product meeting a Class 6 gradation can perform well when trying to achieve stabilization. Specific stabilization requirements should be evaluated at the time of construction.

Drainage: The collection and diversion of surface drainage away from paved areas is extremely important to the satisfactory performance of the pavement. Drainage design should provide for the removal of water from paved areas and reduce wetting of the subgrade soils.

Maintenance: Periodic maintenance will be required in paved areas, consisting of grading to remove ruts and potholes created by the environment and traffic, and to replace material that has been washed away or contaminated. During the lifetime of the pavement, the aggregate surfacing may need to be scarified, with additional aggregate added to restore the thickness to the design depth. The subgrade soils should be prepared according to the "Site Grading" section of this report.

Pavement Materials: Aggregate Base Course should conform to the requirements of AASHTO M147 and to Section 703.03 of the Colorado Department of Transportation (CDOT) Standard Specifications for Road and Bridge Construction and should meet Class 5 or 6 grading and quality as defined by the CDOT specifications. Crushed concrete meeting these requirements may also be used, and may be more resistant to rutting.

EXCAVATION CONSIDERATIONS

In our opinion, the overburden soils encountered in the exploratory borings drilled for this study can be excavated with conventional construction equipment. In accordance with OSHA criteria, the on-site clays will classify as a Type B material, and the sands will classify as an OSHA Type C material. Per OSHA criteria, unless excavations are shored, temporary unretained excavations should have slopes no steeper than the following for each soil type encountered.

Type A…………………...3/4:1 (H:V)
Type B………………….1:1 (H:V)
Type C……………….1½:1 (H:V)

A properly braced excavation or the use of a trench box should be used where the indicated unretained slopes cannot be accommodated. Flatter slopes will be required where groundwater seepage is encountered. OSHA regulations require that excavations greater than 20 feet in depth be designed by a professional engineer. If subsurface conditions vary from those indicated in this report are encountered, the OSHA soil type may vary, and the required cut slopes may need to be adjusted. The contractor's "competent person" should make all decisions regarding excavation slopes.

As noted in this report, groundwater was encountered at a depth of about 15.3 feet during the subsurface investigation, and shallow perched water may also be present within the site soils. If groundwater is present above the depth of excavation, flatter slopes will be required. It is assumed site dewatering would occur in advance of the excavation and be maintained the entire duration that the excavation is open. Surface drainage should be diverted away from all temporary cut slopes in order to reduce the potential for slope erosion and instability. OSHA regulations require that excavations greater than 20 feet in depth and excavations that extend below the ground water level be designed by a professional engineer.

DESIGN AND SUPPORT SERVICES

Kumar & Associates, Inc. should be retained to review the project plans and specifications for conformance with the recommendations provided in this report. We are also available to assist the design team in preparing specifications for geotechnical aspects of the project and, if necessary, perform additional studies to accommodate any changes in the proposed construction.

We recommend that Kumar & Associates, Inc. be retained to provide construction observation and testing services to document that the intent of this report and the requirements of the plans and specifications are being followed during construction. This will allow us to identify possible variations in subsurface conditions from those encountered during this study and to allow us to re-evaluate our recommendations, if needed. We will not be responsible for implementation of the recommendations presented in this report by others, if we are not retained to provide construction observation and testing services.

LIMITATIONS

This study has been conducted for exclusive use by the client for geotechnical related design and construction criteria for the project. The conclusions and recommendations submitted in this report are based upon the data obtained from the exploratory borings at the locations indicated on Fig. 1 or as described in the report, and the proposed type of construction. This report may not reflect subsurface variations that occur between the exploratory borings, and the nature and extent of variations across the site may not become evident until site grading and excavations are performed. If during construction, fill, soil, rock or water conditions appear to be different from those described herein, Kumar & Associates, Inc. should be advised at once so that a re-evaluation of the recommendations presented in this report can be made. Kumar & Associates, Inc. is not responsible for liability associated with interpretation of subsurface data by others.

Swelling soils occur on this site. Such soils are stable at a fixed moisture content but will undergo high volume changes with changes in moisture content. The extent and amount of perched water beneath the building site as a result of area irrigation and inadequate surface drainage is difficult, if not impossible, to foresee.

The recommendations presented in this report are based on current theories and experience of our engineers on the behavior of swelling soil in this area. Standards of practice in this area evolve over time. The owner should be aware that there is a risk in constructing a building in an expansive soil area. Following the recommendations given by a geotechnical engineer, careful construction practice and prudent maintenance by the owner can, however, decrease the risk of foundation movement due to expansive soils.

The scope of services for this project does not include any environmental assessment of the site or identification of contaminated or hazardous materials or conditions. If the owner is concerned about the potential for such contamination, other studies should be undertaken.

