ARROYO CROSSING - PHASE II INFRASTRUCTURE PROJECT

ADDENDUM #1 TO RFB

Clarification #1:
Q: Has there been a geotechnical report conducted on the intended site?

A: Yes. See 2019 geotechnical report on p. 2 of this document.

Clarification #2:
Q: Can the Project Manager provide CAD files of the site?

A: All Civil CAD base files may be accessed via this link.

Clarification #3:
Q: In terms of project timeline, does the Owner/Project Manager have an idea of when a Notice to Proceed will be issued to the selected Contractor? Is there a target substantial completion date?

A: Following the bid opening and award on October 27, 2025, the project owner intends to move forward with executing a contract with the Contractor and issuing a Notice to Proceed by mid to late November. Depending on contractor availability and/or winter weather conditions, the owner intends for substantial completion to be achieved by March or April 2026, or sooner if possible.

Clarification #4:
Q: Is there an engineer's estimate for this project?

A: A current Opinion of Probable Cost from the project engineer is approximately $1.5mil. This is inclusive of a 25% contingency cushion.

Clarification #5:
Q: Is there a required pre-bid meeting for this project?

A: No, there will only be a contract meeting after bids are opened between the Owner/Project managers and the selected contractor.

TRAUTNER GEOTECH LLC
GEOTECHNICAL ENGINEERING, MATERIAL TESTING
AND ENGINEERING GEOLOGY

FEASIBILITY LEVEL GEOTECHNICAL ENGINEERING STUDY/DESIGN LEVEL PAVEMENT RECOMMENDATIONS

ARROYO CROSSING PROJECT

Moab, Utah

April 26, 2019

Prepared For:
Ms. Audrey Graham
Moab Area Community Land Trust
Project Number: 55599GE

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1.0 REPORT INTRODUCTION

This report presents our feasibility level geotechnical engineering recommendations for the proposed Arroyo Crossing Development, with design level pavement section recommendations for the project. This report was requested by Ms. Audrey Graham, Moab Area Community Land Trust. The field study was completed on March 28 and 29, 2019. The laboratory study was completed on April 24, 2019.

This report provides feasibility level geotechnical engineering recommendations that includes comments regarding the site geology as it pertains to the geotechnical engineering evaluation and associated recommendations for this site. This report should not be misinterpreted as a comprehensive geological report or geologic hazard report. We are available to provide a comprehensive geologic/geologic hazard report at your request.

Geotechnical engineering is a discipline which provides insight into natural conditions and site characteristics such as; subsurface soil and water conditions, soil strength, swell (expansion) potential, consolidation (settlement) potential, and often slope stability considerations (when needed). The information provided by the geotechnical engineer is utilized by many people including the project owner, architect or designer, structural engineer, civil engineer, the project builder and others. Feasibility level information, such as that provided in this report, is typically used to help develop a pre-design plan as part of the conceptual development for larger scale projects.

This report does not provide design level foundation recommendations for the project, rather this report provides general discussion and general/cursory geotechnical engineering related parameters that may be used by the project design team to assist with the initial project design and development. This report does include design level asphalt pavement section design recommendations for the proposed project roadways. As the project plans progress, design level foundation studies for the various structures associated with the project should be performed.

It is common for unforeseen, or otherwise variable subsurface soil and water conditions to be encountered during construction. As discussed in our proposal for our services, it is imperative that we be contacted during the roadway subgrade excavation stage of this project to verify that the conditions encountered in our field exploration are representative of those encountered during construction. Compaction testing of the various roadway materials, including embankment fill, aggregate materials, and asphalt pavement are equally important tasks that should be performed by the geotechnical engineering consultant during construction. We should be contacted during the construction phase of the project, and/or if any questions or comments arise as a result of the information presented below.

The following outline provides a synopsis of the various portions of this report;

- Sections 1.0 and 2.0 provide an introduction and an establishment of our scope of service.
- Sections 3.0 and 4.0 of this report present our geotechnical engineering field and laboratory studies
- Sections 5.0 through 8.0 presents our feasibility level comments/recommendations for foundation systems that may be considered to support the proposed structures associated with the development.
- Section 9.0 provides our design level pavement section thickness analyses for the project roadways and parking areas.
- Section 10.0 provides a brief discussion of construction sequencing and strategies which may influence the geotechnical engineering characteristics of the site. The construction considerations section is not intended to address all of the construction planning and needs for the project site, but is intended to provide an overview to aid the owner, design team, and contractor in understanding some construction concepts that may influence some of the geotechnical engineering aspects of the site and proposed development.

The data used to generate our recommendations are presented throughout this report and in the attached figures.

1.1 *Scope of Project*

We understand that the proposed project will consist of designing and constructing an approximate 60-acre mixed use development. The development will include a mixture of single-family and multi-family residential structures, commercial structures, and community park/garden areas. A network of asphalt paved roadways and parking areas will be included with the development.

2.0 FEASIBILITY LEVEL GEOTECHNICAL ENGINIEERING STUDY

This section of this report presents the scope of services as outlined in our July 20, 2016 proposal for our feasibility level geotechnical engineering study and design level pavement recommendations.

2.1 *Geotechnical Engineering Study Scope of Service*

The scope of service and the associated order of presentation of the information within this report, is outlined below.

Field Study

- We advanced fourteen continuous flight auger test borings in a grid-like pattern across the approximate 60-acre development area.
- It is possible that the subsurface information obtained from some of our test borings may be used to supplement future design level studies for specific structures.
- Select driven sleeve and bulk soil samples were obtained from the test borings and returned to our laboratory for testing.

Laboratory Study

Our laboratory study consisted of a feasibility level study as discussed in our proposal. It is possible that the some of the laboratory information obtained may be used to supplement future design level studies for specific structures. The laboratory testing and analysis of the samples obtained included;

- Moisture content and dry density,
- Swell/consolidation tests to provide general information regarding the expansion and consolidation potential of the support soils on this site,
- Plastic and liquid limit tests to determine the Plasticity Index of the soil,
- Sieve analysis tests,
- Soluble sulfates tests to help generally assess the corrosion potential of the site soils on Portland cement concrete.
- Modified proctor tests to assess the maximum dry density and moisture content relationship of the site soils for use as subgrade support materials for asphalt paved roadways and parking areas, and,
- California Bearing Ratio (CBR) tests to assess the strength characteristics of the native site soils for support of flexible asphalt pavement or rigid concrete pavement.

Geotechnical Engineering Comments

- This report addresses geotechnical engineering aspects of the site which may influence future development and foundation planning including;
    - Subsurface soil and water conditions,
    - Comments on viable foundation systems for the conditions encountered,
    - Preliminary bearing capacity values for the foundation concepts that are viable for the project based on the subsurface conditions encountered.
- This report provides design level pavement section recommendations for the project roadways are parking areas.

- Our subsurface exploration, laboratory study and engineering analysis do not address environmental or geologic hazard issues other than general/cursory information regarding potential expansive soil conditions or collapsible soil conditions.

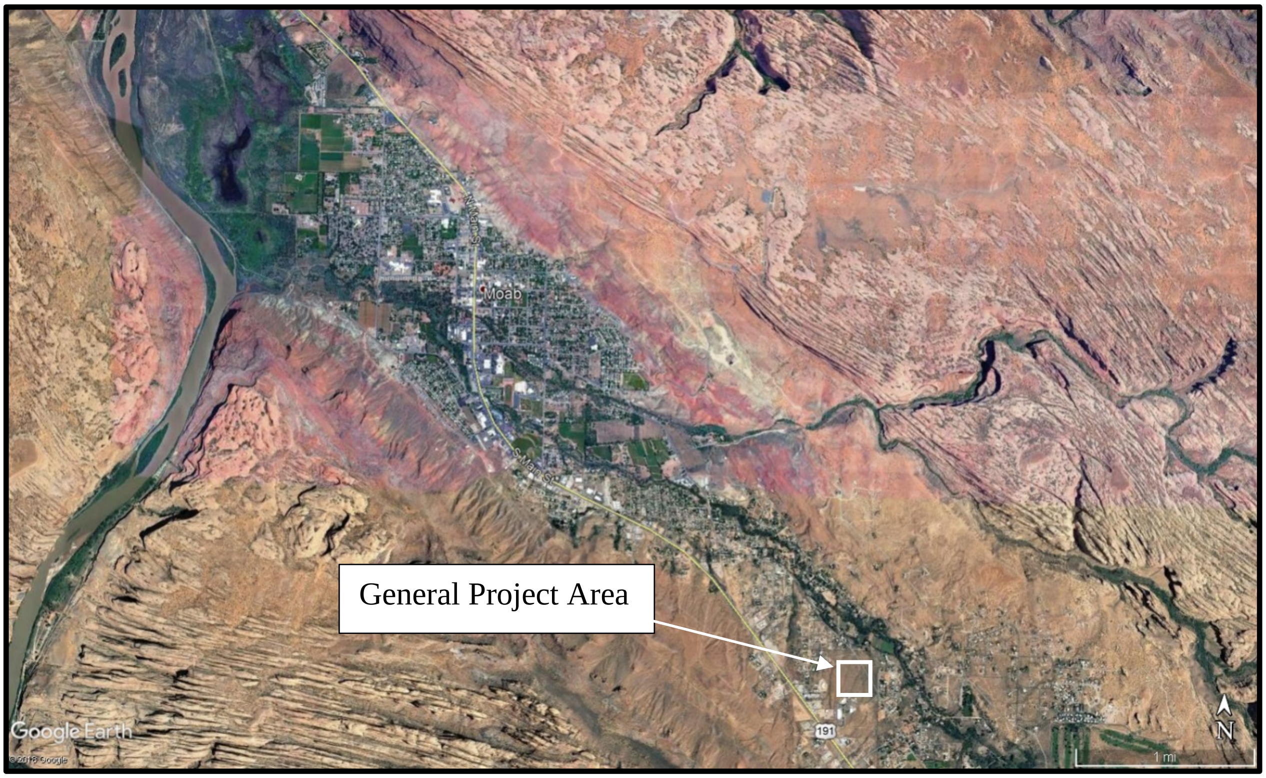
3.0 FIELD STUDY

3.1 *Project Location*

The proposed approximate 60-acre development is located east of State Highway 191, just south of the Moab City Limits within Grand County, Utah. More specifically, the development is generally bounded on the south by a future extension of South Plateau Road (extension of road proposed as part of the subject development), and generally bounded on the north by a future extension of East Starbuck Lane (extension of road proposed as part of the subject development). Spanish Valley Drive bisects the eastern to north-central area of the project site.

Figure 3.1 presented below indicates the general location of the project site. The imagery used for Figure 3.1 was obtained from Google Earth (imagery date: 7/27/2015). A more detailed aerial view of the project site in relationship to existing roadways/structures may be found on Figure 3.2 presented in Section 3.2 below.

Figure 3.1: Approximate Project Location

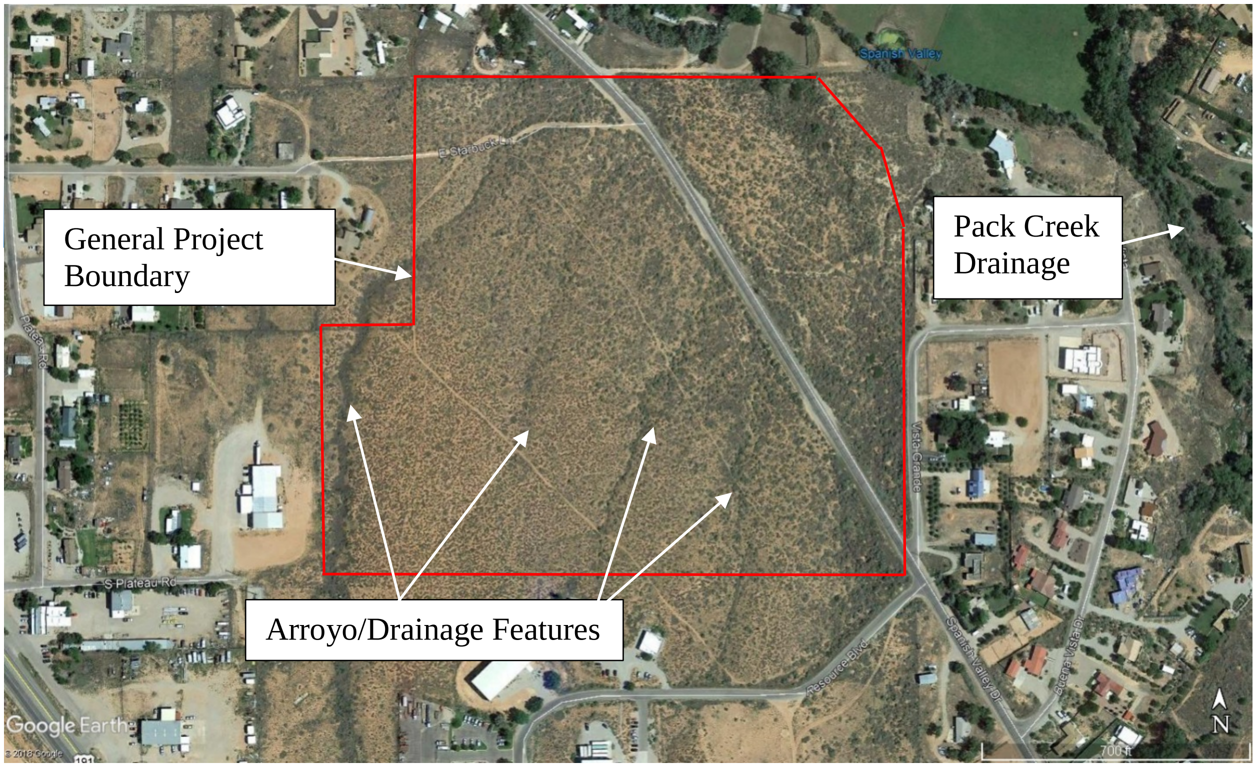


3.2 *Site Description and Geomorphology*

Figure 3.2 presented below may be referenced to help clarify the following discussion (Google Earth imagery date 7/27/2015).

As discussed above, the project site is generally 60-acres in area. The approximate geometry of the project boundaries is shown on Figure 3.2 below. The project area currently consists of undeveloped land. The ground surface on the project site generally slopes down to the northeast with slope inclinations in the range of about 10:1; horizontal to vertical (h:v) to 15:1; h:v with interspersed areas with little to no slope inclination. Numerous shallow to moderate sized arroyo’s/drainages run through the project site with a gradient down to the northeast. The depth of the drainage features range from about 5 to 15 feet with side slope inclinations in the range of about 5:1; h:v. Pack Creek is located below and to the east-northeast of the subject property.

Figure 3.2: Site Characteristics



We observed numerous test holes throughout the project site, likely excavated for a previous geotechnical engineering study. We do not know the particulars of the test holes regarding the disturbed area and depth of the test holes. We recommend that information regarding the test holes be obtained such as the logs of the test holes and any photographs that may be available to help further assess the limits and depths of the previous disturbance. The locations of the previously excavated test holes should be mapped/surveyed in the near future for future identification as the development progresses. We anticipate that the soil backfill within the test holes was not monitored for compaction and may exhibit a high consolidation potential. The test holes will need to be re-excavated to the previous bottom of the test holes and carefully backfilled (placement of compacted fill materials is generally discussed later in this report). We recommend that the backfill placement and compaction be carefully monitored. A photograph of one of the test hole locations that we observed is provided below.

Photograph of Previously Excavated Test Hole

The geomorphology in the vicinity of the project site generally consists of variable depth silty sand soil deposits overlying alluvial deposits of gravels and sands. Based on the information obtained from our test borings, the depth of the sand deposits on the project site generally ranges from about 10 to 25 feet thick at which point the underlying alluvial sand and gravel deposits were encountered.

Variable quantities of calcium sulfate (gypsum) material are often encountered within the soil deposits in the vicinity of the project site. The presence of gypsum within the soil deposits often contributes to high “dry strength” conditions, however when wetted the gypsum material may dissolve, causing the soils to consolidate. The potential consolidation can occur rapidly. For this reason, this phenomena is commonly referred to as “soil collapse” or the presence of “collapsible soils”. The magnitude of potential settlement or “collapse” is dependent on the depth of soils that may become wetted in the future, the void ratio characteristics within the cemented soils, and the effective pressures due to soil overburden pressures and potential structure loads that act on the cemented or "collapsible" soil deposits. The presence of gypsum deposits in the subject site soils and the potential influence of these deposits are discussed in more detail in Sections 3.3, 4.0, and 5.0 of this report.

As discussed in more detail in Section 3.3 below, subsurface free water was encountered in some of our test borings within the alluvial gravel and sand deposits that underlie the project site. The subsurface water elevation in the vicinity of the project site typically fluctuates with varying seasonal water runoff conditions.

3.3 *Subsurface Soil and Water Conditions*

We advanced fourteen test borings in a grid-like pattern across the project site. The approximate locations of our test borings are provided on Figures 3.3 and 3.4 below. Figure 3.3 indicates the approximate test boring locations relative to aerial photography (Google Earth imagery date 7/27/2015). Figure 3.4 indicates the approximate test boring locations relative to the conceptual project layout that was provided to us prior to our field work. The logs of the soils encountered in our test borings are presented in Appendix A.

Figure 3.3: Approximate Test Boring Locations Relative to Google Earth Imagery

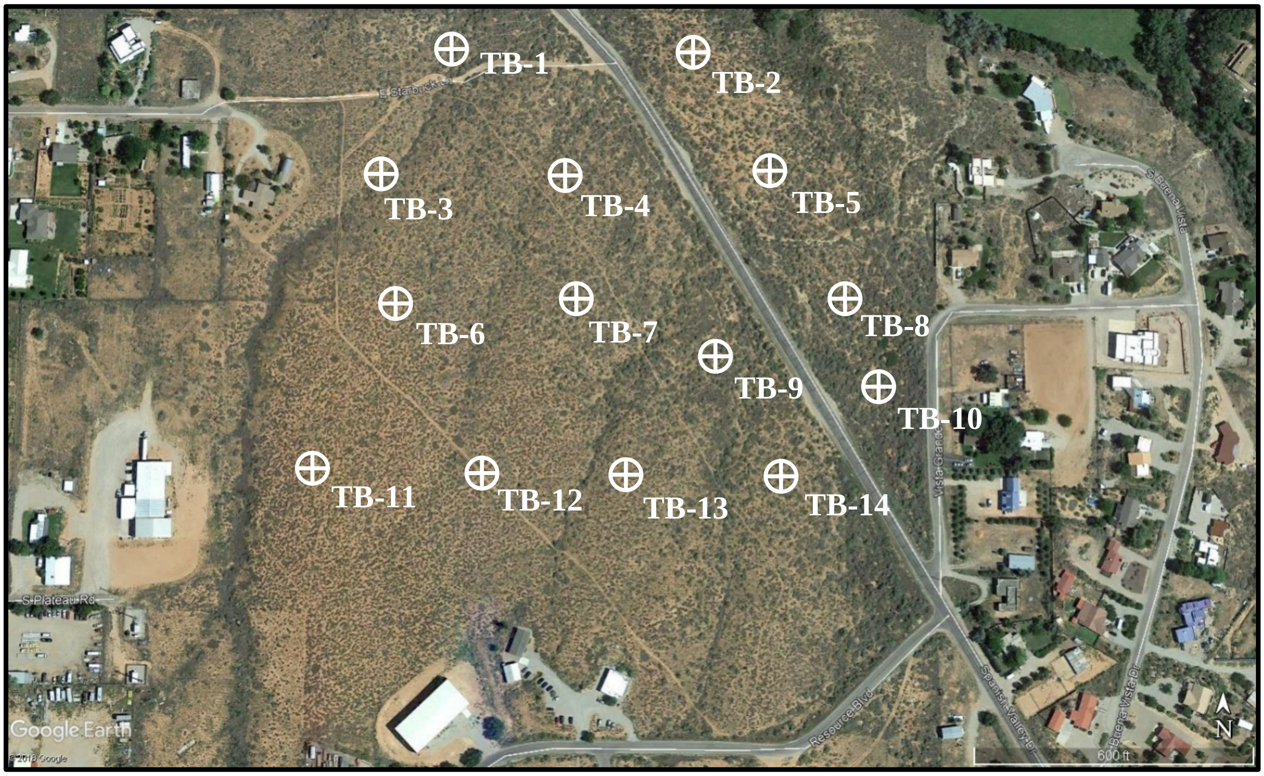


Figure 3.4: Approximate Test Boring Locations Relative to Preliminary Project Layout

The approximate test boring locations shown on the figures above were prepared using notes taken during the field work and are intended to show the approximate test boring locations for reference purposes only. We marked each of the test boring locations in the field if it is desired to obtain surveyed locations of the borings.

The following text provides a general description of the subsurface conditions that we encountered in our test borings. The logs of the test borings provided in Appendix A should be consulted for more detailed subsurface conditions.

Test Borings TB-1, 3, 4, 6, 7, 9, 11, 12, 13, and 14 were advanced within the project area west of Spanish Valley Drive. In these test borings we generally encountered silty sand material from the ground surface to depths ranging from about 8 to 20 feet below the ground surface elevation. The silty sand materials encountered from the ground surface to depths ranging from about 4 to 7 feet below the ground surface elevation were moist to very moist and medium dense to dense. The high moisture conditions within the upper soils have likely been influenced by relatively heavy precipitation during the past winter and spring months. At depths below about 4 to 7 feet below the ground surface elevation the moisture conditions of the subsurface soils decreased to being moist to slightly moist, while the density of the silty sand materials generally increased to being dense to very dense.

We encountered variable quantities of white colored chemical deposits within the silty sand soils in a number of our test borings at depths below about 5 feet below the ground surface elevation. Notable white chemical deposits were encountered in Test Borings TB-4 and TB-7. Standard penetration tests (N-values) within the chemically “cemented” soil deposits encountered in Test Borings TB-4 and TB-7 ranged from about N=50 to N=70 or greater, indicating the very high dry strength characteristics of the chemically cemented soil deposits. Based on the laboratory chemical analyses performed, we anticipate that the white chemical deposits are predominantly composed of calcium sulfate (gypsum). We anticipate that the presence and degree of gypsum deposits below the project site will be variable and may change significantly over relatively short distances.

