GEO-TEST

GEOTECHNICAL ENGINEERING SERVICES REPORT

NO. 1-30401

JOHN STREET
STORM DRAIN
ENGINEERING

ALBUQUERQUE, NEW MEXICO

PREPARED FOR:

RESPEC

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GEO-TEST

August 14, 2023
Job No. 1-30401

RESPEC
7770 Jefferson St. NE, Suite 100
Albuquerque, NM 87109

ATTN: Christopher Archuleta

RE: Geotechnical Engineering Services Report
John Street Storm Drain Engineering
Albuquerque, NM

Dear Mr. Archuleta:

Submitted herein is the Geotechnical Engineering Services Report regarding the above referenced project. The report contains the results of our field investigation, laboratory testing and recommendations regarding pond and embankment construction as well as building foundation, slab on grade and retaining structure design as well as excavation, fill and general site grading criteria.

It has been a pleasure to serve you on this project. If you should have and questions or concerns regarding the report or aspects of the investigation please contact our office.

Respectfully submitted:
GEO-TEST, INC.

PATRICK R. WHORTON
26342

Patrick R. Whorton, PE

Reviewed By:

Patrick J. Byres, PE

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INTRODUCTION

This report presents the results of the geotechnical engineering services investigation performed by this firm for the proposed City of Albuquerque John Street Storm Drain project to be constructed in Albuquerque, New Mexico.

The objectives of this investigation were to:

1) Evaluate the nature and engineering properties of the subsurface soils underlying the site.
2) Provide recommendations for the design and construction of detention ponds including embankment foundation preparation and construction, considering slope stability, settlement, seepage and erodibility.
3) Provide recommendations for building foundation design, slab support and retaining wall design as well as excavation, fill and general site grading criteria.

The investigation includes subsurface exploration, selected soil sampling, laboratory testing of the samples, performing an engineering analysis and preparation of this report.

PROPOSED CONSTRUCTION

It is understood that the project consists of the construction of two new earth embankment storm water detention ponds on an undeveloped parcel of land located within a residential neighborhood located west of the intersection of Gibson Blvd. and Broadway Blvd SE. The ponds are intended to take in storm water from 2 separate sources, the northern pond from a 72-inch storm drain coming in from Hinkle Street SE and the southern pond from a 66-inch storm drain coming in from John Street SE. The ponds will be designed for temporary storage of storm water, designed to completely drain within 96 hours. The ponds will be be constructed by excavation from approximate existing site grades. Cuts on the order of 23 feet for the northern pond and about 14 feet for the southern pond will be needed to bring the bottom of the ponds to their proposed grades. The northern pond will have shotcrete lined 3:1 (horizontal to vertical) slopes and a 14 foot tall embankment and the southern pond will have 3:1 slopes. The southern pond will have a 30-inch diameter outlet pipe in the southwest corner of the pond. In addition, a lift station consisting of a wet well and valve vault as well as a separate electrical/utility building will be constructed on the north side of the northern pond, at or near existing site grades.

Should project details very significantly from those outlines above, this firm should be notified for review and possible revision of the recommendations contained herein.

FIELD EXPLORATION

A total of eight (8) exploratory borings were drilled at the site to depths ranging from 10 to 30 feet below existing surfaces grades. Locations of the borings are shown on the attached Boring Location Map, Figure 2. The soils encountered in the borings were continuously examined, visually classified and logged during the drilling operation. The boring logs are presented in a following section of this report. Drilling was accomplished using a truck mounted drill rig equipped with 3.25 inch inner diameter hollow stem auger. Subsurface soils within the building footprint were sampled at five-foot intervals or less utilizing an open tube split barrel sampler driven by a standard penetration test hammer.

LABORATORY TESTING

Selected samples were tested in Geo-Test, Inc. laboratories to determine certain engineering properties of the subsurface soils encountered in the field investigation. Moisture contents were determined to evaluate the various soil deposits with depth. The results of these tests are shown on the Boring Logs.

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Sieve analysis and Atterberg limits testing was performed to aid in soils classification. Constant head permeability testing was performed on relatively undisturbed tube samples. The results of these tests are presented in the Summary of Laboratory Results and on individual test reports presented in a following section of this report.

SURFACE CONDITIONS

The project site is located west of John St. SE and north of Englewood Dr. within a residential neighborhood. The site is undeveloped and with the exception of John St. on the eastern property line, bordered on all sides by single family homes. The site is graded relatively flat and sparsely populated with native shrubs and grasses with a few medium sized trees.

SUBSURFACE SOIL CONDITIONS

As indicated by the exploratory borings, the subsurface soils beneath the site consisted of two soil types. The first is a loose to medium dense non-plastic silty sand and the second is a moderately firm to firm medium to high plasticity clay. The sand was encountered at the surface throughout the site as is the primary soil type encountered in the upper 20 feet. The clay is present in the upper 20 feet primarily as 3 to 6 foot thick layers which were encountered at about 10 feet below surface grade in most borings. Below a depth of 20 feet, the clay was found to be the primary soil type, extending to the full depth explored in the deeper borings.

Free groundwater was encountered at depths of 27 to 29 feet below existing surface grades. Fluctuations in the level of the ground water may occur due to variations in rainfall, snow melt, underground drainage patterns, and may also be affected by outside factors such as pumping of local wells and the Rio Grande, as well as other factors not evident at the time our measurements were made.

The silty sand was found to be relatively dry throughout the depths explored. The clay was found to have a generally high moisture content, becoming saturated at depths of 27 to 29 feet below existing surface grades.

POND EXCAVATION & CONSTRUCTION

North Pond

The site of the North Pond is underlain by silty sand which was encountered at the surface and extended to a depth of 8 feet where a 6 foot thick layer of medium plasticity clay was encountered. Silty sand was encountered beneath the clay extended to the proposed pond bottom elevation, 23 feet below surface grade, where medium plasticity clay was encountered and extended to the full depth explored with groundwater encountered at a depth of 27 feet, or 5 feet below the proposed pond bottom. These soils may be readily excavated using normal earth moving equipment.

The stability of the North Pond excavation was analyzed using the two-dimensional limit equilibrium stability program STABLPRO by Ensoft. Bishop's Method of Slices was used to develop factors of safety against slip on a circular failure plane for both static and pseudo-static loading conditions for three anticipated slopes to be used in pond construction are presented below in terms of factors of safety with a factor of 1 being stable, less than 1 unstable, 2 recommended for static conditions and 1.3 recommended for pseudo-static conditions.

Slope (H:V)	Height (ft)	Static FS	Pseudo-Static FS
2:1	23	3.076	2.207

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For the purposes of this analysis, a horizontal pseudo-static coefficient of 0.20g was utilized in the analyses based on 100% of the predicted peak ground acceleration within 50 years with a 2 percent probability of exceedance.

South Pond

The site of the South Pond is underlain by silty sand which was encountered at the surface and extended to the full depth explored. A layer of clay was encountered between 8 and 10 feet below surface grade in some areas but was absent in other borings. It is anticipated silty sand will be encountered at the base of the excavation, however given the sporadic placement of clay in the upper 20 feet, clay could be present as well.

The stability of the South Pond excavation was analyzed using the two-dimensional limit equilibrium stability program STABLPRO by Ensoft. Bishop's Method of Slices was used to develop factors of safety against slip on a circular failure plane for both static and pseudo-static loading conditions for three anticipated slopes to be used in pond construction are presented below in terms of factors of safety with a factor of 1 being stable, less than 1 unstable, 2 recommended for static conditions and 1.3 recommended for pseudo-static conditions.

Slope (H:V)	Height (ft)	Static FS	Pseudo-Static FS
3:1	14	4.681	2.737

For the purposes of this analysis, a horizontal pseudo-static coefficient of 0.20g was utilized in the analyses based on 100% of the predicted peak ground acceleration within 50 years with a 2 percent probability of exceedance.

These results indicate that the proposed ponds may be excavated to the proposed depths at the proposed side slopes. However, the construction of the ponds as proposed will result in excessive seepage through the soils separating the two ponds. Given these soils primarily consist of non-plastic, low cohesive silty sand which has a relatively high hydraulic conductivity and that a hold time of 96 hour is enough time to develop steady state flow; excessive seepage through the soils from the higher elevation South Pond to the lower elevation North Pond will occur. Over time, this condition will lead to internal piping and saturation of these soils which could ultimately lead to excessive settlement, erosion and slope collapse.

Therefore, it is recommended that during construction, both ponds be excavated to their design depths, or to such an extent as to construct a earthen embankment between the two ponds which will be completely founded on the deeper underlying clay layer and constructed with processed low plasticity embankment fill according to the methods recommended in the Site Grading section of this report.

The purpose of the constructed embankment is to provide a uniform barrier between the two pond which will have a lower hydraulic conductivity than the surrounding native soils that will be resistant to internal drainage, piping and erosion. The extents of the full depth embankment area should be the area between the two ponds extending a minimum of 20 feet beyond the outside perimeter of the North Pond.

In addition to the use of a low conductive embankment fill, it is further recommended the the side slopes of the embankment be protected with shotcrete or grouted boulders.

It is also recommended that the South Pond be graded to drain toward the south, away from the embankment in order to minimize the time water will be detained at the embankment in order to further minimize seepage through the embankment.

EMBANKMENT CONSTRUCTION

The native soils encountered within the pond excavation areas may be readily excavated using normal earthmoving equipment and may be reused as embankment fill for the construction of the embankments between and surrounding the proposed ponds, although processing, including blending and moisture conditioning, will be required to meet the specifications for structural fill.

The embankment foundation area located between the ponds, extending laterally a minimum of 20 feet beyond the outer perimeter of the North Pond should be excavated to their design depths or to such an extent as to provide for a minimum of 3.0 feet of native clay beneath the embankment base whichever is the greater depth of excavation. The depth of clay should be determined by overexcavation or through investigation by the geotechnical engineer. Based on the results of this investigation, the clay should be present at depths ranging from 18 to 23 feet and extends to depths greater than 3.0 feet such that it is anticipated that overexcavation will not be required, however, given the sporadic depths at which the clay was encountered throughout the site, overexcavation of sand to be replaced with onsite or imported clay may be required on some areas. Once the required excavations have been completed, the native cut surface should be densified, and the embankment constructed in accordance with the Site Grading section of this report.

Embankments surrounding the pond perimeter which will not be directly exposed to ponded water do not require overexcavation or placement on clay, however the native cut surface should be densified before embankment fill is placed according to the method in the Site Grading section of this report.