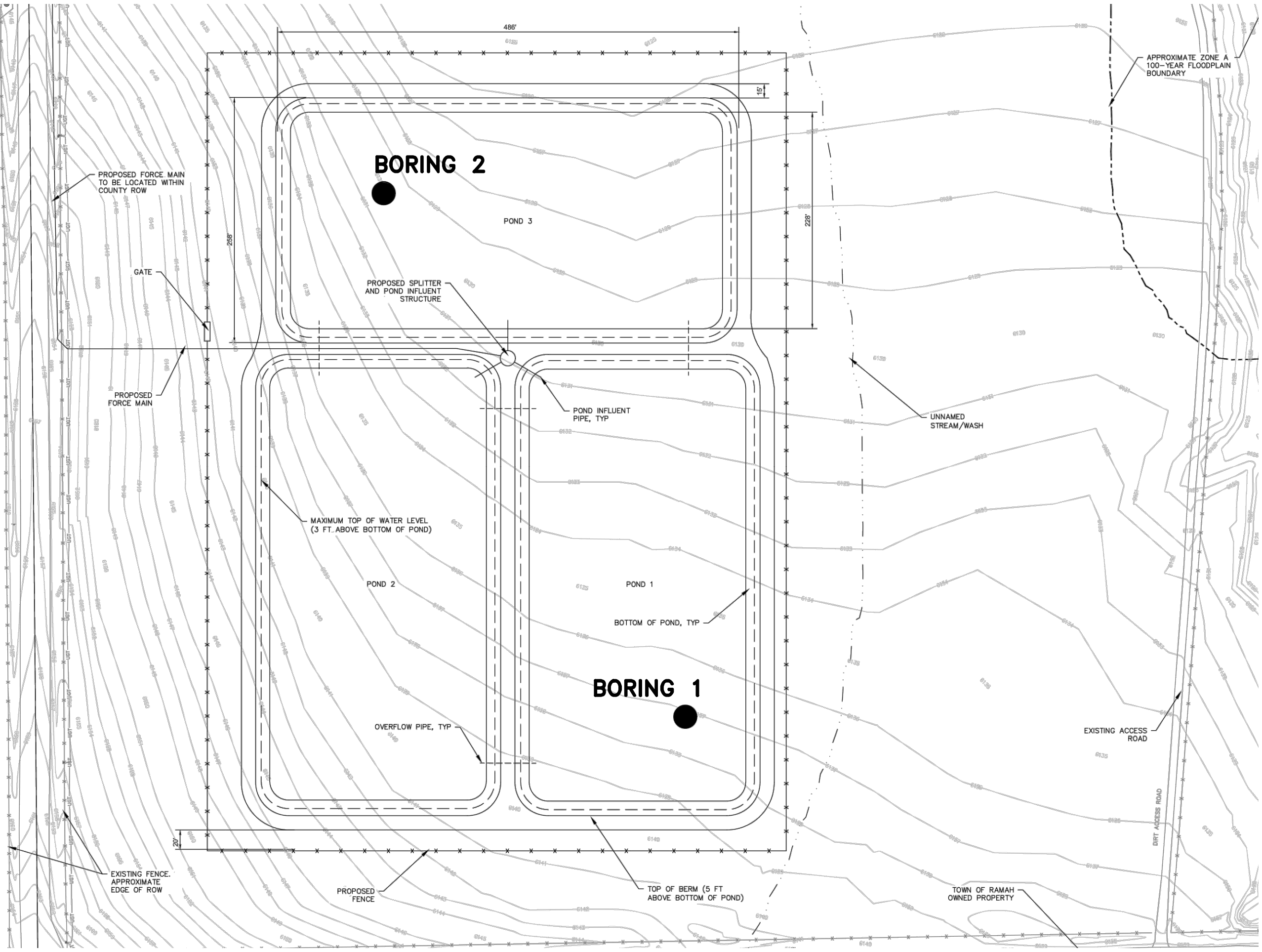
AFK:th

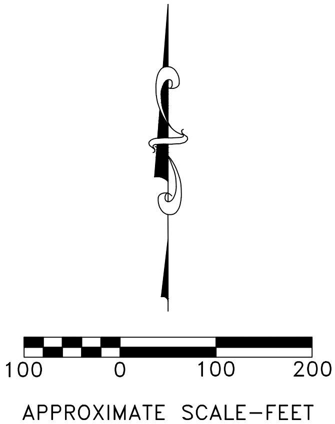
250 0 250 500

APPROXIMATE SCALE-FEET

VICINITY MAP
NOT TO SCALE

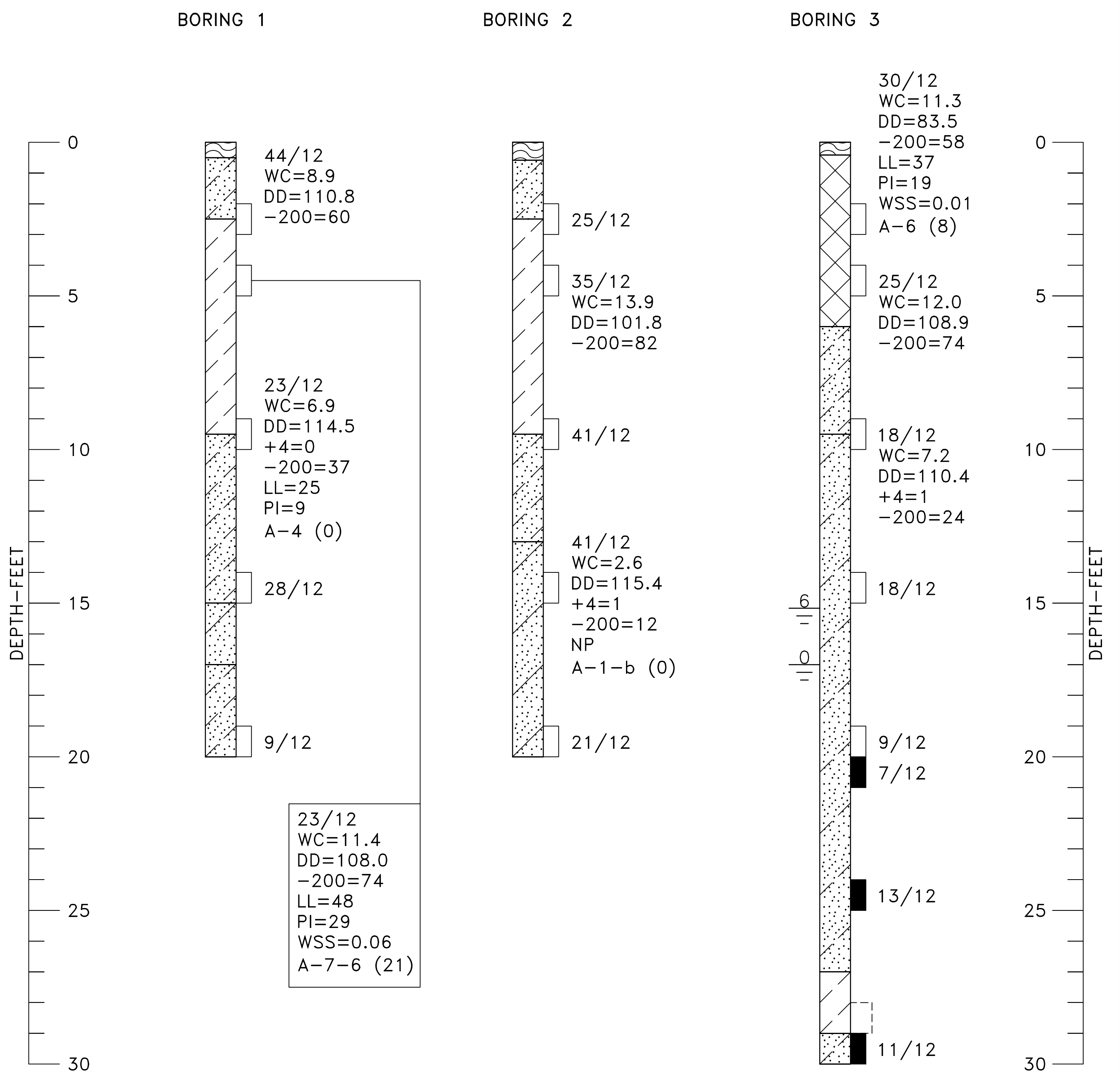
22-2-102	Kumar & Associates	LOCATION OF EXPLORATORY BORINGS	Fig. 1