At depths ranging from about 8 to 20 feet below the ground surface elevation we encountered a mixture of dense to very dense sand, gravel, and cobbles with a silt soil matrix. In general, the depth to the dense sand and gravel deposits increased towards the southern side of the project site.

Test Borings TB-2, TB-5, TB-8, and TB-10 were advanced on the portion of the project site located to the east of Spanish Valley Drive. In these test borings we generally encountered medium dense and moist to very moist silty sand soils from the ground surface to depths ranging from about 2 to 5 feet below the ground surface elevation where we encountered a mixture of dense to very dense sand, gravel and cobbles with a silt soil matrix. We did not encounter evidence of heavy gypsum deposits within our test borings that were advanced to the east of Spanish Valley Drive.

We encountered subsurface free water at depths ranging from about 27 to 37 feet below the ground surface in the portion of the project site located west of Spanish Valley Drive (Test Borings TB-6 and TB-14), and at depths ranging from about 11 to 13 feet below the ground surface elevation in the portion of the project site located east of Spanish Valley Drive (Test Borings TB-2 and TB-5). We suspect that the subsurface water elevation and soil moisture conditions will be influenced by seasonal conditions such as snow melt and/or precipitation and local irrigation. We anticipate that the ground water elevation will vary by plus or minus a few feet depending on seasonal precipitation or snowmelt conditions.

The logs of the subsurface soil conditions encountered in our test borings are presented in Appendix A. The logs present our interpretation of the subsurface conditions encountered in our test borings at the time of our field work. Subsurface soil and water conditions are often variable across relatively short distances. It is likely that variable subsurface soil and water conditions will be encountered during construction. Laboratory soil classifications of samples obtained may differ from field classifications.

3.4 *General Seismic Site Class Considerations*

This section of the report provides general comments regarding the seismic site class for the development. It must be noted that the actual seismic site class as defined by the International Building Code will likely vary somewhat across the project site, and should be defined as a structure/project specific evaluation. The seismic site class as defined by the International Building Code is based on various average values of select soil characteristics such as shear wave velocity, standard penetration test result values, undrained shear strength, and plasticity index.

In general, we feel that the subsurface soils across the project site will meet the criteria for a Site Class D designation based on the limited standard penetration testing that we performed as part of our field work. However, it is possible that isolated areas of the project site may meet the criteria for a Site Class E designation.

4.0 LABORATORY STUDY

We performed the following tests on select samples obtained from the test borings.

Moisture content and dry density; the moisture content and in-situ dry density of some of the Modified California Barrel liner samples was assessed. The results of these tests may be found on the consolidation test results presented on Figures 4.9 through 4.24 of Appendix B. These test results are also tabulated below.

Atterberg Limits and Sieve Analysis Tests; the plastic limit, liquid limit and plasticity index as well as the gradation of select soil samples was determined. The results of the sieve analysis and Atterberg Limits tests are presented on Figures 4.1 through 4.8 of Appendix B. In general, the soils tested classify as USCS type “SM” silty sand with various quantities or gravel, or AASHTO type A-1 to A-4 material.

Swell-Consolidation Tests; the one-dimensional swell-consolidation potential of some of the soil samples obtained was determined in general accordance with constant volume methodology. The soil samples tested were exposed to varying loads and inundated with water at various surcharge pressures to assess the swell potential and/or consolidation potential with water inundation. We did not obtain any measurable swell potential for the site soils, rather the samples consolidated when exposed to water. The one-dimensional consolidation response of the soil samples tested to the loads and inundation with water is represented graphically on Figures 4.9 through 4.24 of Appendix B. We have tabulated some of the pertinent information obtained from the swell-consolidation testing below.

Sample Designation	Moisture Content (percent)	Dry Density (pcf)	Measured Swell Pressure/Potential* (psf)/(percent)	Consolidation/Collapse Potential when wetted (percent)
TB-1 @ 3 feet	4.0	106.2	0.0 (100 psf surcharge)	0.2 (100 psf surcharge)
TB-3 @ 3 feet	8.2	116.7	0.0 (500 psf surcharge)	0.1 (500 psf surcharge)
TB-4 @ 3 feet	8.4	111.2	0.0 (500 psf surcharge)	0.1 (500 psf surcharge)
TB-4 @ 8 feet	3.0	118.4	0.0 (1,000 psf surcharge)	0.5 (1,000 psf surcharge)
TB-6 @ 4 feet	7.0	122.8	0.0 (500 psf surcharge)	0.2 (500 psf surcharge)
TB-6 @ 9 feet	2.6	110.1	0.0 (1,000 psf surcharge)	0.8 (1,000 psf surcharge)
TB-7 @ 3 feet	7.6	110.3	0.0 (100 psf surcharge)	0.0 (100 psf surcharge)
TB-7 @ 8 feet	5.0	122.0	0.0 (1,000 psf surcharge)	0.2 (1,000 psf surcharge)
TB-7 @ 13 feet	3.9	120.8	0.0 (2,000 psf surcharge)	0.9 (2,000 psf surcharge)
TB-8 @ 3 feet	5.8	91.9	0.0 (100 psf surcharge)	0.8 (100 psf surcharge)
TB-11 @ 3 feet	8.7	122.2	0.0 (100 psf surcharge)	0.1 (100 psf surcharge)
TB-11 @ 8 feet	1.5	106.4	0.0 (1,000 psf surcharge)	2.6 (1,000 psf surcharge)
TB-12 @ 3 feet	7.4	121.0	0.0 (100 psf surcharge)	0.1 (100 psf surcharge)
TB-13 @ 4 feet	8.0	112.7	0.0 (500 psf surcharge)	0.3 (500 psf surcharge)
TB-13 @ 9 feet	3.4	111.2	0.0 (1,000 psf surcharge)	1.9 (1,000 psf surcharge)
TB-14 @ 4 feet	1.6	109.3	0.0 (500 psf surcharge)	1.5 (500 psf surcharge)

The samples tested generally exhibit a low to moderate overall consolidation potential when exposed to loads beyond the historic effective pressures that have acted on the soils. In addition, in general the samples that were tested exhibit a low to moderate collapse potential at the point the samples were inundated with water. However, a number of the samples exhibited a moderate to high collapse potential. Based on the laboratory test results, the collapse potential due to cementing from chemical depositions such as gypsum across the project site appears to be generally low to moderate, however some areas on the project site may exhibit a high potential for settlement due to collapse of cemented soils deposits and/or typical settlement reactions to increased loading. Site/structure specific geotechnical analysis should be performed to address areas that may exhibit a high potential for settlement from either collapsible soil conditions or poorly consolidated soil conditions.

Moisture content-dry density relationship (Proctor) tests; We performed laboratory moisture content-dry density tests to assess the relationship between the soil moisture content and dry density. We performed two modified Proctor tests to assess potential differences in the maximum dry density and optimum moisture content across the project site. The Proctor tests were performed in general accordance with ASTM D1557. The results of the laboratory Proctor tests are presented on Figures 4.25 and 4.26. The maximum dry density obtained for the two tests ranged from about 125.0 to 126.5 pounds per cubic foot, with both tests exhibiting an optimum moisture content of about 10.0 percent. In general, the shallow silty sand soil materials appear to exhibit a relatively uniform maximum dry density characteristics as determined by the modified Proctor test.

California Bearing Ratio (CBR) Tests; We assessed the pavement section support characteristics of select composite soil samples in general accordance with ASTM D1883. The results of the CBR tests are presented on Figure 4.27. We obtained a CBR of about 20 for the native silty sand soil materials that are compacted to at least 90 percent of the maximum dry density as defined by the modified Proctor test (ASTM D1557).

Soluble Sulfates Tests; The soluble sulfate quantity of four test samples was assess in order to help estimate the corrosion potential of the site soils on Portland cement concrete. The test results are tabulated below.

Sample Designation	**Water Soluble Sulfate in Soil (percent by weight)**
TB-4, 0'-3'	0.035
TB-5 0'-4'	0.01
TB-7 4'-8'	0.34
TB-7 @ 13'	.049
TB-12 0'-3'	0.01

The American Concrete Institute (ACI) indicates that soil with a soluble sulfate content greater than 0.1 percent constitutes a moderate exposure to sulfate attack on Portland cement concrete. For soils with a moderate potential for sulfate attack the ACI recommends that a maximum water/cement ratio of 0.45 and either a type II, IP(MS), IS(MS), P(MS), I(PM)(MS), or a I(SM)(MS) cement be used for the project. Some of the test samples exhibit a severe potential for sulfate attack (soluble sulfate levels greater than 0.20 percent). The ACI recommends that a Type V Portland cement be used for soils with a severe potential for sulfate attack. Based on our experience, Type V Portland cement is extremely difficult to obtain at this time. Alternative methods for helping to alleviate sulfate attack on Portland cement concrete such as high compressive strength characteristics, and low water to cement ratio mix designs and/or mix designs that include alternative cementitious products may need to be explored for some areas of the project site. Again, site specific geotechnical engineering studies that include soil chemical analysis is recommended for the project to help identify areas that may exhibit a moderate or high exposure risk to sulfate attack on Portland cement concrete.

5.0 VIABLE FOUNDATION SYSTEMS

This section of the report provides feasibility level comments which may be used to help assess the potential geotechnical engineering related challenges associated with the development of this project site. Our comments are based on our feasibility level field study, laboratory study, and our experience with similar subsurface soil conditions.

There are two general types of foundation system concepts, “shallow” and “deep”, with the designation being based on the depth of support of the system. Shallow foundation system concepts include mats or rafts, and conventional spread footings with stem walls. More common deep foundation system concepts include driven piles, drilled piers and steel helical piers. Helical piers or possibly driven pipe piles are likely the most applicable deep foundation systems for the project in areas that exhibit problematic soil conditions such as potential collapsible soils. There are numerous similar foundation design concepts, but the concepts listed above are of the more common types used in the vicinity of the project.

In general, based on our limited scope feasibility level study, we anticipate that conventional spread footing or mat type foundation systems may be considered to support the structures associated with the proposed development. However, as discussed in Section 4.0 above, some of the soils encountered and tested exhibit a moderate to high consolidation potential from either collapse of cemented soils and/or more conventional settlement due to poorly consolidated soil deposits. The consolidation test results obtained from Test Borings TB-11, TB-13, and TB-14 represents the highest collapse potential soils (when inundated with water) that we encountered in our test borings. Based on these test results we anticipate that the magnitude of post construction settlement for shallow supported foundation systems (such as spread footings or mat foundations) may be in the range of about ½ to over 1 inch if about 2 to 3 feet of the site soils below the foundation system were to become wetted after construction. Additional magnitudes of settlement could occur if additional depths of cemented/collapsible soils below the foundation systems were to become wetted. As discussed above, we recommend that site/structure specific geotechnical engineering studies be performed to address potential problematic areas with regard to collapsible soils or soils with a high settlement potential.

Areas of the development where collapsible soils or otherwise high consolidation potential soils are identified during the future site/structure specific geotechnical engineering study may be addressed with mitigation strategies such as;

- Utilize a deep foundation system such as helical piers or possibly driven piles to support structures in areas with a high to severe potential for soil collapse or general settlement.
- Utilize a conventional spread footing foundation system that is supported by a relatively thick layer of reconditioned and compacted native soil subgrade materials in conjunction with imported structural fill materials to support structures in areas that exhibit a less severe potential for soil collapse or general settlement.

The primary factor that contributes to soil settlement, particularly to the collapse of cemented soils, is the introduction of water or higher moisture conditions relative to the moisture conditions that have historically existed within the soil mass. We highly recommend that landscaping that requires even moderate irrigation be generally avoided for the development, particularly in areas immediately surrounding the proposed development structures. The civil design for the project roadways and parking areas and/or storm water runoff from structures with a large roof area should be carefully planned to limit the accumulation of water (such as from storm water detention/retention ponds or subsurface storm water infiltration systems) in areas adjacent to the structures associated with the development. Park areas that require extensive irrigation should be carefully located to limit the potential influence of these irrigated areas on the structures associated with the development.

The integrity and long-term performance of any type of foundation system is influenced by the quality of workmanship which is implemented during construction. It is imperative that all excavation and fill placement operations be conducted by qualified personnel using appropriate equipment and techniques to provide suitable support conditions for the foundation system.

5.1 *Spread Footings*

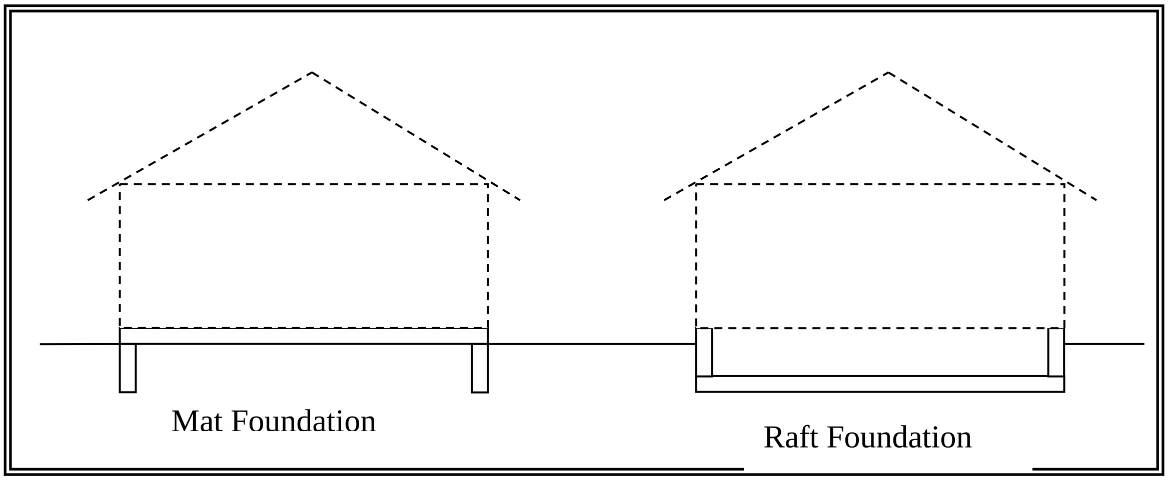
We anticipate that conventional spread footings may be used to support the majority of the structures associated with the development. Particular attention should be addressed during future design level geotechnical engineering studies to identify areas where potential collapsible soils are present. In areas where the soil collapse/settlement potential is determined to be high it may be recommended to utilize a deep foundation system such as helical piers or driven piles. For soils with a low to moderate collapse/settlement potential, it may be recommended to recondition and compact a portion of the subgrade soils (perhaps in the range of about 2 to 3 feet) in conjunction with the placement of a layer of compacted structural fill to help decrease potential post construction consolidation of a spread footing foundation system to a tolerable magnitude.

The following provides a list of design and construction items that we anticipate can be expected for spread footing designs in areas where potential collapsible soils are not a concern;

- We anticipate that in general, it will be recommended to support the spread footings on a composite fill blanket which consists of a layer of the native soils which have been scarified, moisture conditioned, and compacted, followed by the placement of a layer of imported compacted granular structural fill material to help reduce post construction settlement of the foundation system.
- In general, we anticipate that bearing capacity values for spread footings will be in the range of at least 1,500 pounds per square foot or greater for most structure applications. Additional allowable bearing capacity may be provided for footings located on more granular soils (such as those encountered in our test borings that were advanced in areas east of Spanish Valley Drive) or for larger footings or footings with a substantial depth of embedment. Conversely, lower allowable bearing capacity values may be provided for some areas of the project site where soft and/or unconsolidated soil conditions are present.
- It is possible that foundation drains will be recommended, particularly in areas where potential collapsible soil deposits are encountered.
- It may be recommended to avoid isolated footings on some structures that exhibit a potential for moderate to high differential settlement between continuous and isolated footings.

5.2 *Mat Foundations*

Mat or raft foundations are commonly used to support structures on sites with soft and/or wet soil conditions. The design concepts of either system are similar, but their configurations are slightly different. This is shown in the sketch below.



Depending on the subsurface conditions, the depth of the support elevation of a raft foundation may be varied as needed to alter the design capacity of the system.

A mat foundation system must be designed with sufficient rigidity to effectively distribute the structural loads across the mat area. This typically requires a relatively thick steel reinforced concrete section to accomplish. A preliminary modulus of subgrade reaction in the range of about 150 pounds per cubic inch may be used to assess the viability of a mat foundation system for the project structures.

5.3 *General Shallow Foundation Considerations*

Some movement and settlement of any shallow foundation system will occur after construction. Some movement associated with swelling soils could also occur in isolated areas of the development. Utility line connections through and foundation or structural component should be appropriately sleeved to reduce the potential for damage to the utility line. Flexible utility line connections will further reduce the potential for damage associated with movement of the structure.

Deep Foundation Concepts

As discussed above, deep foundation systems are applicable to support structures in areas where high consolidation potential soils and/or collapsible soils have been identified, or to support structures where post construction settlement must be minimized as much as possible.

Deep foundation systems are less susceptible to movement from potential consolidation of shallow soils and/or collapsible soil conditions since the support elevation of the deep foundation system is extended to a stable bearing stratum such as very dense gravel/cobble deposits. Site specific geotechnical evaluation must be performed to determine the suitable bearing elevation and anticipated bearing capacity characteristics for deep foundation system components. Of the various deep foundation systems, we feel that helical piers and driven pipe pile foundation systems are the most viable based on the subsurface conditions that we encountered for this feasibility level study. In either case the deep foundation system is capped with a grade beam or similar structural component which is intended to distribute the imposed structural loads to each deep foundation system component.

5.4 *Helical Piers*

A helical pier is a foundation element consisting of a central shaft with at least one helix plate located on the shaft with its axis positioned parallel to the shaft's length. The helical pier is rotated while being advanced to the proper bearing stratum. The correct rotational rate versus advancement rate is critical for proper performance of the pier. Typically, the installation torque is monitored during installation and utilized to assess the load carrying capacity of the pier. The torque versus load carrying capacity relationship is established by the pier manufacturer and/or from actual load testing data performed on the project site. The number and diameter of helixes can be increased to improve the load carrying capacity of helical piers.

There are many types and brands of helical piers available. Since there are numerous proprietary helical pier suppliers, each manufacturer has different techniques to estimate the load carrying capacity of their product. It should be noted that hard soil, dense gravel soils and cobbles often prevent installation of helical piers to appropriate bearing depths. Helical piers which are not installed to appropriate bearing elevations may not provide sufficient support for the structure.

Helical piers should be extended to bear in the dense alluvial gravel, cobble and sand deposits that underlie the project site. It may be recommended to advance the helical piers below the subsurface free water elevation where gypsum cements soils should not be a concern.

We anticipate that an allowable capacity in the range of at least 40 kips per pier may be obtained. The allowable capacity of the piers will be partially dependent on the size and number of helices used for the helical pier components. We do not recommend attributing any resistance to lateral forces or moments to the helical piers. Battered piers will likely be needed to resolve lateral forces. In general, we recommend that a number of test piers be installed and load tested to obtain site/structure specific correlations of installation torque versus load carrying capacity of the piers.

5.5 *Driven Pipe Piles*

Based on our subsurface exploration we feel that a driven pipe pile foundation system is a technically viable option for the project. As with helical piers, pipe piles should be extended to bear in the dense alluvial gravel, cobble and sand deposits that underlie the project site. It may be recommended to advance extend the pipe piles below the subsurface free water elevation where gypsum cements soils will present less of a concern. Site specific geotechnical evaluation must be performed to determine the suitable bearing elevation and anticipated bearing capacity characteristics for pipe pile foundation systems.

We anticipate that an allowable capacity in the range of 60 kips per pile (or greater) may be obtained for 10 to 12-inch diameter closed end pipe piles. The bearing capacity and driving characteristics will need to be determined at the onset of pile driving operations. Dynamic load testing with PDA sensors/equipment during the driving operations of some of the piles may be recommended to help verify the bearing capacity characteristics of driven pipe piles.

6.0 RETAINING STRUCTURES

We anticipate that laterally loaded exterior site walls and walls that are associated with the actual structures (such as basement walls) will be constructed as part of this site development. Conventional cantilever walls or other types of exterior wall systems such as geosynthetically reinforced earth (GRE) walls or stacked boulder (rockery) walls may be used to support retained soils on this project site.

It should be noted that the site soils are highly susceptible to erosion and/or piping due to the non-cohesive nature of the silty sand soils on this project site. Restraining excavation cut or fill slope surfaces with a retaining wall system should be considered for excavation or fill placements that exhibit a substantial height and/or exhibit steep inclinations. The use of geotextile fabrics may be needed in stacked boulder (rockery) type wall systems to avoid the loss of the retained soils between the individual boulders due to erosion or piping.

In general, the site soils will exhibit relatively low lateral earth pressure design values due to the granular nature of the site soils.

7.0 SUBSURFACE DRAIN SYSTEMS

A subsurface drain system and/or weep holes should be included in the retaining structure design. Exterior retaining structures may be constructed with weep holes to allow subsurface water migration through the retaining structures.

A drain system constructed with a free draining aggregate material and a perforated pipe should be constructed adjacent to interior retaining structures. We anticipate that it may be recommended to construct foundation drain systems on sites that exhibit potential collapsible soil conditions. Drain system details will depend on the site and structure specific conditions.

In general, the site soils are relatively permeable and do not exhibit substantial swell potential. Therefore, a foundation level drain system may not be needed for many of the structures associated with the development (other than in areas that exhibit collapsible soil concerns).

8.0 CONCRETE FLATWORK

We anticipate that both interior and exterior concrete flatwork will be included in the project design. Concrete flatwork is typically lightly loaded and has a limited capability to resist shear forces associated with volume changes in the support soils, including frost heave. It is prudent for the design and construction of concrete flatwork to be able to accommodate some movement due to volume changes in the support soils.

8.1 *Interior Concrete Slab-on-Grade Floors*

Design level recommendations for interior concrete flatwork will likely include the recommendation to recondition and compact a portion of the native subgrade soils materials followed by the placement of a layer of imported granular structural fill to directly support the flatwork. It should be noted that the only way to completely mitigate the potential for movement in the floor system from the site soil conditions is to utilize a structurally supported floor.