Total settlement of the embankment is a function of internal embankment settlement and foundation settlement. Maximum foundation settlements are estimated to be on the order of 1½ inches, and internal embankment settlements are estimated to be on the order of 1 inch or less. However, since the vast majority of both foundation and embankment settlements will be elastic and occur during

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construction, only minor settlement, less than ¾ inch is anticipated upon completion of construction.

The stability of the proposed embankment was analyzed using the two-dimensional limit equilibrium stability program STABLPRO by Ensoft. Bishop's Method of Slices was used to develop factors of safety against slip on a circular failure plane for both static and pseudo-static loading conditions for three anticipated slopes to be used in pond construction are presented below in terms of factors of safety with a factor of 1 being stable, less than 1 unstable, 2 recommended for static conditions and 1.3 recommended for pseudo-static conditions.

Slope (H:V)	Height (ft)	Static FS	Pseudo-Static FS
2:1	23	3.791	2.757
3:1	14	5.687	3.339

Based on these results the embankment will be stable at the proposed slopes, provided the embankment is constructed according the method detailed the Site Grading section of this report. However, the embankment, as well as other cut slopes, will be susceptible to erosion, seepage and other factors as discussed in the following seciton.

Given the embankment will be founded on a layer of clay and constructed with relatively low conductivity embankment fill, excessive seepage beneath the embankment is not considered an issue such that internal drainage within the embankment will not be required. In addition, filter/drain material around pipes within the embankment itself to control seepage is not considered necessary provided the embankment fill is carefully placed around the pipe as recommended in the Site Grading section of this report.

The onsite soils anticipated to be used as dam embankment fill and which will comprise cut slopes throughout the project consist in part of relatively low cohesive granular soils. As such, slopes constructed with these soils will be subject to both sheet and rill erosion. As a general rule the amount of erosion to be expected is directly related to how steep the slope in question is with steeper slopes experiencing more erosion that flatter slopes.

As stated in previous sections, permanent project slopes may be as steep as 2:1. Although stable, these steeper slopes will likely experience greater erosion than flatter slopes which may compromise the stability and integrity of the slopes. Therefore, the stability of all slopes recommended herein should be considered contingent upon the implementation of proper erosion protection. It is recommended that all 2:1 slopes and the 3:1 embankment slopes be protected with either shotcrete or grouted boulders/cobbles to prevent surface erosion as well as erosion associated with the rise an fall of detained water as part of the normal operation of the ponds.

OUTLET STRUCTURES & PIPE INSTALLATION

The principal spillway pipe (outlet) for the southern pond will consist of a ported riser structure connected to a 30-inch diameter reinforced concrete pipe which will connect to an existing sewer located at the southwest corner of the site. The ported riser and RCP will be placed within the cut area of the pond. It is recommended overexcavation be conducted to provide for a minimum of 2.0 feet of embankment fill beneath the riser structure.

The area immediately surrounding the ported riser and primary spillway pipe drop will be particularly susceptible to erosion which may compromise the adjacent cut slope. As such, it is recommended that riprap or other erosion protection be installed around the base of the ported riser and placed to protect the adjacent cut slope.

The subsurface soils within proposed trench excavations, both for inlet and outlet piping may be readily excavated using normal earthmoving equipment. Excavated soils are generally suitable for use as trench backfill and pipe bedding but should meet the specification and be placed according to the method detailed in the Site Grading section of this report.

ELECTRICAL BUILDING FOUNDATION

Near surface soils underlying the building site were found to be loose in their present condition and not considered suitable to provide reliable support of the proposed electrical building. Foundations bearing on these soils would be susceptible to excessive differential settlements, particularly upon significant moisture increases. However, with site preparation and very careful moisture protection, as recommended in a following section of this report, the proposed structure may be supported on a turn-down monolithic slab bearing directly on properly compacted structural fill.

The site preparation would involve overexcavation of the existing soils throughout the building area to such an extent as to provide for a minimum of 3.0 feet of properly compacted, non-expansive structural fill below the base and turn-down edge of the slab. The limits of the overexcavation should also extend laterally from the slab perimeter a distance equal to the depth of fill beneath the base. The exposed native soils at the base of the excavations should be densified prior to placement of structural fill. Detailed recommendations for foundation design and the required site grading are presented below.

Post-construction moisture increases in the supporting soils could cause some differential foundation movements. Therefore, moisture protection is considered a critical design consideration and should be reflected in overall site grading and drainage details as recommended in the Moisture Protection section of this report.

An allowable bearing pressure of 2,500 pounds per square foot is recommended for footing design. This bearing pressure applies to full dead load plus realistic live loads and can be safely increased by one-third for total loads including wind and seismic forces.

The turn-down edge of the slab should be established a minimum of 1.5 feet below lowest adjacent finish grade. The minimum recommended width of continuous footings is 1.33 feet.

Adequate support for lightly loaded slab-on-grade floors will be provided by the structural fill when compacted as recommended in the Site Grading section of this report. Thus, the use of a granular base for structural support of lightly loaded the slab is not considered necessary, however, should a gravel base be desired as a working surface or to increase the modulus of subgrade reaction, a course of granular base may be placed beneath concrete floor slabs on grade.

Where granular base is used beneath the slabs, it should have a plasticity index no greater than 3 and meet in following gradation:

Sieve Size Square Openings	Percent Passing by Dry Weight
1.0 inch	100
¾ inch	70-100
No. 4	35-85
No. 200	0-10

The granular base should be compacted to at least 95 percent of maximum dry density as determined in accordance with ASTM D-1557.

Any heavily loaded slabs on the project bearing on structural fill should be designed using a modulus of subgrade reaction of 200 pounds per square inch per inch of deflection. If a 6 inch thick layer of granular base is placed and compacted below the slab, the modulus of subgrade reaction may be increased to 300 pounds per square inch per inch of deflection.

Resistance to lateral forces will be provided by soil friction between the base of floor slabs, footings and the soils as well as by passive earth pressure acting against the side of the footings and stem walls. A coefficient of friction of 0.40 should be used for computing the lateral resistance between bases of footing and slabs and the soil. With backfill placed as recommended in the Site Grading section of this report, a passive soil pressure of equivalent to a fluid weighing 375 pounds per cubic foot should be used for analysis.

Total settlements of foundations designed and constructed as recommended herein are estimated not to exceed ¾ inch for the soil moisture contents encountered during this investigation or moisture contents introduced during construction. Differential movements should be less than 75 percent of total movements. Significant post-construction moisture increases in the supporting soils could create additional movements and could cause excessive movements, at least in some areas of the site. Accordingly, the moisture protection provisions as recommended in the Moisture Protection section of this report are considered critical for the satisfactory performance of the structures.

LIFT STATION FOUNDATION

Both the wet well and valve vault may be supported by a precast or cast-in-place reinforced manhole base (mat foundation). An allowable soil bearing pressure of 2,500 pounds per square foot (psf) is recommended for use in foundation design. This bearing pressure applies to full dead plus realistic live loads and can be safely increased by one-third for totals loads including wind and seismic forces.

A modulus of subgrade reaction of 200 pounds per square inch per inch of deflection (pci) is recommended for use in a non-rigid design such as a two-dimensional finite element method.

Groundwater was encountered at a depth of 29 feet below existing surface grade within the exploratory borings in the vicinity of the proposed well. Based on these measurements, it is not anticipated that groundwater will infiltrate the lift station excavation during construction, however, this is dependent on groundwater levels during excavation such that dewatering may be required. Post construction, the groundwater level is not anticipated to rise to a level that will require anti-flotation precautions.

Total settlement of the foundations designed and constructed as recommended herein are estimated not to exceed ½ inch for the soil moisture contents encountered during this investigation or moisture contents introduced during construction. Differential movements should be less than 75 percent of total movements.

Significant post-construction moisture increases in the supporting soils could create additional movements and could cause excessive movements of foundation bearing soils. Accordingly, the moisture protection provisions as recommended in a following section of this report are considered critical for the satisfactory performance of lift station elements.

Cut surface preparation prior to the placement of foundations elements and backfill of lift station wells should adhere to the methods and specifications outlined in the Site Grading section of this report.

Resistance to lateral forces will be provided by soil friction between the manhole bases and the soil and by passive earth resistance against the sides of the structures. A coefficient of friction of 0.40 should be used for computing the lateral resistance between bases of foundations and the soil or crushed stone. A passive soil resistance equivalent to a fluid weighing 375 pounds per cubic foot should be used for analysis.

Lateral pressure against the walls of the wet well should be designed for an ‘at rest’ earth pressure of 60 pounds per square foot per foot of depth. This lateral soil pressure is applicable to a condition of horizontal backfill without surcharge loads. Analysis of earth pressures produced by sloping backfill or surcharge loads can be provided by this firm upon request.

Well backfill should meet the structural fill specifications outlined in the Site Grading section of this report. During backfilling, the contractor should be limited to the use of hand operated compaction equipment within a zone of about 3 feet horizontally from the perimeter of the wells. The use of heavier equipment could apply lateral pressures well in excess of the recommended design earth pressure, particularly over the upper portions of the wells.

Excavated slopes for lift station construction should be designed and constructed in accordance with 29 CFR 1926, Subpart P, and any applicable state or local regulations. Excavated temporary slopes should not exceed 1.5 to 1 (horizontal to vertical). Shoring is recommended and should be designed by a qualified engineer utilizing the results of this investigation. Should additional soil data be required for shoring design; it may be provided by this firm upon request. Benching of temporary slopes may be conducted at the contractor’s discretion provided all applicable safety standards are followed and the average slope is no steeper than 1.5:1.

SITE GRADING

The following general guidelines should be included in the project construction specifications to provide a basis for quality control during site grading. It is recommended that all structural fill and backfill be placed and compacted under engineering observation and in accordance with the following:

1 After clearing, grubbing the two ponds should be excavated to their design depths or to such an extent as to provide for a minimum of 3.0 feet of clay below the base of the embankment. The (over)excavation limits should extend laterally a minimum of 20.0 feet laterally beyond the outer perimeter of the North Pond. The soils exposed at the base of the overexcavation should be densified before placement of embankment fill.

2 Densification of native cut surface shall consist of moisture conditioning to the optimum moisture content or above. The upper 12 inches should then be compacted to a minimum of 90 percent of the maximum dry density at or above the optimum moisture content as determined in accordance with ASTM D-698.

3 The results of this investigation indicate that most of the native soils will be suitable for use as structural fill; however due to variation in soil type throughout the site, blending and moisture conditioning will be required to meet the specifications below. Should imported fill be required, it should also meet the specifications for structural fill.