April 22, 2022 - 10:30am
C:\Users\mromero\AppData\Local\Temp\AcPublish_26424\222102-01.dwg

22-2-102	Kumar & Associates	LOCATION OF EXPLORATORY BORINGS	Fig. 1



22-2-102	Kumar & Associates	LOGS OF EXPLORATORY BORINGS	Fig. 2

April 22, 2022 - 10:30am
V:\Projects\2022\22-2-102 Ramah Water Treatment Plant Improvements (CMT)\Drafting\222102-02 to 03.dwg

LEGEND

TOPSOIL.

FILL: SANDY LEAN CLAY AND LEAN CLAY WITH SAND (CL), FINE TO COARSE GRAINED SAND FRACTION, MOIST, MOTTLED BROWNS.

CLAYEY SAND (SC), FINE TO COARSE GRAINED, MEDIUM DENSE TO DENSE, SLIGHTLY MOIST TO VERY MOIST, LIGHT BROWN TO GRAY-BROWN.

SANDY LEAN CLAY TO LEAN CLAY WITH SAND (CL), FINE TO COARSE GRAINED SAND FRACTION, VERY STIFF TO HARD, SLIGHTLY MOIST TO MOIST, LIGHT TO DARK BROWN.

WELL GRADED SAND WITH SILT (SW-SM), FINE TO COARSE GRAINED, MEDIUM DENSE TO DENSE, MOIST, LIGHT BROWN.

WELL GRADED SAND WITH CLAY (SW-SC), FINE TO COARSE GRAINED, LOOSE TO MEDIUM DENSE, MOIST TO WET, LIGHT BROWN TO GRAY-BROWN.

DRIVE SAMPLE, 2-INCH I.D. CALIFORNIA LINER SAMPLE.

DRIVE SAMPLE, 1 3/8-INCH I.D. SPLIT SPOON STANDARD PENETRATION TEST.

DISTURBED BULK SAMPLE.

44/12 DRIVE SAMPLE BLOW COUNT. INDICATES THAT 44 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE THE SAMPLER 12 INCHES.