It may be recommended to utilize a structural floor (crawl space area) in areas where collapsible soil conditions are a concern, or in areas where the placement of substantial fill is needed to establish the finished floor elevation of the structure due to potential settlement of the soil materials below the floor system(s).

8.2 Exterior Concrete Flatwork Considerations

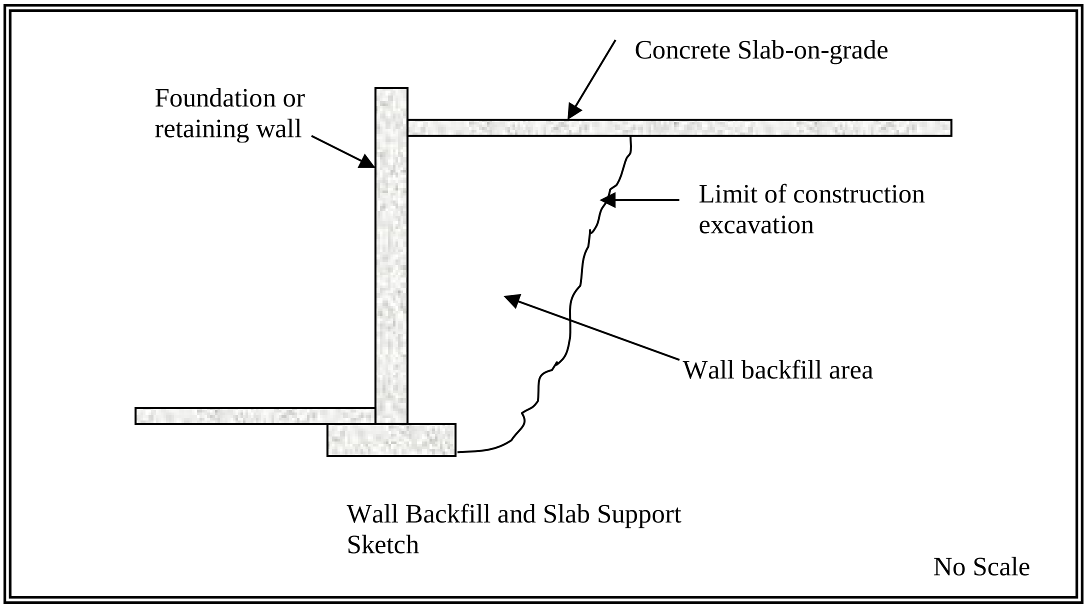
Exterior concrete flatwork includes concrete driveway slabs, aprons, patios, and walkways. The desired performance of exterior flatwork typically varies depending on the proposed use of the site and each owner's individual expectations. As with interior flatwork, exterior flatwork is particularly prone to movement and potential damage due to movement of the support soils. Unlike interior flatwork, exterior flatwork may be exposed to frost heave, particularly on sites with high silt-content soils.

If some movement of exterior flatwork is acceptable, we generally suggest that the support areas be prepared by scarification, moisture conditioning and re-compaction of about 8 inches of the natural soils followed by placement of about 4 to 6 inches of compacted granular fill material.

Exterior flatwork should not be placed on soils prepared for support of landscaping vegetation. Cultivated soils will not provide suitable support for concrete flatwork.

8.3 General Concrete Flatwork Comments

It is relatively common that both interior and exterior concrete flatwork is supported by areas of fill adjacent to either shallow foundation walls or basement retaining walls. A typical sketch of this condition is shown below.



Settlement of the backfill shown above will create a void and lack of soil support for the portions of the slab over the backfill. Settlement of the fill supporting the concrete flatwork is likely to cause damage to the slab-on-grade. Settlement and associated damage to the concrete flatwork may occur when the backfill is relatively deep, even if the backfill is compacted.

If this condition is likely to exist on this site it may be prudent to design the slab to be structurally supported on the retaining or foundation wall and designed to span to areas away from the backfill area as designed by the project structural engineer.

9.0 PAVEMENT SECTION THICKNESS DESIGN RECOMMENDATIONS

We performed a California Bearing Ratio (CBR) test on a composite sample of soil obtained from the project site. Based on the results of the CBR test we used an effective roadbed subgrade resilient modulus M_R of about 10,000 pounds per square inch (psi).

We recommend that the subgrade soils be proof-rolled prior to the scarification and processing operations. Any soft areas observed during the proof-rolling operations should be removed and replaced with properly processed materials and/or granular aggregate materials as part of the subgrade preparation.

The site subgrade pavement section support soils must be scarified to a depth of 12 inches, moisture conditioned and compacted prior to placement of the overlying aggregate pavement section materials. The material should be moisture conditioned to within optimum to about 2 percent above the optimum moisture content and compacted to at least 90 percent of maximum dry density as determined by the modified Proctor test, ASTM D1557. The surface of the subgrade soil should be graded and contoured to be approximately parallel to the finished grade of the pavement surface. All embankment fill materials associated with the establishment of the roadway subgrade elevations must be compacted to at least 90 percent of the maximum dry density as defined by the modified Proctor test (ASTM D1557).

It should be noted that it may be difficult to compact the imported aggregate base course materials over some areas of the project site subgrade soils due to the sandy and therefore un-cohesive nature of the subgrade materials. It may be necessary to utilize a layer of geotextile fabric such as Mirafi RS280i between the subgrade soils and the overlying aggregate base course materials in some areas of the project site if obtaining proper compaction of the aggregate base course materials (discussed below) becomes problematic.

We recommend that the aggregate materials comply with the following specifications

	Percent Passing Each Sieve	
	1” Aggregate Base Course	3” Aggregate Sub-Base-Course
Sieve Size		
4”	--	100
3”	--	95-100
1”	100	
#4	30-65	--
#8	25-55	--
#200	3-12	3-15
Liquid Limit	less than 30	less than 35
Plasticity Index	less than 6	less than 9

Aggregate base-course and sub-base course locally available that do not meet the above specifications may be suitable for this project. We are available to review other aggregate gradations produced by local gravel producers to insure the suitability of the gravel products for the project. The aggregate base course should have a minimum R-value of 72, and the aggregate sub-base course should have a minimum R-Value of 65.

The imported aggregate subbase and/or basecourse materials must be moisture conditioned to plus or minus 2 percent of optimum moisture content, and compacted to at least 95 percent of the maximum dry density as defined by the modified Proctor test (ASTM D1557 or AASHOT T-180).

9.1 *Asphalt Pavement Recommendations*

We recommend that the asphalt concrete used on this project be mixed in accordance with a design prepared by a licensed professional engineer, or an asphalt concrete specialist. We should be contacted to review the mix design prior to placement at the project site. We recommend that the asphalt concrete be compacted to between 92 and 96 percent of the maximum theoretical density.

We have provided several pavement section design thicknesses below. The structural support characteristics of each section are approximately equal. The project civil engineer, or contractor can evaluate the best combination of materials for economic considerations.

We have provided pavement section thickness recommendations for 50,000-18,000 pound equivalent single axle loads (18k ESAL) for low volume parking areas that will not be subjected to appreciable truck traffic below. For the project roadways, we have provided 75,000, 150,000, and 250,000 18k ESAL pavement design sections. The sections provided for 75,000 and 150,00 18k ESAL values may be used for higher volume parking areas and/or parking areas that will be subjected to heavy truck traffic. We are available to provide additional design sections, if these are desired.

Pavement Section Design Thickness
Light Duty Standard Passenger Car/Truck Use Parking Areas
50,000 18k ESAL

Pavement Section Component	**Alternative Thicknesses of Each Component (inches)**		
Asphalt Concrete	2½	2½	3
1” minus aggregate base course	4	8	6
3” minus aggregate subbase	5	0	0
Reconditioned Subgrade	12	12	12

Pavement Section Design Thickness
75,000 18k ESAL

Pavement Section Component	**Alternative Thicknesses of Each Component (inches)**		
Asphalt Concrete	3	3	4
1” minus aggregate base course	4	8	5
3” minus aggregate subbase	6	0	0
Reconditioned Subgrade	12	12	12

Pavement Section Design Thickness
150,000 18k ESAL

Pavement Section Component	**Alternative Thicknesses of Each Component (inches)**		
Asphalt Concrete	3	3	4
1” minus aggregate base course	6	11	7
3” minus aggregate subbase	6	0	0
Reconditioned Subgrade	12	12	12

Pavement Section Design Thickness
250,000 18k ESAL

Pavement Section Component	**Alternative Thicknesses of Each Component (inches)**	
Asphalt Concrete	4	4
1” minus aggregate base course	6	9
3” minus aggregate subbase	5	0
Reconditioned Subgrade	12	12

The pavement section thicknesses tabulated above are appropriate for the post-construction traffic use associated with the development. Heavy construction equipment traffic will have a significant influence on the quality, character, and design life of the pavement sections tabulated above. If possible, we recommend that the asphalt pavement not be placed until completion of the major construction activity. We are available to discuss this with you as the project progresses.

9.2 *Rigid Concrete Pavement Recommendations for Parking Areas*

This section of the report provides ridged concrete pavement section designs for parking areas. These recommendations are not suitable for concrete pavement roadways. Please contact us if additional information is desired for rigid concrete pavement roadways.

The following recommendations are based on the American Concrete Institute (ACI) Guide for Design and Construction of Concrete Parking Lots report 330R-01, strength characteristics (CBR-value) of the native subgrade soils, and on our experience with concrete paved parking lots in the region. A modulus of subgrade reaction of 200 pounds per cubic inch may be used for design purposes. Our recommendations are as follows;

- For parking areas that will service light vehicular traffic only, a minimum concrete mat thickness of 5.0 inches should be used. This section is valid for an average daily truck (ADTT) of less than 10.
- For areas that will be exposed to heavy truck traffic on a regular basis a minimum concrete mat thickness of 6.0 inches should be used.

The concrete mat should be supported by a minimum thickness of 8 inches of 1-inch minus aggregate base course material that meets the specifications tabulated in Section 9.0 above. We are available to review aggregate gradations produced by local gravel producers to insure the suitability of the gravel products for the project. The aggregate materials should be compacted to at least 95 percent of maximum dry density as defined by AASHTO T-180 or ASTM D1557, modified Proctor test. The upper 12 inches of subgrade support soils should be scarified and compacted to at least 90 percent of maximum dry density as defined by AASHTO T-180 or ASTM D1557, modified Proctor test prior to placement of the aggregate base course.

A concrete with a minimum 28-day compressive strength of 4,500 pounds per square inch (psi) should be used. Joint spacing should not exceed 12½ feet for a slab thickness of 5 inches and should not exceed 15 feet for a slab thickness of 6 inches. The recommendations for minimum concrete strength presented here are based on the ACI recommendations. The site specific structural engineering design recommendations should take precedent over those listed in this report. The project structural engineer should be contacted for steel reinforcement design. We are available to discuss our recommendations with the project structural engineer as needed.

9.3 General Pavement Considerations

Water intrusion into the pavement section support materials will negatively influence the performance of the pavement surface. Water from irrigation, water from natural sources that migrates into the soils beneath landscape surfaces, and water from any source that gains access to the support materials can all decrease the life of the pavement section. Care should be taken along curbs and any edge of the asphalt pavement to develop an interface between the material that will reduce subsurface and surface water migration into the support soil and pavement section materials. Landscape islands and other irrigated features often promote water migration into adjacent pavement structures.

Asphalt shrinkage as well as minor shrinkage of concrete is common. Joints between the two materials typically widen with time due to continued shrinkage. This can cause premature damage to the pavement section in the following ways;

- The gap between the asphalt pavement and adjacent curb and gutter can introduce water into the pavement support aggregate and subgrade materials.
- If a high water flow velocity exists in the curb areas of roadways, the gap between the asphalt pavement and adjacent curb and gutter can lead to erosion and loss of the pavement support aggregate materials and underlying subgrade materials. We have observed this phenomena in the Moab area.

Future maintenance of the project should include observations and periodic sealing of any location where surface water may gain access to the pavement section support materials. As previously discussed, the native silty sand soil materials exhibit a very high erosion potential due to the un-cohesive nature of the native soil materials.

In areas where high water flow in the curb and gutter are expected it may be prudent to consider placing a layer of geotextile fabric between the asphalt pavement section and adjacent curb and gutter (below the abutment of the asphalt pavement to the adjacent curb and gutter) due to the high erodibility potential of the native silt sand materials. If a geotextile fabric is used, we recommend that it extend completely below the width of the curb-and-gutter with the remaining width of the fabric extending directly below the asphalt pavement. A fabric such as Mirafi rs280i or equivalent is suitable for this purpose. Alternatively, the curb and gutter section may be designed to be supported over a relatively thick aggregate base course section that provides a buffer between the silty sand subgrade soils and potential gap that can develop between the asphalt pavement section and adjacent curb and gutter. We recommend that this “buffer” or curb-and-gutter support section consist of at least 8 inches of aggregate base course. These recommendations are likely more critical for street sections with grades greater than about 2 to 3 percent.

10.0 CONSTRUCTION CONSIDERATIONS

This section of the report provides feasibility level comments, considerations and recommendations for aspects of the site construction which may influence, or be influenced by the geotechnical engineering considerations discussed above. The information presented below is not intended to discuss all aspects of the site construction conditions and considerations that may be encountered as the project progresses. If any questions arise as a result of our recommendations presented above, or if unexpected subsurface conditions are encountered during construction we should be contacted immediately.

10.1 *Fill Placement Recommendations*

There are several references throughout this report regarding both natural soil and compacted structural fill recommendations. The recommendations presented below are appropriate for the fill placement considerations discussed throughout the report above.

All areas to receive fill, structural components, or other site improvements should be properly prepared and grubbed at the initiation of the project construction. The grubbing operations should include scarification and removal of organic material and soil. No fill material or concrete should be placed in areas where existing vegetation or fill material exist.

We observed evidence of previous site use and existing man-placed fill during our field work. All existing fill material should be removed from areas planned for support of structural components or the project roadways and other infrastructure related to the development. Excavated areas and subterranean voids should be backfilled with properly compacted fill material as discussed below. As discussed in Section 3.2 above, we observed numerous test holes likely advanced as part of a previous geotechnical engineering study within the project area. The aerial extent and depth of these test holes are not known at this time. We anticipate that the test hole backfill materials were not compacted. The backfill soils placed in these test holes should be removed and replaced with well monitored compacted fill materials.

10.1.1 *Natural Soil Fill*

Any natural soil used for any fill purpose should be free of all deleterious material, such as organic material and construction debris. Natural soil fill includes excavated and replaced material or in-place scarified material.

10.1.2 *Granular Compacted Structural Fill*

Granular compacted structural fill is referenced in numerous locations throughout the text of this report. Granular compacted structural fill should be constructed using an imported commercially produced rock product such as aggregate road base. Many products other than road base, such as clean aggregate or select crusher fines may be suitable, depending on the intended use. If a specification is needed by the design professional for development of project specifications, a material conforming to the gradation presented below may be used.

	Percent Passing Each Sieve	
	1" Aggregate Base Course	3" Aggregate Sub-Base-Course
Sieve Size		
4"	--	100
3"	--	95-100
1"	100	
#4	30-65	--
#8	25-55	--
#200	3-12	3-15
Liquid Limit	less than 30	less than 35
Plasticity Index	Less than 6	Less than 9

10.2 *Excavation Considerations*

Unless a specific classification is performed, the site soils should be considered as an Occupational Safety and Health Administration (OSHA) Type C soil and should be sloped and/or benched according to the current OSHA regulations. Excavations should be sloped and benched to prevent wall collapse. Any soil can release suddenly and cave unexpectedly from excavation walls, particularly if the soils is very moist, or if fractures within the soil are present. Daily observations of the excavations should be conducted by OSHA competent site personnel to assess safety considerations.

10.2.1 *Excavation Cut Slopes*

We anticipate that some permanent excavation cut slopes may be included in the site development. Temporary cut slopes should not exceed 5 feet in height and should not be steeper than about 1:1, horizontal to vertical for most soils. Permanent cut slopes of greater than 5 feet or steeper than 2½:1, h:v must be analyzed on a site specific basis.

10.3 *Utility Considerations*

Subsurface utility trenches will be constructed as part of the site development. Utility line backfill often becomes a conduit for post construction water migration. If utility line trenches approach the individual structures associated with the development from above, water migrating along the utility line and/or backfill may have direct access to the portions of the proposed structures where the utility line penetrations are made through the foundation system. The foundation soils in the vicinity of the utility line penetration may be influenced by the additional subsurface water. There are a few options to help mitigate water migration along utility line backfill. Backfill bulkheads constructed with high clay content soils and/or placement of subsurface drains to promote utility line water discharge away from the foundation support soil are some concepts that may be used for the development.

Some movement of all structural components is normal and expected. The amount of movement may be greater on sites with problematic soil conditions. Utility line penetrations through any walls or floor slabs should be sleeved so that movement of the walls or slabs does not induce movement or stress in the utility line. Utility connections should be flexible to allow for some movement of the floor slab.

10.4 *Radon Issues*

The requested scope of service of this report did not include assessment of the site soils for radon production. Many soils and formational materials in the region produce Radon gas. Site/structure specific radon evaluation may be performed to assess the radon exposure level for the individual structures associated with the development. Several Federal Government agencies including the Environmental Protection Agency (EPA) have information and guidelines available for Radon considerations and home construction.

11.0 CONSTRUCTION MONITORING AND TESTING

Construction monitoring including engineering observations and materials testing during construction is a critical aspect of the geotechnical engineering contribution to any project. Unexpected subsurface conditions are often encountered during construction. The site excavations should be observed by the geotechnical engineer or a representative during the early stages of the site construction to verify that the actual subsurface soil and water conditions were properly characterized as part of field exploration, laboratory testing and engineering analysis. If the subsurface conditions encountered during construction are different than those that were the basis of the geotechnical engineering report then modifications to the design may be implemented.

Compaction testing of fill material should be performed throughout the project construction so that the engineer and contractor may monitor the quality of the fill placement techniques being used at the site. Generally we recommend that compaction testing be performed for any fill material that is placed as part of the site development. Compaction tests should be performed on each lift of material placed in areas proposed for support of structural components. In addition to compaction testing we recommend that the grain size distribution, clay content and swell potential be evaluated for any imported materials that are planned for use on the site. Concrete tests should be performed on foundation concrete and flatwork. We should be contacted to provide testing services for the various asphalt pavement components associated with the project. We are available to develop a testing program for soil, aggregate materials, concrete and asphaltic concrete for this project.

12.0 CONCLUSIONS AND CONSIDERATIONS

We feel that it is feasible to develop this site for the proposed use. The information presented in this report is based on our understanding of the proposed construction that was provided to us and on the data obtained from our field and laboratory studies. We recommend that we be contacted during the design and construction phases of this project to aid in the implementation of our recommendations. Please contact us immediately if you have any questions, or if any of the information presented above is not appropriate for the proposed site construction.

The recommendations presented above are intended to be used only for this project site and the proposed construction which was provided to us. The recommendations presented above are not suitable for adjacent project sites, or for proposed construction that is different than that outlined for this study.

Our recommendations are based on limited field and laboratory sampling and testing. Unexpected subsurface conditions encountered during construction may alter our recommendations. We should be contacted during construction to observe the exposed subsurface soil conditions to provide comments and verification of our recommendations. We are available to review and tailor our recommendations as the project progresses and additional information which may influence our recommendations becomes available.

PN: 55599GE
April 26, 2019

Please contact us if you have any questions, or if we may be of additional service.