4 All embankment fill should be free of vegetation and debris and contain no rocks larger than 3 inches. The gradation of the embankment fill material, as determined in accordance with ASTM D-422, should be as follows:

Size	Percent Passing
3 inch	100
No. 4	60 - 100
No. 200	25 - 65

5 The plasticity index of embankment fill should be between 5 and 15 when tested in accordance with ASTM D-4318.

6 All trench backfill should be free of vegetation and debris and contain no rocks larger than 3 inches. Gradation of the backfill material, as determined in accordance with ASTM D-422, should be as follows:

Size	Percent Passing
3 inch	100
No. 4	60 - 100
No. 200	5 - 45

7 Embankment fill and trench backfill, consisting of soil approved by the geotechnical engineer, should be placed in controlled compacted layers not exceeding 8 inches (compacted) with approved compaction equipment. All structural fill material should be blended as necessary to produce a homogeneous embankment. No lifts of high permeability material or material differing substantially from the lift below should be permitted. Sheepsfoot or vibratory sheepsfoot or segmented steel wheel type compactors should be used. If the compactors “walk out” during compaction, or if it is desired to use flat wheel compactors, the upper 1 to 2 inches of the lift should be scarified prior to placing a subsequent lift. The embankment should be raised uniformly. All compaction should be accomplished to a minimum of 95

percent of maximum dry density as determined in accordance with ASTM D-1557. The moisture content of the structural fill during compaction should be at or 3 percent above the optimum moisture content. With any vibratory compactor, vibrations should be controlled or eliminated to avoid damage to adjacent structures or infrastructure.

8 Tests for degree of compaction should be determined in accordance with ASTM D-1556 or ASTM D-6938. Continuous, full time observation and field tests should be conducted during fill and backfill placement by a representative of the geotechnical engineer to assist the contractor in evaluating the required degree of compaction. If less than the required compaction is required, additional compaction effort should be made with adjustment of the moisture content as necessary until 95 percent compaction is obtained.

EXCAVATIONS

The results of this investigation indicate that the surficial soils can be readily excavated using normal earth moving and excavation equipment. Temporary construction excavations should be maintained at slopes of 1.5:1 (H:V) or flatter. Surcharge loads including construction traffic and excavated spoil materials should be maintained at least 10 feet from the crest of any excavation slope. Surface water should be routed such that it does not flow down the face of the excavation slopes. Where insufficient space exists for open cut excavations, a shoring system will be required. All excavations should comply with all applicable safety regulations.

Experience dictates shrinkage factors greater than calculated values. Stripping, subgrade preparation, hauling and wind losses, and ground compaction, both in the borrow (reservoir) areas and within the embankment foundation areas are all factors in shrinkage. We recommend using a shrinkage factor on the order of 25 percent.

REVIEW AND INSPECTION

This report has been prepared to aid in the evaluation of this site and to assist in the design of this project. It is recommended that the geotechnical engineer be provided the opportunity to review the final design drawings and specifications in order to determine whether the recommendations in this report are applicable to the final design. Review of the final design drawings and specifications should be noted in writing by the geotechnical engineer.

In order to permit correlation between the conditions encountered during construction and to confirm recommendations presented herein, it is recommended that the geotechnical engineer be retained to perform continuous observations and testing during the earthwork portion of this project. Observation and testing should be performed during construction to confirm that suitable fill and embankment soils are placed upon competent materials.

CLOSURE

Our conclusions, recommendations and opinions presented herein are:

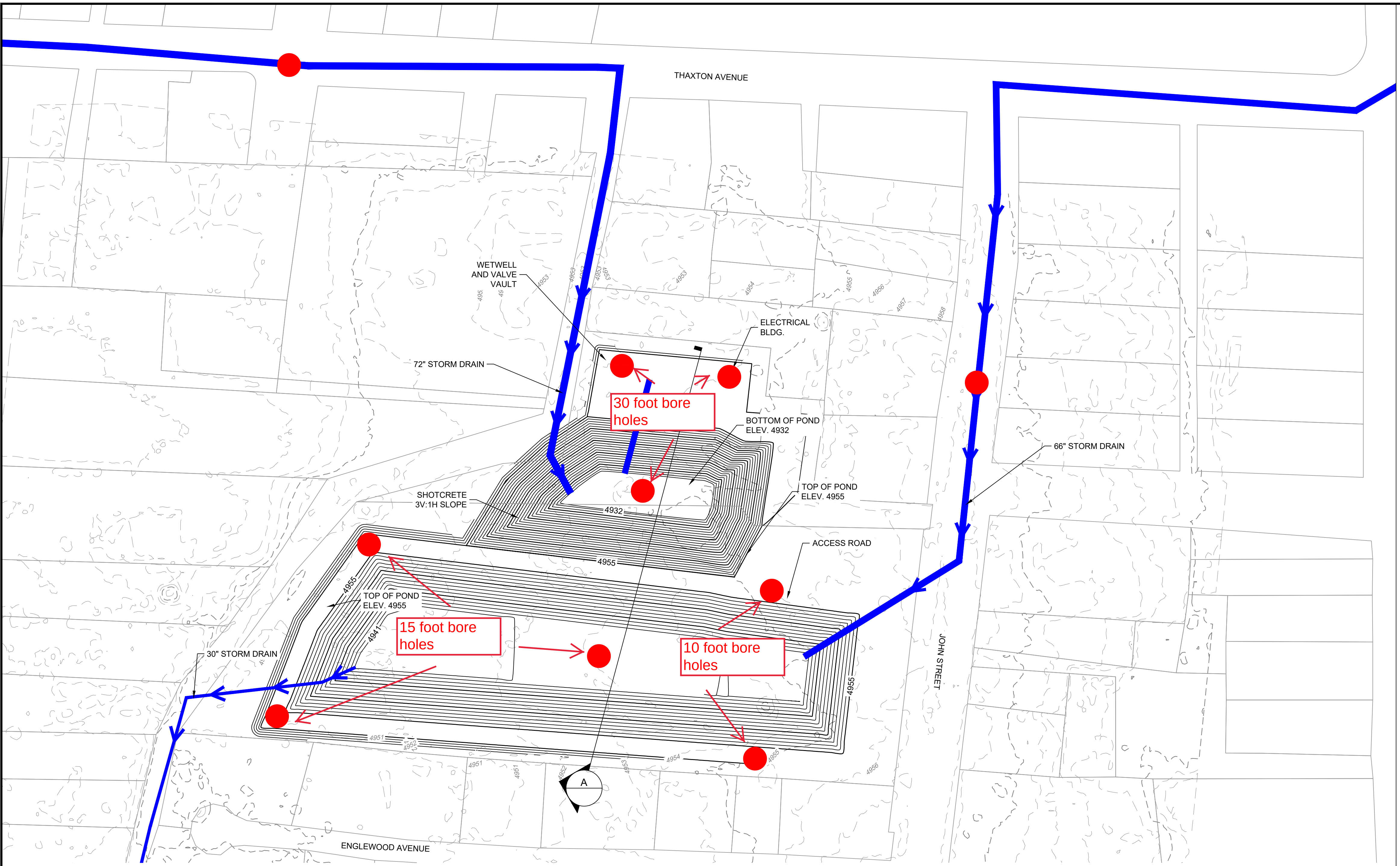
1. Based upon our evaluation and interpretation of the findings of the field and laboratory program.
2. Based upon an interpolation of soil conditions between and beyond the explorations.
3. Subject to confirmation of the conditions encountered during construction.
4. Based upon the assumption that sufficient observation will be provided during construction.
5. Prepared in accordance with generally accepted professional geotechnical engineering principles and practice.

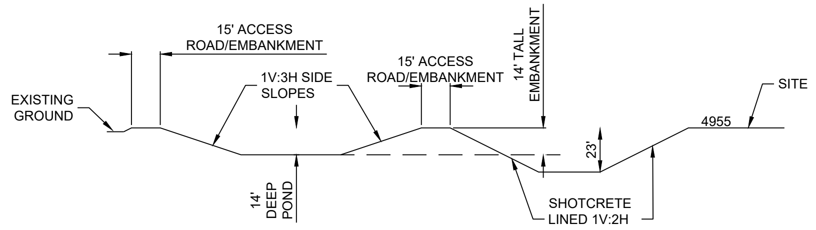
This report has been prepared for the sole use of RESPEC, specifically to aid in the design of the proposed John Street Storm Drain Engineering project in Albuquerque, New Mexico, and not for use by any third parties without consent.

We make no other warranty, either expressed or implied. Any person using this report for bidding or construction purposes should perform such independent investigation as they deem necessary to satisfy themselves as to the surface and subsurface conditions to be encountered and the procedures to be used in the performance of work on this project. If conditions encountered during construction appear to be different than indicated by this report, this office should be notified.

All soil samples will be discarded 60 days after the date of this report unless we receive a specific request to retain the samples for a longer period of time.

NAME: N:\Projects\04270 COA Transportation and Drainage On Call\Task 4-South Broadway DMP Update\3. DWG\Exhibits\04270.0004 Borehole Locations-Option 4.dwg PLOT DATE: Feb 07, 2023 9:36am



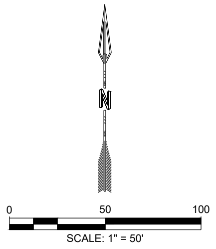


SECTION A
Scale: NTS

Geo-Test
1-30401
Figure 1

LEGEND

STORM DRAIN	—
BOREHOLE LOCATIONS	●



JOHN STREET POND
OPTION 4 BOREHOLE LOCATIONS
EXHIBIT 1

February 2023

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7770 JEFFERSON STREET NE, SUITE 200
ALBUQUERQUE, NEW MEXICO 87109
WWW.RESPEC.COM PHONE: (505)253-9718

BORING LOCATION MAP

John Street Ponds
Albuquerque, New Mexico
Job No. 1-30401

Figure 2

GEO-TEST
GEOTECHNICAL ENGINEERING
AND MATERIAL TESTING

GEO-TEST

Project: John Street Ponds
Date: 06/07/2023
Project No: 1-30401
Elevation:
Type: 3.25" ID HSA