6 DEPTH TO WATER LEVEL AND NUMBER OF DAYS AFTER DRILLING MEASUREMENT WAS MADE.

NOTES

1. THE EXPLORATORY BORINGS WERE DRILLED ON MARCH 15, 2022 WITH A 4-INCH-DIAMETER CONTINUOUS-FLIGHT POWER AUGER.

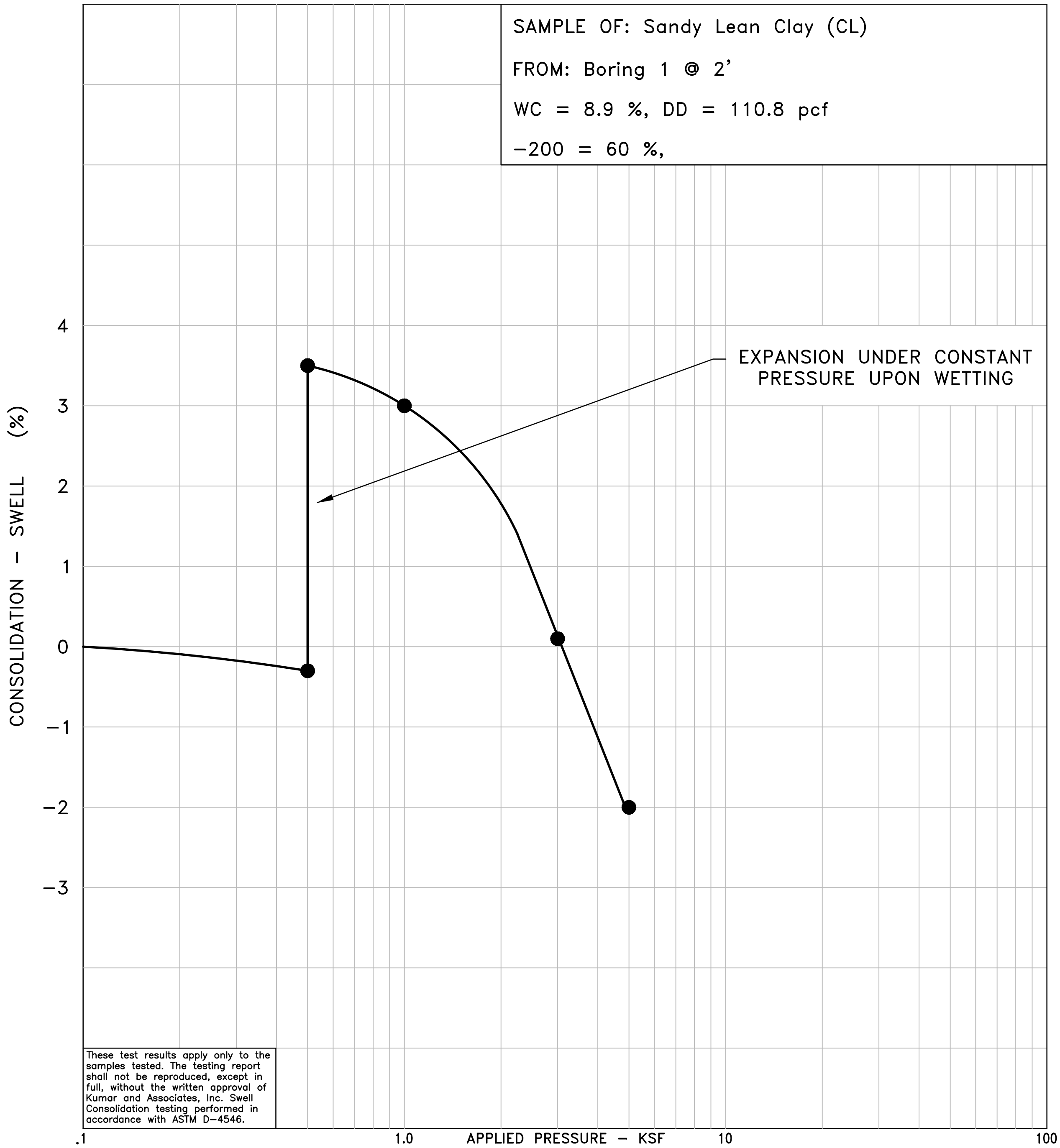
2. THE LOCATIONS OF THE EXPLORATORY BORINGS WERE MEASURED APPROXIMATELY BY PACING FROM FEATURES SHOWN ON THE SITE PLAN PROVIDED AND SHOULD BE CONSIDERED ACCURATE ONLY TO THE DEGREE IMPLIED BY THE METHOD USED.

3. THE ELEVATIONS OF THE EXPLORATORY BORINGS WERE NOT MEASURED AND THE LOGS OF THE EXPLORATORY BORINGS ARE PLOTTED TO DEPTH.

4. THE LINES BETWEEN MATERIALS SHOWN ON THE EXPLORATORY BORING LOGS REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN MATERIAL TYPES AND THE TRANSITIONS MAY BE GRADUAL.

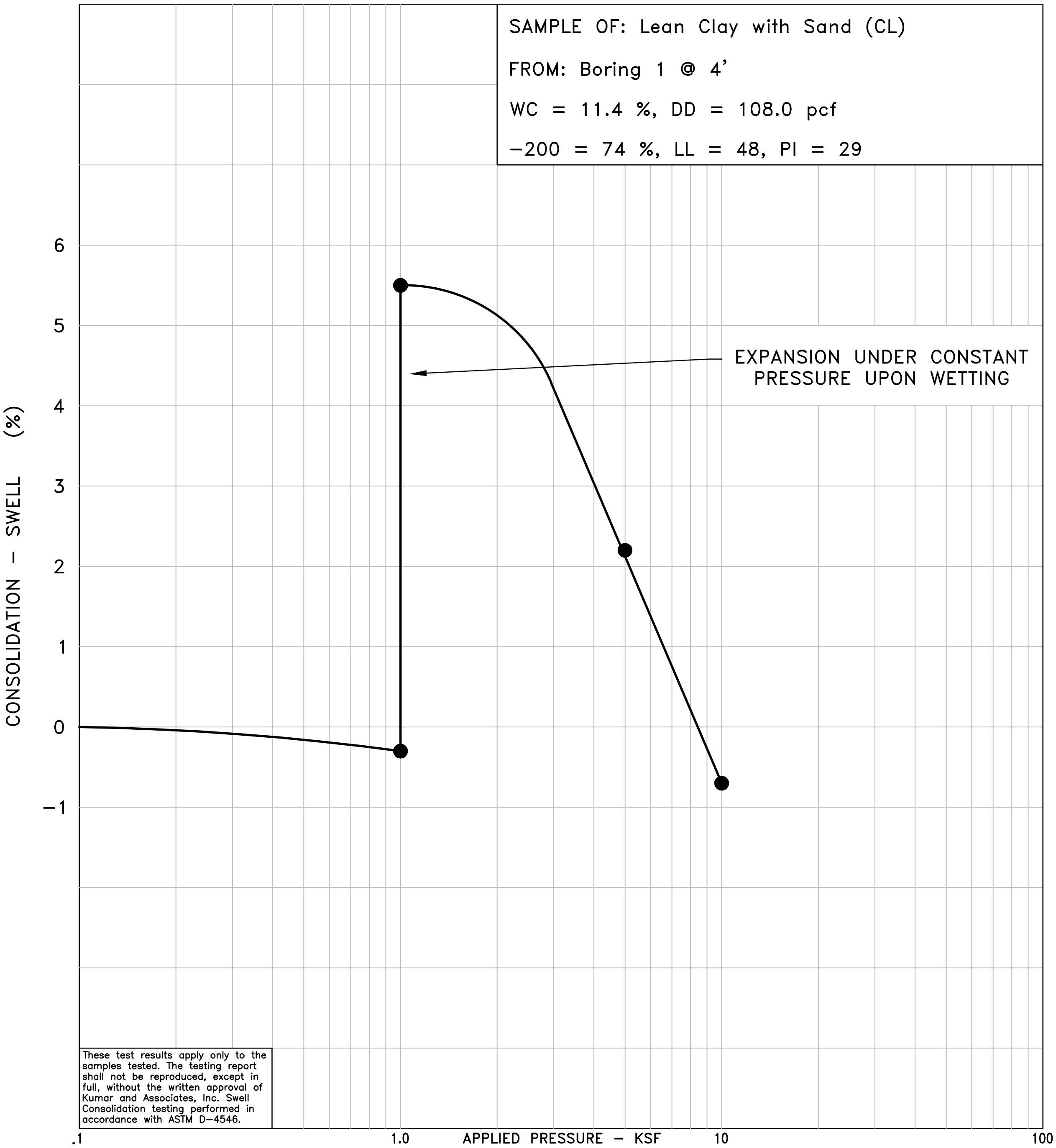
5. GROUNDWATER LEVELS SHOWN ON THE LOGS WERE MEASURED AT THE TIME AND UNDER CONDITIONS INDICATED. FLUCTUATIONS IN THE WATER LEVEL MAY OCCUR WITH TIME.

