Geotechnical Evaluation

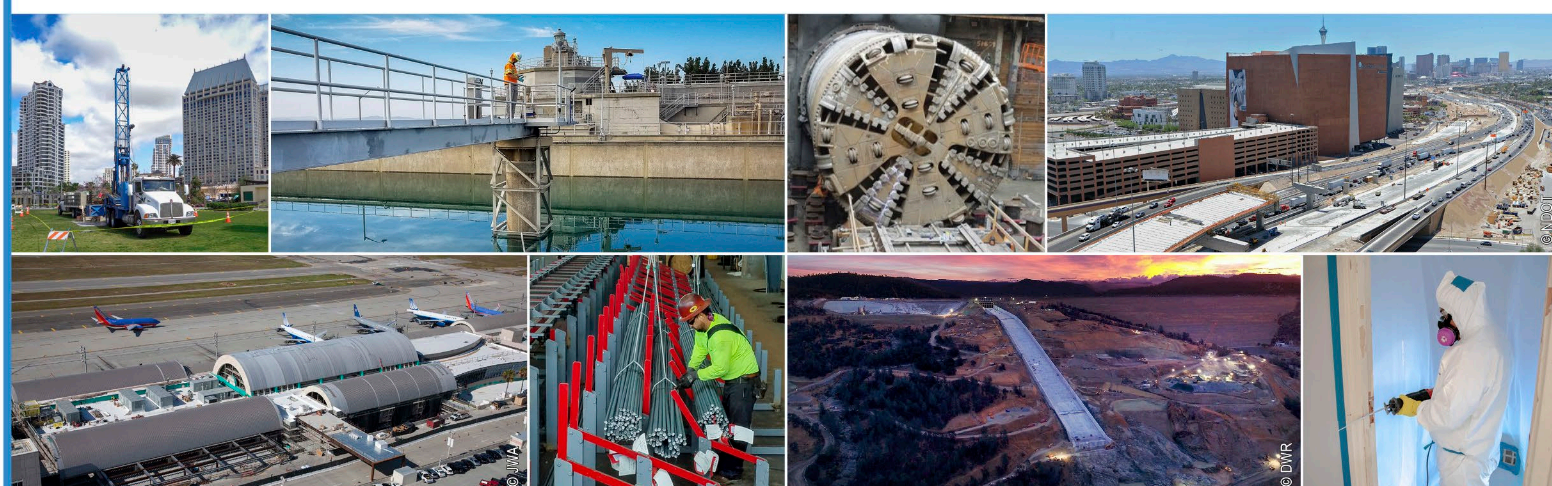
Green Canyon Water Treatment Plant
Chlorine Contact Tank
Green Canyon Along Green Canyon Road

North Logan, Utah

Hazen

10619 South Jordan Gateway, Suite 130 | South Jordan, Utah 84095

March 11, 2024 | Project No. 800331001



Geotechnical | Environmental | Construction Inspection & Testing | Forensic Engineering & Expert Witness

Geophysics | Engineering Geology | Laboratory Testing | Industrial Hygiene | Occupational Safety | Air Quality | GIS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

35+ YEARS

Geotechnical Evaluation

Green Canyon Water Treatment Plant
Chlorine Contact Tank
Green Canyon Along Green Canyon Road

North Logan, Utah

Mr. Parry Osborn, P.E.

Hazen

10619 South Jordan Gateway, Suite 130 | South Jordan, Utah 84095

March 11, 2024 | Project No. 800331001

Joseph F. Spendlove, GIT
Senior Staff Geologist

Robert E. Gambrell, PE
Senior Engineer

PROFESSIONAL ENGINEER
No. 11256609
ROBERT E.
GAMBRELL
Exp. 3/31/25
STATE OF UTAH
3/11/24

JFS/REG/kgg

CONTENTS

TABLES

FIGURES

APPENDICES

1 INTRODUCTION

In accordance with your request, Ninyo & Moore has performed a geotechnical evaluation for the Chlorine Contact Tank project to be constructed at the Green Canyon Water Treatment Plant along Green Canyon Road in North Logan, Utah. The approximate location of the site is indicated on Figure 1. The purposes of our geotechnical study were to evaluate subsurface soil conditions at the project site and to provide design and construction recommendations regarding geotechnical aspects of the project. This report presents the findings of our subsurface exploration, results of laboratory testing, conclusions regarding subsurface conditions at the project site, and geotechnical recommendations for design and construction of this project.

2 SCOPE OF SERVICES

The scope of our services included the following:

- Review of pertinent background information, including in-house geotechnical data, aerial photographs, and published regional and local geologic maps and soils data.
- Coordination and mobilization for subsurface exploration. Mark-out of existing utilities was conducted through Blue Stakes of Utah.
- Performance of a refraction microtremor (ReMi) survey to evaluate the shear wave velocity profile at the project site to a depth of approximately 100 feet.
- Drilling, logging, and sampling of two exploratory borings to depths up to approximately 26.5 feet. The purpose of the soil borings was to evaluate the subsurface soil and groundwater conditions, including obtaining soil samples for laboratory testing.
- Performance of laboratory tests to evaluate the geotechnical characteristics of the subsurface soils, including in-place moisture content and density, gradation, Atterberg limits (plasticity), shear strength, and chemical (corrosivity) considerations, including pH, reduction-oxidation potential, electrical resistivity, water-soluble sodium content, water-soluble sulfate content, water-soluble chloride content, and total dissolved solids (solubility).
- Compilation and analysis of the field and laboratory data.
- Preparation of this report presenting our findings, conclusions, and recommendations.

3 PROJECT DESCRIPTION

The project will include design and construction of a new below-grade tank, which will be located adjacent to the west side of the existing water treatment plant. The tank will have a capacity of approximately 300,000 gallons and will be approximately 40 feet wide by 50 feet long by 20 feet deep. A small vault approximately 8 to 10 feet deep may be located near the tank. The project site is shown on Figure 2.

4 GENERAL SITE CONDITIONS

The project site was located at the existing Green Canyon Water Treatment Plant. At the time of our field activities, the site was developed with a single-story, concrete masonry building with a compacted soil parking area in the eastern portion of the site, two below-grade concrete water tanks in the central portion of the site, and one small, concrete masonry maintenance building in the southern portion of the site. The site was covered with sparse grasses, vegetation, and trees along the southern boundary. A 6-foot tall chain-link fence topped with barbed wire was observed around the perimeter of the site. Adjacent properties include Green Canyon Road followed by Kings Nature Center and trails to the north; and undeveloped land to the east, south, and west. The topography at the site generally slopes down to the west with a total relief of approximately 30 feet. Above-ground power lines were observed along the northern side of Green Canyon Road. Indications of underground utilities were also observed, including power and communication lines. Additional underground utilities may also be present at or near the site.

5 GEOLOGY

Based on our field observations, subsurface exploration, and review of referenced geologic and soils data, the project site is underlain primarily by Quaternary-age alluvial and deltaic soil deposits (native soil) consisting primarily of gravel, sand, silt, and clay deposits with occasional cobbles and boulders. Ninyo & Moore's findings regarding the geologic setting, potential geologic hazards, ground motions, and liquefaction potential at the project site are provided in the following sections.

5.1 Geologic Setting

The project site is located in the eastern portion of the Cache Valley along the western base of the Bear River Range. The Bear River Range is located on the eastern edge of the Great Basin, which is made up of many naturally formed structural basins resulting from block faulting, which is a fundamental characteristic of the Basin and Range physiographic province.

The Bear River Range was formed during tectonic mountain building activity known as the Sevier Orogeny approximately 160 to 50 million years ago (mya). Compressional tectonic forces resulted in folding and thrusting of the rocks of the Bear River Range. Normal faulting in the Bear River Range associated with the Basin and Range extension followed approximately 17 mya.

Sediment deposition in the Cache Valley is largely attributed to the Pleistocene-age Lake Bonneville, a freshwater lake covering much of northern and western Utah and depositing thousands of feet of clay, silt, and sand. After failure of a natural earthen dam on the northern portion of the Lake, water levels dropped over 300 feet rapidly to water levels known as the Provo shoreline. Water levels continued dropping slowly primarily due to drier climate conditions to the modern-day levels of the Great Salk Lake and Utah Lake. After the recession of Lake Bonneville, sediment deposition in the area of the site was influenced by periods of heavy runoff and erosion of Green Canyon.

The Bear River Range extends in a north-south direction and generally drains toward the west through rivers and washes into the Bear River and Cutler Marsh. The referenced geologic map titled *Provisional Geologic Map of the Smithfield Quadrangle, Cache County, Utah* (Lowe, M., 1993) indicates that the project area is underlain primarily by Quaternary-age lacustrine deposits that are composed primarily of fine to coarse grained sand and silts with minor gravel deposits.

5.2 Potential Geologic Hazards

Ninyo & Moore's geotechnical study included an evaluation of the possible presence of geologic hazards, such as faults and ground fissures, in the site area. This evaluation included visual observation of the site for indications of adverse geologic features and review of published geologic and soils maps and literature, and other data listed in the references section of this report. Referenced geologic data were also reviewed to evaluate seismic activity levels, and associated potential earthquake hazards, for faults in the site vicinity.

Based on our review of referenced data, no faults traverse the project site. Surficial disturbance associated with active faulting was not observed at the site during our field evaluation. Review of referenced geologic data indicates that the nearest active fault (i.e., a fault that has experienced ground surface rupture within the past 10,000 years) to the site is the Central segment of the East Cache Fault Zone. Table 1 lists the principal, known active faults that may affect the project site along with approximate fault-to-site distances and anticipated maximum moment magnitudes (M_{max}). The approximate fault-to-site distances, M_{max} values, and activity levels were obtained using the referenced USGS web-based programs (USGS, 2014; USGS, 2024).

Table 1 – Principal Active Faults in Vicinity of Project Site

Fault Name	Approximate Distance From Project Site to Fault (miles)	Maximum Moment Magnitude (M_{max})
East Cache Fault Zone, Central Segment	0.1	7.3
West Cache Fault Zone, Junction Hills Segment	10.2	6.7
West Cache Fault Zone, Wellsville Segment	12.3	6.6
Wasatch Fault Zone, Collinston Segment	16.0	--
Wasatch Fault Zone, Brigham City Segment	16.1	7.0
West Cache Fault Zone, Clarkston Segment	17.4	6.7

Review of the referenced geologic data does not indicate the presence of ground fissures at the project site and no ground fissures were observed during our field activities. Additionally, our review indicates that the site is not located in a *Surface Fault Rupture Hazard Special Study Zone* (UGS, 2008b).

5.3 Ground Motions

Using the Applied Technology Council (ATC) Hazard Tool (https://hazards.atcouncil.org), estimated maximum considered earthquake spectral response accelerations for short (0.2 second) and long (1.0 second) periods were obtained for the project site, which is located at approximately 41.7674 degrees north latitude and -111.7784 degrees west longitude. Based on the results of our field exploration and ReMi survey, American Society of Civil Engineers (ASCE) Standard 7-16 (ASCE, 2016), and a review of available geologic information, Seismic Site Class C is appropriate for the project site. The parameters presented in the following table are characteristic of the site for design purposes.