Respectfully submitted,
TRAUTNER GEOTECH

PROFESSIONAL ENGINEER
No. 10272171-2202
JONATHAN P.
BUTLER
4/26/19
STATE OF UTAH

Jonathan P. Butler, P.E.
Senior Geotechnical Engineer

APPENDIX A

Logs of Test Borings

TRAUTNER GEOTECH LLC

Field Engineer : J. Butler
Hole Diameter : Four inch Solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 3/28/2019
Total Depth (approx.) : 15 feet
Location : See Figure 3.3 & 3.4

LOG OF BORING TB-1

Arroyo Crossing Project
Moab, Utah
Ms. Audrey Graham, Chair
Moab Area Community Land Trust

Project Number: 55599GE

Sample Type
- ☒ Mod. California Sampler
- ▨ Bag Sample
- ■ Standard Split Spoon

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	SAND, silty, medium dense to dense, moist, red	SM		▨			
3				☒	2/6 6/6		
4				▨			
6	SAND, silty, dense, moist, red	SM		▨			
8	SAND, GRAVEL, silty, dense, moist, brown	SM/GM					
15	Bottom of test boring at 15 feet						
16							
17							

TRAUTNER GEOTECH LLC

Field Engineer : J. Butler
Hole Diameter : Four inch Solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 3/28/2019
Total Depth (approx.) : 14.5 feet
Location : See Figure 3.3 & 3.4

LOG OF BORING TB-2

Arroyo Crossing Project
Moab, Utah
Ms. Audrey Graham, Chair
Moab Area Community Land Trust

Project Number: 55599GE

Sample Type
- ☒ Mod. California Sampler
- ▨ Bag Sample
- ■ Standard Split Spoon

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0–2.5	SAND, silty, slightly clayey, medium dense, moist to very moist, red	SM	⁙	▨			
2.5–3	GRAVEL, SAND, silty, cobbles, dense, moist, brown	GM	⁘	▨			
3–4			⁘	☒	27/6 40/6		
4–8			⁘	▨			
8–9.5			⁘	■	6/6 4/6 8/6		
9.5–11			⁘	▨			
11–13	GRAVEL, SILT, sandy, few cobbles, medium dense, wet, brown	GM	⁘	▨		▽	Water Level After Drilling at 11 feet
13–14.5			⁘	■	6/6 5/6 5/6		
14.5	Bottom of test boring at 14.5 feet						

TRAUTNER GEOTECH LLC

Field Engineer : J. Butler
Hole Diameter : Four inch Solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 3/28/2019
Total Depth (approx.) : 18 feet
Location : See Figure 3.3 & 3.4

LOG OF BORING TB-3

Arroyo Crossing Project
Moab, Utah
Ms. Audrey Graham, Chair
Moab Area Community Land Trust

Project Number: 55599GE

Sample Type
- ☒ Mod. California Sampler
- ▨ Bag Sample
- ■ Standard Split Spoon

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	SAND, silty, gravels, few cobbles, medium dense to dense, moist, red	SM		▨			
3				☒	4/6 25/6		
4	GRAVEL, SAND, cobbles, silty, dense, moist, tan to brown	GM		▨			
7.5	GRAVEL, COBBLES, sandy, slightly silty, very dense, moist, brown	GP		▨			
8				■	50/4		
10	GRAVEL, SAND, silty, cobbles, dense, moist, brown	GM/SM		▨			
13	SAND, gravels, silty, few cobbles, dense, moist, brown	SM		▨			
18	Bottom of test boring at 18 feet						

TRAUTNER GEOTECH LLC

Field Engineer : J. Butler
Hole Diameter : Four inch Solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 3/28/2019
Total Depth (approx.) : 18 feet
Location : See Figure 3.3 & 3.4

LOG OF BORING TB-4

Arroyo Crossing Project
Moab, Utah
Ms. Audrey Graham, Chair
Moab Area Community Land Trust

Project Number: 55599GE

Sample Type
- ☒ Mod. California Sampler
- ▨ Bag Sample
- ■ Standard Split Spoon

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	SAND, silty, medium dense, moist to very moist, red			▨			
1				▨			
2		SM		▨			
3				☒	2/6 4/6		
4				▨			
5	GRAVEL, SAND, silty, dense, moist, tan	GM		▨			
6				▨			*notable gypsum cemented soils 6.5 feet to 10 feet
7	SAND, silty, white chemical deposits, dense, moist, light tan			▨			
8		SM		☒	14/6 32/6		
9				▨			
10	SAND, silty, dense, moist, red			▨			
11		SM		▨			
12				▨			
13	SAND, silty, few gravels, soft, moist, brown						
14							
15		SM					
16							
17							
18	Bottom of test boring at 18 feet						
19							
20							

TRAUTNER GEOTECH LLC

Field Engineer : J. Butler
Hole Diameter : 3.25 inch hollow
Drilling Method : Continuous Flight Auger
Sampling Method : Standard Split Spoon
Date Drilled : 3/29/2019
Total Depth (approx.) : 18 feet
Location : See Figure 3.3 & 3.4

LOG OF BORING TB-5

Arroyo Crossing Project
Moab, Utah
Ms. Audrey Graham, Chair
Moab Area Community Land Trust

Project Number: 55599GE

Sample Type
- ☒ Mod. California Sampler
- ▨ Bag Sample
- ■ Standard Split Spoon

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	Samples	Blow Count	Water Level	REMARKS
0	SAND, silty, medium dense, moist, red	SM	▨			
2.5	GRAVEL, SAND, silty, cobbles, dense to very dense, moist, brown	GM	▨ ■ (4)	14/6 14/6 21/6		
5	GRAVEL, COBBLES, sandy, slightly silty, very dense, moist, brown	GP-GM	■ ▨ ■ (9)	 15/6 39/6 46/6		
11	GRAVEL, SAND, cobbles, silty, dense, very moist, brown	GM				
13	GRAVEL, SAND, cobbles, silty, dense, wet, brown	GM	■ (14)	3/6 6/6 9/6	▽	Water Level After Drilling at 13 feet
18	Auger refusal at 18 feet on dense cobbles					

TRAUTNER GEOTECH LLC

Field Engineer : J. Butler
Hole Diameter : 3.25 inch hollow
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 3/28/2019
Total Depth (approx.) : 39 feet
Location : See Figure 3.3 & 3.4

LOG OF BORING TB-6

Arroyo Crossing Project
Moab, Utah
Ms. Audrey Graham, Chair
Moab Area Community Land Trust

Project Number: 55599GE

Sample Type
- ☒ Mod. California Sampler
- ▨ Bag Sample
- ■ Standard Split Spoon

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	SAND, silty, medium dense to dense, moist, red	SM		▨ ☒ ▨	7/6 13/6		
7	SAND, silty, few gravels, dense, slightly moist, red	SM		▨ ☒ ▨	17/6 21/6		
12	GRAVEL, COBBLES, sandy, silty, dense to very dense, moist, brown	GM		▨ ■ ■	50/6 13/6 19/6 12/3 20/0		
23	SAND, slightly silty, few gravels, dense, moist to very moist, tan	SM		■ ▨	6/6 7/6 13/6	▽	Water Level After Drilling at 37 feet
39	Bottom of test boring at 39 feet						

TRAUTNER GEOTECH LLC

Field Engineer : J. Butler
Hole Diameter : Four inch solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 3/28/2019
Total Depth (approx.) : 25 feet
Location : See Figure 3.3 & 3.4

LOG OF BORING TB-7

Arroyo Crossing Project
Moab, Utah
Ms. Audrey Graham, Chair
Moab Area Community Land Trust

Project Number: 55599GE

Sample Type
- ☒ Mod. California Sampler
- ▨ Bag Sample
- ■ Standard Split Spoon

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	SAND, silty, medium dense, moist, red	SM		▨			
4	SAND, GRAVEL, silty, cobbles, dense, moist, tan	SM-GM		☒	2/6 3/6		
5				▨			
6	SAND, silty, few gravels, dense, moist to slighly moist, tan	SM		▨			*notable gypsum cemented soils 6 feet to 21 feet
8	SAND, silty, white chemical deposits, very dense, slightly moist, light tan	SM		▨			
9				☒	10/6 48/6		
10				▨			
14				☒	20/6 50/5		
21	GRAVEL, COBBLES, sandy, slightly silty, dense to very dense, moist, brown	GM/GP					
25	Auger refusal at 25 feet on dense cobbles						

TRAUTNER GEOTECH LLC

Field Engineer : J. Butler
Hole Diameter : Four inch solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 3/29/2019
Total Depth (approx.) : 7 feet
Location : See Figure 3.3 & 3.4

LOG OF BORING TB-8

Arroyo Crossing Project
Moab, Utah
Ms. Audrey Graham, Chair
Moab Area Community Land Trust

Project Number: 55599GE

Sample Type
- ☒ Mod. California Sampler
- ▨ Bag Sample
- ■ Standard Split Spoon

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	SAND, silty, soft to medium dense, very moist, red	SM		▨			
3				☒	1/6 6/6		
4				▨			
5	GRAVEL, COBBLES, sandy, silty, very dense, moist, brown	GM		▨			
7	Auger refusal at 7 feet on dense cobbles						
8							
9							
10							

TRAUTNER GEOTECH LLC

Field Engineer : J. Butler
Hole Diameter : Four inch solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 3/28/2019
Total Depth (approx.) : 9 feet
Location : See Figure 3.3 & 3.4

LOG OF BORING TB-9

Arroyo Crossing Project
Moab, Utah
Ms. Audrey Graham, Chair
Moab Area Community Land Trust

Project Number: 55599GE

Sample Type
- ☒ Mod. California Sampler
- ▨ Bag Sample
- ■ Standard Split Spoon

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	SAND, silty, medium dense to dense, moist to very moist, red	SM		▨ ☒	4/6 25/6 rock		
4	SAND, silty, gravels, dense, moist, red	SM		▨			
6	SAND, silty, few gravels, dense, moist to slightly moist, orange	SM		▨ ☒	17/6 27/6		
9	Bottom of test boring at 9 feet						

TRAUTNER GEOTECH LLC

Field Engineer : J. Butler
Hole Diameter : Four inch solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 3/29/2019
Total Depth (approx.) : 7 feet
Location : See Figure 3.3 & 3.4

LOG OF BORING TB-10

Arroyo Crossing Project
Moab, Utah
Ms. Audrey Graham, Chair
Moab Area Community Land Trust

Project Number: 55599GE

Sample Type

- ⊠ Mod. California Sampler
- ▨ Bag Sample
- ■ Standard Split Spoon

Water Level

- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	SAND, silty, soft to medium dense, very moist, red	SM		▨			
1							
2							
3				▨			
4							
5	GRAVEL, COBBLES, sandy, silty, very dense, moist, brown	GM					
6	Auger refusal at 6 feet on dense cobbles						
7							
8							
9							
10							

TRAUTNER GEOTECH LLC

Field Engineer : J. Butler
Hole Diameter : Four inch solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 3/28/2019
Total Depth (approx.) : 9 feet
Location : See Figure 3.3 & 3.4

LOG OF BORING TB-11

Arroyo Crossing Project
Moab, Utah
Ms. Audrey Graham, Chair
Moab Area Community Land Trust

Project Number: 55599GE

Sample Type
- ☒ Mod. California Sampler
- ▨ Bag Sample
- ■ Standard Split Spoon

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	SAND, silty, medium dense, moist, red	SM		▨			
1	SAND, CLAY, stiff, very moist, red	SC/CL		▨			
3				☒	5/6 10/6		
4				▨			
6	SAND, gravels, slightly silty, few cobbles, dense, slightly moist, tan	SM		▨			
8				☒	6/6 12/6		
9	Bottom of test boring at 9 feet						
10							

TRAUTNER GEOTECH LLC

Field Engineer : J. Butler
Hole Diameter : Four inch solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 3/28/2019
Total Depth (approx.) : 18 feet
Location : See Figure 3.3 & 3.4

LOG OF BORING TB-12

Arroyo Crossing Project
Moab, Utah
Ms. Audrey Graham, Chair
Moab Area Community Land Trust

Project Number: 55599GE

Sample Type
- ☒ Mod. California Sampler
- ▨ Bag Sample
- ■ Standard Split Spoon

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	SAND, silty, medium dense, moist to very moist, red			▨			
1							
2		SM					
3				☒	5/6 9/6		
4				▨			
5	SAND, silty, few gravels, dense, slightly moist, tan						
6							
7		SM					
8				▨			
9							
10	SAND, silty, few gravels, dense, moist, tan						
11							
12		SM					
13							
14							
15	SAND, silty, few gravels, dense, moist, brown						
16		SM					
17							
18	Bottom of test boring at 18 feet						
19							
20							

TRAUTNER GEOTECH LLC

Field Engineer : J. Butler
Hole Diameter : 3.25 inch hollow
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 3/28/2019
Total Depth (approx.) : 28 feet
Location : See Figure 3.3 & 3.4

LOG OF BORING TB-13

Arroyo Crossing Project
Moab, Utah
Ms. Audrey Graham, Chair
Moab Area Community Land Trust

Project Number: 55599GE

Sample Type
- ☒ Mod. California Sampler
- ▨ Bag Sample
- ■ Standard Split Spoon

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	SAND, silty, few gravels, medium dense to dense, moist, red	SM		▨			
4				☒	7/6 9/6		
5	SAND, silty, few gravels, dense to very dense, slightly moist to moist, tan	SM		▨			
9				☒	16/6 22/6		
11	GRAVEL, cobbles, clayey, slightly silty, very dense, moist, tan	GM					
14				■	40/6 50/6		
20	GRAVEL, SAND, silty, dense, moist, brown	GM/SM					
25	GRAVEL, COBBLES, sandy, slightly silty, very dense, moist, brown	GP-GM					
28	Auger refusal at 28 feet on dense cobbles						

TRAUTNER GEOTECH LLC

Field Engineer : J. Butler
Hole Diameter : 3.25 inch hollow
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 3/28/2019
Total Depth (approx.) : 30.5 feet
Location : See Figure 3.3 & 3.4

LOG OF BORING TB-14

Arroyo Crossing Project
Moab, Utah
Ms. Audrey Graham, Chair
Moab Area Community Land Trust

Project Number: 55599GE

Sample Type
- ☒ Mod. California Sampler
- ▨ Bag Sample
- ■ Standard Split Spoon

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

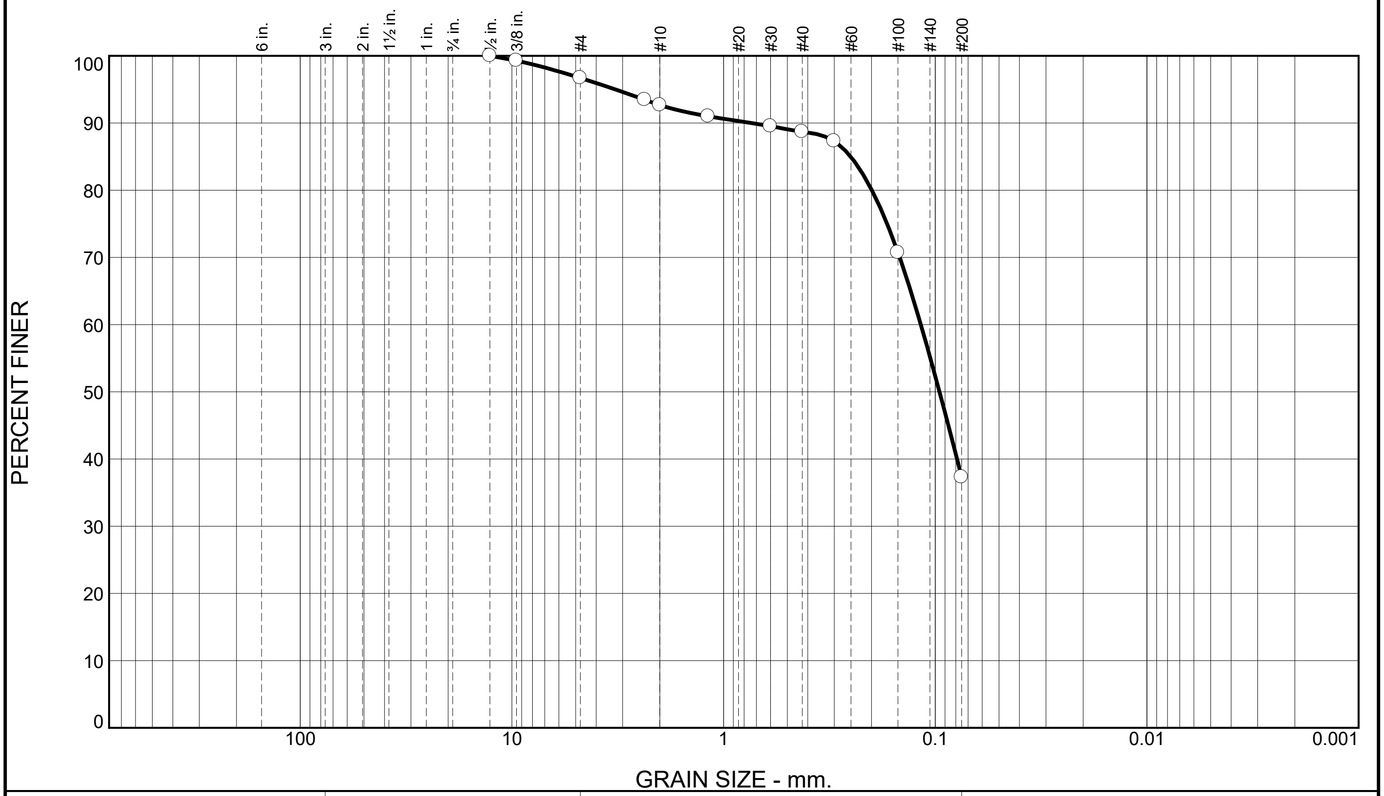
Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	SAND, silty, few gravels, medium dense to dense, very moist, red	SM		▨ ☒	12/6 17/6		
5	SAND, silty, few gravels, dense, moist, red	SM		▨ ■	4/6 15/6 14/6		
10	SAND, silty, dense, moist, red	SM		■	11/6 16/6 13/6		
16	GRAVEL, COBBLES, sandy, slightly silty, dense to very dense, moist to very moist, brown	GP-GM					
21	SAND, silty, gravels, few cobbles, very dense, very moist, brown	SM					
27	SAND, silty, gravels, few cobbles, dense, wet, brown	SM				▽	Water Level After Drilling at 27 feet
29	GRAVEL, SAND, slightly silty, few cobbles, dense, wet, brown	GM		■	12/6 27/6 30/6		
31	Bottom of test boring at 30.5 feet						

APPENDIX B

Laboratory Test Result

Atterberg Limits and Sieve Analysis: Figures 4.1 through 4.8
Swell-Consolidation Tests: Figures 4.9 through 4.24
Modified Proctor Tests: Figures 4.25 and 4.26
California Bearing Ratio: Figure 4.27

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	3.3	4.0	4.0	51.4	37.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.50	100.0		
.375	99.3		
#4	96.7		
#8	93.5		
#10	92.7		
#16	91.0		
#30	89.5		
#40	88.7		
#50	87.3		
#100	70.7		
#200	37.3		

* (no specification provided)

Soil Description

SM Silty Sand

Atterberg Limits

PL= 0 LL= 0 PI= 0

Coefficients

D_{90}= 0.7419 D_{85}= 0.2510 D_{60}= 0.1170
D_{50}= 0.0955 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-4(0)

Remarks

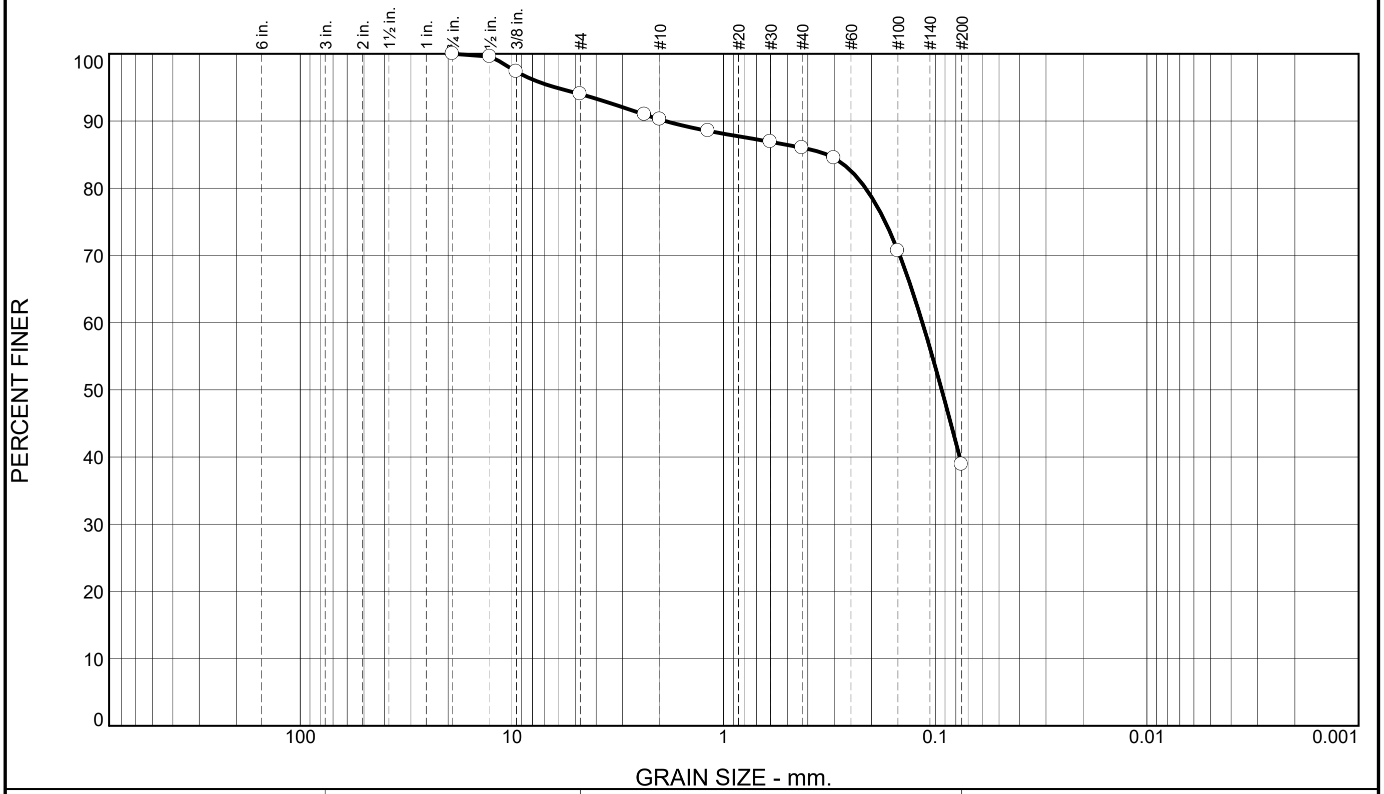
Location: Bulk TB-3,6,11,12 and 13
Sample Number: C10219-A **Depth:** 0'-4' **Date:** 4/2/19

TRAUTNER GEOTECH LLC

Client: Moab Area Community Land Trust, Audrey Graham, Chair
Project: Arroyo Crossing, Moab, UT
Project No: 55599GE **Figure** 4.1

Tested By: R. Barrett **Checked By:** J. Butler P.E.