6. LABORATORY TEST RESULTS:
   WC = WATER CONTENT (%) (ASTM D2216);
   DD = DRY DENSITY (pcf) (ASTM D2216);
   +4 = PERCENTAGE RETAINED ON NO. 4 SIEVE (ASTM D6913);
   -200= PERCENTAGE PASSING NO. 200 SIEVE (ASTM D1140);
   LL = LIQUID LIMIT (ASTM D4318);
   PI = PLASTICITY INDEX (ASTM D4318);
   NP = NON-PLASTIC (ASTM D4318);
   WSS = WATER SOLUBLE SULFATES (%) (CP-L 2103);
   A-7-6 (21) = AASHTO CLASSIFICATION (GROUP INDEX) (AASHTO M 145).



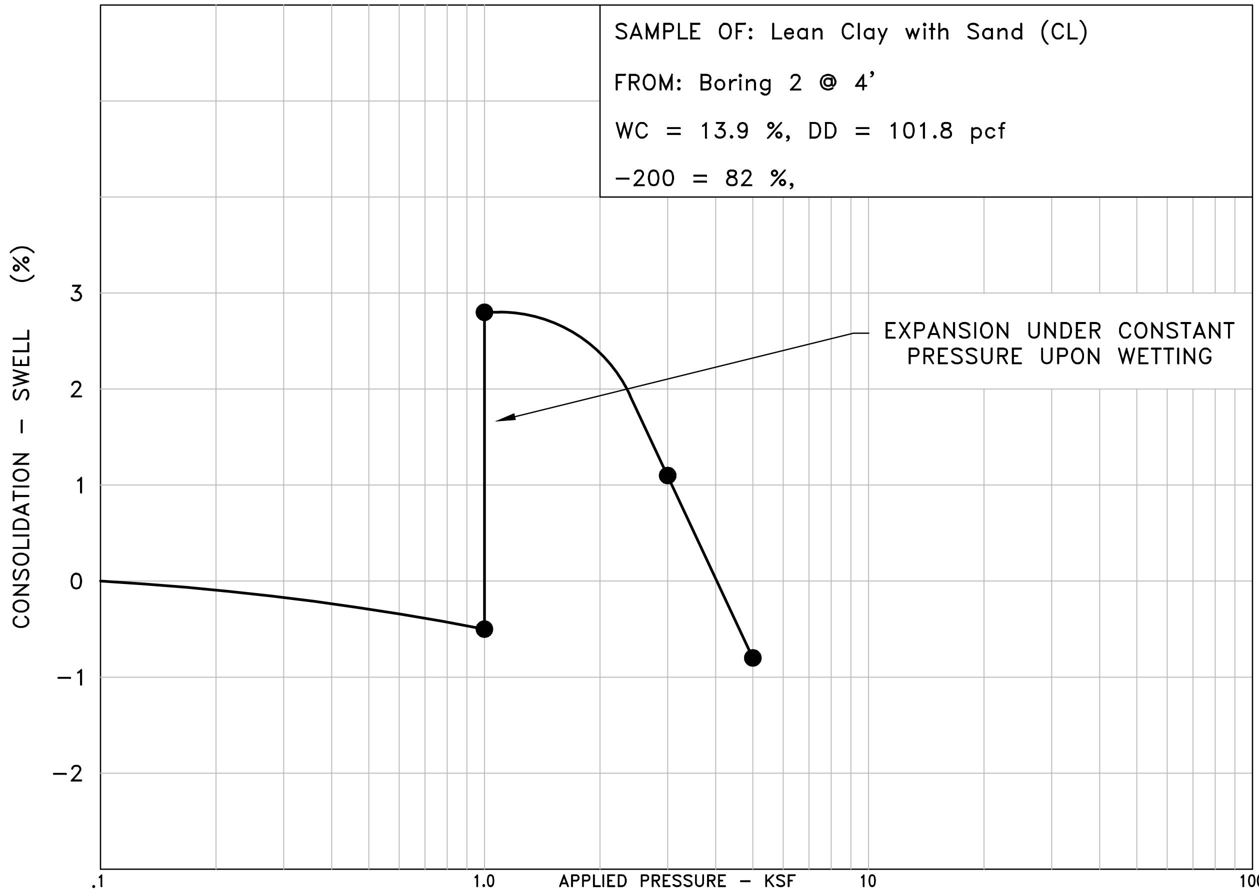
April 22, 2022 − 10:30am
V:\Projects\2022\22-2-102 Ramah Water Treatment Plant Improvements (CMT)\Drafting\222102-04 to 06.dwg