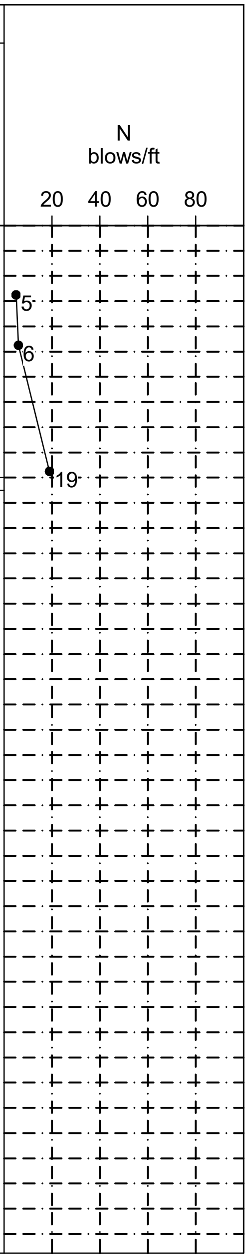
LOG OF TEST BORINGS

NO: 1

GROUNDWATER DEPTH

During Drilling: none
After 24 Hours:

DEPTH (Ft)	LOG	SAMPLE: SAMPLE INTERVAL	SAMPLE: TYPE	SAMPLE: N. BLOWS/FT	SAMPLE: MOISTURE %	SAMPLE: DRY DENSITY (pcf)	SAMPLE: USC	SUBSURFACE PROFILE: DESCRIPTION
			SS	2-2-3 5	4			
5			SS	4-3-3 6	2		SM	SILTY SAND, non-plastic, loose, dry, brown
10			SS	9-10-9 19	8		CL	CLAY with SAND, medium plasticity, firm, dry, dark brown
								Stopped Auger @ 9 feet Stopped Sampler @ 10.5 feet
15								
20								
25								
30								
35								
40								



LEGEND

SS - Split Spoon
AC - Auger Cuttings
UD/SL - Undisturbed Sleeve

AMSL - Above Mean Sea Level
CS - Continuous Sampler
UD - Undisturbed
ST - Shelby Tube

Stratification lines represent approximate boundaries between soil types. Transitions may be gradual. Water level readings have been made at times and under conditions stated. Fluctuations of groundwater may occur due to factors other than those present at the time measurments were made.

GEO-TEST

Project: John Street Ponds
Date: 06/07/2023
Project No: 1-30401
Elevation:
Type: 3.25" ID HSA

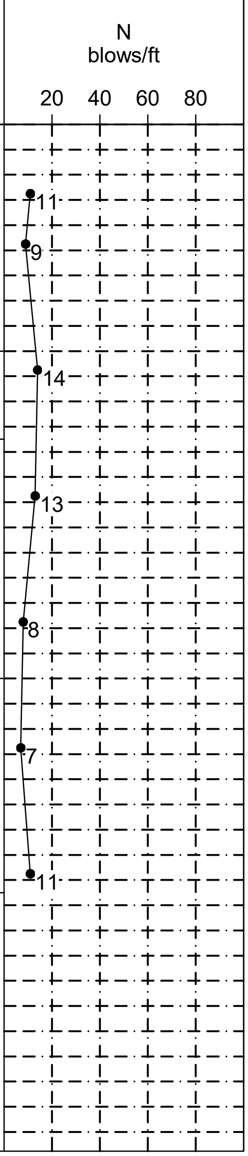
LOG OF TEST BORINGS

NO: 2

GROUNDWATER DEPTH

During Drilling: 29.0
After 24 Hours:

DEPTH (Ft)	LOG	SAMPLE						SUBSURFACE PROFILE
		SAMPLE INTERVAL	TYPE	N. BLOWS/FT	MOISTURE %	DRY DENSITY (pcf)	USC	DESCRIPTION
		☒	SS	4-5-6 11	4		SM	SILTY SAND, non-plastic, medium dense to loose, dry, brown
5		☒	SS	5-5-4 9	3			
10		☒	SS	5-6-8 14	12		CL	CLAY with SAND, medium plasticity, moderately firm, moist, dark brown
15		☒	SS	3-6-7 13	4		SM	SILTY SAND, non-plastic, medium dense to loose, dry, light brown
20		☒	SS	3-4-4 8	10			
25		☒	SS	2-4-3 7	29		CL	CLAY, medium plasticity, soft to stiff, very wet to saturated, dark brown
30		☒	SS	4-5-6 11	47			∇
								Stopped Auger @ 29 feet Stopped Sampler @ 30.5 feet
35								
40								



LEGEND

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GEO-TEST

Project: John Street Ponds
Date: 06/07/2023
Elevation:
Project No: 1-30401
Type: 3.25" ID HSA

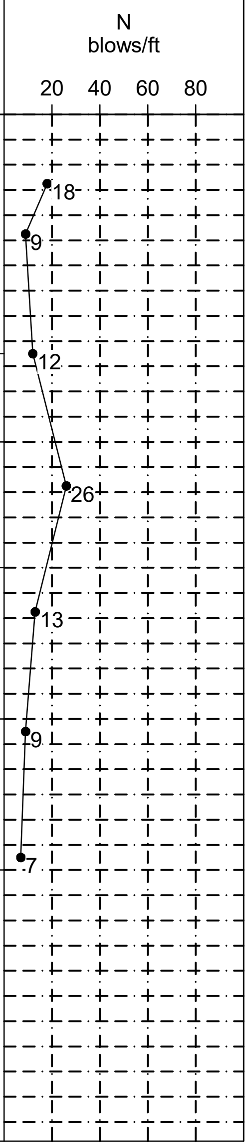
LOG OF TEST BORINGS

NO: 3

GROUNDWATER DEPTH

During Drilling: 29.0
After 24 Hours:

		SAMPLE					SUBSURFACE PROFILE	
DEPTH (Ft)	LOG	SAMPLE INTERVAL	TYPE	N. BLOWS/FT	MOISTURE %	DRY DENSITY (pcf)	USC	DESCRIPTION
		☒	SS	5-8-10 18	2			
5		☒	SS	5-5-4 9	2		SM	SILTY SAND, non-plastic, medium dense to loose, dry, light brown
10		■	UD	4-8 12	20	91	CL	CLAY with SAND, medium plasticity, firm, very moist, dark brown
15		☒	SS	9-12-14 26	7		SM	SILTY SAND, non-plastic, medium dense, slightly moist, light brown
20		☒	SS	4-6-7 13	14		CL	CLAY with SAND, medium plasticity, moderately firm, moist, dark brown
25		■	UD	2-7 9	24	104	CH	CLAY, high plasticity, moderately firm to medium stiff, very moist to saturated, dark brown
30		■	UD	3-4 7	32	90	∇	Stopped Auger @ 29 feet Stopped Sampler @ 30 feet
35								
40								



LEGEND

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GEO-TEST

Project: John Street Ponds
Date: 06/07/2023
Elevation:
Project No: 1-30401
Type: 3.25" ID HSA

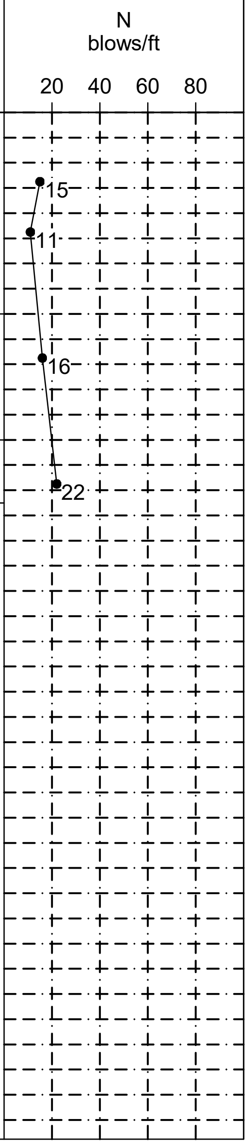
LOG OF TEST BORINGS

NO: 4

GROUNDWATER DEPTH

During Drilling: none
After 24 Hours:

		SAMPLE						SUBSURFACE PROFILE
DEPTH (Ft)	LOG	SAMPLE INTERVAL	TYPE	N. BLOWS/FT	MOISTURE %	DRY DENSITY (pcf)	USC	DESCRIPTION
	⁙	☒	SS	4-7-8 15	2		SM	SILTY SAND, non-plastic, medium dense, dry, light brown
5	⁙	☒	SS	7-6-5 11	3			
10	▨	☒	SS	6-7-9 16	17		CL	CLAY with SAND, medium plasticity, firm, moist, dark brown
15	⁙	☒	SS	9-11-11 22	2		SM	SILTY SAND, non-plastic, medium dense, dry, light brown
								Stopped Auger @ 14 feet Stopped Sampler @ 15.5 feet
20								
25								
30								
35								
40								



LEGEND

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GEO-TEST

Project: John Street Ponds
Date: 06/08/2023
Project No: 1-30401
Elevation:
Type: 3.25" ID HSA

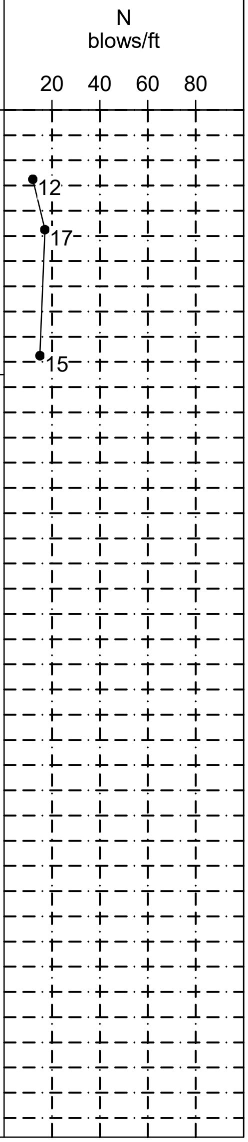
LOG OF TEST BORINGS

NO: 5

GROUNDWATER DEPTH

During Drilling: none
After 24 Hours:

DEPTH (Ft)	LOG	SAMPLE INTERVAL	TYPE	N. BLOWS/FT	MOISTURE %	DRY DENSITY (pcf)	USC	DESCRIPTION
		SAMPLE					SUBSURFACE PROFILE	
	▨	☒	SS	6-5-7 12	3			
5	▨	☒	SS	6-8-9 17	2		SM	SILTY SAND, non-plastic, medium dense, dry, light brown
10	▨	☒	SS	4-8-7 15	1			
								Stopped Auger @ 9 feet Stopped Sampler @ 10.5 feet
15								
20								
25								
30								
35								
40								



LEGEND

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GEO-TEST

Project: John Street Ponds
Date: 06/08/2023
Elevation:
Project No: 1-30401
Type: 3.25" ID HSA