Table 2 – Seismic Design Criteria

Site Coefficients and Spectral Response Acceleration Parameters	Values
Site Class	C
Site Coefficient at 0.2-second Period, F_a	1.2
Site Coefficient at 1.0-second Period, F_v	1.5
Mapped Spectral Response Acceleration at 0.2-second Period, S_S	0.977g
Mapped Spectral Response Acceleration at 1.0-second Period, S_1	0.326g
Spectral Response Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	1.173g
Spectral Response Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.49g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	0.782g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.326g
Site Amplification Factor, F_{PGA}	1.2
Peak Ground Acceleration, PGA	0.423g
Modified Peak Ground Acceleration, PGA_M	0.507g

5.4 Liquefaction Potential

Liquefaction is a phenomenon in which loose, saturated soils lose shear strength under short-term (dynamic) loading conditions. Ground shaking of sufficient duration results in the loss of grain-to-grain contact in potentially liquefiable soils due to a rapid increase in pore water pressure, causing the soil to behave as a fluid for a short period of time.

To be potentially liquefiable, a soil is typically cohesionless with a grain-size distribution generally consisting of sand and silt. It is generally loose to medium dense and has relatively high moisture content, which is typical near or below groundwater level. The potential for liquefaction decreases with increasing clay and gravel content, but increases as the ground acceleration and duration of shaking increase. Potentially liquefiable soils need to be subjected to sufficient magnitude and duration of ground shaking for liquefaction to occur.

An in-depth evaluation of the potential for liquefaction at the site was outside the scope of this geotechnical evaluation. However, review of the referenced geologic data indicates that the project site is mapped in a zone with a low liquefaction potential. Accordingly, liquefaction is not a design concern.

6 FIELD EXPLORATION AND SUBSURFACE CONDITIONS

Ninyo & Moore's subsurface exploration at the project site was performed on February 7 and February 8, 2024. This exploration consisted of drilling, logging, and sampling of two exploratory test borings (B-1 and B-2). The borings were drilled with a Mobile B-80 drill rig utilizing hollow-stem augers and ODEX drilling techniques. The borings were drilled to depths up to approximately 26.5 feet. The purpose of the borings was to evaluate subsurface conditions at the project site and to collect soil samples for laboratory testing. The elevations of the borings based on Mean Sea Level (MSL) were estimated from Google Earth (Google Earth Website, 2024) data. Accordingly, the ground elevations that are recorded on the boring logs in Appendix A should be considered approximate. The approximate locations of the borings are shown on Figure 2.

Laboratory tests were performed on representative soil samples collected from the borings to evaluate the in-place moisture content and density, gradation, Atterberg limits (plasticity), shear strength, and chemical (corrosivity) considerations, including pH, reduction-oxidation potential, electrical resistivity, water-soluble sodium content, water-soluble sulfate content, water-soluble chloride content, and total dissolved solids (solubility). The results of the in-place moisture content and density tests are provided on the boring logs in Appendix A. The other laboratory test results and descriptions of testing procedures utilized are presented in Appendix B and Appendix C.

6.1 Subsurface Soil Encountered

Generalized descriptions of the subsurface soils encountered in the exploratory borings are provided in the following sections.

6.1.1 Fill Soil

Fill materials were encountered in our exploratory borings extending to depths up to approximately 3.5 feet below the ground surface. The encountered fill soils consisted primarily of medium dense, silty, clayey gravel; and stiff, lean clay with sand and varying amounts of gravel and organics. Existing fill materials should be considered undocumented fill and unsuitable for support of structures and improvements in their present condition. The term undocumented fill refers to fill placed without engineering control and documentation. Fill soils may be left in place where documentation can be provided showing that the soils were engineered.

6.1.2 Native Soil

Native soil was encountered below the fill to the total depths of our exploratory borings. The encountered native soil consisted primarily of dense to very dense sand with varying amounts of silt and gravel; and stiff to hard, lean clay with varying amounts of sand and gravel. The encountered native soils were generally dry to moist. It should be noted that the soil samples were collected using samplers with an inside diameter of approximately 1.4 to 2.4 inches. Accordingly, in-situ soils may have higher concentrations of gravel, cobbles, and/or boulders than indicated on the boring logs.

6.2 Laboratory Testing

Laboratory tests were performed on representative samples of soil obtained from the exploratory borings. Results of these tests are summarized in the following table and presented in Appendix B and Appendix C.

Table 3 – Summary of Laboratory Test Results

Test Type	Test Results	Remarks
In-Place Moisture Content	1.7 to 12.1 percent	--
In-Place Dry Density	107.9 to 135.8 pcf	--
Atterberg Limits Liquid Limit Plastic Limit Plasticity Index	 15 and 17 12 and 13 3 and 4	 Low plasticity
pH	8.4 to 8.6	--
Oxidation-Reduction Potential	296 to 298 mV	--
Electrical Resistivity	15.3 and 21.7 Ohm-m	Moderate to severe corrosion potential to normal grade steel.
Water-Soluble Sodium	144 and 172 mg/kg (ppm)	--
Water-Soluble Sulfate	<10 mg/kg (ppm)	Sulfate Exposure Class S0 – Negligible corrosion potential to concrete.
Water-Soluble Chloride	<10 and 24 mg/kg (ppm)	Low corrosivity potential to normal grade steel.
Total Dissolved Solids (Solubility)	2,150 and 6,670 mg/kg (ppm)	Low to moderate solubility potential.

6.3 Groundwater

Groundwater was not encountered in our borings at the time of drilling. Groundwater levels are influenced by seasonal factors, variations in ground surface topography, precipitation, irrigation practices, soil/rock types, groundwater pumping, and other factors and are subject to fluctuations. Evaluation of factors associated with groundwater fluctuations was beyond the scope of this study.

6.4 Refraction Microtremor (ReMi) Survey

Ninyo & Moore performed a ReMi survey to obtain the shear wave velocity profile to a nominal depth of approximately 100 feet at the subject site to evaluate Seismic Site Class in general accordance with ASCE 7-16 (ASCE, 2016). The approximate location of the survey is indicated on Figure 2. Data was collected using a 24-Channel Geometrics Geode, exploration seismograph coupled with 24 vertical component 4.5 Hertz geophones spaced approximately 20 feet apart. Ambient noise (microtremors) was recorded for a total period length of 30 seconds. The average shear wave velocity for the upper 100 feet was 1,313 feet per second, which indicates a Seismic Site Class C. The one-dimensional shear wave velocity structure is provided in Appendix D.

7 FINDINGS AND CONCLUSIONS

Based on the findings of this study, it is our opinion that there are no known geotechnical or geologic conditions that would preclude construction of the proposed project, provided the recommendations presented herein are implemented and appropriate construction practices are followed. Geotechnical design and construction considerations for the proposed project include the following:

- **Existing Fill:** Fill material, which is considered undocumented/non-engineered, was encountered to depths up to approximately 3.5 feet in our exploratory excavations. Deeper areas of fill at the project site should be anticipated. Since undocumented/non-engineered fill is not suitable for support of proposed project improvements, this soil will need to be removed in areas of proposed structures and improvements. The existing fill may be left in-place if documentation can be provided indicating that the fill was "engineered."
- **Structural Fill and Backfill:** The findings of our study indicate that the soils encountered in our exploratory borings generally should be suitable for use as structural fill and backfill material for the project. The excavated on-site soils may be used as structural fill and backfill provided they comply with the recommendations presented in Section 8.2.2.
- **Subgrade Support:** Structure foundations and other project improvements should be supported on medium dense to very dense native granular soils, on stiff to hard fine-grained native soils, or on a zone of adequately placed and compacted structural fill.
- **Groundwater:** Groundwater was not encountered in our exploratory borings at the time of drilling, and is not anticipated to be a design or construction concern.
- **Undermining:** There is a potential for damage to the existing building due to undermining and loss of lateral support for the building's foundation during construction of adjacent footings for the planned addition. We strongly emphasize that the contractor for the project take adequate

precautions during construction so that movement of the existing building does not occur. Precautions may include placement of shoring, installation of sacrificial jacks, underpinning, and/or performance of soil removal adjacent to the foundation in sections up to several feet in length.

- **Seismic Parameters:** In accordance with ASCE 7-16, the seismic parameters provided in Table 2 are characteristic of the site and should be considered, where appropriate, in design of the proposed structures.
- **Liquefaction:** The project site is mapped in a zone with a low liquefaction potential. Accordingly, liquefaction is not a design concern.
- **Geologic Hazards:** Review of published geologic data and our field observations do not indicate the presence of adverse on-site geologic hazards, such as faults and ground fissures, which may affect proposed site development.
- **Corrosion Potential:** Chemical test results indicate that the tested soils have a low to severe corrosion potential to metal and concrete.
- **Underground Utilities:** Indications of several underground utilities were observed at the site during our field activities. Existing utilities at the site should be located and marked prior to earthwork operations, and they should be removed from proposed building and other site improvement areas or abandoned in-place.

8 RECOMMENDATIONS

The following sections provide geotechnical recommendations for design and construction of proposed project improvements.

8.1 Construction Vibrations

Existing utilities in the proposed project area may include rerouting, removal, or in-place abandonment of underground utilities. Utilities should be adequately capped or rerouted at the project perimeter in accordance with the requirements of the governing authorities and the recommendations of the geotechnical consultant at the time of construction operations. Abandoned underground utility pipes under proposed building limits should be removed from the site, or, if the pipes are left in place, they should be filled with flowable fill, such as grout or controlled low strength material (CLSM). The contractor should take adequate precautions when grading the site to reduce the potential for damage to existing utilities that are to remain in service.

Construction activities will be in close proximity to adjacent structures and surrounding improvements. Vibrations created from construction operations may cause distress to adjacent utilities, structures, and improvements. Prior to the initiation of the construction activities, we recommend performing a detailed pre-construction survey of the adjacent buildings and other surface improvements. In addition, a vibration monitoring plan should be prepared and implemented during construction activities. The vibration monitoring plan should provide a description of planned scope of monitoring services, and should establish a schedule and protocol for the vibration monitoring services once the demolition and construction of the project begins. Ninyo & Moore may be retained to provide such services, upon request.

8.2 Earthwork

The following subsections provide recommendations for earthwork, including site grading, structural fill and backfill, import soil, excavations and dewatering, and temporary excavations and shoring.

8.2.1 Site Grading

Prior to grading, areas of proposed structures and improvements should be cleared of any surface obstructions, pavement, debris, topsoil, vegetation, undocumented fill, and other deleterious material. Existing fill materials should be considered undocumented/non-engineered and unsuitable for support of structures and improvements in the present condition. The term undocumented fill refers to fill placed without engineering control and documentation. Such materials generated from clearing operations should be removed and disposed of in non-structural areas or at a legal landfill. Fill soils may be left in place where documentation can be provided showing that the soils were engineered. Findings of our study indicate that the soils encountered in our exploratory borings generally should be suitable for use as structural fill and backfill material for the project. Soils excavated in areas of proposed project improvements may be re-used as structural fill and backfill provided they conform to recommendations provided in Section 8.2.2.

After the removals described above have been made, the exposed native soils should be scarified to approximately 6 inches, moisture-conditioned to approximately optimum moisture content, and compacted to 95 percent or more relative compaction, as evaluated by ASTM International (ASTM) Standard D1557. The project's geotechnical consultant should observe excavation bottoms and areas to receive fill at the time of grading to assess the suitability of the exposed material and to evaluate if removals down to more competent soils are needed.