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	6.0	3.8	4.2	47.1	38.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.75	100.0		
.50	99.6		
.375	97.3		
#4	94.0		
#8	91.0		
#10	90.2		
#16	88.5		
#30	86.9		
#40	86.0		
#50	84.5		
#100	70.7		
#200	38.9		

* (no specification provided)

Soil Description

SM Silty Sand

Atterberg Limits

PL= 0 LL= 0 PI= 0

Coefficients

D_{90}= 1.8935	D_{85}= 0.3249	D_{60}= 0.1150
D_{50}= 0.0933	D_{30}=	D_{15}=
D_{10}=	C_u=	C_c=

Classification

USCS= SM AASHTO= A-4(0)

Remarks

Location: Bulk TB-1,2,4,7,9 and 14
Sample Number: C10219-B **Depth:** 0'-4' **Date:** 4/2/19

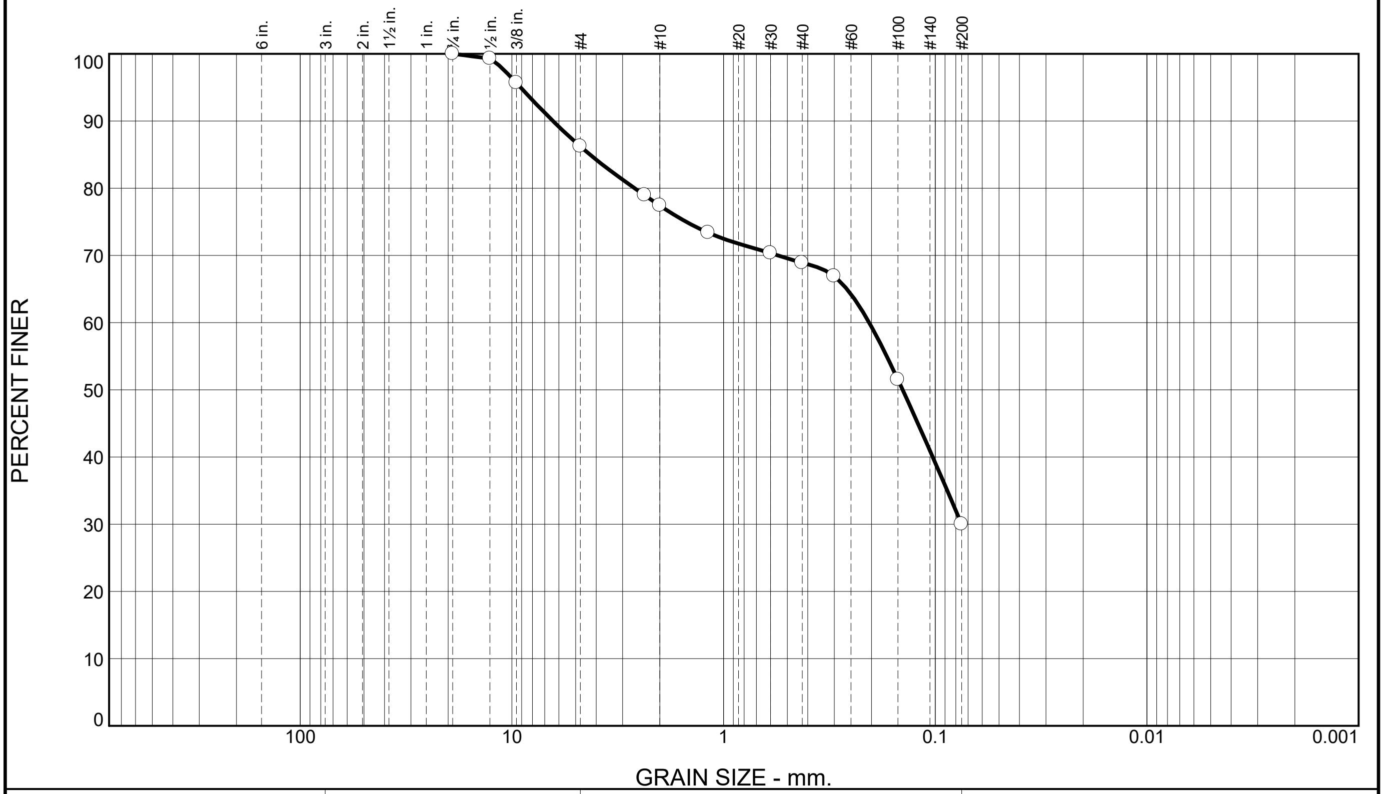
TRAUTNER GEOTECH LLC

Client: Moab Area Community Land Trust, Audrey Graham, Chair
Project: Arroyo Crossing, Moab, UT

Project No: 55599GE **Figure** 4.2

Tested By: R. Barrett **Checked By:** J. Butler P.E.

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	13.8	8.8	8.5	38.9	30.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.75	100.0		
.50	99.3		
.375	95.7		
#4	86.2		
#8	79.0		
#10	77.4		
#16	73.4		
#30	70.3		
#40	68.9		
#50	66.9		
#100	51.5		
#200	30.0		

* (no specification provided)

Soil Description

SM Silty Sand

Atterberg Limits

PL= 0 LL= 0 PI= 0

Coefficients

D_{90}= 6.3966 D_{85}= 4.2786 D_{60}= 0.2052
D_{50}= 0.1426 D_{30}= 0.0750 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

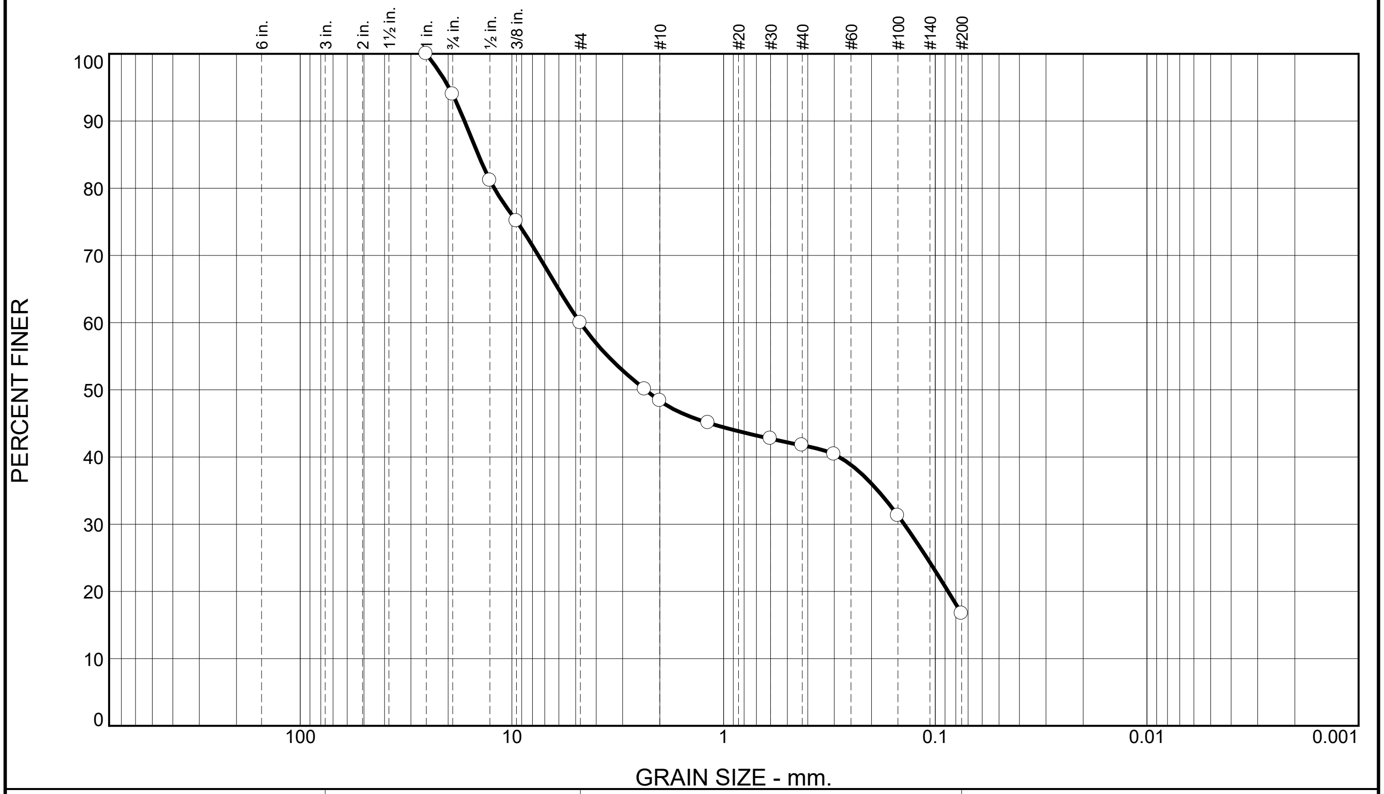
Location: TB-3
Sample Number: C10219-L **Depth:** 0'-3' **Date:** 4/2/19

TRAUTNER GEOTECH LLC

Client: Moab Area Community Land Trust, Audrey Graham, Chair
Project: Arroyo Crossing, Moab, UT
Project No: 55599GE **Figure** 4.3

Tested By: R. Barrett **Checked By:** J. Butler

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	6.0	34.0	11.6	6.7	25.0	16.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.0	100.0		
.75	94.0		
.50	81.1		
.375	75.1		
#4	60.0		
#8	50.1		
#10	48.4		
#16	45.1		
#30	42.7		
#40	41.7		
#50	40.4		
#100	31.3		
#200	16.7		

* (no specification provided)

Soil Description

SM Silty Sand with Gravel

Atterberg Limits

PL= 0 LL= 0 PI= 0

Coefficients

D_{90}= 16.7490 D_{85}= 14.4319 D_{60}= 4.7617
D_{50}= 2.3403 D_{30}= 0.1402 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-1-b

Remarks

Location: TB-3
Sample Number: C10219-N **Depth:** 4'-8' **Date:** 4/2/19

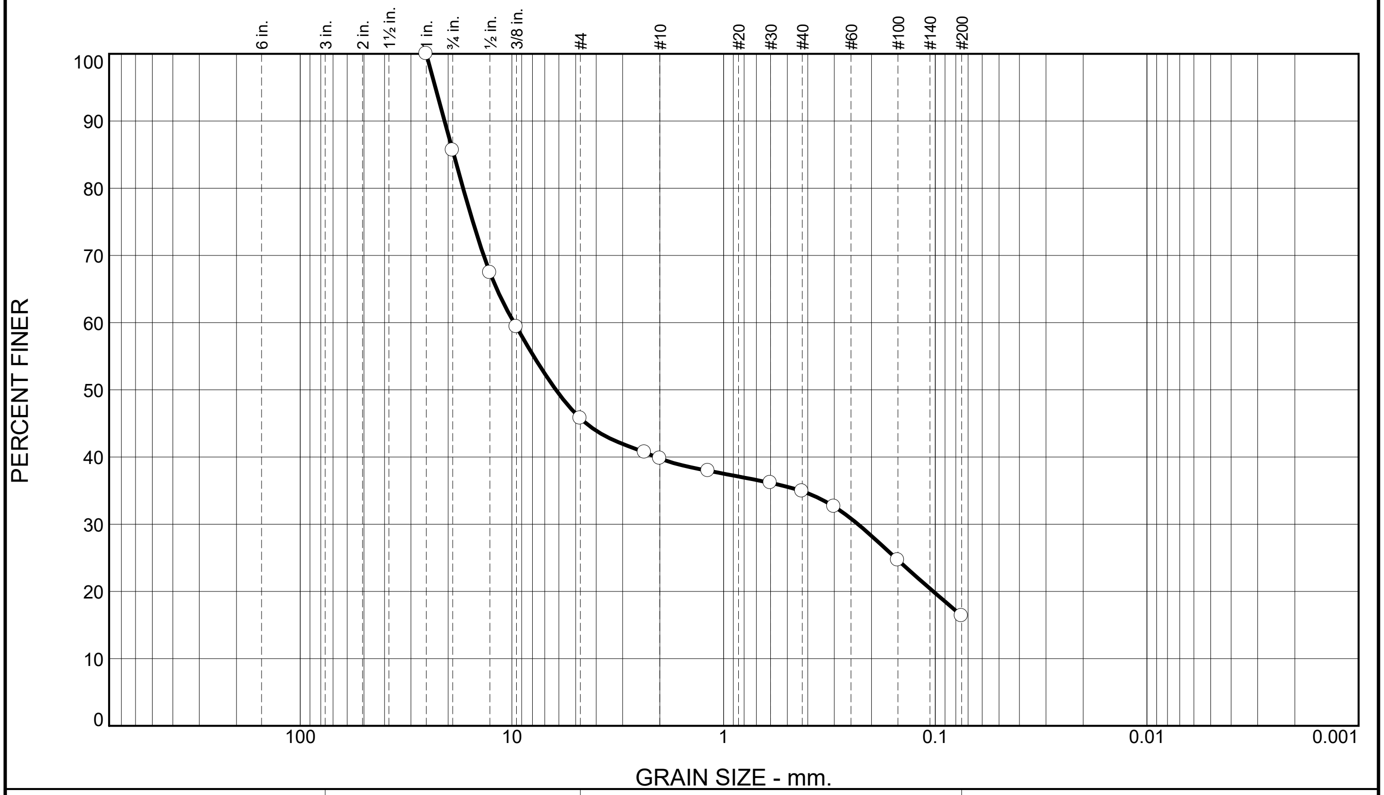
TRAUTNER GEOTECH LLC

Client: Moab Area Community Land Trust, Audrey Graham, Chair
Project: Arroyo Crossing, Moab, UT

Project No: 55599GE **Figure** 4.4

Tested By: R. Barrett **Checked By:** J. Butler P.E.

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	14.4	39.8	6.0	4.9	18.5	16.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.0	100.0		
.75	85.6		
.50	67.4		
.375	59.4		
#4	45.8		
#8	40.7		
#10	39.8		
#16	37.9		
#30	36.1		
#40	34.9		
#50	32.6		
#100	24.7		
#200	16.4		

* (no specification provided)

Soil Description

GM Silty Gravel with Sand

Atterberg Limits

PL= 0 LL= 0 PI= 0

Coefficients

D_{90}= 20.7986 D_{85}= 18.8166 D_{60}= 9.7758

D_{50}= 6.1977 D_{30}= 0.2318 D_{15}=

D_{10}= C_u= C_c=

Classification

USCS= GM AASHTO= A-1-b

Remarks

Location: TB-5

Sample Number: C10219-X **Depth:** 5.5'-9' **Date:** 4/2/19

TRAUTNER GEOTECH LLC

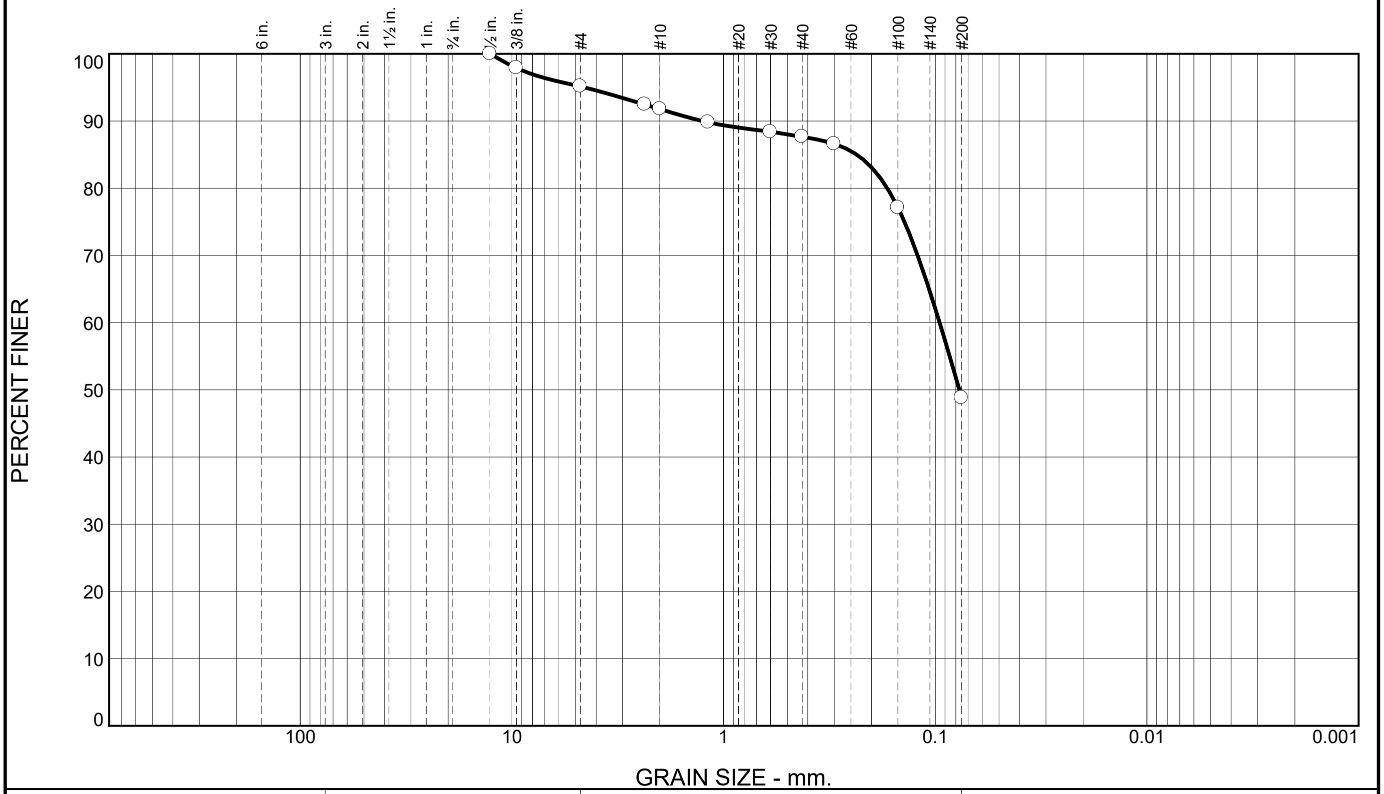
Client: Moab Area Community Land Trust, Audrey Graham, Chair

Project: Arroyo Crossing, Moab, UT

Project No: 55599GE **Figure** 4.5

Tested By: R. Barrett **Checked By:** J. Butler P.E.

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	4.9	3.3	4.2	38.8	48.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.50	100.0		
.375	97.9		
#4	95.1		
#8	92.4		
#10	91.8		
#16	89.8		
#30	88.4		
#40	87.6		
#50	86.6		
#100	77.1		
#200	48.8		

* (no specification provided)

Soil Description

SM Silty Sand

Atterberg Limits

PL= 16 LL= 17 PI= 1

Coefficients

D_{90}= 1.2625 D_{85}= 0.2353 D_{60}= 0.0954
D_{50}= 0.0769 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-4(0)

Remarks

Location: TB-7
Sample Number: C10219-LL **Depth:** 4'-8' **Date:** 4/2/19

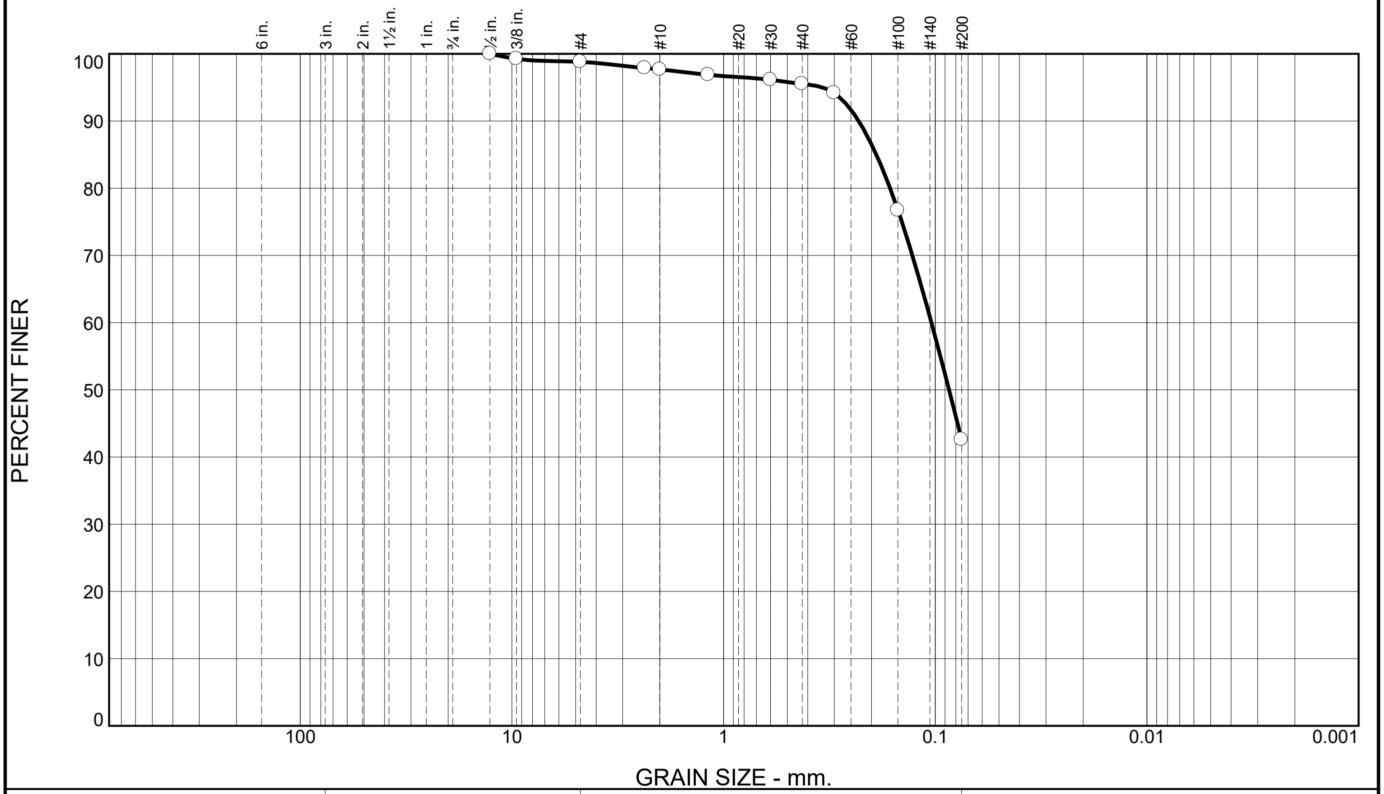
TRAUTNER GEOTECH LLC

Client: Moab Area Community Land Trust, Audrey Graham, Chair
Project: Arroyo Crossing, Moab, UT

Project No: 55599GE **Figure** 4.6

Tested By: R. Barrett **Checked By:** J. Butler P.E.