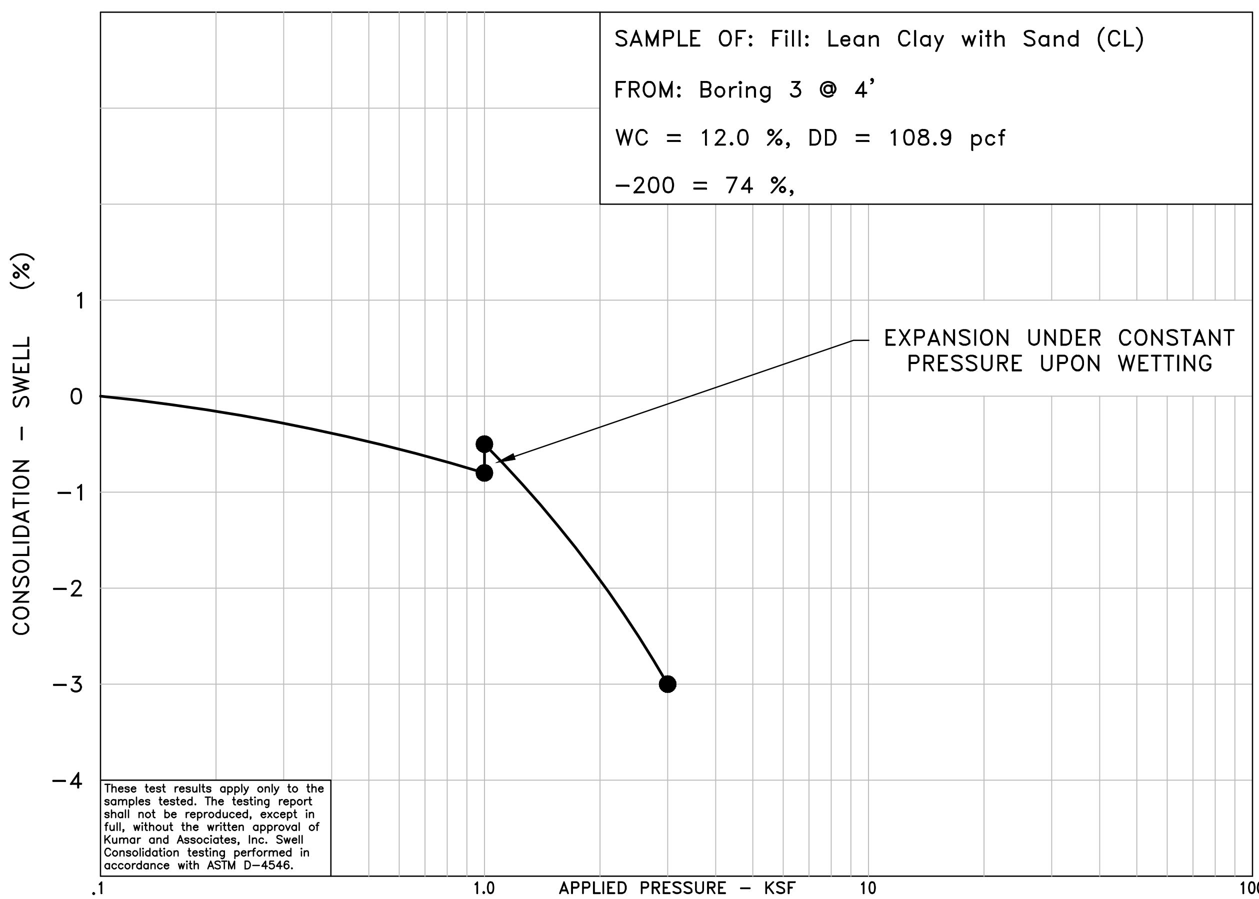
22−2−102	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULTS	Fig. 4



April 22, 2022 − 10:30am
V:\Projects\2022\22-2-102 Ramah Water Treatment Plant Improvements (CMT)\Drafting\222102-04 to 06.dwg