Surface preparations should extend 5 feet or more beyond the exterior edges of planned structure foundations and 2 feet or more beyond planned exterior concrete flatwork, pavement areas, and retaining/screen walls, or to a lateral distance that is equivalent to the depth of compacted structural fill, whichever is greater.

Based on the density/consistency of the existing native soils at the site, some shrinkage should be anticipated when these soils are excavated, processed, and compacted. For planning purposes, an estimated shrinkage factor of approximately 20 percent may be used for on-site soils encountered in the upper 5 feet.

The geotechnical consultant should be retained to observe the remedial excavations, and the elevations of the excavation bottoms should be surveyed by the project civil engineer.

8.2.2 Structural Fill and Backfill

The following sections include recommendations regarding soil suitability, placement, and compaction of structural fill and backfill.

8.2.2.1 Soil Suitability

Based on the findings of our subsurface evaluation and laboratory test results, the soils encountered during our exploration should generally be suitable for use as structural fill and backfill material. The excavated on-site soils may be used as structural fill and backfill provided they comply with the recommendations presented in this section.

Structural fill and backfill soil should not contain organic matter, debris, other deleterious matter, or rocks or hard chunks larger than approximately 4 inches in nominal diameter. These soils should have a low solubility potential of 1.0 percent or less, as evaluated by SM2540C at an extraction ratio of 1:5 (soil to water) and corrected for dilution, and a very low to low expansion potential (Expansion Index, EI, less than 50, as evaluated by ASTM D4829).

8.2.2.2 Placement and Compaction

Soils used as structural fill and backfill should be moisture-conditioned to approximately optimum moisture content and placed and compacted in uniform horizontal lifts to a relative compaction of 95 percent, as evaluated by the ASTM D1557. The optimal lift thickness of fill will depend on the type of soil and compaction equipment used, but should generally not exceed approximately 8 inches in loose thickness. Placement and compaction of structural fill should be performed in accordance with applicable building codes.

Earthwork operations should be observed and compaction of structural fill and backfill materials should be tested by the project's geotechnical consultant. Typically, one field test should be performed per lift for each approximately 2,500 square feet of fill placement in structural areas. Additional field tests may also be performed in structural and non-structural areas at the discretion of the geotechnical consultant.

8.2.3 Import Soil

Import soil should consist of coarse-grained material (50 percent or more retained on the No. 200 sieve). Import soil should have a low solubility potential of 1.0 percent or less, as evaluated by SM2540C at an extraction ratio of 1:5 (soil to water) and corrected for dilution, a low sulfate content (less than 0.1 percent), and a very low to low expansion potential (EI less than 50, as evaluated by ASTM D4829). Import soil should not contain organic matter, debris, other deleterious matter, or rocks or hard chunks larger than approximately 4 inches in nominal diameter. We further recommend that proposed import material be evaluated by the project's geotechnical consultant at the borrow source for its suitability prior to being imported to the project site. Import soil should be moisture-conditioned, placed, and compacted in accordance with the recommendations set forth in the previous section.

8.2.4 Excavations and Dewatering

Groundwater was not encountered at the time of our subsurface exploration. Accordingly, dewatering measures are not anticipated. Groundwater levels will fluctuate due to seasonal variations from precipitation, irrigation, groundwater withdrawal or injection, and other factors.

8.2.5 Temporary Excavations and Shoring

Temporary slope configurations should be consistent with the requirements provided in the referenced Occupational Safety and Health Administration (OSHA) regulations (OSHA, 2022) document. Temporary slope surfaces should be kept moist to retard raveling and sloughing. Water should not be allowed to flow over the top of excavations in an uncontrolled manner. Stockpiled material and/or equipment should be kept back from the top of excavations a distance equivalent to the depth of the excavation or more. Workers should be protected from falling debris, sloughing, and raveling in accordance with OSHA regulations (OSHA, 2022). Temporary excavations should be observed by the project's geotechnical consultant so that appropriate additional recommendations may be provided based on the actual field conditions. Temporary excavations are time sensitive and failures are possible.

Shoring systems should be designed for the contractor by a professional engineer registered in the State of Utah. In addition to lateral earth pressures, shoring design should include surcharge loads exerted by adjacent existing roadways, structure foundations, construction equipment, construction traffic, material stockpiles, etc. located within a 1:1 (H:V) plane extending upward from the toe of the excavation. Shoring design should discuss the anticipated top deflection of the shoring components. Depending on the anticipated top deflection of the shoring components, settlement of buildings, buried utility lines, exterior flatwork, and other improvements located within close proximity (approximately 10 feet or more) of the temporary shoring should be considered.

8.3 Utility Installation

The contractor should take particular care to achieve and maintain adequate compaction of the backfill soils around manholes, valve risers, and other vertical pipeline elements where settlements are commonly observed. Use of controlled low strength material (CLSM) or a similar material should be considered in lieu of compacted soil backfill in areas with low tolerances for surface settlement. This may also reduce permeability of the utility trench backfill.

Pipe bedding materials, placement, and compaction should meet the specifications of the pipe manufacturer and applicable municipal standards. Materials proposed for use as pipe bedding should be tested for suitability prior to use.

Special care should be exercised to avoid damaging the pipe or other structures during the compaction of the backfill. In addition, the underside (or haunches) of the buried pipe should be supported on bedding material that is compacted as described above. This may need to be performed with placement by hand or small-scale compaction equipment.

Surface drainage should be designed to divert surface water away from utility trenches. Where topography, site constraints, or other factors limit or preclude adequate surface drainage, granular bedding materials should be surrounded by a non-woven geotextile fabric (e.g., TenCate Mirafi® 140N or equivalent) to reduce the migration of fines into bedding material, which can result in severe, isolated settlements.

8.4 Structure Foundations

We anticipate the tank may be supported on conventional spread foundations and/or mat foundations. The allowable bearing capacity, which was developed considering a factor of safety of 2.5, may be increased by one-third for short duration loads, such as wind or seismic. Additionally, shallow foundations should have an embedment depth of 32 inches or more below adjacent finished grade (for frost protection), and a width of 12 inches or more. Seismic parameters for design of structures at the site are provided in Table 2 in Section 5.3. Foundations should be designed in accordance with structural considerations and the following recommendations. In addition, requirements of the appropriate governing jurisdictions and applicable building codes should be considered in design of the structures.

8.4.1 Conventional Spread Footings

Lightly loaded structures, such as retaining walls, may be supported by conventional spread foundations utilizing an allowable bearing capacity of 1,500 pounds per square foot (psf). Spread footings should be founded on medium dense to very dense native granular soils, on stiff to hard native fine-grained soils, or on a zone of adequately placed and compacted structural fill (reworked native or import soils). The allowable bearing capacity may be increased by 200 psf for each additional 1 foot of width and 500 psf for each additional 1 foot of embedment up to 2,000 psf.

Lateral resistance for footings is presented in Section 8.6. From a geotechnical standpoint, we recommend that footings be reinforced with two No. 4 or larger reinforcing bars, one placed near the top and one near the bottom of the footings. Additional reinforcement may be recommended by the structural engineer.

8.4.2 Mat Foundations

Mat foundations should be established on at least 6 inches of Untreated Base Course placed on exposed native subgrade soils scarified and re-compacted to at least 95 percent as evaluated by ASTM D1557, or on adequately placed and compacted structural fill. Mat foundations established as recommended above may be designed for an allowable bearing capacity of 3,000 psf.

Bending of the mat foundation from imposed foundation loads and resulting stresses within the mat foundation should be estimated using the subgrade modulus. The recommended vertical modulus of subgrade reaction, kv1, for use in design of a flexible mat foundation is 75 pounds per cubic inch (pci) applicable for a 12-inch-square loaded area. For actual mat foundation sizes, the subgrade modulus should be reduced using the following formula:

$$Kv = Kv1(B+1)/2B \text{ (Equation 1)}$$

Where, for a uniformly loaded mat:

Kv = vertical modulus of subgrade reaction for actual mat foundation width

Kv1 = vertical modulus of subgrade reaction for 1-foot-square loaded area in pci

B = mat foundation width in feet

For point loads on the mat, the vertical modulus of subgrade reaction need not be reduced using the formula above for the entire width of the mat or slab but rather some equivalent width which is related to the flexural stiffness of the mat relative to the underlying soil subgrade stiffness and may be estimated using the following formula:

B’ = 14T (Equation 2)

Where:

B’ = equivalent foundation width in feet to be used in Equation 1 for B

T = thickness of mat in feet

8.5 Settlement

Based on our evaluation of spread footing bearing capacity, we anticipate that static settlement of conventional spread foundations will be on the order of 1 inch or less. We estimate static spread footing differential settlement of about ½-inch over a horizontal span of about 40 feet.

8.6 Lateral Earth Pressures

Lateral earth pressures may be estimated using the values provided below. These values are based on our observation of the on-site soils, considered no groundwater, and assume that the ground surface is horizontal for a distance of 10 feet, or three times the height generating the passive pressure, whichever is more. These values also assume that retaining walls will have a height of approximately 6 feet or less.

For passive resistance to lateral loads, we recommend a passive lateral earth pressure of 300 psf per foot of depth up to a value of 2,000 psf. We recommend that the upper 12 inches of soil not protected by pavement or a concrete slab be neglected when calculating passive resistance. The passive lateral earth pressure may be increased by one-third when considering loads of short duration such as wind or seismic forces. For active and at-rest lateral earth pressures, we recommend equivalent fluid pressures of 40 pcf and 59 pcf, respectively. In addition, for seismic active lateral earth pressures, an additional equivalent fluid pressure of 12 pcf should be added to the static active equivalent fluid pressure provided herein.

For frictional resistance to lateral loads, we recommend that a coefficient of friction of 0.51 be used between soil/soil contacts. A coefficient of friction of 0.32 may be used between soil and concrete contacts. Passive and frictional resistances may be used in combination, provided the passive resistance does not exceed one-half of the total allowable resistance.

Measures should be taken so that hydrostatic pressure does not build up behind retaining walls. Drainage measures should include free-draining granular backfill material and perforated drain pipes, or weep holes lined with polyvinyl chloride (PVC) pipe. Drain pipes should outlet away from structures and retaining walls should be waterproofed in accordance with the recommendations of a qualified civil engineer.

8.7 Construction in Cold or Wet Weather

During construction, the site should be graded such that surface water can drain readily away from the structure and improvement areas. It is important to avoid ponding of water in or near excavations. Water that accumulates in excavations should be promptly pumped out or otherwise removed and these areas should be allowed to dry out before resuming construction. Berms, ditches, and similar means may be used to decrease stormwater entering the work area and to efficiently convey it to appropriate outlets off site.

Earthwork activities undertaken during the cold weather season may be difficult and should be done by an experienced contractor. Fill should not be placed on top of frozen soils. The frozen soils should be removed prior to placement of new engineered fill or other construction material. Frozen soil should not be used as structural fill or backfill. The frozen soil may be reused (provided it meets the selection criteria) once it has thawed completely. In addition, compaction of the soils may be more difficult due to the viscosity change in water at lower temperatures.

If construction proceeds during cold weather, foundations, slabs, or other concrete elements should not be placed on frozen subgrade soil. Frozen soil should either be removed from beneath concrete elements, or thawed and re-compacted. To limit the potential for soil freezing, the time between excavation and construction should be minimized. Blankets, straw, soil cover, or heating may be used to decrease the potential of soil freezing.