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	1.2	1.1	2.2	52.9	42.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.50	100.0		
.375	99.3		
#4	98.8		
#8	97.9		
#10	97.7		
#16	96.9		
#30	96.1		
#40	95.5		
#50	94.2		
#100	76.7		
#200	42.6		

* (no specification provided)

Soil Description

SM Silty Sand

Atterberg Limits

PL= 14 LL= 16 PI= 2

Coefficients

D_{90}= 0.2299 D_{85}= 0.1898 D_{60}= 0.1043
D_{50}= 0.0861 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-4(0)

Remarks

Location: TB-11
Sample Number: C10219-WW **Depth:** 0'-3' **Date:** 4/2/19

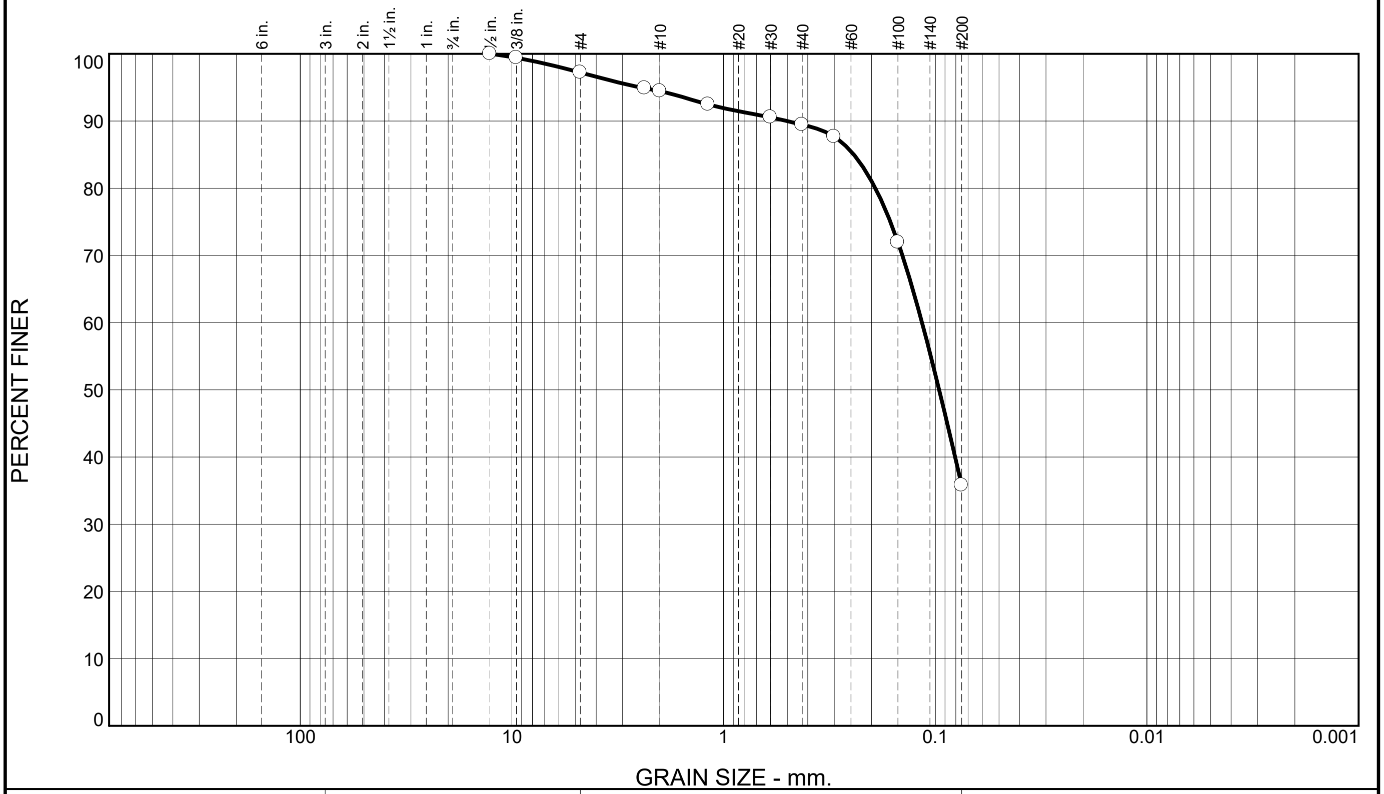
TRAUTNER GEOTECH LLC

Client: Moab Area Community Land Trust, Audrey Graham, Chair
Project: Arroyo Crossing, Moab, UT

Project No: 55599GE **Figure** 4.7

Tested By: R. Barrett **Checked By:** J. Butler P.E.

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	2.8	2.8	4.9	53.7	35.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.50	100.0		
.375	99.4		
#4	97.2		
#8	94.9		
#10	94.4		
#16	92.5		
#30	90.5		
#40	89.5		
#50	87.7		
#100	71.9		
#200	35.8		

* (no specification provided)

Soil Description

SM Silty Sand

Atterberg Limits

PL= 0 LL= 0 PI= 0

Coefficients

D_{90}= 0.5031 D_{85}= 0.2425 D_{60}= 0.1155
D_{50}= 0.0959 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-4(0)

Remarks

Location: TB-14
Sample Number: C10219-K3 **Depth:** 0'-4' **Date:** 4/2/19

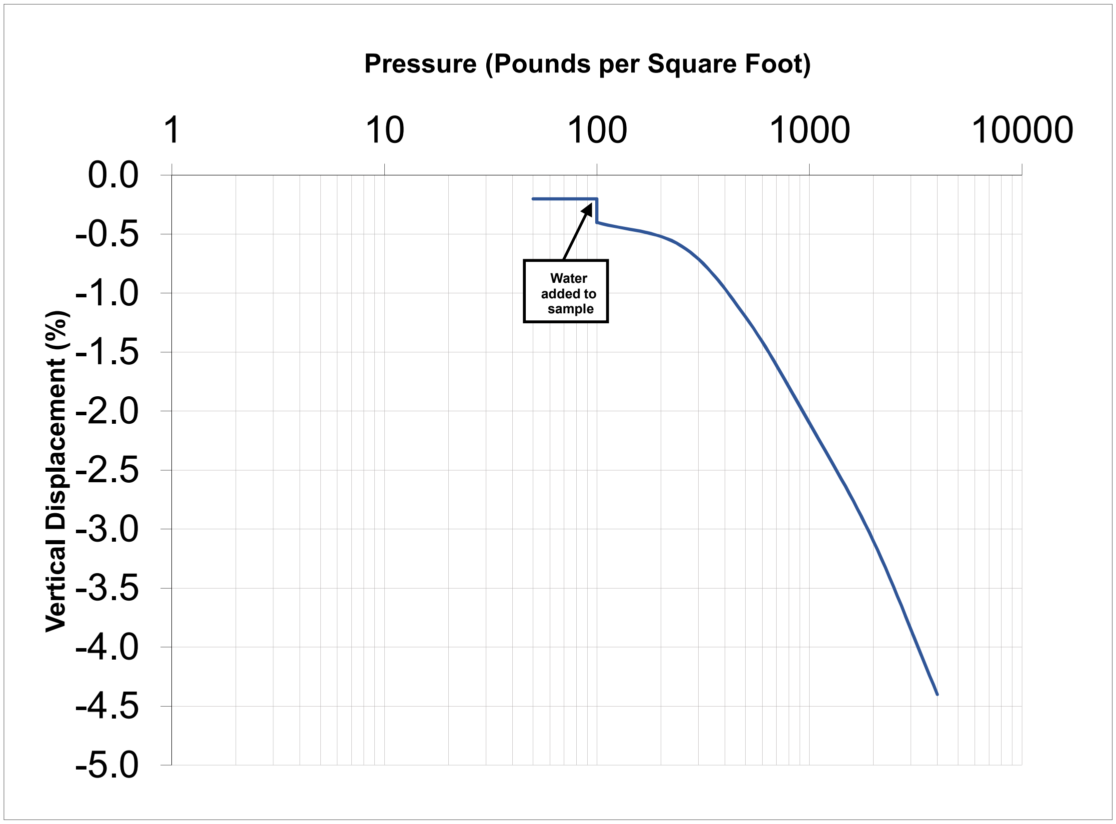
TRAUTNER GEOTECH LLC

Client: Moab Area Community Land Trust, Audrey Graham, Chair
Project: Arroyo Crossing, Moab, UT

Project No: 55599GE **Figure** 4.8

Tested By: R. Barrett **Checked By:** J. Butler P.E.

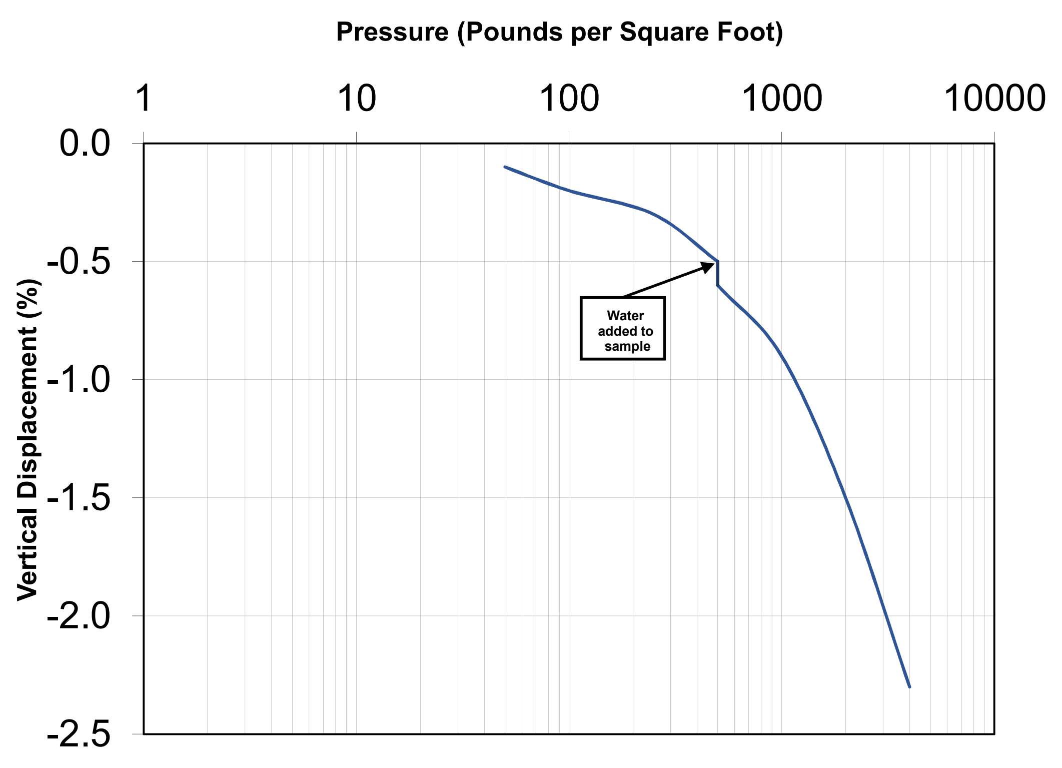
SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-1@3'	
Visual Soil Description:	SM Silty Sand	
Swell Potential (%)	Consolidated	
Constant Voume Swell Pressure (lb/ft^2):	N/A	
	Initial	**Final**
Moisture Content (%):	4.0	19.1
Dry Density (lb/ft^3):	106.2	110.3
Height (in.):	1.000	0.956
Diameter (in.):	1.94	1.94

Project Number:	55599GE
Sample ID:	C10219-d
Figure:	4.9

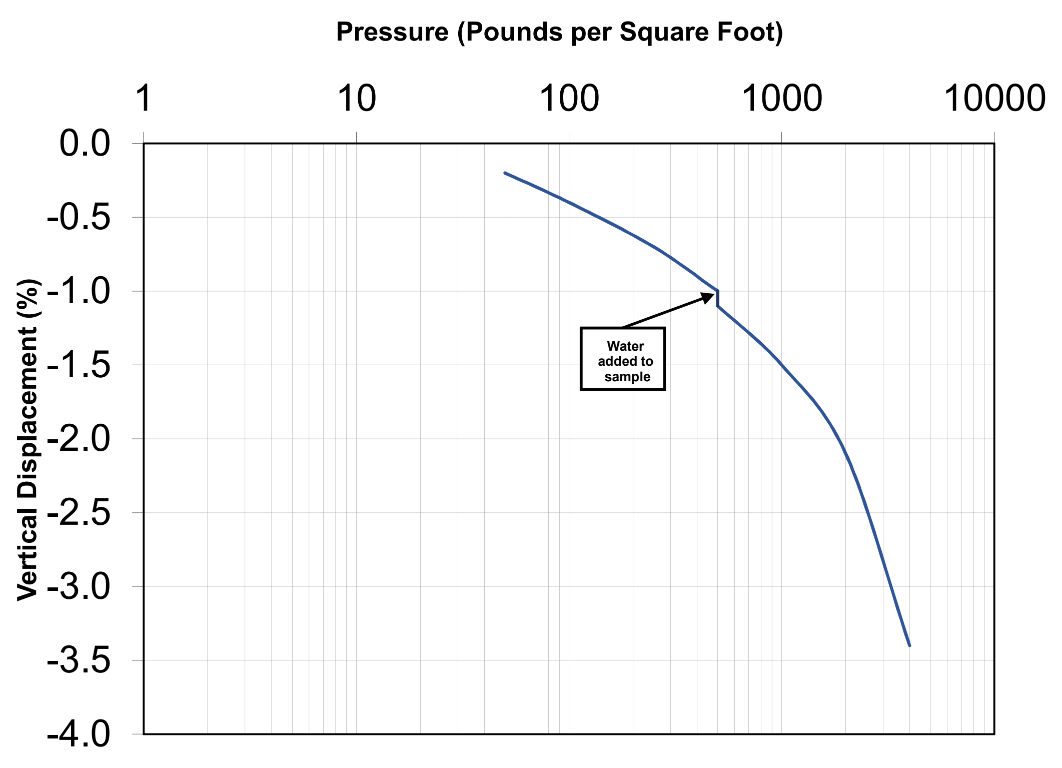
SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-3@3'	
Visual Soil Description:	SM Silty Sand	
Swell Potential (%)	Consolidated	
Constant Voume Swell Pressure (lb/ft^2):	N/A	
	Initial	**Final**
Moisture Content (%):	8.2	14.3
Dry Density (lb/ft^3):	116.7	119.1
Height (in.):	1.000	0.977
Diameter (in.):	1.94	1.94

Project Number:	55599GE
Sample ID:	10219-M
Figure:	4.10

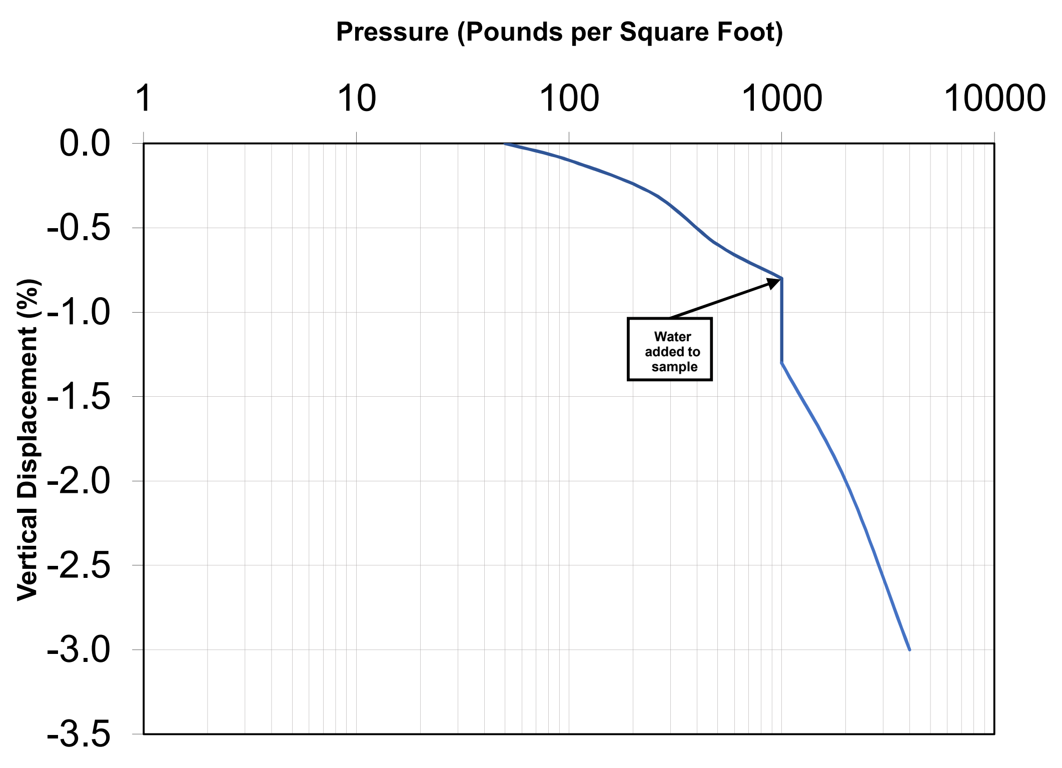
SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-4@3'	
Visual Soil Description:	SM Silty Sand	
Swell Potential (%)	Consolidated	
Constant Voume Swell Pressure (lb/ft^2):	N/A	
	Initial	**Final**
Moisture Content (%):	8.4	17.7
Dry Density (lb/ft^3):	111.2	114.1
Height (in.):	1.000	0.966
Diameter (in.):	1.94	1.94

Project Number:	55599GE
Sample ID:	10219-R
Figure:	4.11

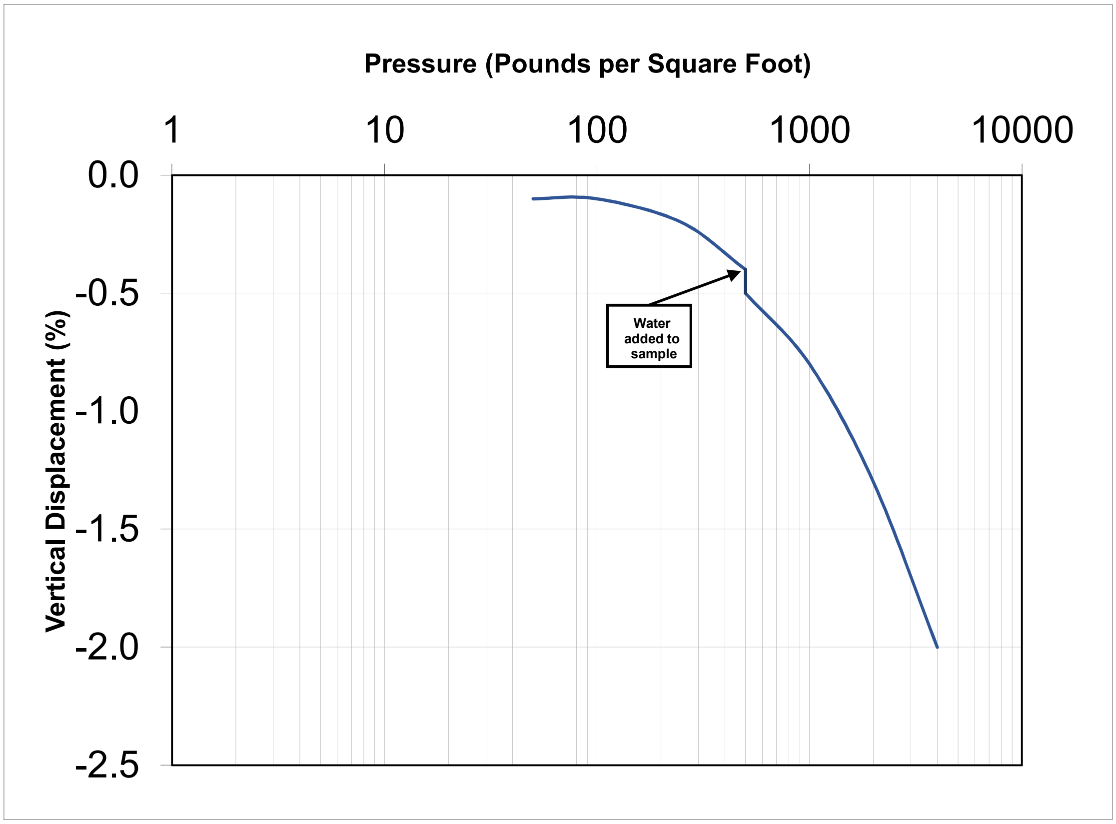
SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-4@8'	
Visual Soil Description:	SM Silty Sand	
Swell Potential (%)	Consolidated	
Constant Voume Swell Pressure (lb/ft^2):	N/A	
	Initial	**Final**
Moisture Content (%):	3.0	14.0
Dry Density (lb/ft^3):	118.4	120.6
Height (in.):	1.000	0.970
Diameter (in.):	1.94	1.94

Project Number:	55599GE
Sample ID:	C10219-T
Figure:	4.12

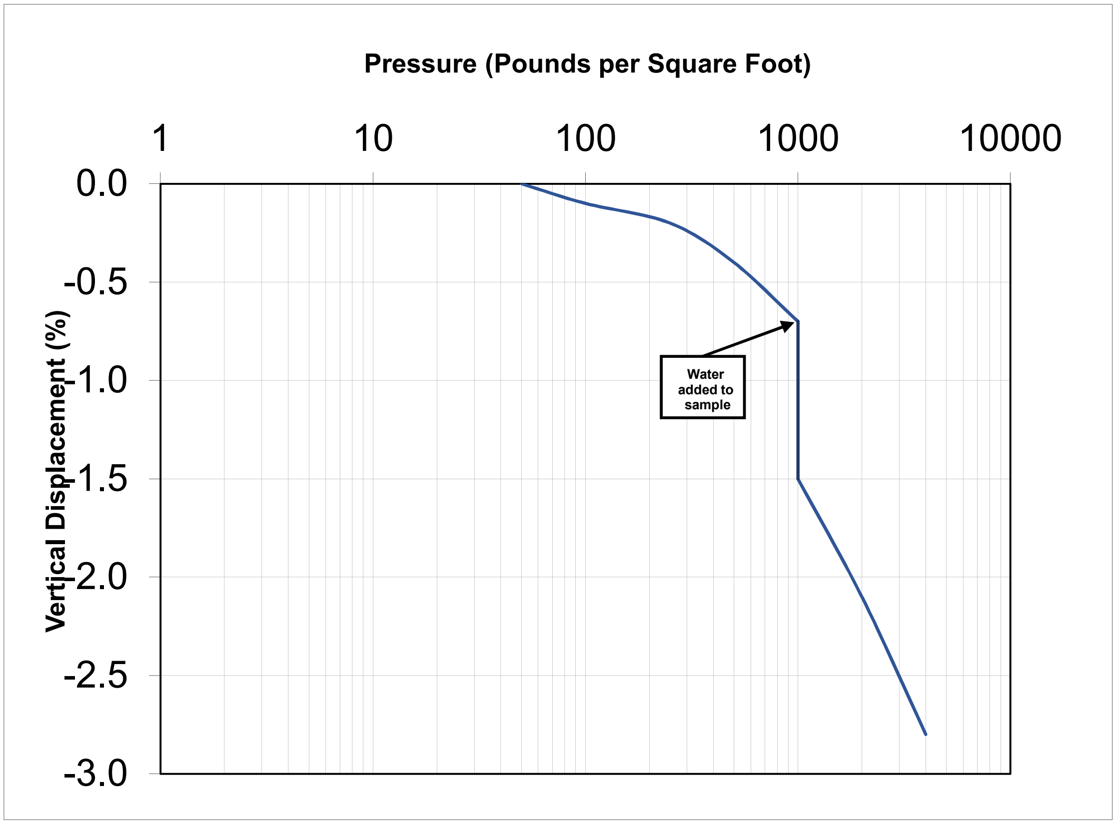
SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-6@4'	
Visual Soil Description:	SM Silty Sand	
Swell Potential (%)	Consolidated	
Constant Voume Swell Pressure (lb/ft^2):	N/A	
	Initial	**Final**
Moisture Content (%):	7.0	12.3
Dry Density (lb/ft^3):	122.8	124.5
Height (in.):	1.000	0.980
Diameter (in.):	1.94	1.94