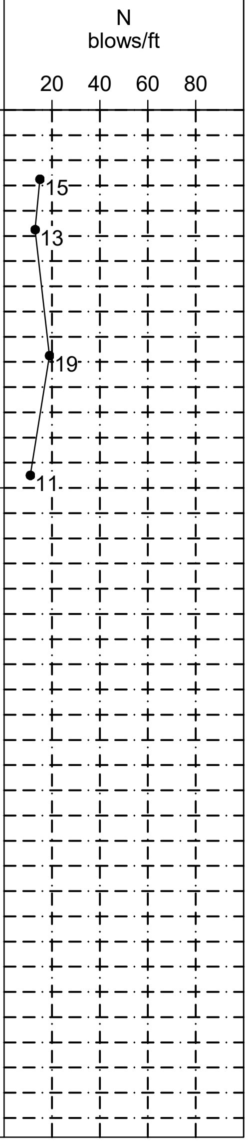
LOG OF TEST BORINGS

NO: 6

GROUNDWATER DEPTH

During Drilling: none
After 24 Hours:

DEPTH (Ft)	LOG	SAMPLE SAMPLE INTERVAL	 TYPE	 N. BLOWS/FT	 MOISTURE %	 DRY DENSITY (pcf)	SUBSURFACE PROFILE USC	 DESCRIPTION
	▨	☒	SS	3-7-8 15	3		SM	SILTY SAND, non-plastic, medium dense, dry to moist, brown
5	▨	☒	SS	7-7-6 13	3			
10	▨	☒	SS	7-9-10 19	9			
15	▨	■	UD	4-7 11	3			
20 25 30 35 40								Stopped Auger @ 14 feet Stopped Sampler @ 15 feet



LEGEND

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GEO-TEST

Project: John Street Ponds
Date: 06/08/2023
Elevation:
Project No: 1-30401
Type: 3.25" ID HSA

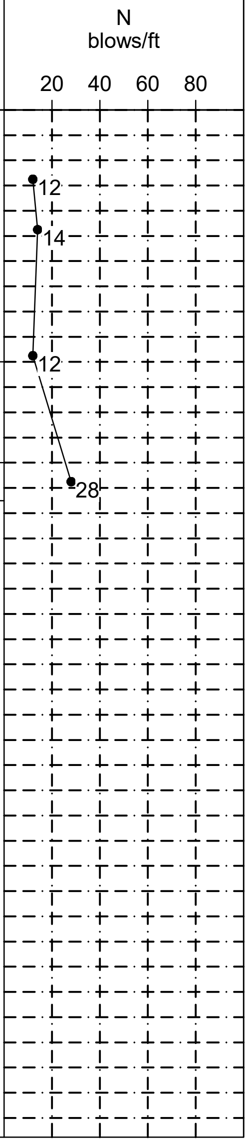
LOG OF TEST BORINGS

NO: 7

GROUNDWATER DEPTH

During Drilling: none
After 24 Hours:

DEPTH (Ft)	LOG	SAMPLE INTERVAL	TYPE	N. BLOWS/FT	MOISTURE %	DRY DENSITY (pcf)	USC	DESCRIPTION
		☒	SS	3-5-7 12	2			
5		☒	SS	3-6-8 14	1		SM	SILTY SAND, non-plastic, medium dense, dry, light brown
10		☒	SS	4-5-7 12	20			
							CL	CLAY, medium plasticity, moderately firm, moist, dark brown
15		☒	SS	7-13-15 28	2		SM	SILTY SAND, non-plastic, medium dense, dry, light brown Stopped Auger @ 14 feet Stopped Sampler @ 15.5 feet
20								
25								
30								
35								
40								



LEGEND

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GEO-TEST

Project: John Street Ponds
Date: 06/08/2023
Elevation:
Project No: 1-30401
Type: 3.25" ID HSA

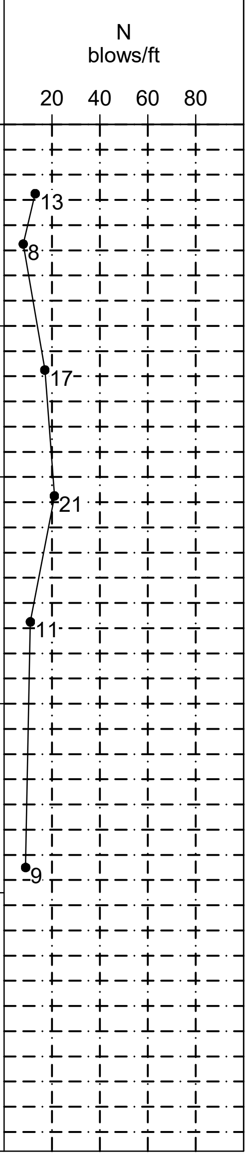
LOG OF TEST BORINGS

NO: 8

GROUNDWATER DEPTH

During Drilling: 27.0
After 24 Hours:

DEPTH (Ft)	SAMPLE TYPE	N. BLOWS/FT	MOISTURE %	DRY DENSITY (pcf)	USC	DESCRIPTION
	SS	3-5-8 13	3		SM	SILTY SAND, non-plastic, medium dense to loose, dry, light brown
5	SS	3-3-5 8	5			
10	SS	5-7-10 17	12		CL	CLAY with SAND, medium plasticty, firm, slightl moist, dark brown
15	SS	5-8-13 21	4		SM	SILTY SAND, non-plastic, medium dense, dry, light brown
20	SS	5-6-5 11	5			
25	SS	50/6" 50/6"	21	105	CL	∇ CLAY with SAND, medium plasticity, hard to stiff, very wet to saturated, brown
30	UD	2-7 9	40	82		
						Stopped Auger @ 29 feet Stopped Sampler @ 30.5 feet



LEGEND

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UD/SL - Undisturbed Sleeve
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SUMMARY OF LABORATORY RESULTS

						SIEVE ANALYSIS PERCENT PASSING											
TEST HOLE	DEPTH (FEET)	UNIFIED CLASS	(%) MOIST	LL	PI	NO 200	NO 100	NO 40	NO 10	NO 4	3/8"	1/2"	3/4"	1"	1 1/2"	2"	4"
1	3.0		3.5														
1	5.0	SM	2.4	NP	NP	29	75	94	98	99	100						
1	10.0		7.6														
2	3.0	SM	3.6	NP	NP	30	66	88	98	99	100						
2	5.0		3.3														
2	10.0	CL	11.8	31	16	74	95	98	99	99	100						
2	15.0	SM	4.0	NP	NP	19	68	93	99	99	100						
2	20.0		9.9														
2	25.0	CL	28.5	40	22	97	99	99	100								
2	30.0		47.5														
3	3.0		2.2														
3	5.0	SM	2.3	NP	NP	24	70	94	99	99	100						
3	10.0	CL	19.9	43	24	82	94	97	99	99	100						
3	15.0		7.2														
3	20.0		13.8														
3	25.0	CH	24.5	51	34	99	99	99	100								
3	30.0		32.2														
4	3.0	SM	2.3	NP	NP	17	59	90	100								
4	5.0		3.2														

GEO-TEST

LL = LIQUID LIMIT
PI = PLASTICITY INDEX
NP = NON PLASTIC or NO VALUE

Project: John Street Ponds
Location: Albuquerque, NM
Number: 1-30401

SUMMARY OF LABORATORY RESULTS

						SIEVE ANALYSIS PERCENT PASSING											
TEST HOLE	DEPTH (FEET)	UNIFIED CLASS	(%) MOIST	LL	PI	NO 200	NO 100	NO 40	NO 10	NO 4	3/8"	1/2"	3/4"	1"	1 1/2"	2"	4"
4	10.0	CL	17.4	44	28	78	92	95	96	97	97	97	100				
4	15.0		2.4														
5	3.0		3.0														
5	5.0	SM	2.2	NP	NP	28	72	90	96	97	97	100					
5	10.0		1.5														
6	3.0	SM	2.7	NP	NP	20	61	90	99	99	99	100					
6	5.0		3.5														
6	10.0		8.7														
6	15.0	SM	3.2	NP	NP	23	75	94	100								
7	3.0		2.3														
7	5.0		1.5														
7	10.0	CL	19.5	46	26	91	97	98	98	99	100						
7	15.0		2.2														
8	3.0	SM	3.0	NP	NP	24	61	88	100								
8	5.0		5.2														
8	10.0	CL	11.7	31	16	70	95	99	100								
8	15.0		3.7														
8	20.0		5.1														
8	25.0	CL	20.8	41	25	85	94	98	100								

GEO-TEST

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Project: John Street Ponds
Location: Albuquerque, NM
Number: 1-30401

SUMMARY OF LABORATORY RESULTS

						SIEVE ANALYSIS PERCENT PASSING											
TEST HOLE	DEPTH (FEET)	UNIFIED CLASS	(%) MOIST	LL	PI	NO 200	NO 100	NO 40	NO 10	NO 4	3/8"	1/2"	3/4"	1"	1 1/2"	2"	4"
8	30.0		39.9														
Bulk	5.0	SM		NP	NP	14	48	82	98	99	100						

GEO-TEST

LL = LIQUID LIMIT
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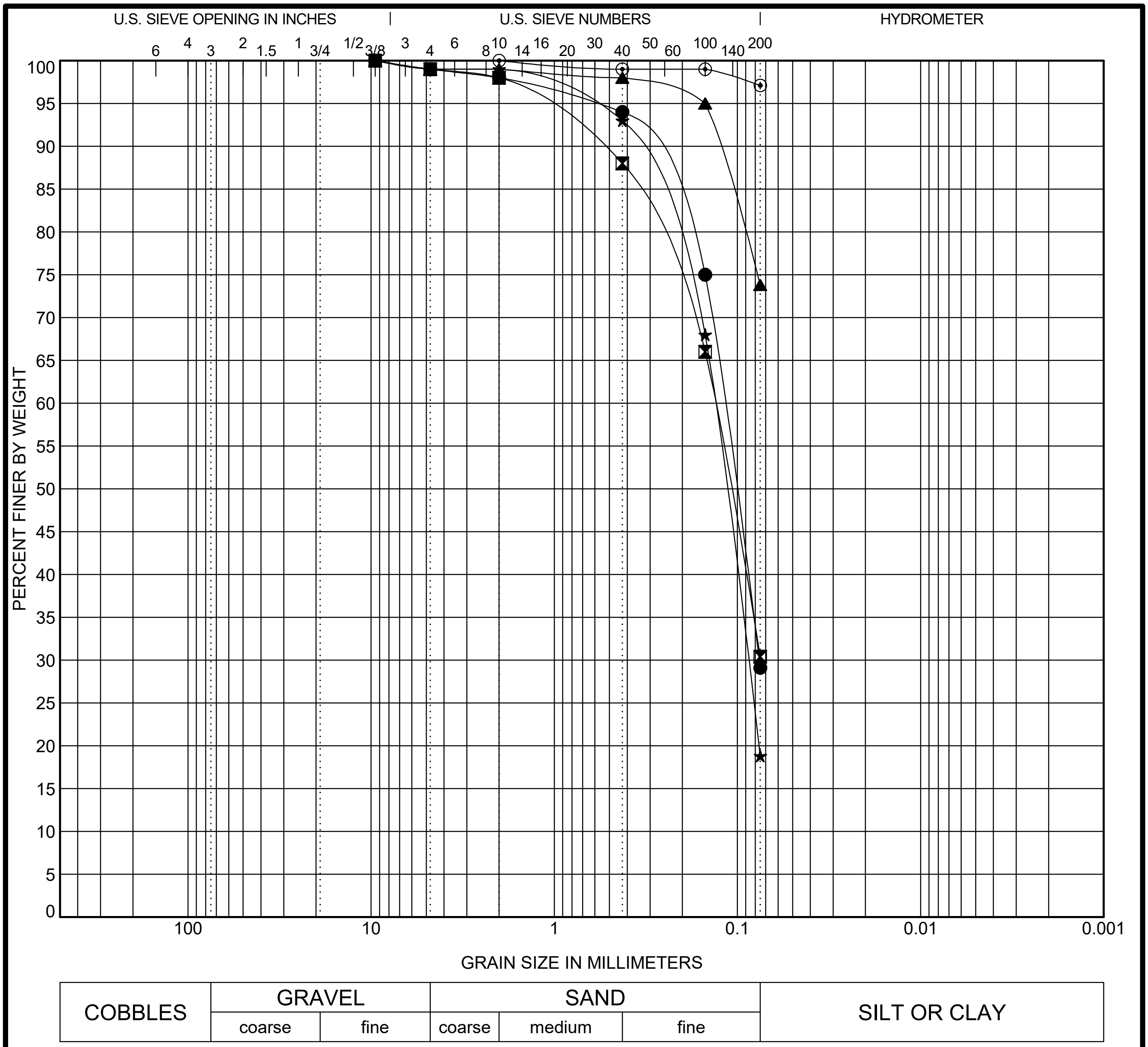
Project: John Street Ponds

Location: Albuquerque, NM

Number: 1-30401



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	1	5.0	**SILTY SAND(SM)**	**NP**	**NP**	**NP**		
☒	2	3.0	**SILTY SAND(SM)**	**NP**	**NP**	**NP**		
▲	2	10.0	**LEAN CLAY with SAND(CL)**	**31**	**15**	**16**		
★	2	15.0	**SILTY SAND(SM)**	**NP**	**NP**	**NP**		
⊙	2	25.0	**LEAN CLAY(CL)**	**40**	**18**	**22**		

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	1	5.0	9.5	0.12	0.076		1.0	69.9	29.1	
☒	2	3.0	9.5	0.133			1.0	68.6	30.4	
▲	2	10.0	9.5				1.0	25.2	73.8	
★	2	15.0	9.5	0.134	0.088		1.0	80.2	18.8	
⊙	2	25.0	2				0.0	2.9	97.1	

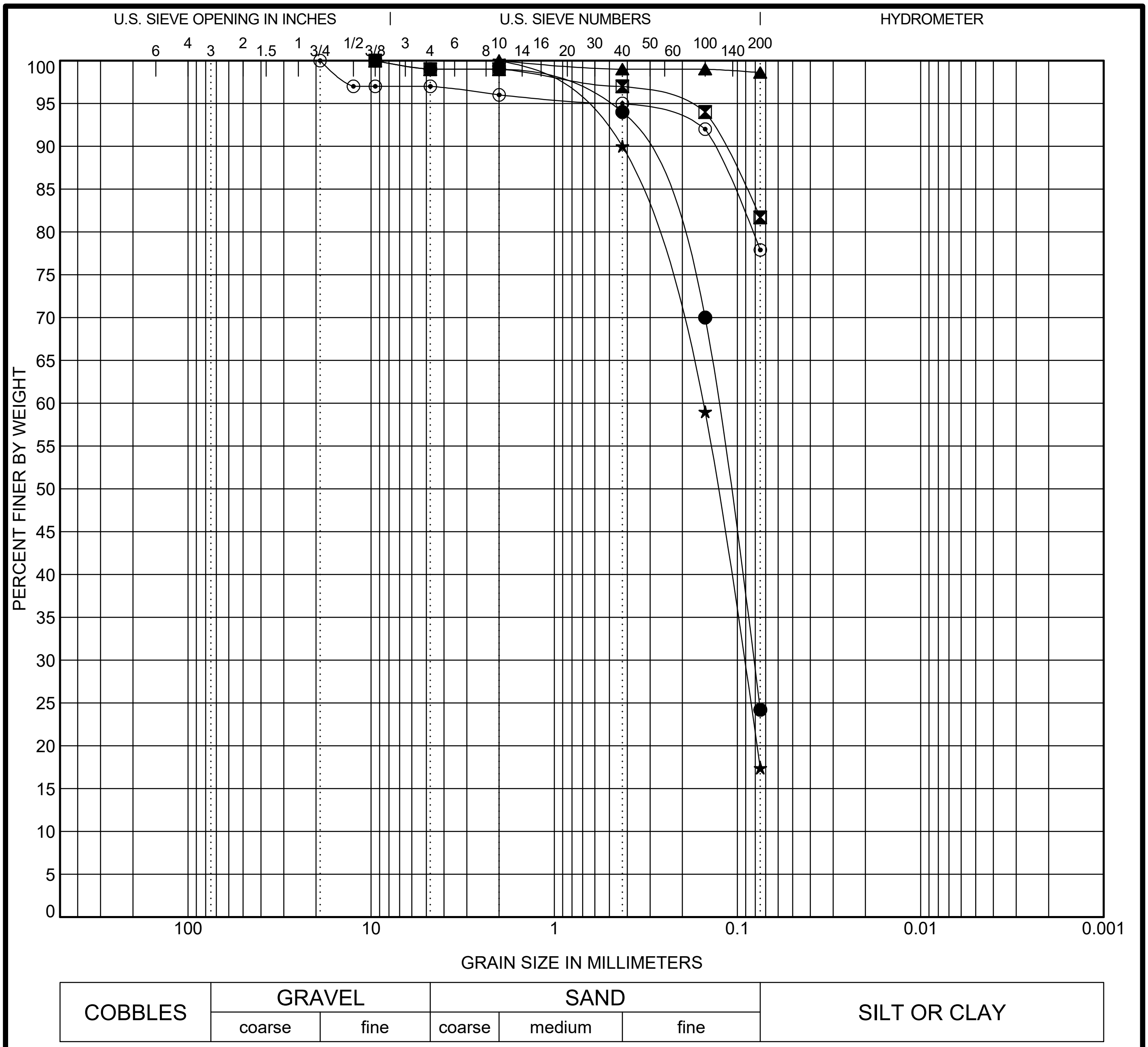
GEO-TEST

GRAIN SIZE DISTRIBUTION

Project: John Street Ponds

Location: Albuquerque, NM

Number: 1-30401



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	3	5.0	**SILTY SAND(SM)**	**NP**	**NP**	**NP**		
⧗	3	10.0	**LEAN CLAY with SAND(CL)**	**43**	**19**	**24**		
▲	3	25.0	**FAT CLAY(CH)**	**51**	**17**	**34**		
★	4	3.0	**SILTY SAND(SM)**	**NP**	**NP**	**NP**		
⊙	4	10.0	**LEAN CLAY with SAND(CL)**	**44**	**16**	**28**		

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	3	5.0	**9.5**	**0.129**	**0.082**		**1.0**	**74.8**	**24.2**	
⧗	3	10.0	**9.5**				**1.0**	**17.3**	**81.7**	
▲	3	25.0	**2**				**0.0**	**1.4**	**98.6**	
★	4	3.0	**2**	**0.155**	**0.093**		**0.0**	**82.6**	**17.4**	
⊙	4	10.0	**19**				**3.0**	**19.1**	**77.9**	