8.8 Frost Heave

Site soils are susceptible to frost heave if allowed to become saturated and exposed to freezing temperatures and repeated freeze/thaw cycling. The formation of ice in the underlying soils can result in 2 or more inches of heave of pavements, flatwork, and other hardscaping in sustained cold weather. A portion of this movement may be recovered when the soils thaw, but due to loss of soil density, some degree of displacement will remain. Frost heave of hardscaping could also result in areas of fine-grained subgrade soils.

In areas where hardscape movements are a design concern (i.e. exterior flatwork located adjacent to the building within the doorway swing zone), replacement of the subgrade soils with 3 or more feet of clean, coarse sand or gravel, or supporting the element on foundations similar to the building, or spanning over a void should be considered. Detailed recommendations in this regard can be provided upon request.

8.9 Concrete and Corrosion Considerations

The corrosion potential of on-site soils to concrete and metal was evaluated in the laboratory using representative samples obtained from the exploratory borings. Results of these tests are presented in Appendix C. Recommendations regarding concrete to be utilized in construction of proposed improvements and for metal in contact with on-site soils are provided in the following sections.

8.9.1 Concrete

Chemical tests performed on selected samples of on-site soils indicated sulfate contents of less than 10 mg/kg (ppm). Based on review of the referenced International Building Code (ICC, 2018) and American Concrete Institute manual (ACI, 2019), the tested soils are considered to have a sulfate exposure class of S0. Additionally, concrete in contact with on-site soil is anticipated to have a freeze/thaw exposure class of F2. Accordingly, we recommend that concrete in contact with on-site soils, along with subsurface walls up to 12 inches above finished grade have a design compressive strength of 4,500 psi or more, a water-cement ratio of 0.45 percent or less by weight, contain Type II cement, and contain 5.5 to 7.5 percent air-entrainment, as specified by ACI 318-19 (ACI, 2019). It is recommended that reinforcing bars in cast-against-grade concrete be covered by approximately 3 inches or more of concrete. Concrete should be placed with an approximate 4-inch slump and good densification procedures should be used during placement to reduce the potential for honeycombing. Concrete samples should be obtained, as indicated by ACI manual Section 318 (ACI, 2019), and the slump should be tested at the site by the project's geotechnical consultant. Structural concrete should be placed in accordance with American Concrete Institute (ACI, 2019) and project specifications.

8.9.2 Metal in Contact with On-Site Soils

Chemical tests performed on selected samples of on-site soils indicated low to severe corrosion potential to normal grade steel. Accordingly, Ninyo & Moore recommends that corrosion reduction methods be implemented for this project for metal in contact with soil. These corrosion reduction methods may include utilization of protective coatings, pipe sleeving, and/or appropriate cathodic protection as recommended by a qualified corrosion engineer. Where permitted by jurisdictional building codes, the use of plastic pipes for buried utilities should also be considered.

8.10 Moisture Infiltration Reduction and Surface Drainage

Infiltration of water into subsurface soils can lead to soil movement and associated distress, and chemically and physically related deterioration of concrete structures. To reduce the potential for infiltration of moisture into subsurface soils at the site, we recommend the following:

- Positive drainage should be established and maintained away from the proposed structure. Positive drainage may be established by providing a surface gradient for paved areas of 2 percent or more for a distance of 10 feet or more away from structure perimeters. For unpaved areas, positive drainage may be established by a slope of 5 percent or more for a distance of 10 feet or more away from structure perimeters, where possible.
- Adequate surface drainage should be provided to channel surface water away from on-site structures and to a suitable outlet such as a storm drain or the street. Adequate surface drainage may be enhanced by utilization of graded swales, area drains, and other drainage devices. Surface run-off should not be allowed to pond near structures.
- Building roof drains should have downspouts tightlined to an appropriate outlet, such as a storm drain or the street. If tightlining of the downspouts is not practicable, they should discharge 5 feet or more away from the building or onto paved areas that slopes away from the structure. Downspouts should not be allowed to discharge onto the ground surface adjacent to building foundations or concrete flatwork.
- Ninyo & Moore recommends that low-water use (drip irrigated) landscaping be utilized on site, particularly within 5 feet of the building and exterior site improvements, including areas of concrete flatwork and masonry block walls. Spray irrigation should not be used within 5 feet of the building. For drip irrigated foundation plating located within 5 feet of the building, we recommend incorporating a drainage system that drains the excess irrigation water away from this zone or soil moisture probes to prevent over watering.
- Irrigation heads should be oriented so that they spray away from building and block wall surfaces.
- Utility trenches should be backfilled with compacted, low permeability fill (i.e. permeability of 5-10 cm/s or less) within 5 feet of the building. Planters, if any, should be maintained 10 feet or more from the building and constructed with closed bottoms or with drainage systems to drain excess irrigation away from the building.
- The facility owner should develop a program for the continued maintenance of the irrigation systems, which should be performed periodically, to prevent overwatering of landscaping within 5 to 10 feet of the building perimeter.

8.11 Observation and Testing

The geotechnical consultant should perform appropriate observation and testing services during fill placement, grading, and construction operations. These services should include observation of removal of soft, loose, undocumented fill, or otherwise unsuitable soils, evaluation of subgrade conditions where soil removals are performed, and performance of observation and testing services during placement and compaction of structural fill and backfill soils. The geotechnical consultant should also perform observation and testing services during placement of concrete, mortar, grout, asphalt concrete, and steel reinforcement.

The recommendations provided in this report are based on the assumption that Ninyo & Moore will provide geotechnical observation, testing, and inspection services during grading and construction. In the event that it is decided not to utilize the services of Ninyo & Moore during construction, we request that the selected consultant provide the client with a letter (with a copy sent to Ninyo & Moore) indicating that they fully understand Ninyo & Moore's recommendations, and that they are in full agreement with the design parameters and recommendations contained in this report.

8.12 Plan Review

The recommendations presented in this report are based on preliminary design information for the proposed project, as provided by Hazen personnel, and on the findings of our geotechnical evaluation. When finished, project plans and specifications should be reviewed by the geotechnical consultant prior to submitting the plans and specifications for bid. Additional field exploration and laboratory testing may be needed upon review of the project design plans.

8.13 Pre-Construction Meeting

We recommend that a pre-construction meeting be held. The owner or the owner's representative, the architect, the civil engineer, the contractor, and the geotechnical consultant should be in attendance to discuss the plans and the project.

9 LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified and additional recommendations, if warranted, will be provided upon request. The conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. Changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

10 REFERENCES

American Association of State Highway and Transportation Officials (AASHTO), 1993, AASHTO Guide for Design of Pavement Structures: Fourth Edition, Volume 1 and Volume 2.

American Concrete Institute (ACI), 2019, Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary (ACI 318R-19).

American Society of Civil Engineers (ASCE), 2016, Minimum Design Loads for Building and Other Structures, Standard 7-16.

Applied Technology Council (ATC), 2023, Hazards by Location, https://hazards.atcouncil.org/#/.

ASTM International (ASTM), 2022, Annual Book of ASTM Standards, Section 4 – Construction.

CivilTech, 2005, Allpile Version 7.2i.

Google Earth Website, Aerial photographs of the Logan area, Utah: http://earth.google.com accessed on January 10, 2024.

International Code Council (ICC), 2018, International Building Code (IBC).

Lowe, M., 1993, Provisional geologic map of the Smithfield quadrangle, Cache County, Utah. M-143. UGS. 1:24,000 scale.

Ninyo & Moore proprietary in-house data.

Occupational Safety and Health Administration (OSHA), 2022, OSHA Standards for the Construction Industry, 29 CFR Part 1926: dated January 1.

United States Geological Survey (USGS), 2014, National Seismic Hazards Maps – Source Parameters: https://earthquake.usgs.gov/cfusion/hazfaults_2014_search.

United States Geological Survey (USGS), 2024, Quaternary Faults and Fold Database of the United States: http://earthquakes.usgs.gov/qfaults.

United States Geological Survey (USGS), 1984, Logan Quadrangle 30 X 60 minute Topographic Map, Utah-Wyoming-Idaho, 1:100,000 scale.

Utah Department of Transportation (UDOT), 2022, 2023 Standard Specifications for Road and Bridge Construction: dated May 10.

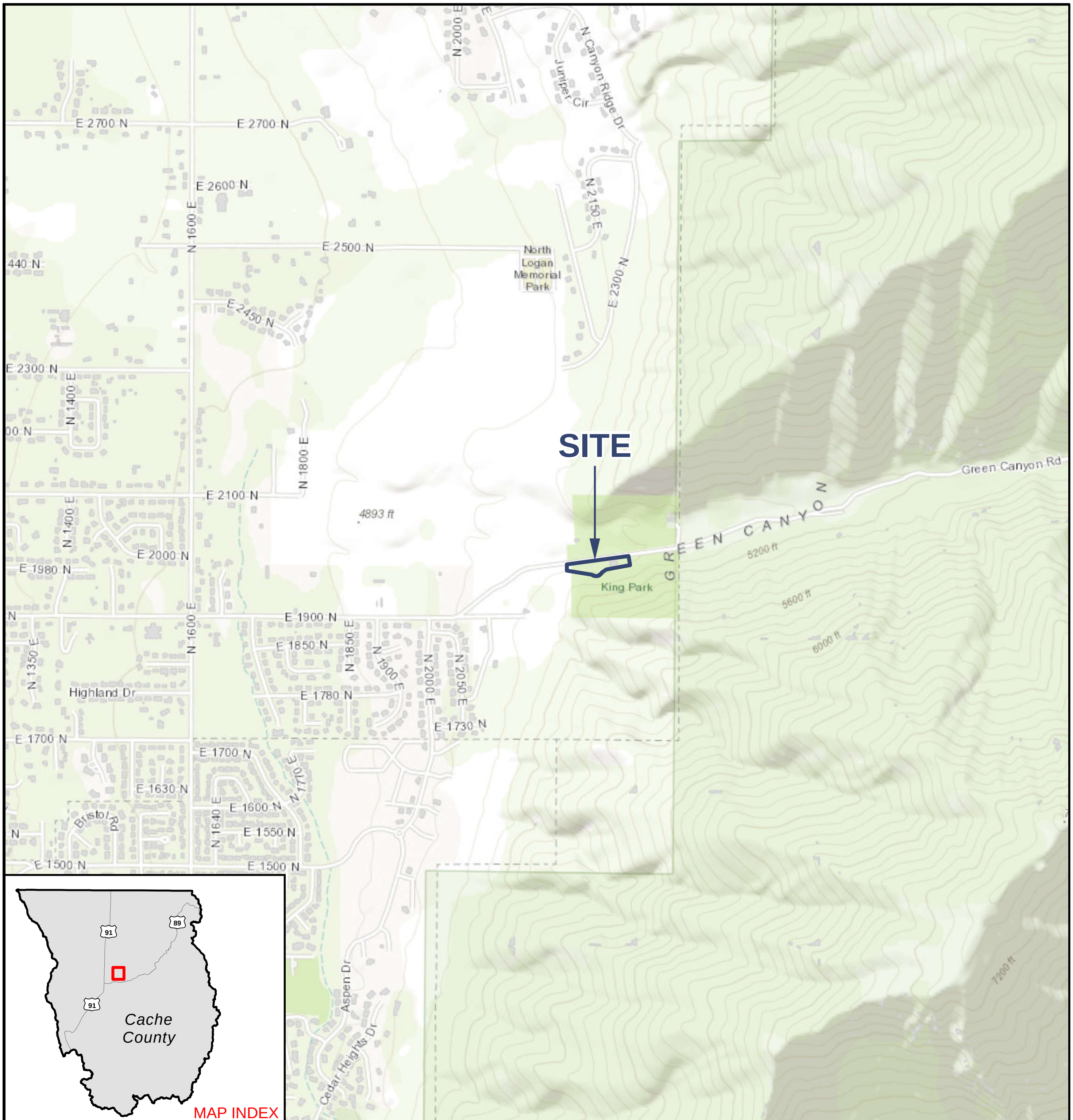
Utah Geological Survey (UGS), 2008a, Liquefaction Special Study Areas, Wasatch Front and Nearby Areas, Utah, compiled by Christenson, G. E. and Shaw, L.M.