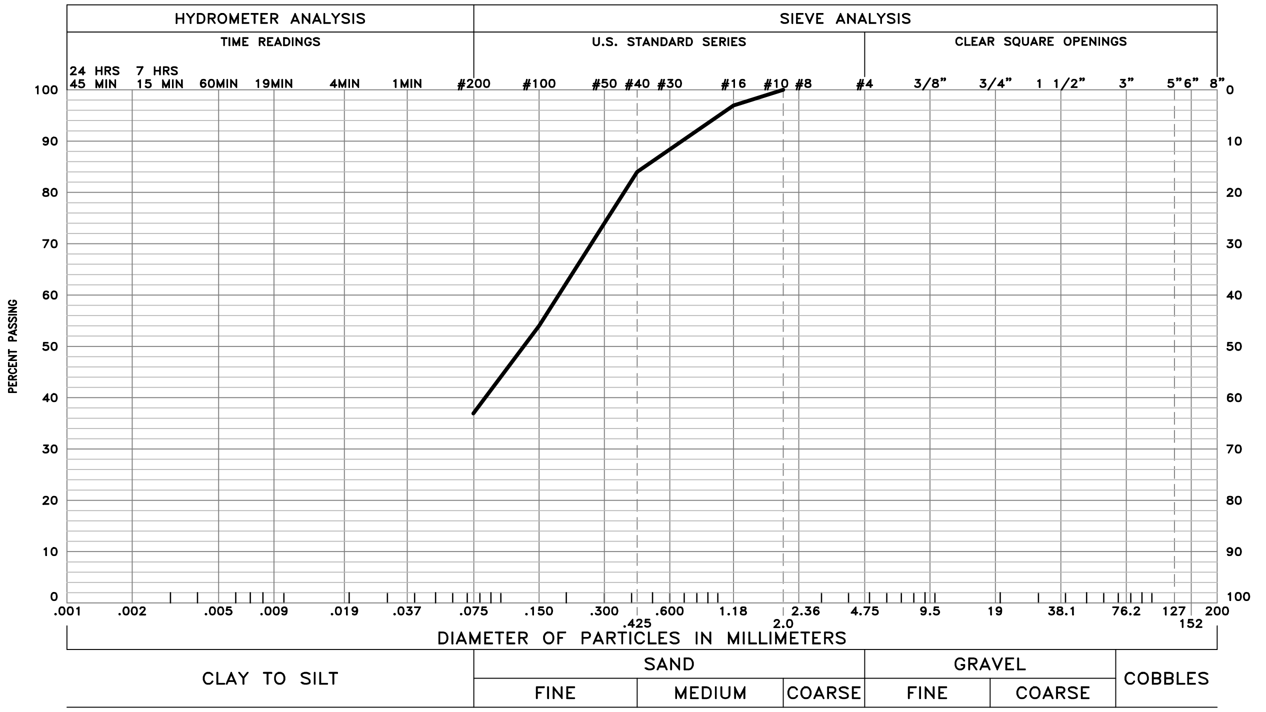
22−2−102	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULTS	Fig. 5





22−2−102	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULTS	Fig. 6

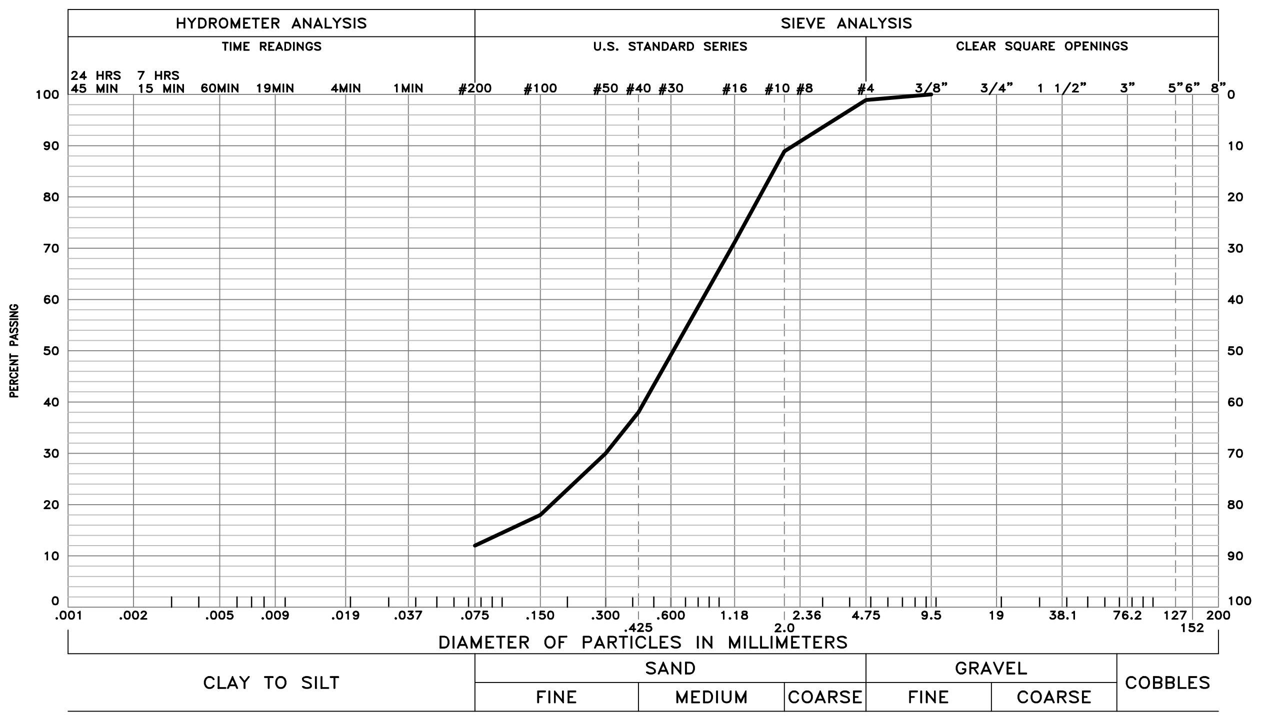
April 22, 2022 − 10:30am
V:\Projects\2022\22-2-102 Ramah Water Treatment Plant Improvements (CMT)\Drafting\222102-04 to 06.dwg



GRAVEL 0 % SAND 63 % SILT AND CLAY 37 %

LIQUID LIMIT 25 PLASTICITY INDEX 9

SAMPLE OF: Clayey Sand (SC) FROM: Boring 1 @ 9'

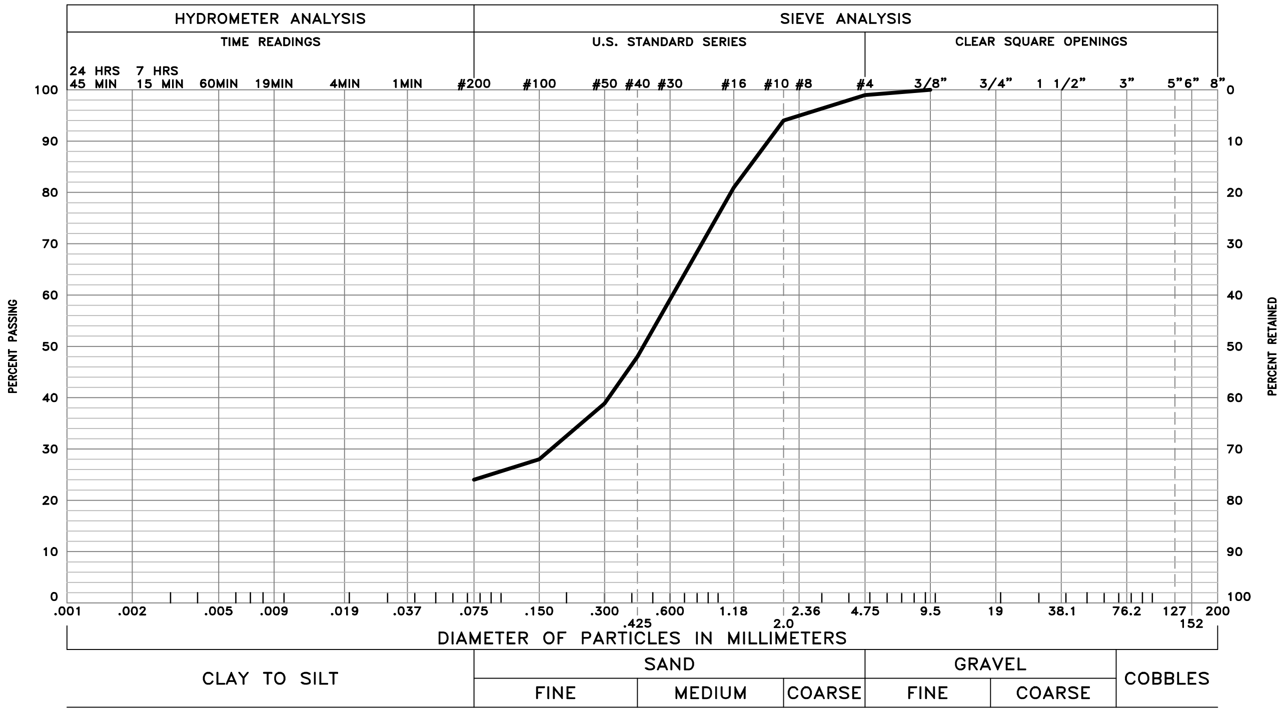


GRAVEL 1 % SAND 87 % SILT AND CLAY 12 %

LIQUID LIMIT – PLASTICITY INDEX NP

SAMPLE OF: Well Graded Sand with Silt (SW–SM) FROM: Boring 2 @ 14'

22–2–102	Kumar & Associates	GRADATION TEST RESULTS	Fig. 7



GRAVEL 1 % SAND 75 % SILT AND CLAY 24 %

LIQUID LIMIT – PLASTICITY INDEX –

SAMPLE OF: Clayey Sand (SC) FROM: Boring 3 @ 9'

These test results apply only to the samples which were tested. The testing report shall not be reproduced, except in full, without the written approval of Kumar & Associates, Inc. Sieve analysis testing is performed in accordance with ASTM D6913, ASTM D7928, ASTM C136 and/or ASTM D1140.

22–2–102	Kumar & Associates	GRADATION TEST RESULTS	Fig. 8

April 22, 2022 – 10:30am
V:\Projects\2022\22-2-102 Ramah Water Treatment Plant Improvements (CMT)\Drafting\222102-07 to 08.dwg

Kumar and Associates, Inc.

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

Project No.: 22-2-102
Project Name: Ramah Water Treatment Plant
Date Sampled: 03/15/2022
Date Received: 3/15/22

SAMPLE LOCATION		DATE TESTED	NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (pcf)	GRADATION		PERCENT PASSING NO. 200 SIEVE	ATTERBERG LIMITS		WATER SOLUBLE SULFATES (%)	AASHTO CLASSIFICATION (Group Index)	SOIL OR BEDROCK TYPE (Unified Soil Classification)
BORING	DEPTH (ft)				GRAVEL (%)	SAND (%)		LIQUID LIMIT	PLASTICITY INDEX			
1	2	3/30/22	8.9	110.8			60					Sandy Lean Clay (CL)
1	4	3/30/22	11.4	108.0			74	48	29	0.06	A-7-6 (21)	Lean Clay with Sand (CL)
1	9	3/30/22	6.9	114.5	0	63	37	25	9		A-4 (0)	Clayey Sand (SC)
2	4	3/30/22	13.9	101.8			82					Lean Clay with Sand (CL)
2	14	3/30/22	2.6	115.4	1	87	12		NP		A-1-b (0)	Well Graded Sand with Silt (SW-SM)
3	2	3/30/22	11.3	83.5			58	37	19	0.01	A-6 (8)	Fill: Sandy Lean Clay (CL)
3	4	3/30/22	12.0	108.9			74					Fill: Lean Clay with Sand (CL)
3	9	3/30/22	7.2	110.4	1	75	24					Clayey Sand (SC)