GEO-TEST

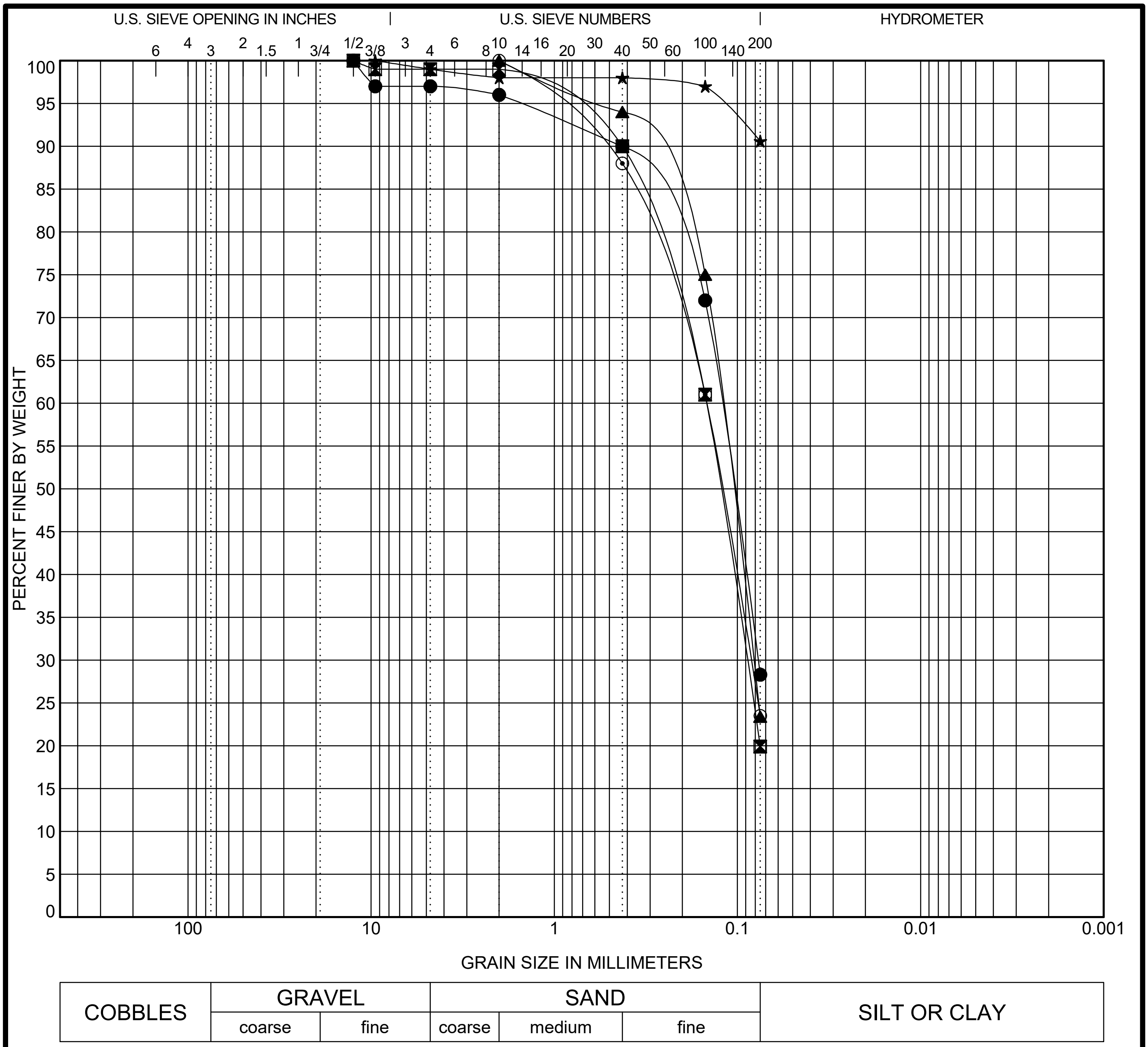
GRAIN SIZE DISTRIBUTION

Project: John Street Ponds

Location: Albuquerque, NM

Number: 1-30401

US GRAIN SIZE 1-30401.GPJ GEO TEST.GDT 7/6/23



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	5	5.0	**SILTY SAND(SM)**	**NP**	**NP**	**NP**		
⧗	6	3.0	**SILTY SAND(SM)**	**NP**	**NP**	**NP**		
▲	6	15.0	**SILTY SAND(SM)**	**NP**	**NP**	**NP**		
★	7	10.0	**LEAN CLAY(CL)**	**46**	**20**	**26**		
⊙	8	3.0	**SILTY SAND(SM)**	**NP**	**NP**	**NP**		

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	5	5.0	12.5	0.124	0.077		3.0	68.7	28.3	
⧗	6	3.0	12.5	0.147	0.089		1.0	79.1	19.9	
▲	6	15.0	2	0.123	0.082		0.0	76.6	23.4	
★	7	10.0	9.5				1.0	8.4	90.6	
⊙	8	3.0	2	0.147	0.085		0.0	76.5	23.5	

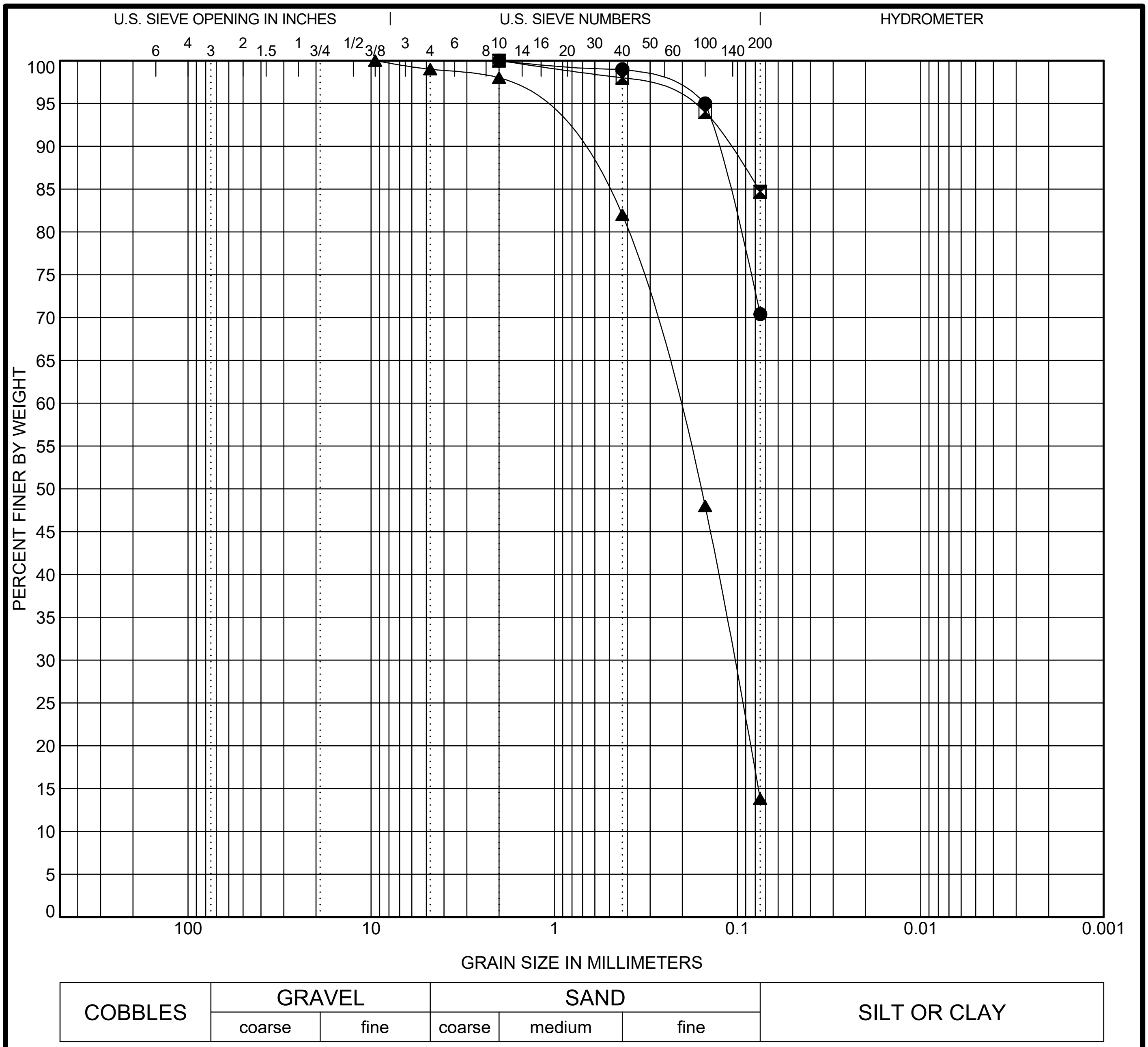
GEO-TEST

GRAIN SIZE DISTRIBUTION

Project: John Street Ponds

Location: Albuquerque, NM

Number: 1-30401



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**8**	**10.0**	**LEAN CLAY with SAND(CL)**	**31**	**15**	**16**		
⧗	**8**	**25.0**	**LEAN CLAY with SAND(CL)**	**41**	**16**	**25**		
▲	**Bulk**	**5.0**	**SILTY SAND(SM)**	**NP**	**NP**	**NP**		

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**8**	**10.0**	**2**				**0.0**	**29.6**	**70.4**	
⧗	**8**	**25.0**	**2**				**0.0**	**15.3**	**84.7**	
▲	**Bulk**	**5.0**	**9.5**	**0.217**	**0.104**		**1.0**	**85.2**	**13.8**	

GEO-TEST

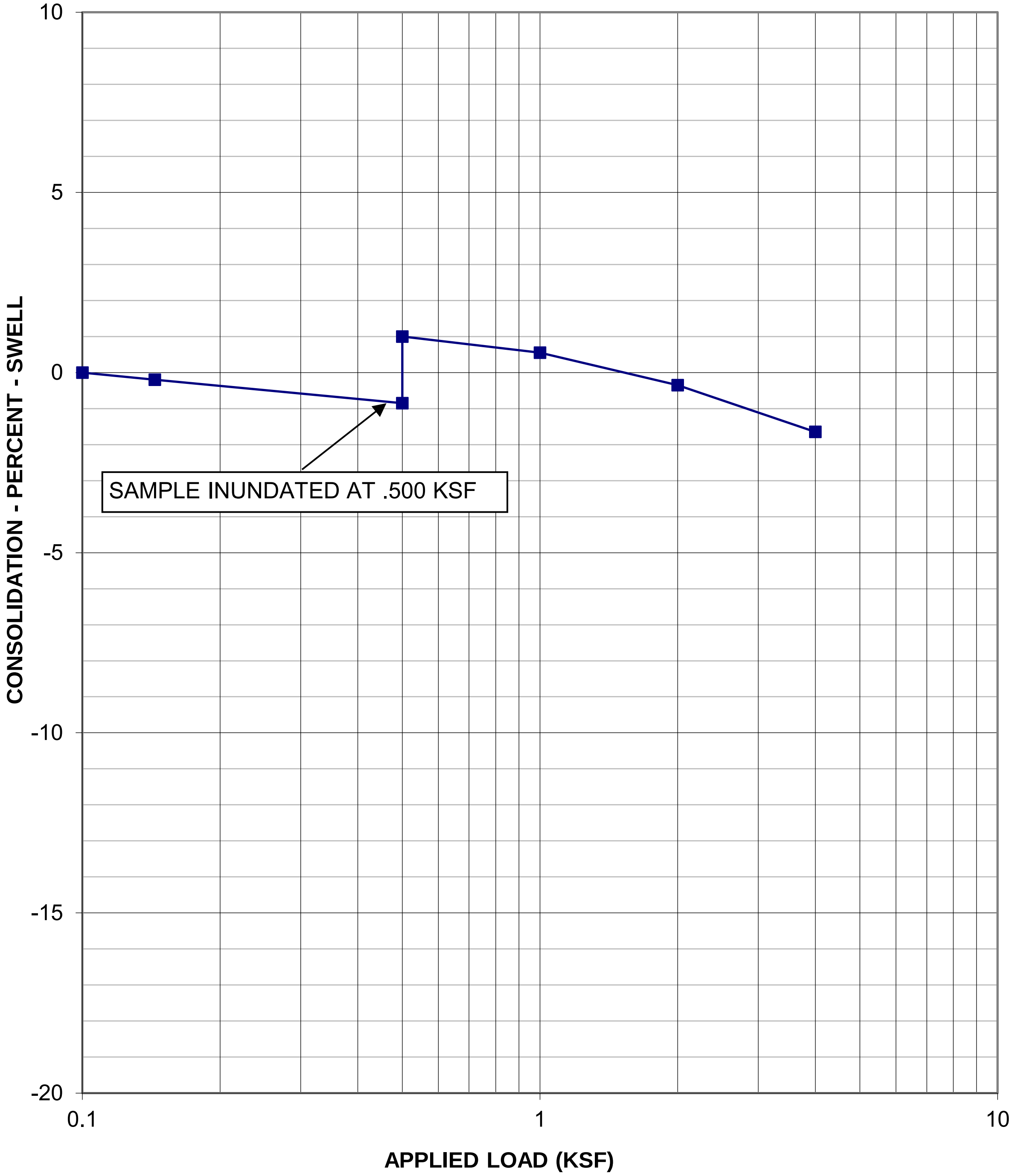
GRAIN SIZE DISTRIBUTION

Project: John Street Ponds

Location: Albuquerque, NM

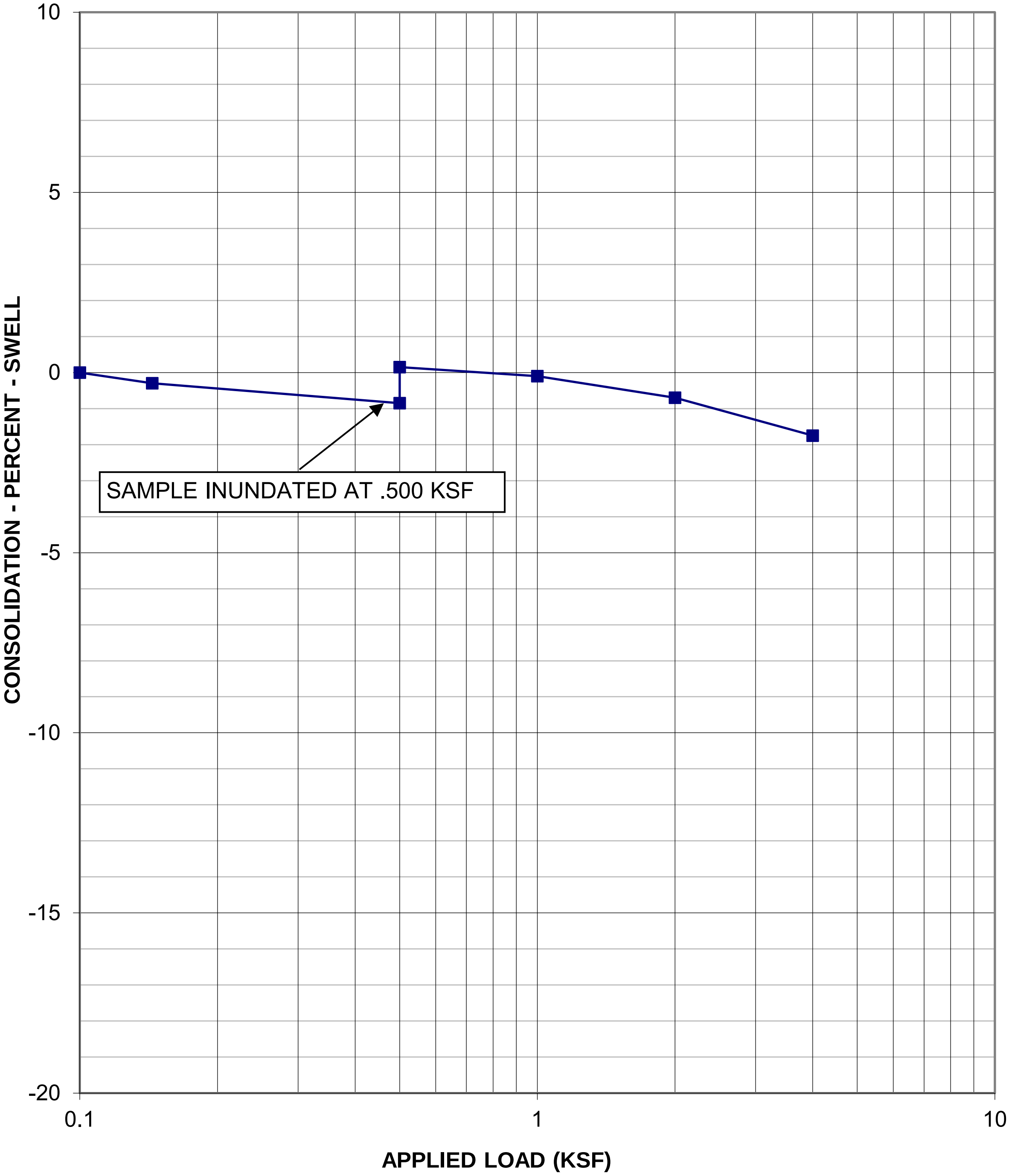
Number: 1-30401

CONSOLIDATION TEST RESULT
John Street Ponds
JOB NO. 1-30401
Boring #3 @ 25 ft



INITIAL MOISTURE CONTENT =	22.3 %
INITIAL DRY DENSITY =	103.9 PCF

CONSOLIDATION TEST RESULT
John Street Ponds
JOB NO. 1-30401
Boring #8 @ 25 ft



INITIAL MOISTURE CONTENT = 22.0 %
INITIAL DRY DENSITY = 104.6 PCF

Rigid Wall Constant Head Permeability

Project: John Street Ponds
Job #: 1-30401
Boring/Location: Boring 3
Sample Depth: 30 feet
Soil Description: High Plasticity Clay (CL)
Remolded to: N/A In-Situ Tube Sample

Aparatus Weight Empty:	211.9	grams	Weight of Sample:	687.3	grams
Aparartus Weight + Soil:	899.2	grams	Weight of Sample:	1.515212	lb
Mold Diameter:	6.182	cm	Mold Area:	30.01566	cm^2
Pipe Diameter:	1.27	cm	Pipe Area:	1.266769	cm^2
Length of Sample	11.984	cm	Area Factor:	0.042204	
Pressure Head Applied 1psi = 70.34 cm:	1406.8	cm	Volume of Sample:	359.7077	cm^3
Can #:			Volume of Sample:	0.012703	ft^3
Wet Weight:	174.7	grams	**Unit Weight:**	**119.3**	**lb/ft^3**
Dry Weight:	132.1	grams	**Moisture Content:**	**32.2**	**%**
			Dry Unit Weight:	**90.2**	**lb/ft^3**

Time	Trial 1		Trial 2		Trial 3	
Hour	1		1		1	
Minute	7		7		7	
Second	55		22		37	
Total (hr)	1.131944		1.122778		1.126944	
h_0	65	cm	65	cm	65	cm
h_1	10	cm	10	cm	10	cm
$Head_0$	1483.784	cm	1483.784	cm	1483.784	cm
$Head_1$	1428.784	cm	1428.784	cm	1428.784	cm
Ks (cm/hour)	0.02	cm/hr	0.02	cm/hr	0.02	cm/hr
Ks (cm/sec)	4.69E-06	cm/s	4.73E-06	cm/s	4.71E-06	cm/s

Saturated Hydraulic Conductivity, $K_{s:}$ **0.02 cm/hr**

Saturated Hydraulic Conductivity, K_s: **4.71E-06 cm/s**

Rigid Wall Constant Head Permeability

GEO-TEST

Project: John Street Ponds
Job #: 1-30401
Boring/Location: Boring 6
Sample Depth: 15 feet
Soil Description: Silty Sand (SM)
Remolded to: N/A In-Situ Tube Sample

Aparatus Weight Empty:	213	grams	Weight of Sample:	609.3	grams
Apparartus Weight + Soil:	822.3	grams	Weight of Sample:	1.343254	lb
Mold Diameter:	6.195	cm	Mold Area:	30.14203	cm^2
Pipe Diameter:	1.27	cm	Pipe Area:	1.266769	cm^2
Length of Sample	11.654	cm	Area Factor:	0.042027	
Pressure Head Applied 1psi = 70.34 cm:	0	cm	Volume of Sample:	351.2752	cm^3
Can #:			Volume of Sample:	0.012405	ft^3
Wet Weight:	179.4	grams	**Unit Weight:**	**108.3**	**lb/ft^3**
Dry Weight:	176.4	grams	**Moisture Content:**	**1.7**	**%**
			Dry Unit Weight:	**106.5**	**lb/ft^3**

Time	Trial 1	Trial 2	Trial 3
Hour	0	0	0
Minute	1	1	1
Second	17	17	17
Total (hr)	0.021389	0.021389	0.021389
h_0	65 cm	65 cm	65 cm
h_1	10 cm	10 cm	10 cm
$Head_0$	76.654 cm	76.654 cm	76.654 cm
$Head_1$	21.654 cm	21.654 cm	21.654 cm
Ks (cm/hour)	28.95 cm/hr	28.95 cm/hr	28.95 cm/hr
Ks (cm/sec)	8.04E-03 cm/s	8.04E-03 cm/s	8.04E-03 cm/s

Saturated Hydraulic Conductivity, $K_{S:}$ **28.95 cm/hr**

Saturated Hydraulic Conductivity, K_s: **8.04E-03 cm/s**

Rigid Wall Constant Head Permeability

Project: John Street Ponds
Job #: 1-30401
Boring/Location: Boring 8
Sample Depth: 30 feet
Soil Description: Clay with Sand (CL)
Remolded to: N/A In-Situ Tube Sample

Aparatus Weight Empty:	211.7	grams	Weight of Sample:	623.6	grams
Aparartus Weight + Soil:	835.3	grams	Weight of Sample:	1.37478	lb
Mold Diameter:	6.187	cm	Mold Area:	30.06423	cm^2
Pipe Diameter:	1.27	cm	Pipe Area:	1.266769	cm^2
Length of Sample	11.356	cm	Area Factor:	0.042135	
Pressure Head Applied 1psi = 70.34 cm:	1406.8	cm	Volume of Sample:	341.4094	cm^3
Can #:			Volume of Sample:	0.012057	ft^3
Wet Weight:	175.1	grams	**Unit Weight:**	**114.0**	**lb/ft^3**
Dry Weight:	125.2	grams	**Moisture Content:**	**39.9**	**%**
			Dry Unit Weight:	**81.5**	**lb/ft^3**

Time	Trial 1		Trial 2		Trial 3	
Hour	0		0		0	
Minute	10		10		10	
Second	41		17		53	
Total (hr)	0.178056		0.171389		0.181389	
h_0	65	cm	65	cm	65	cm
h_1	10	cm	10	cm	10	cm
$Head_0$	1483.156	cm	1483.156	cm	1483.156	cm
$Head_1$	1428.156	cm	1428.156	cm	1428.156	cm
Ks (cm/hour)	0.10	cm/hr	0.11	cm/hr	0.10	cm/hr
Ks (cm/sec)	2.82E-05	cm/s	2.93E-05	cm/s	2.77E-05	cm/s

Saturated Hydraulic Conductivity, $K_{s:}$ **0.10 cm/hr**

Saturated Hydraulic Conductivity, K_s: **2.84E-05 cm/s**