Utah Geological Survey (UGS), 2008b, Surface Fault Rupture Special Study Areas, Wasatch Front and Nearby Areas, Utah, compiled by Christenson, G. E. and Shaw, L.M.

Utah Geological Survey (UGS), 2003, Liquefaction Potential Map for Cache Valley, Cache County, Utah, compiled by Bremser, K. and Halseth, D.

Utah Geological Survey (UGS), 2024, Utah Geologic Hazards Portal: https://geology.utah.gov/apps/hazards/

FIGURES



NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: ESRI WORLD TOPO, 2024

P800331001.aprx 2/14/2024 JDL

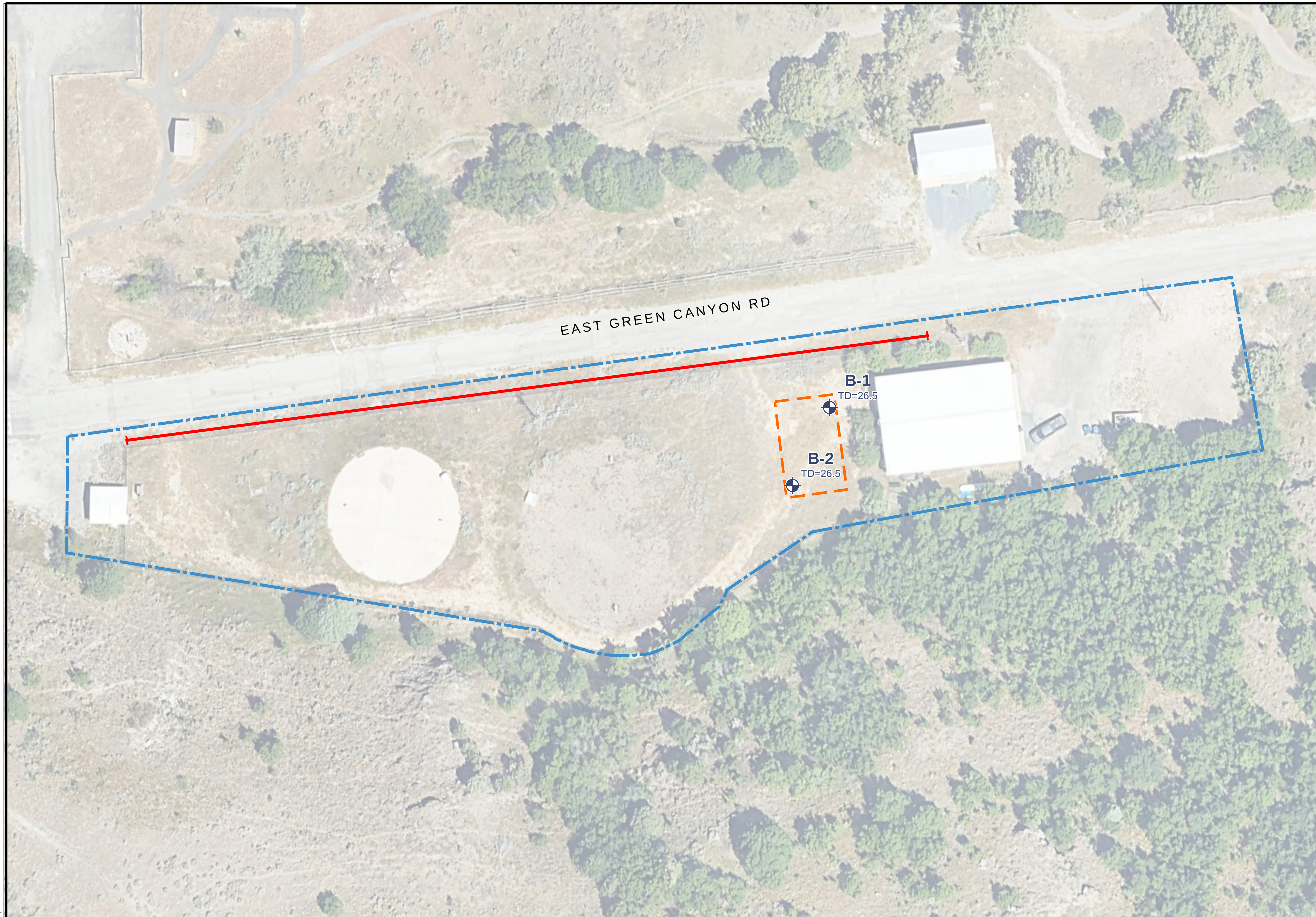
FIGURE 1

SITE LOCATION

GREEN CANYON WTP - CHLORINE CONTACT TANK
GREEN CANYON ALONG GREEN CANYON ROAD, NORTH LOGAN, UTAH

800331001 | 3/24

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



LEGEND

B-2 TD=26.5 BORING TD=TOTAL DEPTH IN FEET

ReMi SURVEY

SITE BOUNDARY

PROPOSED CHLORINE TANK

N

FEET

0 60 120

P800331001.aprx 2/14/2024 JDL

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: GOOGLE EARTH, 2024

FIGURE 2

EXPLORATION LOCATIONS

GREEN CANYON WTP - CHLORINE CONTACT TANK
GREEN CANYON ALONG GREEN CANYON ROAD, NORTH LOGAN, UTAH

800331001 | 3/24

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Soil Samples
Disturbed soil samples were obtained in the field using the following methods.

Bulk Soil Samples
Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test (SPT) Sampler
Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. In general accordance with ASTM D1586, the sampler was driven into the ground with a 140-pound hammer free-falling from a height of 30 inches and the number of blows recorded on the boring logs as an index to the relative resistance of the materials sampled. Soil samples were observed and removed from the sampler, bagged, sealed, and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples
Relatively undisturbed soil samples were obtained in the field using a modified split barrel drive sampler. The sampler, with an external diameter of 3.0 inches, was lined with 1-inch-long, thin, brass rings with inside diameters of 2.4 inches. In general accordance with ASTM D3550, the sampler was driven into the ground with a 140-pound hammer free-falling from a height of 30 inches and the number of blows recorded on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

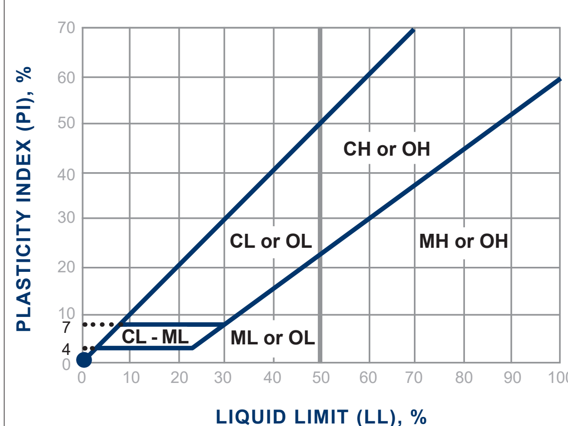
Soil Classification Chart Per ASTM D 2488

Primary Divisions			Group Symbol	Group Name
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	**GRAVEL** more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines	GW	well-graded GRAVEL
			GP	poorly graded GRAVEL
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines	GW-GM	well-graded GRAVEL with silt
			GP-GM	poorly graded GRAVEL with silt
			GW-GC	well-graded GRAVEL with clay
			GP-GC	poorly graded GRAVEL with clay
		GRAVEL with FINES more than 12% fines	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND 50% or more of coarse fraction passes No. 4 sieve	CLEAN SAND less than 5% fines	SW	well-graded SAND
			SP	poorly graded SAND
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines	SW-SM	well-graded SAND with silt
			SP-SM	poorly graded SAND with silt
			SW-SC	well-graded SAND with clay
			SP-SC	poorly graded SAND with clay
		SAND with FINES more than 12% fines	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	**SILT and CLAY** liquid limit less than 50%	INORGANIC	CL	lean CLAY
			ML	SILT
			CL-ML	silty CLAY
		ORGANIC	OL (PI > 4)	organic CLAY
			OL (PI < 4)	organic SILT
	SILT and CLAY liquid limit 50% or more	INORGANIC	CH	fat CLAY
			MH	elastic SILT
		ORGANIC	OH (plots on or above "A"-line)	organic CLAY
			OH (plots below "A"-line)	organic SILT
	Highly Organic Soils		PT	Peat

Grain Size

Description		Sieve Size	Grain Size	Approximate Size
Boulders		> 12"	> 12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	0.079 - 0.19"	Rock-salt-sized to pea-sized
	Medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	< 0.0029"	Flour-sized and smaller

Plasticity Chart



Apparent Density - Coarse-Grained Soil

Apparent Density	Spooling Cable or Cathead		Automatic Trip Hammer	
	SPT (blows/foot)	Modified Split Barrel (blows/foot)	SPT (blows/foot)	Modified Split Barrel (blows/foot)
Very Loose	≤ 4	≤ 8	≤ 3	≤ 5
Loose	5 - 10	9 - 21	4 - 7	6 - 14
Medium Dense	11 - 30	22 - 63	8 - 20	15 - 42
Dense	31 - 50	64 - 105	21 - 33	43 - 70
Very Dense	> 50	> 105	> 33	> 70

Consistency - Fine-Grained Soil

Consis-tency	Spooling Cable or Cathead		Automatic Trip Hammer	
	SPT (blows/foot)	Modified Split Barrel (blows/foot)	SPT (blows/foot)	Modified Split Barrel (blows/foot)
Very Soft	< 2	< 3	< 1	< 2
Soft	2 - 4	3 - 5	1 - 3	2 - 3
Firm	5 - 8	6 - 10	4 - 5	4 - 6
Stiff	9 - 15	11 - 20	6 - 10	7 - 13
Very Stiff	16 - 30	21 - 39	11 - 20	14 - 26
Hard	> 30	> 39	> 20	> 26

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

USCS METHOD OF SOIL CLASSIFICATION

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0								Bulk sample.
								Modified split-barrel drive sampler.
								No recovery with modified split-barrel drive sampler.
								Sample retained by others.
								Standard Penetration Test (SPT).
5								No recovery with a SPT.
			XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
								No recovery with Shelby tube sampler.
								Continuous Push Sample.
								Seepage.
10								Groundwater encountered during drilling.
								Groundwater measured after drilling.
							SM	MAJOR MATERIAL TYPE (SOIL): Solid line denotes unit change.
							CL	Dashed line denotes material change. Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Shear Bedding Surface
15								
20								The total depth line is a solid line that is drawn at the bottom of the boring.