Project Number:	55599GE
Sample ID:	10219-BB
Figure:	4.13

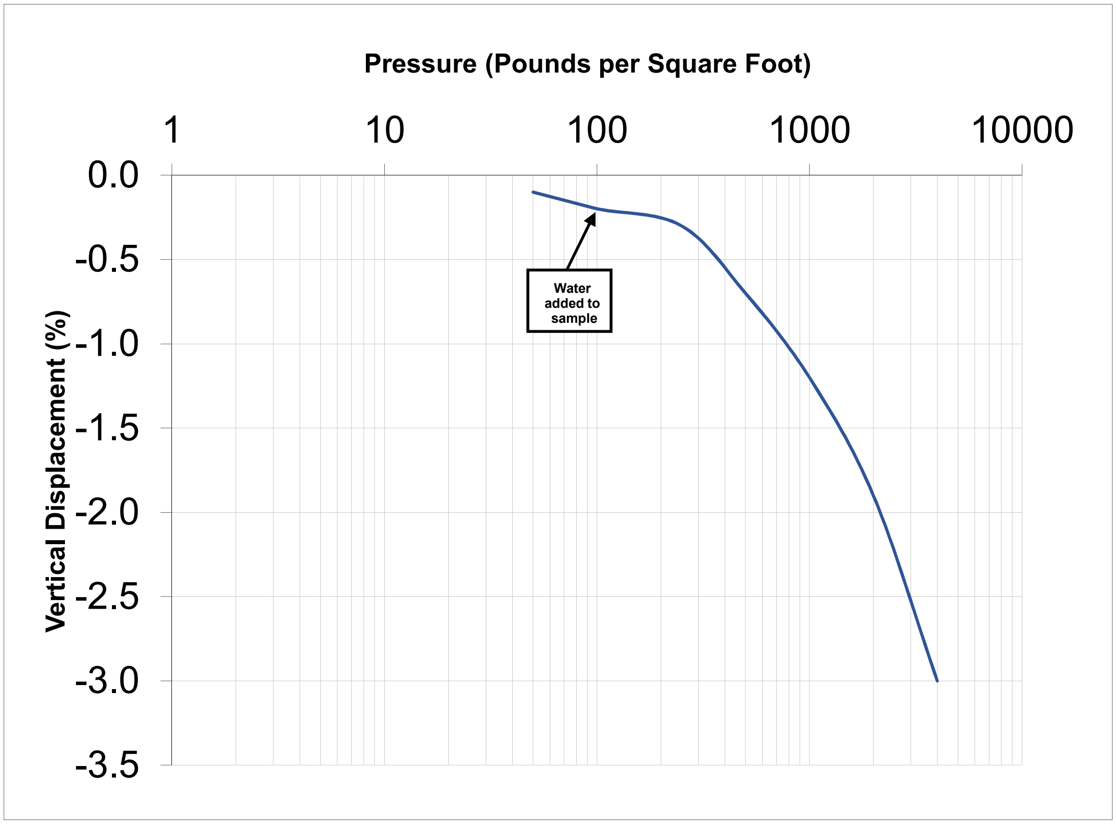
SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-6@9'	
Visual Soil Description:	SM Silty Sand	
Swell Potential (%)	Consolidated	
Constant Voume Swell Pressure (lb/ft^2):	N/A	
	Initial	**Final**
Moisture Content (%):	2.6	17.2
Dry Density (lb/ft^3):	110.1	111.6
Height (in.):	1.000	0.972
Diameter (in.):	1.94	1.94

Project Number:	55599GE
Sample ID:	10219-dd
Figure:	4.14

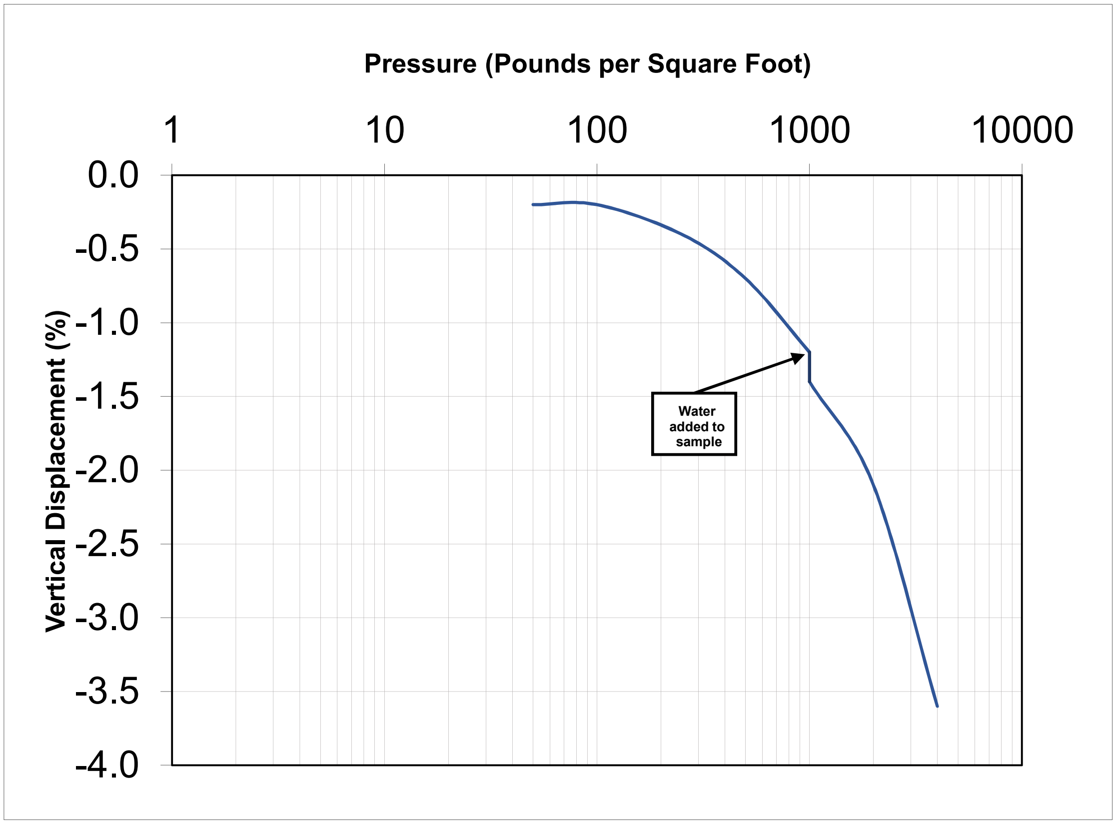
SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-7@3'	
Visual Soil Description:	SM Silty Sand	
Swell Potential (%)	Consolidated	
Constant Voume Swell Pressure (lb/ft^2):	N/A	
	Initial	**Final**
Moisture Content (%):	7.6	17.0
Dry Density (lb/ft^3):	110.3	112.7
Height (in.):	1.000	0.970
Diameter (in.):	1.94	1.94

Project Number:	55599GE
Sample ID:	C10219-KK
Figure:	4.15

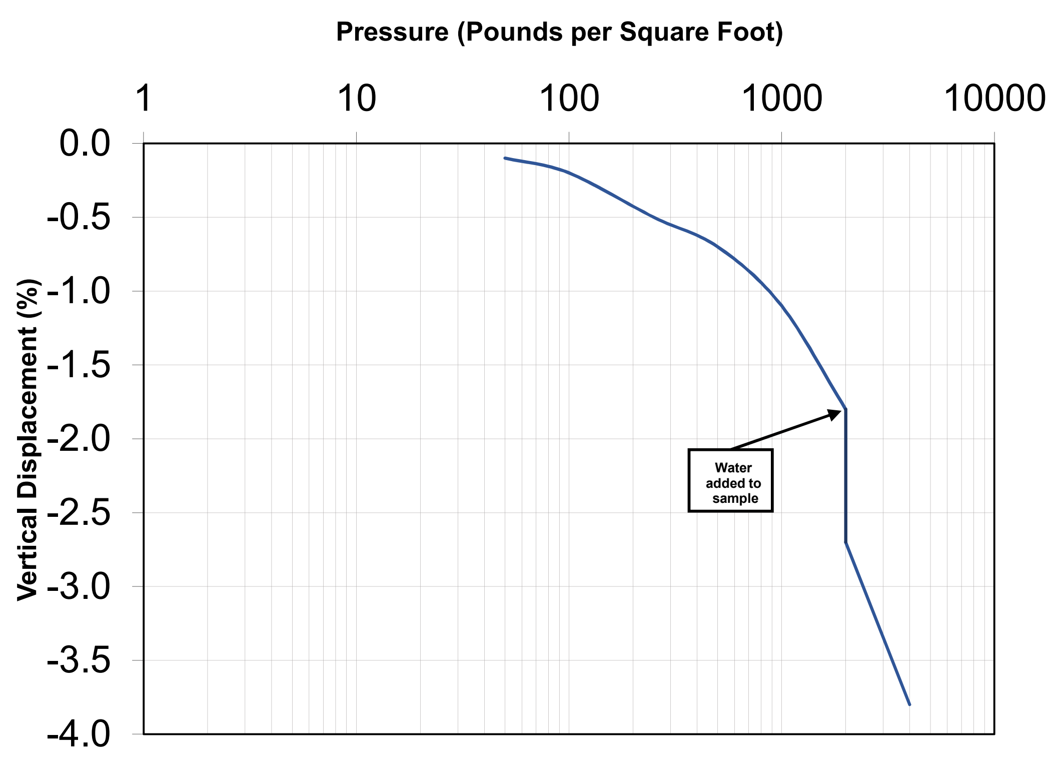
SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-7@8'	
Visual Soil Description:	SM Silty Sand	
Swell Potential (%)	Consolidated	
Constant Voume Swell Pressure (lb/ft^2):	N/A	
	Initial	**Final**
Moisture Content (%):	5.0	13.2
Dry Density (lb/ft^3):	122.0	125.2
Height (in.):	1.000	0.964
Diameter (in.):	1.94	1.94

Project Number:	55599GE
Sample ID:	C10219-MM
Figure:	4.16

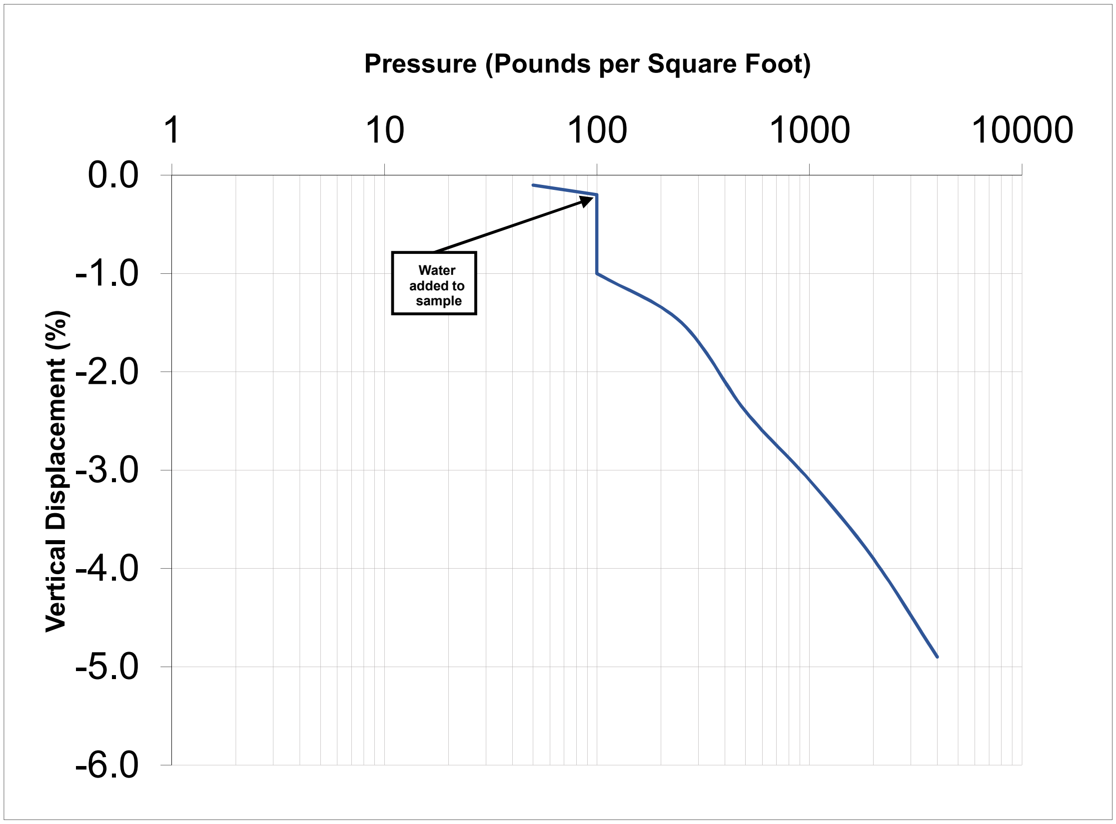
SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-7@13'	
Visual Soil Description:	SM Silty Sand	
Swell Potential (%)	Consolidated	
Constant Voume Swell Pressure (lb/ft^2):	N/A	
	Initial	**Final**
Moisture Content (%):	3.9	14.5
Dry Density (lb/ft^3):	120.8	124.0
Height (in.):	1.000	0.962
Diameter (in.):	1.94	1.94

Project Number:	55599GE
Sample ID:	C10219-OO
Figure:	4.17

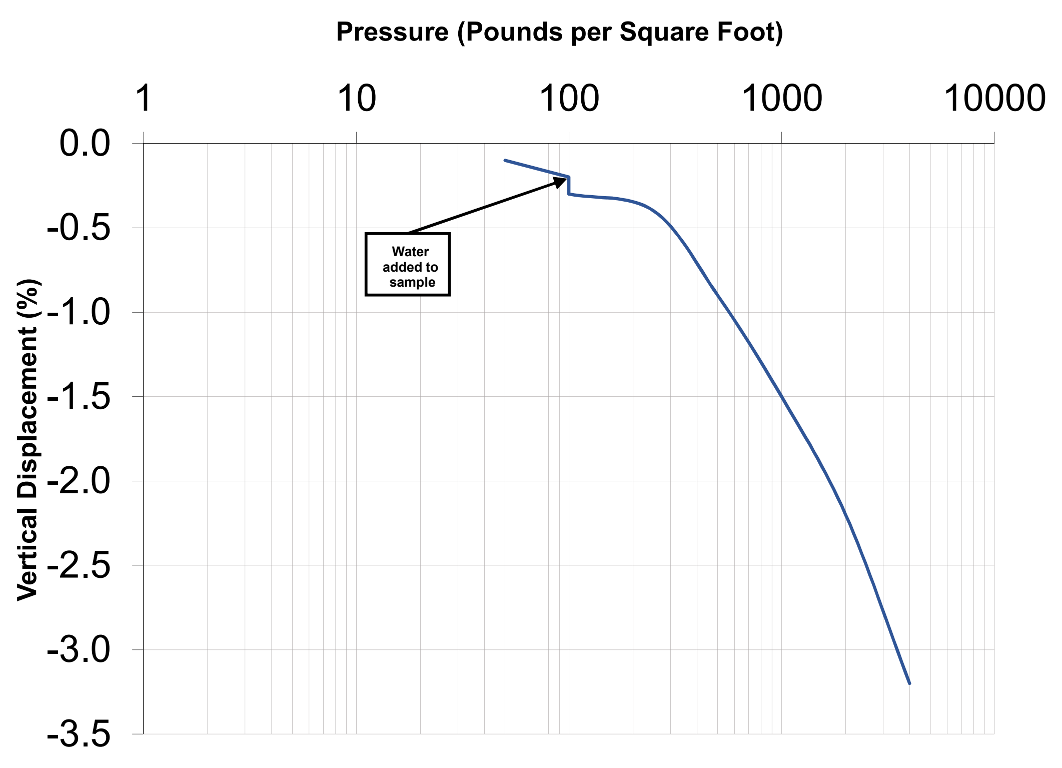
SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-8@3'	
Visual Soil Description:	SM Silty Sand	
Swell Potential (%)	Consolidated	
Constant Voume Swell Pressure (lb/ft^2):	N/A	
	Initial	**Final**
Moisture Content (%):	5.8	24.6
Dry Density (lb/ft^3):	91.8	97.3
Height (in.):	1.000	0.951
Diameter (in.):	1.94	1.94

Project Number:	55599GE
Sample ID:	C10219-QQ
Figure:	4.18

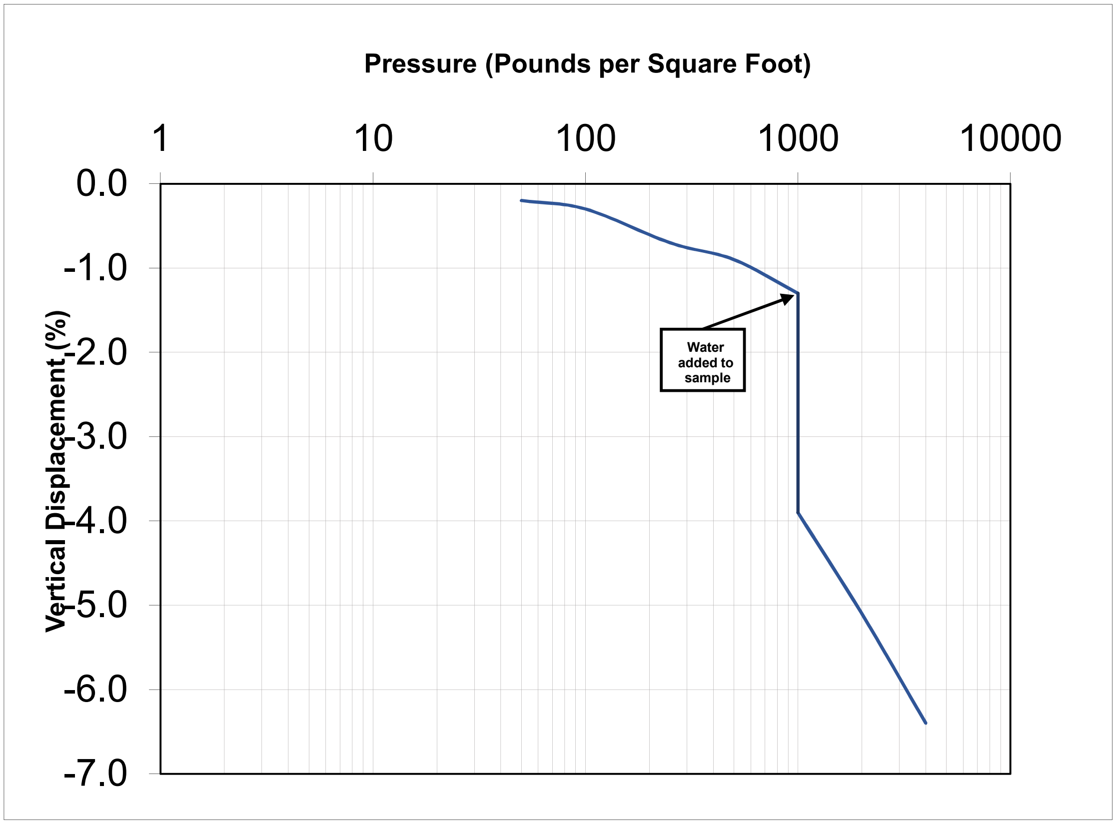
SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-11@3'	
Visual Soil Description:	SM Silty Sand	
Swell Potential (%)	Consolidated	
Constant Voume Swell Pressure (lb/ft^2):	N/A	
	Initial	**Final**
Moisture Content (%):	8.7	12.8
Dry Density (lb/ft^3):	122.2	125.7
Height (in.):	1.000	0.968
Diameter (in.):	1.94	1.94

Project Number:	55599GE
Sample ID:	C10219-XX
Figure:	4.19

SWELL - CONSOLIDATION TEST

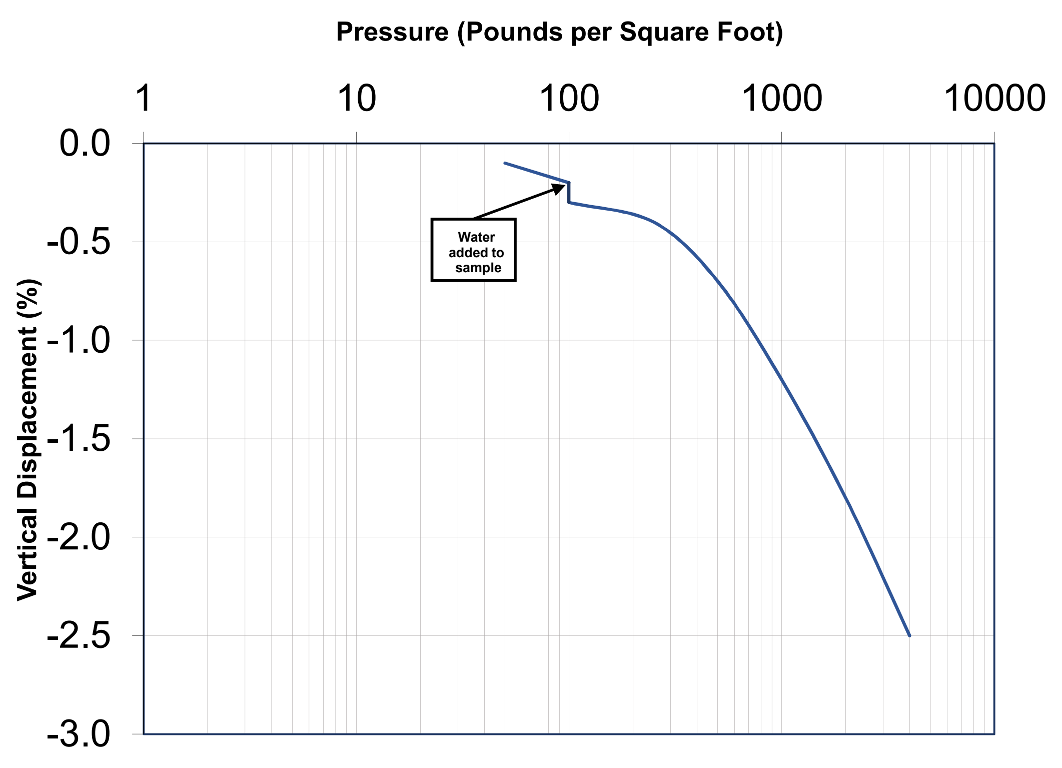


SUMMARY OF TEST RESULTS

Sample Source:	TB-11@8'	
Visual Soil Description:	SM Silty Sand	
Swell Potential (%)	Consolidated	
Constant Voume Swell Pressure (lb/ft^2):	N/A	
	Initial	**Final**
Moisture Content (%):	1.6	19.8
Dry Density (lb/ft^3):	106.3	110.7
Height (in.):	1.000	0.936
Diameter (in.):	1.94	1.94

Project Number:	55599GE
Sample ID:	10219-ZZ
Figure:	4.20

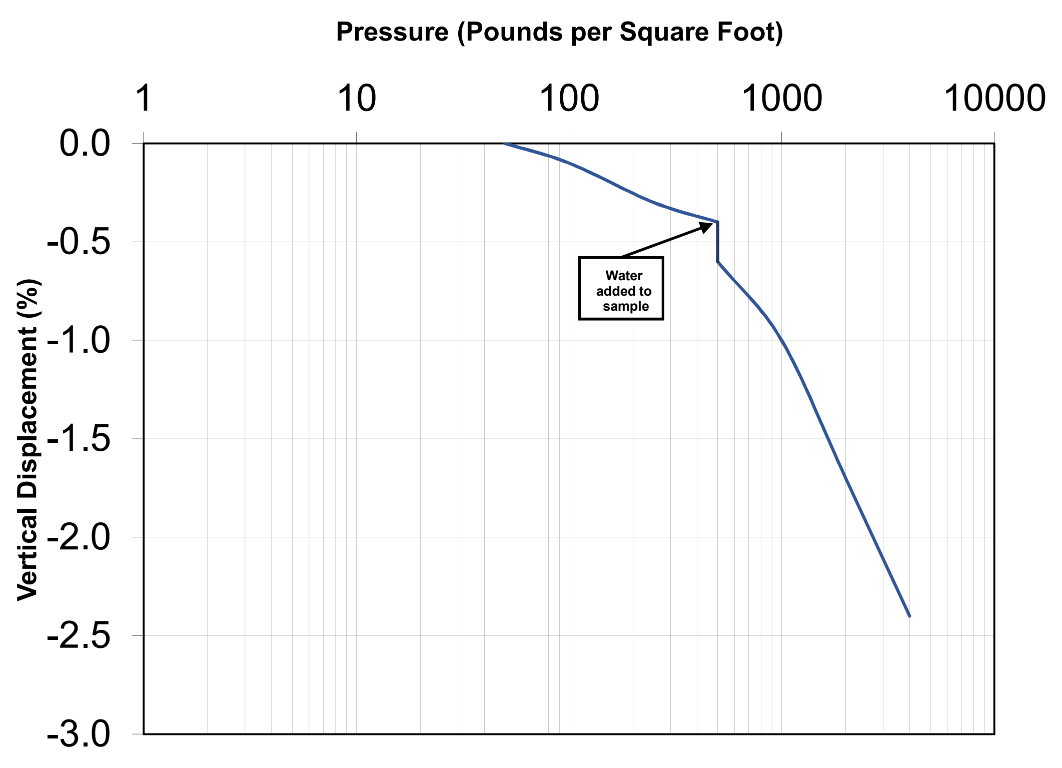
SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-12@3'	
Visual Soil Description:	SM Silty Sand	
Swell Potential (%)	Consolidated	
Constant Voume Swell Pressure (lb/ft^2):	N/A	
	Initial	**Final**
Moisture Content (%):	7.4	13.9
Dry Density (lb/ft^3):	120.9	123.3
Height (in.):	1.000	0.975
Diameter (in.):	1.94	1.94

Project Number:	55599GE
Sample ID:	10219-B3
Figure:	4.21

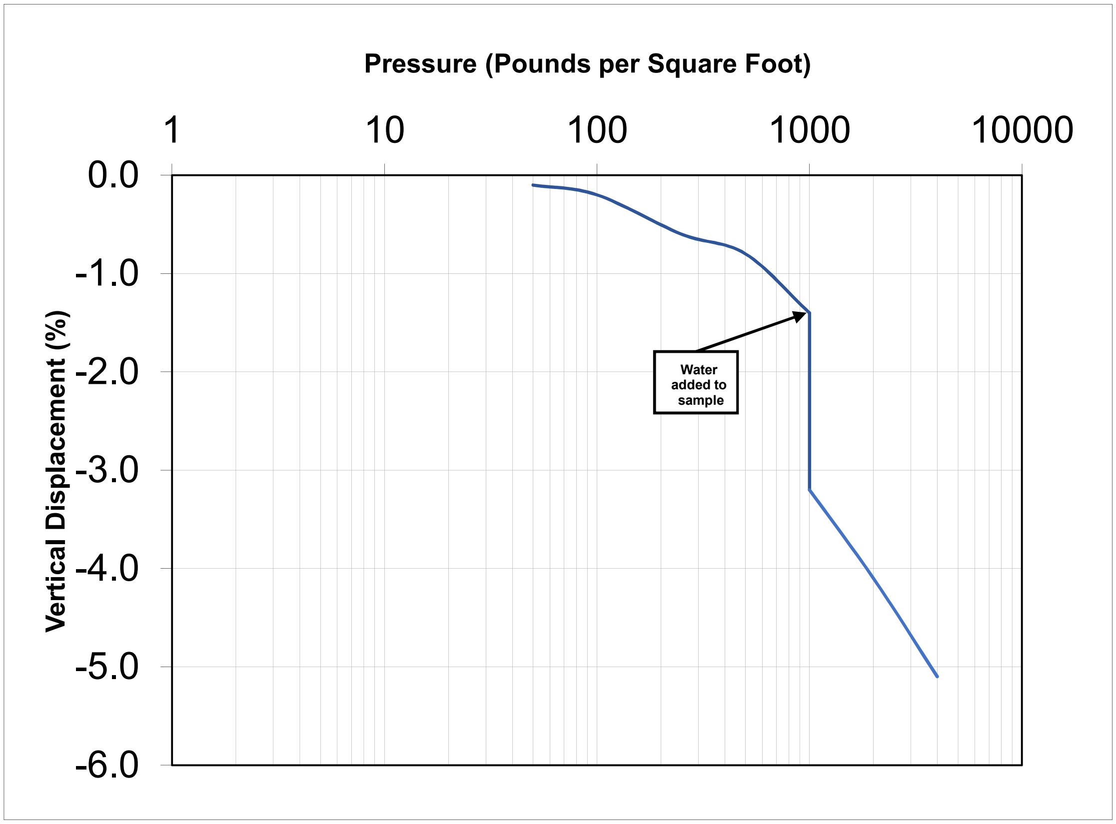
SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-13@4'	
Visual Soil Description:	SM Silty Sand	
Swell Potential (%)	Consolidated	
Constant Voume Swell Pressure (lb/ft^2):	N/A	
	Initial	**Final**
Moisture Content (%):	8.0	18.9
Dry Density (lb/ft^3):	112.7	113.0
Height (in.):	1.000	0.976
Diameter (in.):	1.94	1.94

Project Number:	55599GE
Sample ID:	10219-F3
Figure:	4.22

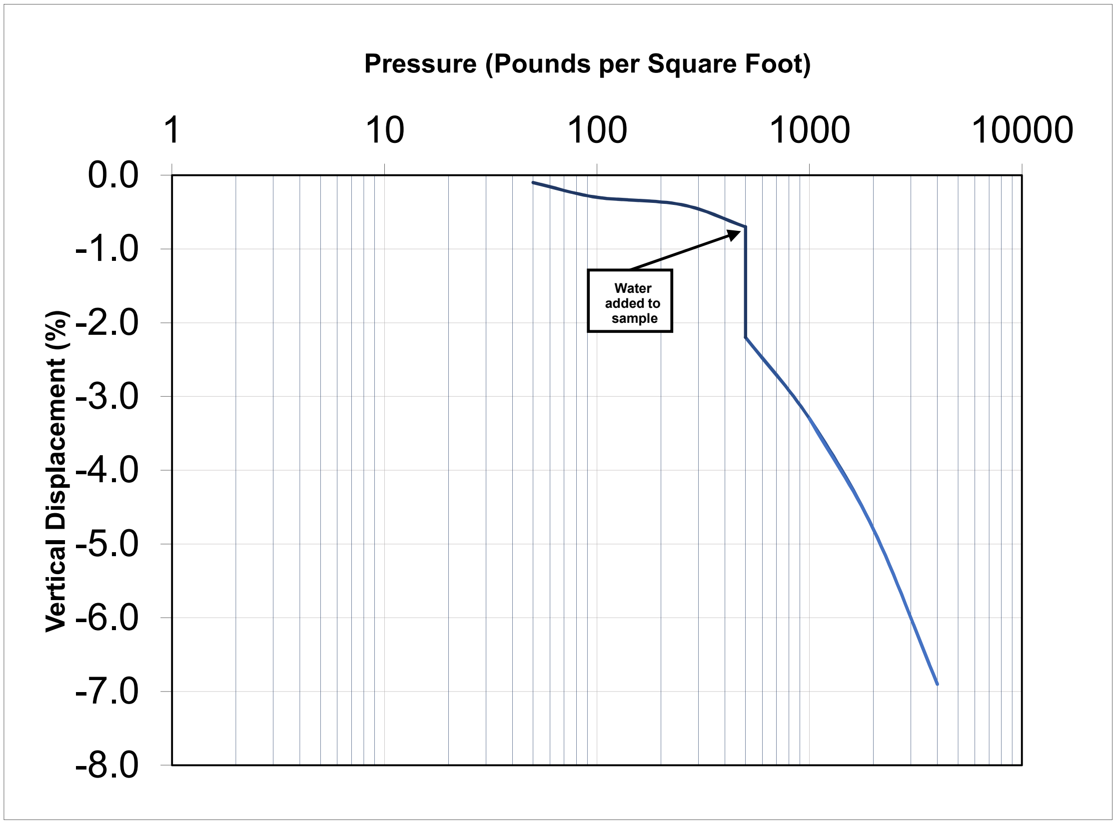
SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-13@9'	
Visual Soil Description:	SM Silty Sand	
Swell Potential (%)	Consolidated	
Constant Voume Swell Pressure (lb/ft^2):	N/A	
	Initial	**Final**
Moisture Content (%):	3.4	16.6
Dry Density (lb/ft^3):	111.2	115.5
Height (in.):	1.000	0.949
Diameter (in.):	1.94	1.94

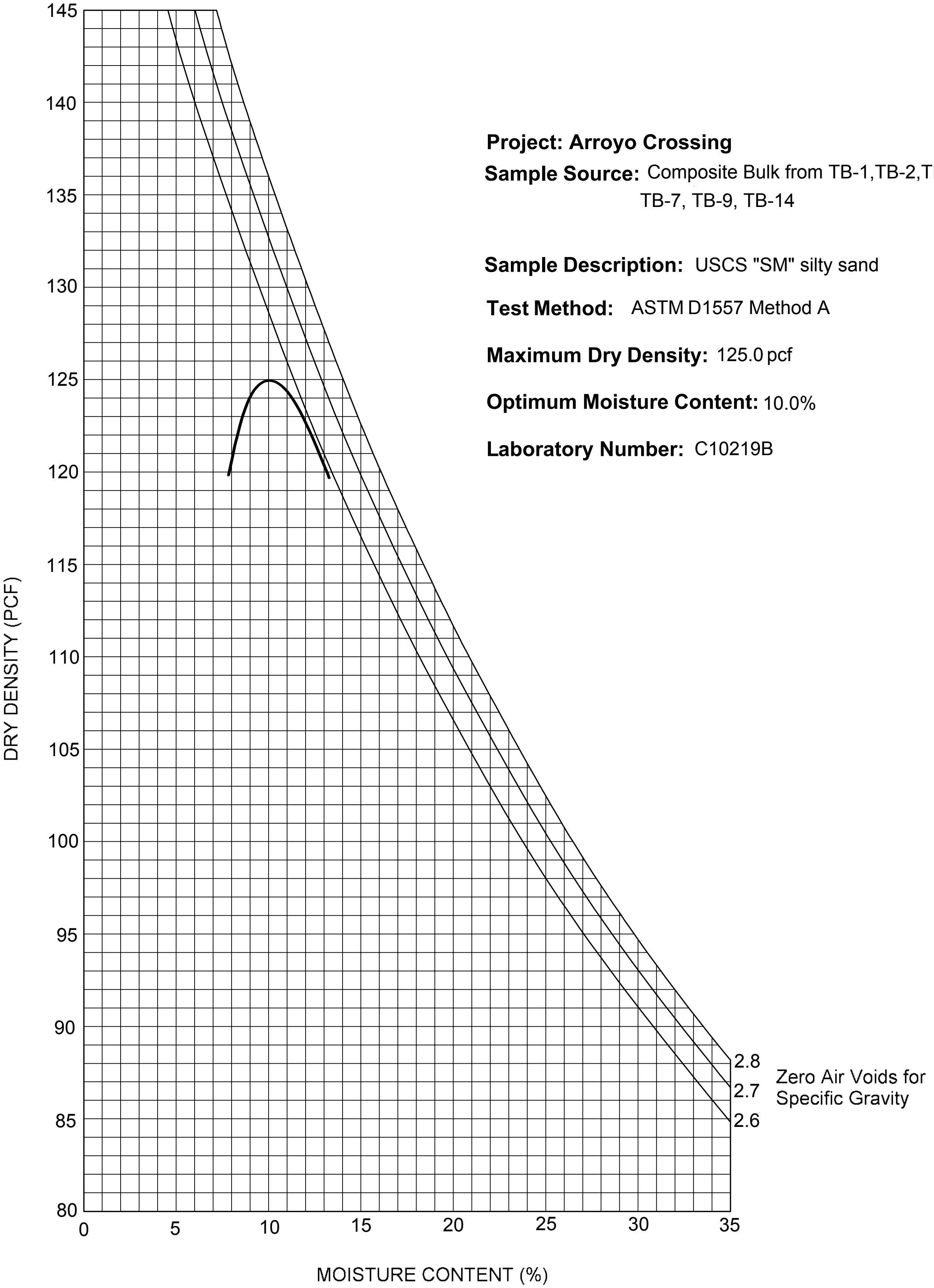
Project Number:	55599GE
Sample ID:	C10219-H3
Figure:	4.23

SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-14@4'	
Visual Soil Description:	SM Silty Sand	
Swell Potential (%)	Consolidated	
Constant Voume Swell Pressure (lb/ft^2):	N/A	
	Initial	**Final**
Moisture Content (%):	1.6	15.5
Dry Density (lb/ft^3):	109.4	115.3
Height (in.):	1.000	0.931
Diameter (in.):	1.94	1.94

Project Number:	55599GE
Sample ID:	C10219-L3
Figure:	4.24



Project: Arroyo Crossing

Sample Source: Composite Bulk from TB-1,TB-2,TB-4,
TB-7, TB-9, TB-14

Sample Description: USCS "SM" silty sand

Test Method: ASTM D1557 Method A

Maximum Dry Density: 125.0 pcf

Optimum Moisture Content: 10.0%

Laboratory Number: C10219B

TRAUTNER GEOTECH LLC

Project No. : 55599GE

Date: 4/9/19

Figure: 4.25

Project: Arroyo Crossing

Sample Source: Composite Bulk from TB-3, TB-6, TB-11, TB-12, TB-13

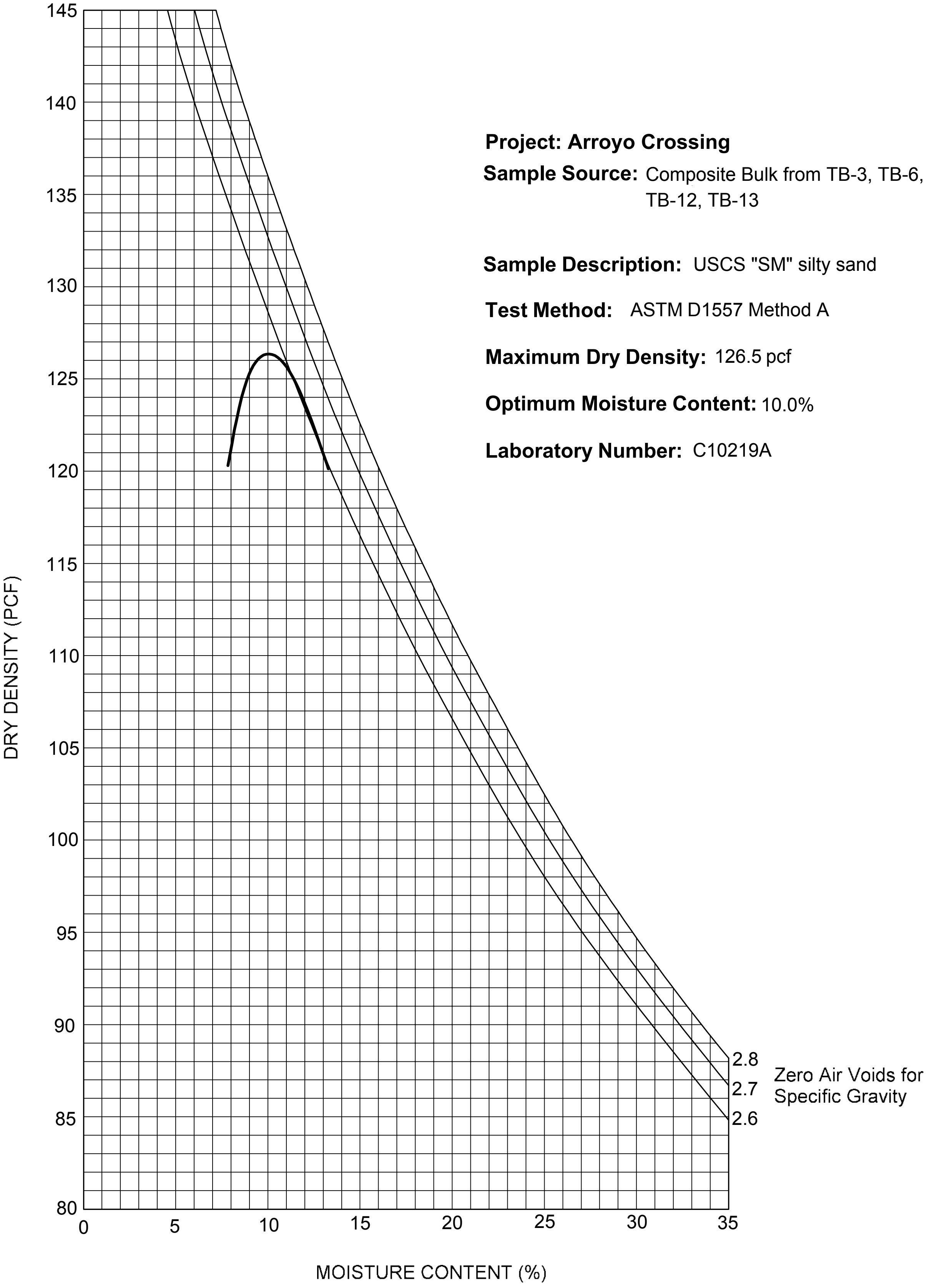
Sample Description: USCS "SM" silty sand

Test Method: ASTM D1557 Method A

Maximum Dry Density: 126.5 pcf

Optimum Moisture Content: 10.0%

Laboratory Number: C10219A



TRAUTNER GEOTECH LLC

Project No. : 55599GE

Date: 4/9/19

Figure: 4.26

TRAUTNER GEOTECH LLC
GEOTECHNICAL ENGINEERING, MATERIAL TESTING
AND ENGINEERING GEOLOGY

California Bearing Ratio Test Results
ASTM D1883

PROJECT NAME: Arroyo Crossing Development PROJ NO: 55599GE Date: 4/10/19
TECHNICIAN: JB

LAB NO: C10219A

Figure 4.27

Proctor Method: \STM D1557-A
Max Dry Density: 126.5 pcf
Optimum Moisure Content: 10.0%

Condition: soaked
Surcharge: 15 Lbs

Sample Source: TB-3,6,11,12,13

Pre-Soak			**After 72 hour Soak**			
Dry Density (PCF)	Moisture Content (%)	Relative Compaction (%)	Dry Density (PCF)	Moisture Content of Top One (1) Inch (%)	Swell (%)	CBR (0.100" penetration)
106.3	10.3	84	109.1	14.5	0.0	3.9
115.6	10.0	91	117.5	11.5	0.0	22.4
119.3	10.1	94	120.9	10.9	0.0	23.6