DATE DRILLED	2/8/24	BORING NO.	B-1				
GROUND ELEVATION	5,034' ± (MSL)	SHEET	1	OF	2		
METHOD OF DRILLING	Mobile B-80 Hollow-Stem Auger and ODEX Drill Rig						
DRIVE WEIGHT	140 lbs. (Auto. Trip)	DROP	30"				
SAMPLED BY	JCH	LOGGED BY	JCH	REVIEWED BY	REG/EDE		

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	X	X	3/6" 3/6" 5/6"				CL	<u>FILL</u>: Dark brown, moist, stiff, lean CLAY with sand; trace organics.
	X	X	9/6" 43/6" 37/6"	12.1	107.9		CL	<u>NATIVE SOIL</u>: Brown, moist, hard, sandy lean CLAY with gravel.
5		X	21/6" 30/6" 33/6"					Light brownish gray; dry.
		X	47/6" 39/6" 50/6"	1.7	121.8		SM	Light brownish gray, dry, very dense, silty SAND with gravel.
10	X			2.0				
15		X	15/6" 21/6" 23/6"					
20								

DATE DRILLED	2/8/24	BORING NO.	B-1				
GROUND ELEVATION	5,034' ± (MSL)	SHEET	2	OF	2		
METHOD OF DRILLING	Mobile B-80 Hollow-Stem Auger and ODEX Drill Rig						
DRIVE WEIGHT	140 lbs. (Auto. Trip)	DROP	30"				
SAMPLED BY	JCH	LOGGED BY	JCH	REVIEWED BY	REG/EDE		

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20			28/6" 28/6" 20/6"	3.0	135.1		SM	NATIVE SOIL: (Continued) Light brownish gray, dry, dense, silty SAND.
25			29/6" 35/6" 32/6"					Very dense.
								Total Depth = 26.5 feet. Groundwater not encountered during drilling. Backfilled on 2/8/24. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
30								
35								
40								

DATE DRILLED 2/7/24 BORING NO. B-2

GROUND ELEVATION 5,035' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING Mobile B-80 Hollow-Stem Auger and ODEX Drill Rig

DRIVE WEIGHT 140 lbs. (Auto. Trip) DROP 30"

SAMPLED BY EAS/JCH LOGGED BY EAS/JCH REVIEWED BY REG/EDE

DEPTH (feet)	SAMPLES: Bulk	SAMPLES: Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							CL	FILL: Brown, moist, stiff, lean CLAY with sand; few gravel.
			8/6" 12/6" 12/6"				GC-GM	Brown, moist, medium dense, silty, clayey GRAVEL.
			5/6" 5/6" 5/6"	2.5	135.5		CL	NATIVE SOIL: Brown, dry, stiff, lean CLAY; trace silt and gravel.
5								
			16/6" 22/6" 50/6"				SC-SM	Light brown, dry, very dense, silty, clayey SAND with gravel.
			50/5"					
10								
				3.3				
15			15/6" 50/5"					
20								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 2/7/24 BORING NO. B-2 GROUND ELEVATION 5,035' ± (MSL) SHEET 2 OF 2 METHOD OF DRILLING Mobile B-80 Hollow-Stem Auger and ODEX Drill Rig DRIVE WEIGHT 140 lbs. (Auto. Trip) DROP 30" SAMPLED BY EAS/JCH LOGGED BY EAS/JCH REVIEWED BY REG/EDE **DESCRIPTION/INTERPRETATION**
20			16/6" 19/6" 36/6"	2.6	135.8		SM	NATIVE SOIL: (Continued) Light brown, dry, dense, silty SAND with gravel.
25			34/6" 34/6" 23/6"					Very dense.
30								Total Depth = 26.5 feet. Groundwater not encountered during drilling. Backfilled on 2/8/24. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
35								
40								

APPENDIX B

Laboratory Test Results

APPENDIX B

LABORATORY TEST RESULTS

Classification
Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D2488. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

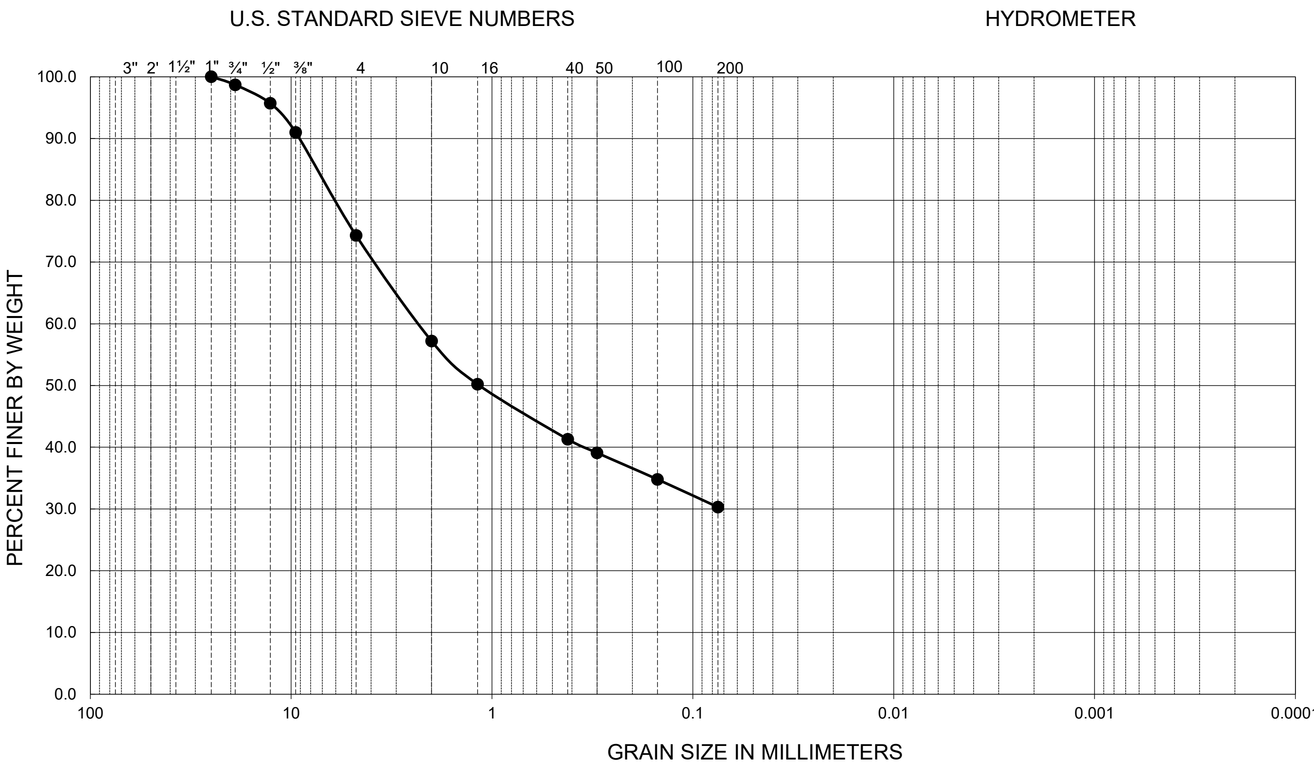
In-Place Moisture and Density
The moisture content and dry density of relatively undisturbed samples obtained from the exploratory excavations were evaluated in general accordance with ASTM D2937. The test results are presented on the logs of the exploratory excavations in Appendix A.

Gradation Analysis
Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D7928, C117, and C136. These test results were utilized in evaluating the soil classifications in accordance with the USCS. The grain-size distribution curves are shown on Figure B-1 and Figure B-2.

Atterberg Limits
Tests were performed on selected representative soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D4318. These test results were utilized to evaluate soil classification in accordance with the USCS. The test results and classifications are shown on Figure B-3.

Direct Shear
Direct shear tests were performed on undisturbed samples in general accordance with ASTM D3080 to evaluate the shear strength characteristics of selected materials. The samples were inundated during shearing to represent adverse field conditions. The results are shown on Figure 4 through Figure 5.

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-1	10.0-15.0	15	12	3	--	--	2.30	--	--	30.3	SM

Material Percent by Weight			Soil Type
Gravel	Sand	Fines	Silty SAND with gravel
25.7	44.0	30.3	
			Moisture Content
			2.0%

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D422

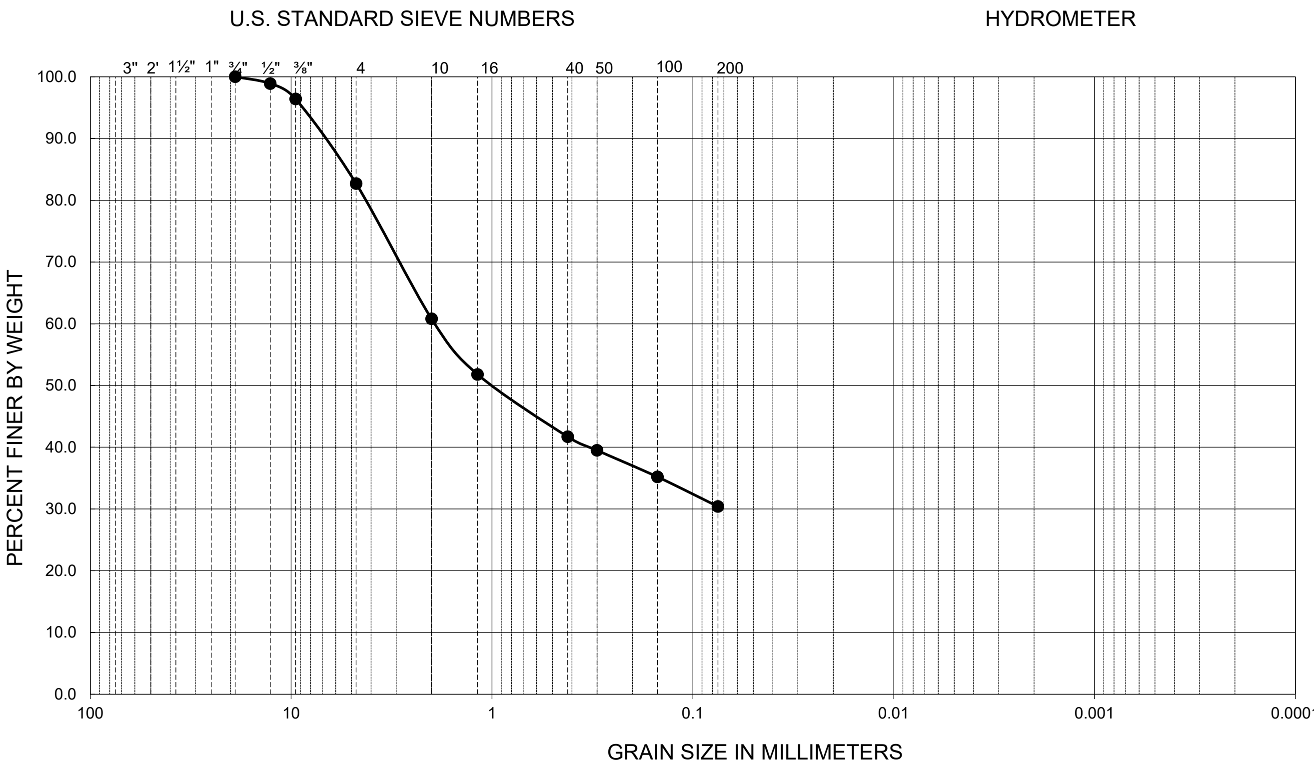
FIGURE B-1

GRADATION TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

GREEN CANYON WTP – CHLORINE CONTACT TANK
GREEN CANYON ALONG GREEN CANYON ROAD, NORTH LOGAN, UTAH

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-2	10.0-15.0	17	13	4	--	--	1.91	--	--	30.4	SC-SM

Material Percent by Weight			Soil Type
Gravel	Sand	Fines	Silty, clayey SAND with gravel
17.3	52.3	30.4	

Moisture Content
3.3%

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D422

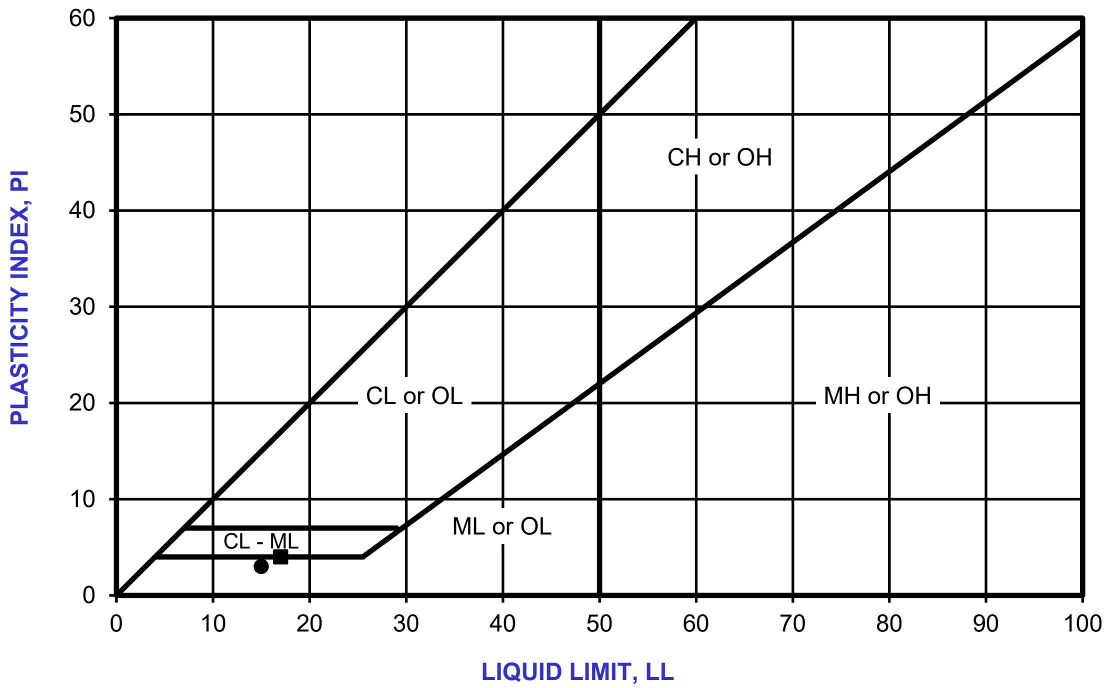
FIGURE B-2

GRADATION TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

GREEN CANYON WTP – CHLORINE CONTACT TANK
GREEN CANYON ALONG GREEN CANYON ROAD, NORTH LOGAN, UTAH

SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-1	10.0-15.0	15	12	3	ML	SM
■	B-2	10.0-15.0	17	13	4	CL-ML	SC-SM



PERFORMED IN GENERAL ACCORDANCE WITH D4318

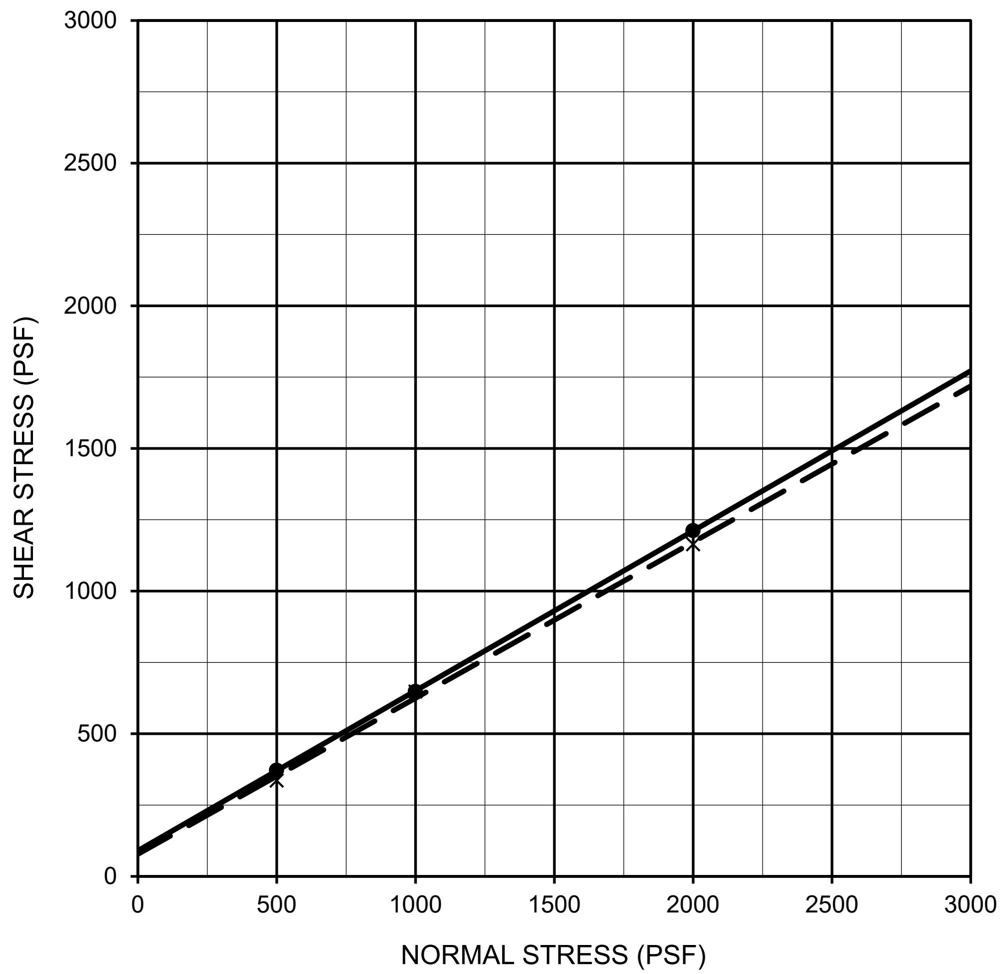
FIGURE B-3

ATTERBERG LIMITS TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

GREEN CANYON WTP – CHLORINE CONTACT TANK
GREEN CANYON ALONG GREEN CANYON ROAD, NORTH LOGAN, UTAH

800331001 | 3/24



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Sandy Lean CLAY	—●—	B-1	3.5-5.0	Peak	90	29	CL
Sandy Lean CLAY	– – X – –	B-1	3.5-5.0	Ultimate	78	29	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D3080

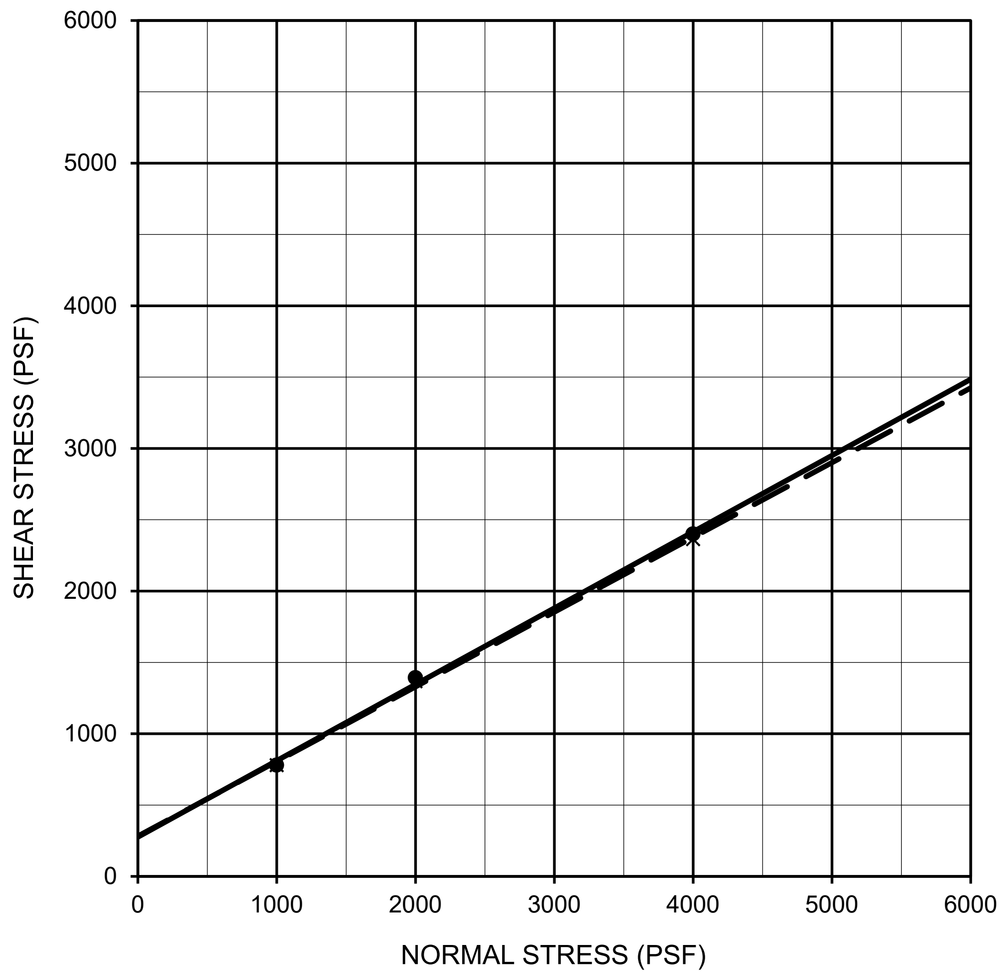
FIGURE B-4

DIRECT SHEAR TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

GREEN CANYON WTP - CHLORINE CONTACT TANK
GREEN CANYON ALONG GREEN CANYON ROAD, NORTH LOGAN, UTAH

800331001 | 3/24



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Poorly Graded SAND with Silt	—●—	B-1	8.5-10.0	Peak	276	28	SM
Poorly Graded SAND with Silt	– – X – –	B-1	8.5-10.0	Ultimate	282	28	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D3080

FIGURE B-5

DIRECT SHEAR TEST RESULTS

GREEN CANYON WTP - CHLORINE CONTACT TANK
GREEN CANYON ALONG GREEN CANYON ROAD, NORTH LOGAN, UTAH

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

APPENDIX C

Chemical Test Results

APPENDIX C

CHEMICAL TEST RESULTS

The results of the chemical tests are provided in this appendix.

Chemtech-Ford Laboratories
Serving the Intermountain West Since 1953

9632 South 500 West
Sandy, UT 84070
O:(801) 262-7299 F: (866) 792-0093
www.ChemtechFord.com

Certificate of Analysis

Ninyo and Moore
Joseph Spendlove
871 Robinson Drive
North Salt Lake, UT 84054

PO#: **800331001**
Receipt: **2/12/24 15:00 @ 17.1** °C
Date Reported: 2/20/2024
Project Name: **800331001**

Sample ID: **B-1 @ 6.0-7.5 ft**

Matrix: **Solid**
Lab ID: **24B0812-01**
Date Sampled: **2/7/24 0:00**
Sampled By: **Joseph Spendlove**

	Result	Units	Minimum Reporting Limit	Method	Preparation Date/Time	Analysis Date/Time	Flag(s)
Inorganic							
Chloride, Soluble (IC)	24	mg/kg dry	10	EPA 300.0	2/14/24	2/14/24	
eH	296	mV	0.1	SM 2580 B	2/13/24	2/13/24	
pH	8.6	pH Units	0.1	EPA 9045D	2/13/24	2/14/24	
Resistivity	15.3	ohm m	1.0	SSSA 10-3.3	2/13/24	2/13/24	
Sulfate, Soluble (IC)	ND	mg/kg dry	10	EPA 300.0	2/14/24	2/14/24	
Total Dissolved Solids, Soluble	2150	mg/kg dry	500	SM 2540 C	2/13/24	2/13/24	
Total Solids	97.9	%	0.1	CTF8000	2/13/24	2/14/24	
Metals							
Sodium, Total	172	mg/kg dry	34.3	EPA 6010D/3050B	2/14/24	2/15/24	

Chemtech-Ford Laboratories
Serving the Intermountain West Since 1953

9632 South 500 West
Sandy, UT 84070
O:(801) 262-7299 F: (866) 792-0093
www.ChemtechFord.com

Certificate of Analysis

Ninyo and Moore
Joseph Spendlove
871 Robinson Drive
North Salt Lake, UT 84054

PO#: **800331001**
Receipt: **2/12/24 15:00 @ 17.1** °C
Date Reported: 2/20/2024
Project Name: **800331001**

Sample ID: **B-2 @ 3.5-5.0 ft**
Matrix: **Solid**
Lab ID: **24B0812-02**
Date Sampled: **2/8/24 0:00**
Sampled By: **Joseph Spendlove**

	Result	Units	Minimum Reporting Limit	Method	Preparation Date/Time	Analysis Date/Time	Flag(s)
Inorganic							
Chloride, Soluble (IC)	ND	mg/kg dry	11	EPA 300.0	2/14/24	2/14/24	
eH	298	mV	0.1	SM 2580 B	2/13/24	2/13/24	
pH	8.4	pH Units	0.1	EPA 9045D	2/13/24	2/14/24	
Resistivity	21.7	ohm m	1.0	SSSA 10-3.3	2/13/24	2/13/24	
Sulfate, Soluble (IC)	ND	mg/kg dry	11	EPA 300.0	2/14/24	2/14/24	
Total Dissolved Solids, Soluble	6670	mg/kg dry	546	SM 2540 C	2/13/24	2/13/24	
Total Solids	91.5	%	0.1	CTF8000	2/13/24	2/14/24	
Metals							
Sodium, Total	144	mg/kg dry	35.7	EPA 6010D/3050B	2/14/24	2/15/24	

Certificate of Analysis

Ninyo and Moore	PO#: **800331001**
Joseph Spendlove	Receipt: **2/12/24 15:00 @ 17.1** °C
871 Robinson Drive	Date Reported: 2/20/2024
North Salt Lake, UT 84054	Project Name: **800331001**

Report Footnotes

Abbreviations

ND = Not detected at the corresponding Minimum Reporting Limit (MRL).
1 mg/L = one milligram per liter or 1 mg/kg = one milligram per kilogram = 1 part per million.
1 ug/L = one microgram per liter or 1 ug/kg = one microgram per kilogram = 1 part per billion.
1 ng/L = one nanogram per liter or 1 ng/kg = one nanogram per kilogram = 1 part per trillion.
On calculated parameters, there may be a slight difference between summing the rounded values shown on the report vs the unrounded values used in the calculation.

APPENDIX D

Refraction Microtremor Survey Results

APPENDIX D

REFRACTION MICROTREMOR SURVEY RESULTS

Ninyo & Moore performed a ReMi survey to obtain the shear wave velocity profile to a nominal depth of approximately 100 feet at the subject site to evaluate Seismic Site Class in general accordance with the 2018 International Building Code (ICC, 2018). The approximate length and orientation of the survey array is indicated on Figure 2. Data was collected to a depth of approximately 100 feet using a using 24-Channel Geometrics Geode exploration seismograph coupled with 24 vertical component 4.5 Hertz geophones spaced approximately 20 feet apart. Ambient noise (microtremors) was recorded for a total period length of 30 seconds. The results of the ReMi survey are provided on Figure D-1.

The calculated average shear wave velocities to a depth of approximately 100 feet at the location of the geophone array measured at 1,313 feet per second. Based on this information, a Seismic Site Class C is characteristic for design purposes for the project site.

Refraction MicroTremor (ReMi) Survey, R-1

Shear Wave Velocity v. Depth

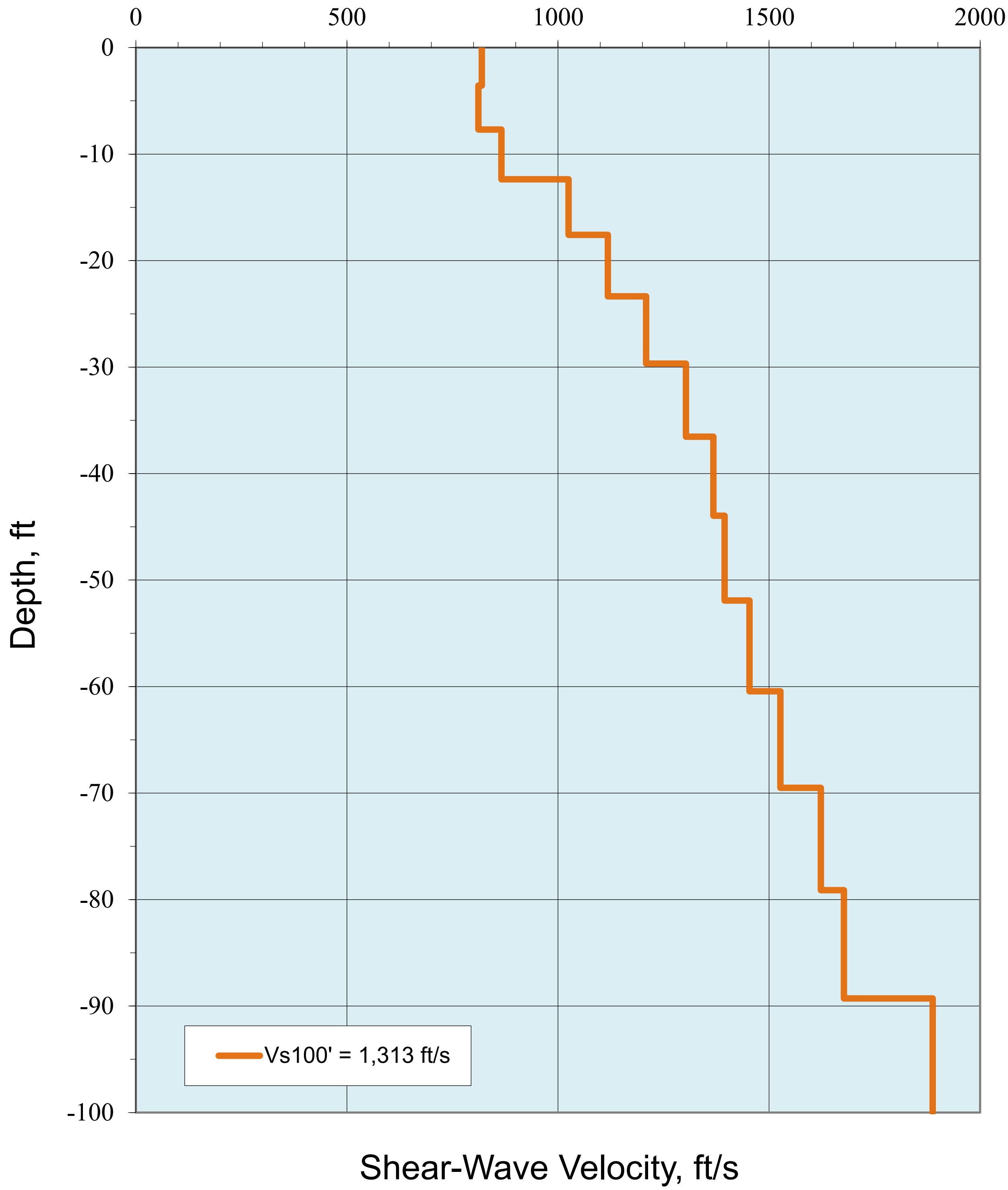
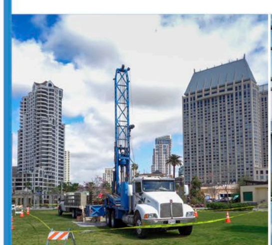


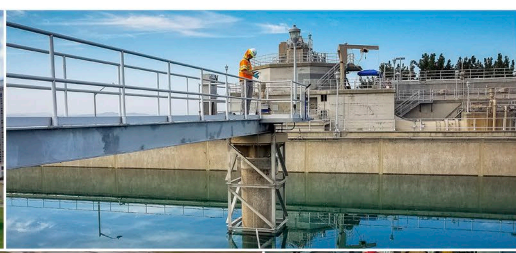
FIGURE D-1

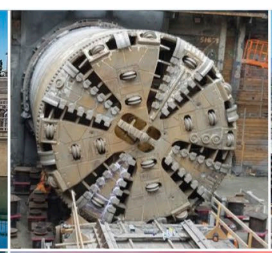
REMI SURVEY RESULTS

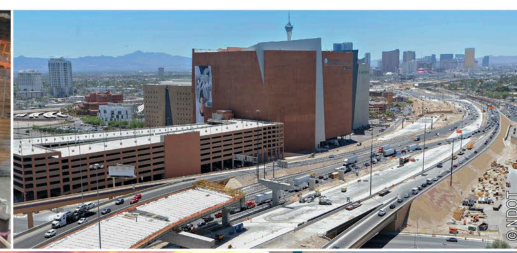
GREEN CANYON WTP - CHLORINE CONTACT TANK
GREEN CANYON ALONG GREEN CANYON ROAD, NORTH LOGAN, UTAH

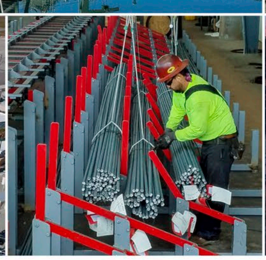
Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

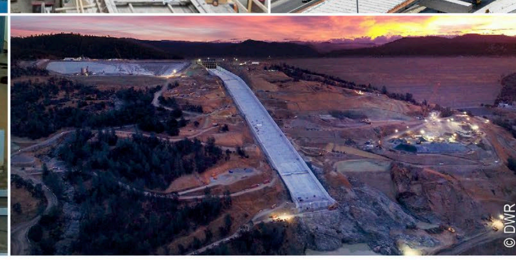












871 Robinson Drive, North Salt Lake, UT 84054 | p. 801.973.2500

ARIZONA | CALIFORNIA | COLORADO | NEVADA | TEXAS | UTAH

ninyoandmoore.com

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants