GEOTECHNICAL ENGINEERING

REPORT

Hobble Creek Canyon Water Tank

Bartholomew Canyon Road

Springville, Utah

Prepared for:

Ted Mickelsen, PE

Jones & DeMille Engineering

Prepared by:

CONSOLIDATED ENGINEERING LABORATORIES

2130 South 3140 West Suite C
Salt Lake City, UT 84119

CEL Project No. 20-06720

March 20, 2018

CONSOLIDATED ENGINEERING
LABORATORIES

April 20, 2018

Jones & DeMille Engineering
775 West 1200 North, Suites A&D
Springville, UT 84663

Attention: Ted Mickelsen, PE

Subject: **Geotechnical Engineering Report**
Hobble Creek Canyon Water Tank
Bartholomew Canyon Road
Springville, Utah
CEL Project No. 20-06720

Dear Mr. Mickelsen:

Consolidated Engineering Laboratories (CEL) has completed a Geotechnical Study for the proposed Hobble Creek Canyon Water Tank to be located on Bartholomew Canyon Road in Springville, Utah. This study has been prepared based on our understanding of the proposed project from information shared with CEL during our site visit on January 17, 2018 and the site explorations done on March 15 and 16, 2018. A summary of our key findings includes the following:

- In general, the proposed construction at the site is considered geotechnically feasible provided the recommendations of this report are implemented in the design and construction of the project.
- Subsurface soil conditions include medium dense to very dense clayey sands with gravel and cobbles, which will require particular attention, and heavy-duty excavation equipment during subgrade preparation and excavation adjacent to and below existing foundations, and utility trench excavation in narrow or limited site access areas.
- Tank foundations may be designed using a net-allowable soil bearing pressure of 5,000 psf. Thickened slab foundations smaller than 4-foot square should be designed using an allowable bearing pressure of 3,000 psf.
- On-site granular soils may be generally suitable for re-use as site grading fill, but may require significant screening to remove oversized or otherwise unsuitable materials to meet requirements for gradation and quality. A compacted unit weight of 130 pcf may be assumed for the on-site granular soils used as Structural or Engineered Fill or Backfill, under drained conditions. An undrained effective unit weight of 65 pcf may be used for submerged conditions.
- The following static lateral earth (equivalent fluid) pressures may be used in the design of the buried water tank with properly compacted Engineered Fill or Backfill:
  - Active: 27 pcf
  - At-rest: 50 pcf
  - Passive: 500 pcf

- For seismic conditions, the active and at-rest equivalent fluid pressures should be increased by 34 pcf.

- The tank may be designed to seismic ground motion parameters corresponding to a Site Class 'C' based on the soil conditions observed and the general location of the proposed tank in the high mountain canyon fill area of Bartholomew Canyon.
- CEL should be retained to provide subgrade observation during construction to observe the exposed subgrade soils beneath the tank foundations and along the site walls to confirm the soil conditions are consistent with the conclusions and recommendations of this report.

This work was performed in accordance with our scope of work included in our proposal number 20-06720 Rev1 dated March 9, 2018 and authorized by you on March 12, 2018.

Please contact the undersigned at 801-891-3786, or at cgarris@ce-labs.com with any questions regarding these recommendations.

Respectfully submitted,

CONSOLIDATED ENGINEERING LABORATORIES

PROFESSIONAL ENGINEER
4-20-18
No. 188305-2202
CHRIS T. GARRIS
STATE OF UTAH

Robert E. Gambrell, P.E.I.
Staff Geotechnical Engineer

Chris Garris, P.E.
Principal Geotechnical Engineer (CEL)

CTG/reg
Distribution:
PDF to Addressee; Ted Mickelsen, PE, ted.m@jonesanddemille.com

TABLE OF CONTENTS

TABLE OF CONTENTS (continued)

1.0 INTRODUCTION

1.1. Validity of Report

This report is valid for three years after publication. If construction begins after this time period, CEL should be contacted to confirm that the site conditions have not changed significantly. If the proposed development differs considerably from that described above, CEL should be notified to determine if additional recommendations are required. Additionally, if CEL is not involved during the geotechnical aspects of construction, this report may become wholly or in part invalid; CEL's geotechnical personnel should be retained to verify that the subsurface conditions anticipated when preparing this report are similar to the subsurface conditions revealed during construction. CEL's involvement should include grading and foundation plan review, grading observation and testing, foundation excavation observation, subgrade preparation, utility trench backfill testing, and concrete placement.

1.2. Purpose and Scope

The purpose of this geotechnical study is to evaluate the subsurface conditions at the site and prepare geotechnical recommendations for the proposed Bartholomew Water Tank located on Bartholomew Canyon Road in Hobble Creek Canyon near Springville, Utah. This study provides geotechnical recommendations for foundations, seismic design parameters, site preparation, grading, temporary and permanent cut slopes, site drainage considerations, utility trench backfilling, and other soil related design and construction recommendations.

The scope of this study included the review of available geotechnical and geologic literature for the site, the drilling of one (1) boring and the excavation of two (2) test pits within the project site, laboratory testing of selected samples retrieved from the exploration, engineering analysis of the accumulated data, and preparation of this report. The conclusions and recommendations presented in this report are based on the data acquired and analyzed during this study, and on prudent engineering judgment and experience. This study did not include an assessment of potentially toxic or hazardous materials that may be present on or beneath the site.

1.3. Site Description

The project site is located in Bartholomew Canyon (Hobble Creek Canyon) east of Springville, Utah as shown on *Figure A-1*, *Site Vicinity Map*. The project site is located in a high mountain narrow canyon, bounded by undeveloped grassed and wooded land to the north and east; by Bartholomew Canyon Road to west; and an existing water tank down-canyon to the south. The existing site has an approximate 25 percent downhill slope to the south with an estimated ground surface elevation ranging from 6235 to 6255 feet, based on Google Earth™ aerial photographs. A hydro-electric water source well head and buried pipeline is located beneath the western portion of the site. A buried spring inlet pipeline is reported to be present to the north (up-canyon) from the site, and feeds the existing lower water tank via a buried pipeline immediately west of the proposed water tank site.

The site has coordinates of approximately 40.2219° north latitude and -111.5080° west longitude (Utah DFCS Coordinates) using Google Earth aerial images.

1.4. Proposed Construction

CEL understands that a new water tank will be constructed below grade in the available space immediately north of the existing 1.5 million-gallon tank on the east side of the Bartholomew Canyon Rd. CEL understands that the proposed water tank will hold approximately 500,000 gallons and will have a diameter up to approximately 80 feet. The exact location was not fully defined during the field exploration phase, however, CEL understands that the tank will be located below the clearing just north of the existing tank. CEL understands that the estimated tank bottom elevation may be approximately Elevation 6222, or approximately 25 feet below existing grade, as shown on Figure A-11.

2.0 PROCEDURES AND RESULTS

2.1. Literature Review

Pertinent geologic and geotechnical literature pertaining to the site area and other project information was reviewed. These included various publications and maps issued by the United States Geological Survey (USGS), Utah Geological Survey.

2.2. Field Exploration

In order to characterize the subsurface conditions beneath the proposed improvement areas, a field exploration program was conducted which consisted of drilling one (1) boring and excavating two (2) test pits at the site. The boring and test pits were performed on March 16, 2018 by Earthcore Drilling and Nelson Contractors, respectively. The boring was drilled using a truck-mounted B-80 drill rig equipped with an ODEX air-rotary casing advancement system. The test pits were excavated using a Kamatsu PC50MR mini-excavator. The boring and test pits were conducted under the full-time observation of a CEL Staff Geotechnical Engineer supervised by a Utah-Registered Professional Engineer. The exploration locations were selected to provide adequate coverage to characterize the subsurface soil conditions and properties below the proposed project improvements. The locations of the exploration locations relative to the existing site conditions are shown on *Figure A-2*, *Exploration Location Map*.

A CEL field engineering representative visually classified the materials encountered in the boring and test pits according to the Unified Soil Classification System as the boring and test pits were advanced. Relatively undisturbed soil samples were recovered at selected intervals within the boring ranging from approximately 3 to 5 feet, using a two-inch outside diameter Standard Penetration Test (SPT) sampler. The samplers were driven by means of a 140-pound safety hammer with an approximate 30-inch fall. Resistance to penetration was recorded as the number of hammer blows required to drive the sampler the final foot of an 18-inch drive. SPT blow counts shown on the final boring logs are directly measured (SPT sampler) blow counts (N-Values).

The boring and test pit logs with descriptions of the various materials encountered in each boring, a key to the boring symbols, and select laboratory test results are included in Appendix A. Actual transitions between soil types and layers may be gradual and may inherently vary or be different than noted on the logs. Ground surface elevations indicated on the soil boring and test pit logs were estimated to the nearest foot using Google Earth. Boring locations were established by estimating distances and angles from site landmarks. Subsurface conditions in unexplored locations or at other times may vary from those encountered at specific boring locations. If such variations are noted during construction or if project development plans are changed, CEL must review the changes and amend our recommendations, if necessary.

2.3. Laboratory Testing

Laboratory tests were performed on selected samples to determine some of the physical and engineering properties of the subsurface soils. The results of the laboratory testing are either presented on the boring logs, and/or are included in Appendix B. The following soil tests were performed for this study:

Moisture Content (ASTM D2216) –Moisture and density tests were conducted on selected samples to measure the in-place moisture content and density of the subsurface materials. These properties provide information for evaluating the physical characteristics of the subsurface soils. Test results are shown on the borings logs in Appendix A.

Particle Size Analysis (Wet and Dry Sieve) and Fines Content (ASTM D422 and D1140) - Sieve analysis or fines content (minus No. 200 sieve) tests were conducted on selected samples to determine the soil particle size distribution. This information is useful for the evaluation of liquefaction potential and characterizing the soil type according to USCS. Test results are presented on the boring logs and in Appendix B.

Chemical Tests - Water-soluble sulfate can affect the concrete mix design for concrete in contact with the ground, such as shallow foundations, piles, piers, and concrete slabs. A representative sample of the fine-grained soils was tested to measure sulfate content, pH, and resistivity. Test results are included in Appendix B and are summarized in Section 5.9 of this report.

3.0 GEOLOGIC AND SEISMIC OVERVIEW

3.1. Regional Geologic Setting

The site is located within Bartholomew Canyon east of Springville, Utah at an elevation of approximately 6250 feet. Available geologic maps suggest that the site is located in an area of Quaternary mass-movement and colluvial deposits, and includes landslides and areas of slope wash and soil creep. Subsurface conditions encountered in the field explorations are generally consistent with the geologic description, and are described in detail in section 4.0. The site is located within a narrow canyon; however, no signs of landslide activity were observed in the immediate vicinity of the proposed tank.

This region is generally bounded by the Wasatch Mountain Range. The Wasatch Fault Zone along the Wasatch Mountain Range, located approximately 5.25 miles to the southeast is capable of producing a magnitude 7.2 earthquake. Potential seismically-induced hazards include severe ground shaking, liquefaction of submerged loose granular soils, slope stability and other hazards as described in Section 3.2.

3.2. Seismic Induced Hazards

3.2.1. Ground Shaking

Published seismic data suggests that the site may experience very strong ground shaking from a major earthquake originating from one or more of the closer or major faults such as the Wasatch Fault, see *Figure A-3, Fault Map*, generally considered to be capable of generating an earthquake as high as Magnitude 7.2 (Arabasz et al., 1992). Refer to Table 2 in Section 5.2 of this report for recommended seismic design parameters based on a 2 percent probability of exceedance in a 50 year time period (recurrence interval of 2,475 years), as published by the USGS.

3.2.2. Liquefaction Induced Phenomena

Research and historical data indicate that soil liquefaction generally occurs in saturated, loose granular soil (primarily fine to medium-grained, clean sand deposits) during or after strong seismic ground shaking and is typified by a loss of shear strength in the affected soil layer, resulting in potential slope instability, densification and ground settlement, or lateral flow. According to the Utah Department of Natural Resources, see Figure A-4, *Liquefaction Potential Map*, the project site is located in a designated area classified as having a "very low" liquefaction potential. This suggests that there is less than a 5 percent chance that the site will experience ground shaking strong enough to induce liquefaction within saturated soils at the site in a 100-year time period. Based on the in-situ density and consistency of the on-site soils, the project site has a low risk of liquefaction-induced hazards for the design earthquake.

3.2.3. Consolidation Settlement

Consolidation is the densification of soil into a more dense arrangement from additional loading, such as from new fills or foundation loads. Consolidation of clayey soils is usually a long-term process, whereby the water is squeezed out of the soil matrix with time. Sandy soils consolidate relatively rapidly with an introduction of a load, typically during the construction process. Consolidation of soft fine-grained clay and silt soil layers can cause settlement of the ground surface and structures. The natural soils encountered in the exploration generally consist of fine to coarse granular soils extending below the proposed water tank bottom elevation, to the maximum depths explored of approximately 31.5 feet. Fine-grained clay soils were encountered, however, near the bottom of Test Pit TP-2. The extent of this clay layer was not determined during the field exploration phase due to limitations of the excavation equipment reach. CEL should be retained to observe the exposed soil conditions at the time of construction to document the location of the clay layer and to confirm anticipated subsurface conditions at the bottom of tank foundation elevation.

4.0 SUBSURFACE CONDITIONS

4.1. Subsurface Soil Conditions

The following paragraphs provide generalized descriptions of the subsurface profiles and soil conditions encountered within the borings conducted during this study. As previously noted, soil conditions may vary in unexplored locations.

The boring and test pits were performed to depths ranging from approximately 11 to 31.5 feet below existing grade. The soil conditions encountered in the boring and each of the test pits generally consists of approximately 7 to 10 feet of undocumented fill, overlaying natural gravel with minor to moderate amounts of clay and silt and significant cobbles and possible boulders to the maximum depths penetrated. A layer of topsoil and low plasticity clay was observed below the fill near the bottom of Test Pit TP-2. The locations of the boring and test pits are presented on *Figure A-2*, *Exploration Location Map*.

The undocumented, non-engineered fill encountered in the boring and test pits primarily consists of clayey SAND and sandy CLAY. This fill is likely the result of site grading and/or previous construction activities at the site. The fill is considered to be undocumented and unsuitable for the support of structures.

The natural clay is very moist, gray in color, soft to medium stiff and are anticipated to exhibit moderately low to moderate strength and moderate to moderately high compressibility characteristics under moderate loading.

The natural gravel is medium dense to very dense, slightly moist to very moist, and light brown and brown in color, and contain variable amounts of cobbles and possibly boulders.

Additional details of the subsurface conditions encountered in the exploratory borings, including laboratory test results, are included in the borings logs in Appendix A, and laboratory test summaries are presented in Appendix B. The lines designating the interface between soil types on the boring logs generally represent approximate boundaries. In situ, the transition between soil types may be gradual. It should also be noted that as the collection was done through a 2-inch diameter sampler, the samples collected in the boring during the exploration may not fully represent the true particle size gradation, such as the presence of over-size materials such as cobbles and boulders.

4.2. Groundwater Conditions

Groundwater was encountered within the boring and test pits at depths ranging from approximately 4 to 8 feet below grade at the time of our field exploration, and are near the original ground surface beneath the observed fill. Perched groundwater may be present above clayey layers observed in the explorations. Groundwater levels may vary with changes in precipitation, seasonal weather, surface water, and other site-specific factors. A detailed investigation of local groundwater conditions, or historical groundwater levels was not performed and is beyond the scope of this study; however, seasonal fluctuations in the groundwater level at the site on the order of at least 2-3 feet may be anticipated. The observed groundwater conditions appear to be consistent with anticipated groundwater elevations considering the location of nearby culinary water spring collection zones located immediately up-canyon from the project site. Based on our observations, a permanent maximum design groundwater depth of zero (existing grade) is recommended for use in the tank design.

Groundwater is anticipated to be encountered within the tank excavation and will require temporary construction dewatering and permanent considerations for saturated backfill and drainage conditions around the proposed water tank.

4.3. Existing Buried Water Pipeline

CEL was instructed to identify the location of the buried water pipeline leading from the spring water collection field to the north of the tank near the western portion of the proposed tank location as shown on Figure A-2 in Appendix A of this report. CEL used the mini-excavator to carefully unearth the top of the buried water pipeline which was encountered at a depth of approximately 7-1/2 feet below grade near the western portion of the site in Test Pit TP-3. The pipeline appeared to consist of a reinforced concrete pipe, approximately 24-inches in diameter. A portion of the pipe bell was damaged during excavation, and the Owner was contacted to verify if repairs were warranted. The depth and location of this utility are only approximate.

5.0 CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are based on our understanding of the proposed construction, site observations, our evaluation and interpretation of the field and laboratory data obtained during this exploration, our experience with similar subsurface conditions, and generally accepted geotechnical engineering principles and practices.

5.1. Seismic Design Parameters

The proposed structures should be designed in accordance with local design practices to resist the lateral forces generated by ground shaking associated with a major earthquake occurring within the Wasatch Range region.

Based on the subsurface conditions encountered in the borings and the geologic conditions of the site, we conservatively judge the site to be represented by IBC Site Class C, representative of generally dense soils observed in the explorations, and anticipated subsurface conditions over the upper 100 feet of the soil profile from geologic mapping and descriptions to be appropriate for this site.

For design of the proposed site structures in accordance with the seismic provisions of International Building Code (IBC 2012) and ASCE 7-10, utilizing a Site Class "C", the following seismic ground motion values should be used for design (see Table 2). The Site Class was determined using the average SPT values obtained during the field exploration and considering the local geology beneath the site.

Table 1. Seismic Coefficients Based on ASCE 7-10

Item	Value	ASCE 7-10 Standard Table/Figure/Eq[R2]
Site Class	"C"	
Mapped Spectral Response Accelerations Short Period, S_s 1-second Period, S_1	 0.769 g 0.264 g	 Figure 1613.3.1(1) Figure 1613.3.1(2)
Site Coefficient, F_a	1.092	Table 1613.3.3(1)
Site Coefficient, F_v	1.536	Table 1613.3.3(2)
MCE (S_{MS})	0.840 g	Equation 16-37
MCE (S_{M1})	0.406 g	Equation 16-38
Design Spectral Response Acceleration Short Period, S_{DS} 1-second Period, S_{D1}	 0.560 g 0.270 g	 Equation 16-39 Equation 16-40

R2 U.S. Seismic "Design Maps" Web Application, https://geohazards.usgs.gov/secure/designmaps/us/application.php

ASCE 7-10 § 11.6-1 and 11.6-2 indicate that the Seismic Design Category for all Occupancy Categories is "C". A PGA_M of <u>0.35</u> g using an F_{PGA} of 1.074 and PGA of 0.326 g was calculated for the site using USGS Seismic Design Tool for ASCE 7-10 ($PGA_M = PGA \times F_{PGA}$). These spectral values can be further modified by performing a Ground Response analysis, in accordance with ASCE 7-10

5.2. Earthwork

In general, earthwork for the project is anticipated to consist of excavation to a depth of approximately 25 feet below existing grade to reach the bottom tank elevation. CEL understands that excavated spoils will be stored onsite and be placed over the existing grade surrounding the tank. Based on the proposed tank elevation relative to the observed groundwater levels at the time of our field exploration, significant dewatering will be required prior to and during construction. Care should be taken not to disturb natural soils below or adjacent to existing pipelines, foundations or other structures to remain in place. Temporary unsupported excavation 'cut' slopes should be designed and constructed as discussed in Section 5.3. Temporary or permanent shoring may be required to accommodate property boundary or other site restrictions.

Up to approximately 10 feet of undocumented fill was encountered in the explorations conducted at the site. Additional undocumented fills may exist in unexplored areas of the site. All surficial loose/disturbed soils and non-engineered fills must be removed below the proposed tank. All existing utility locations should be reviewed to assess their impact on the proposed construction and abandoned and/or relocated as appropriate.

5.2.1. Excavation

Earthwork is generally anticipated to consist of excavation and removal of existing fill and natural soils to the anticipated bottom tank elevation of 6,222 feet, approximately 25 feet below existing grade. The soils found on site contain areas of very dense gravel, cobbles, and boulders. These materials may require excavation using heavy duty or specialized equipment, particularly in narrow, confined excavations such as utility trenches. Excavation may dislodge cobbles and boulders, requiring filling and compaction with Structural Fill. The proposed excavation is anticipated to be near existing underground utilities and pipelines that must be protected from damage during excavation and construction.

5.2.2. Site Preparation and Grading

Site stripping and grading should be performed in accordance with the recommendations contained in this report. A pre-construction conference should be held at the jobsite with representatives from the owner, general contractor, grading contractor, and CEL prior to starting the stripping and demolition operations at the site.

The proposed tank excavation site should be cleared of any debris, vegetation and organics, and other deleterious materials.

Excavated spoils may be evaluated by the Geotechnical Engineer for possible reuse and placement as fill materials. The grading contractor should be aware of any possible buried structures or underground utilities at the site which are to be removed or abandoned appropriately. Voids resulting from the removal of underground obstructions, or dislodged cobbles or boulders extending below the proposed bottom foundation elevation should be cleared and backfilled with properly compacted Structural Fill or other material approved by the Geotechnical Engineer.

Final grading should be designed to provide positive drainage away from the tank. Backfill zones and soil/landscape areas, if any, within 25 feet of the tank should slope at a minimum of four percent away from the tank.

5.2.3. Tank Subgrade Preparation

Following excavation to the required grades, the exposed tank foundation subgrade should be scarified to a depth of 12 inches. Exposed cobbles and boulders greater than 3 inches in size should be removed from along the bottom of the tank excavation. Voids resulting from the removal of over-sized materials or underground obstructions extending below the proposed finish grade should be over-excavated to create a smoothed depression that can be backfilled with properly compacted Structural Fill or other material approved by the Geotechnical Engineer. The scarified and prepared subgrade should be moisture conditioned and compacted as recommended in Section 5.2.7. The finished compacted subgrade surface should be firm and unyielding and should be protected from damage caused by construction activities, surface water, or weather conditions.

As an alternative to scarification and recompaction, the subgrade may be over-excavated and replaced with at least 2-feet of properly moisture conditioned, placed and compacted Structural Fill.

Placement of Controlled Low-Strength Material (CLSM) or flowable fill may also be considered to fill voids and/or to be placed over the subgrade to provide a working surface beneath the tank foundation.

Portions of the exposed subgrade could be unstable under construction equipment loads, and/or during placement and compaction Structural Fill. Possible options for wet subgrade stabilization include over-excavation and replacement with free-draining gravel (e.g. 2-inch minus drain rock) and/or placement of a stabilization geotextile or geogrid. CEL should be retained to observe subgrade construction to evaluate unstable conditions as they may be encountered and to provide recommended stabilization procedures for the conditions observed.

5.2.4. Dewatering

Dewatering is anticipated to occur prior to and during construction. The dewatering system should be designed by a qualified engineer or Contractor experienced the design and construction of dewatering systems in similar subsurface conditions, including potential perched water conditions. In general, dewatering may be accomplished using a system of sumps, and possibly well-points, connected to a system of pipes that collect and discharge water at a suitable location outside the limits of the tank excavation. However, the effectiveness of the dewatering system should be evaluated based on the subsurface soil and groundwater conditions observed. The dewatering system shall be designed to discharge collected water to a suitable location as determined by the Owner or designated project representatives. The dewatering system should be designed to operate continuously during construction to maintain the designed dewatering level and to protect the excavation cut slopes and the tank construction. The effectiveness of the dewatering system may be measured using observation wells placed within the dewatered zone, and should be monitored periodically through the construction period.

5.2.5. Temporary Cut and Fill Slopes

The Contractor should incorporate all appropriate requirements of OSHA into the design, construction, and maintenance of temporary construction slopes for construction. Excavation safety regulations are provided in the OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, Subpart P, and apply to excavations greater than five feet in depth.

The Contractor, or his specialty subcontractor, should design temporary construction slopes to conform to the OSHA regulations and should determine actual temporary slope inclinations based on the subsurface conditions exposed at the time of construction. For pre-construction planning purposes, temporary cut slope excavations greater than 4 feet deep through dewatered, granular soils, including the undocumented fill, may be constructed at 1.5:1 or flatter. Temporary cut slopes through dewatered clay soils or drained dense gravel may be constructed at 1:1 or flatter. Temporary excavations through drained soils, generally less than 5 feet in height may be constructed at near vertical.

If insufficient dewatering occurs, and/or if temporary slopes are left open for extended periods of time, exposure to weather and rain could have detrimental effects such as sloughing and erosion on surficial soils exposed in the excavations. We recommend that all vehicles and other surcharge loads be kept at least 10 feet, or at least the height of the slope away from the top of temporary slopes, whichever is greater. Temporary slopes shall be protected from excessive drying or saturation during construction. Adequate provisions should be made to divert surface water from ponding on top of the slope and from flowing over the slope face. Desiccation or excessive moisture in the excavation could reduce stability and require shoring or laying back side slopes.

A ‘Competent Person’ as defined by OSHA should regularly inspect temporary cut slopes for signs of distress including sloughing, raveling, or cracking. Mitigation plans should be established to allow slopes to be laid back or for buttress fill to be placed against distressed slopes to avoid injuring workers in the tank excavation, or to the construction.

Temporary uncompacted fill slopes up to 10 feet tall should be constructed at 2:1 (horizontal to vertical) or flatter. Steeper and or higher slopes may be possible depending on the type of fill materials used and their relative compaction.

5.2.6. Temporary Shoring

Considering the depth of the excavation and the site boundary constraints with respect to the allowable temporary cut slopes and potential for slope distress during construction, CEL anticipates that shoring may be required at some locations. Shoring systems shall be designed considering the subsurface soil and groundwater conditions described in this report and on any other observations of subsurface conditions that may be available. Possible shoring systems for soil retainage less than approximately 15 feet may include cantilevered soldier piles and lagging, soil nails, or other cost efficient systems that can be constructed in the subsurface conditions observed. Driven sheet piles are not anticipated to be effective due to anticipated resistance in the dense gravel/cobble/boulder environment. Shoring systems should be designed to resist lateral earth pressures consistent with the soil conditions observed along the sides of the excavation, and may include clay as well as the gravel soils as described in Section 5.5. For clay soils, temporary cantilevered shoring systems should be designed to resist a lateral equivalent fluid pressure of 50 pcf for active drained (unsaturated) conditions. Temporary shoring should be designed considering undrained conditions consistent with the anticipated water levels and dewatering activities during construction. CEL should be retained to review all shoring submittals for consistency with the geotechnical recommendations of this report prior to mobilizing equipment or materials to the project site.

5.2.7. Fill Material Requirements

In general, fill materials should be non-expansive, well-graded, having a Plasticity Index of 12 or less and enough fines so the soil can bind together. Fill materials should be free of environmental contaminants, organic materials and debris, and should not contain rocks or lumps greater than three inches in maximum size. Based on the results of the field and laboratory testing programs, portions of the on-site materials may be suitable for use as fill if they meet these requirements and can be effectively screened to remove over-sized materials, segregated, and stock piled for re-use. All fill materials should be sampled, tested and approved by the Geotechnical Engineer prior to use on site to ensure they meet fill material requirements as follows:

Structural Fill

Structural Fill beneath the tank or other structures should consist of clean, well-graded granular materials having a maximum particle size of 3 inches, less than 12% passing the No. 200 Sieve, and non-plastic fines having a Plasticity Index of less than 15. Structural Fill shall extend laterally away from the edge of footings a distance equal to the depth of fill beneath the footings.

Engineered Fill

Engineered Fill may be used as backfill around the tank and as site grading fill and should consist of generally granular soils having a maximum particle size of 4 inches, and no more than 35 percent fines with a Plasticity Index of less than 15%.

On-site soils below any stripped material and having an organic content of less than three percent by weight, free of construction debris and meeting the requirements for Engineered fill may be reused as general fill as approved by the Geotechnical Engineer. However, considering the amounts of gravel and cobbles found in most of soils encountered, these soils may require significant processing to remove the over-sized particles.

5.2.8. Project Compaction Recommendations

Table 3 provides the recommended compaction requirements for this project, based on the Modified Proctor laboratory compaction test (ASTM Test Method D1557). Depending on final project details, some items listed below may not apply to this project. Specific moisture conditioning and relative compaction requirements should comply with approved project specifications and standards, where applicable, if different than recommended herein.

Properly placed and compacted Structural Fill or Engineered Fill or Backfill may be designed to have a minimum compacted unit weight of 130 pounds per cubic foot (pcf) under drained conditions. An effective unit weight of 65 pcf may be used for saturated backfill conditions, in addition to the unit weight of water for hydrostatic conditions.

Table 2. Project Compaction Recommendations

Description	Percent Relative Compaction (ASTM D1557)	Optimum Moisture Content
Subgrade	90	+/- 3

Description	Percent Relative Compaction (ASTM D1557)	Optimum Moisture Content
Structural Fill Beneath Footings and Tank	95	+/- 3
Engineered Fill or Backfill	95	+/- 3
Backfill within 5 feet of Tank Wall or Utility	90	+/- 3
Underground Utility Trench Backfill (< 5 feet)	90	+/- 3
Underground Utility Trench Backfill (> 5 feet)	95	+/- 3
Underground Utility Backfill – Landscape Areas	85	+/- 3
Underground Utility Backfill, Clean Sand	95	+/- 2

Fill materials should be moisture conditioned prior to placement in loose horizontal lifts not to exceed 8 inches, and compacted with appropriately sized compaction equipment in uniform passes until the desired relative compaction levels are achieved.

In order to achieve satisfactory compaction of fill materials, it may be necessary to adjust the water content of the fill soils prior to placement and compaction. Moisture conditioning (drying or wetting) may be done most effectively in hot, dry months (e.g. during summertime), and should be avoided in colder, wet months. On-site excavated spoils are generally anticipated to be very moist to wet and may require significant work, time and area to spread out in wind-rows to achieve a suitable moisture content prior to placement and compaction.

Only hand-operated compaction equipment, such as jumping jacks, should be used to compact backfill within 5 feet of tank walls and buried utilities.

5.3. Utility Trench Design and Construction

5.3.1. Utility Trench Design

Buried utility trenches, including pipelines, should be designed according to approved project standards and specifications for plan/profile, pipe bedding and backfill zones, and minimum relative compaction. Pipelines located beneath the water table should be designed to resist hydrostatic uplift (buoyancy) forces based on the recommended minimum backfill unit weights and the maximum water levels considered at the site.

5.3.2. Trench Backfilling

Utility trenches may be backfilled with approved imported pipe bedding sand meeting project specifications and Engineered Fill or Backfill, in accordance with project approved standards and specifications. Portions of the onsite soils may be suitable for pipe bedding, however, they may require significant screening to remove over-size particles to meet the required gradation.

Pipeline trenches should be backfilled with engineered fill placed in lifts of approximately 8 inches in precompacted thickness, and compacted to the requirements presented in Table 3. Thicker lifts can be considered, provided the method of compaction is approved by the Geotechnical Engineer, and the required minimum degree of compaction is achieved and verified through in-situ moisture-density testing.

5.4. Tank Foundations

The proposed tank may be supported on conventional continuous ring footing and isolated spread footings or thickened slabs bearing on properly prepared natural subgrade or properly placed and compacted structural fill extending down to properly prepared subgrade. Continuous footings should have a minimum width of at least 18 inches, and isolated column footings should have a minimum width of at least 24 inches. Footings located adjacent to other footings or utilities should bear below an imaginary 1:1 (horizontal to vertical) plane projected upward from the bottom edge of the adjacent footings or utility trenches.

The proposed structure may be supported on foundations bearing directly on natural soils, or properly placed and compacted Structural Fill. Footings bearing on natural granular soils or a minimum of 2 feet of properly placed and compacted Structural Fill extending to natural granular soils may be designed for a net allowable bearing capacity as described below:

- 3,000 psf for footings less than 4 feet square
- 5,000 psf for footings 4 feet and larger

These bearing capacities are net values, as the weight of the footing itself has already been accounted for and can be neglected as a load for design purposes. The allowable bearing capacity may be increased by 1/3 for temporary instantaneous loads.

Footings designed for the allowable bearing pressures listed above are anticipated to experience settlements much less than 1 inch, with anticipated differential settlement of less than ¼ inch across the tank floor. The majority of the anticipated settlement will occur instantaneously during tank construction and filling. The tank should be designed to accommodate the estimated total and differential settlements presented in this report.

A geotechnical engineering representative from CEL should be retained to observe the and confirm that footing excavations bear in soils suitable for the recommended maximum design bearing capacity prior to formwork and reinforcing steel placement. If any unsuitable supporting soil is encountered, the footing excavation should be deepened until suitable supporting, undisturbed native material is encountered. The over-excavation should be backfilled using structural fill or lean concrete (or a sand-cement slurry mix acceptable to the Geotechnical Engineer) up to the bottom of the footing concrete.

Footing excavations should have firm bottoms and be free from excessive slough prior to concrete or reinforcing steel placement. Care should also be taken to prevent excessive wetting or drying of the bearing materials during construction. Extremely wet or dry or any loose or disturbed material in the bottom of the footing excavations should be removed prior to placing concrete. If construction occurs during the winter months, a thin layer of concrete (sometimes referred to as a rat slab or mud mat) could be placed at the bottom of the footing excavations if needed to protect the bearing soil and facilitate removal of water and slough if rainwater fills the excavations.

5.5. Lateral Resistance and Earth Pressures

Foundation elements can resist lateral loads with a combination of bottom friction and passive resistance. An allowable coefficient of friction of 0.35 may be utilized for the footing interface with properly prepared natural soils and 0.40 for footing interface with a minimum of 2 feet of properly placed and compacted granular Structural Fill.

The proposed buried tank may be designed using the following static lateral earth pressures for free-draining horizontal Engineered Fill or Backfill conditions with sufficient drainage to preclude the development of hydrostatic pressures behind the wall:

- Cantilevered retaining walls designed to freely move or rotate may be designed using an active earth (equivalent fluid) pressure of 27 pounds per cubic foot (pcf).
- Restrained walls pinned at the top and bottom or otherwise restricted to movements less than 0.005xH, with H being the retained height, may be designed using an at-rest fluid pressure of 50 pcf.
- A passive earth pressure resistance of 500 pcf may be used for walls or structures pushing into the undisturbed natural granular soil, or 350 pcf for undisturbed natural clay soils; however, these values should be reduced by ½ if used in conjunction with the lateral friction factor for the design of retaining structures.
- For seismic conditions, the active and at-rest lateral earth equivalent fluid pressure values should be increased by 28 pcf.

The above earth pressures are based on properly placed and compacted Engineered Fill or Backfill material having a unit weight of 130 pcf and a friction angle of 38 degrees.

Higher lateral earth pressures will be present for retaining walls greater than 6 feet, sloping backfill or for non-free-draining and/or hydrostatic conditions. CEL should be retained to review retaining wall plans, calculations and final site grading information for consistency with the geotechnical recommendations.

5.6. Corrosion Testing

A representative sample of the fine-grained soils was selected for testing to measure sulfate content, pH, and resistivity. Test results are summarized in Table 4 below.

Table 3. Corrosion Test Results

Soil Description	Sample Depth (feet)	Sulfate (mg/kg)	pH	Resistivity
TP-2, Silty CLAY with fine sand	11	8.94	7.85	5,560

Granular soils inherently have lower concentrations of water-soluble sulfates, therefore, are considered to have a negligible sulfate attack potential on concrete or corrosion of buried metal utilities.

Water-soluble sulfate can affect the concrete mix design for concrete in contact with the ground, such as shallow foundations, piles, piers, and concrete slabs. Section 4.3 in American Concrete Institute (ACI) 318, as referenced by the IBC, provides the following evaluation criteria shown on Table 5:

Table 4. Sulfate Evaluation Criteria

Sulfate Exposure	Water-Soluble Sulfate in Soil, Percentage by Weight or (mg/kg)	Sulfate in Water, ppm	Cement Type	Max. Water Cementitious Ratio by Weight	Min. Unconfined Compressive Strength, psi
Negligible	0.00-0.10 (0-1,000)	0-150	NA	NA	NA
Moderate	0.10-0.20 (1,000-2,000)	150-1,500	II, IP (MS), IS (MS)	0.50	4,000
Severe	0.20-2.00 (2,000-20,000)	1,500-10,000	V	0.45	4,500
Very Severe	Over 2.00 (20,000)	Over 10,000	V plus Pozzolan	0.45	4,500

Based on these test results, the site soils are anticipated to have a negligible impact on buried concrete structures at the site. Type I/II Portland Cement may be used without Pozzolan or other minimum cementitious ratio or strength requirements as indicated in the ACI table above.

The corrosion test results are preliminary, and provide information only for the specific soils sampled and tested. Other soils at the site may be more or less corrosive. Providing a complete assessment of the corrosion potential of the site soils is not within our scope of work. For specific long-term corrosion control design recommendations, we recommend that a professional corrosion engineer evaluate the corrosion potential of the soil environment on buried concrete structures, steel pipe coated with cement-mortar, and ferrous metals.

5.7. Drainage

Site grading should be designed to establish and maintain a positive grade of at least 4 percent away from the tank backfill for a distance of at least 25 feet.

Tank walls should be designed with a permanent drainage system to preclude the build-up of hydrostatic water pressures. The drainage system should consist of at least a prefabricated geo-composite drainage panel and/or a chimney drain consisting of free-draining sand or gravel placed directly adjacent to the tank wall. A drainage collection system should be designed to collect and transmit the collected drain water to a suitable discharge location.

5.8. Observation and Testing During Construction

We recommend that CEL be retained to provide observation and construction material testing services during site preparation, tank excavation, site grading, and utility trench backfill to observe compliance with the design concepts, specifications and recommendations, and to allow for possible changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

6.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report are based upon the soil and groundwater conditions encountered in the boring and test pit. If variations or undesirable conditions are encountered during construction, CEL should be contacted so that supplemental recommendations may be provided.

This report is issued with the understanding that it is the responsibility of the owner or his representatives to see that the information and recommendations contained herein are called to the attention of the other members of the design team and incorporated into the plans and specifications, and that the necessary steps are taken to see that the recommendations are implemented during construction.

The findings and recommendations presented in this report are valid as of the present time for the development as currently proposed. However, changes in the conditions of the property or adjacent properties may occur with the passage of time, whether by natural processes or the acts of other persons. In addition, changes in applicable or appropriate standards may occur through legislation or the broadening of knowledge. Accordingly the findings and recommendations presented in this report may be invalidated, wholly or in part, by changes outside our control. Therefore, this report is subject to review by CEL after a period of three (3) years has elapsed from the date of issuance of this report. In addition, if the currently proposed design scheme as noted in this report is altered, CEL should be provided the opportunity to review the changed design and provide supplemental recommendations as needed.

Recommendations are presented in this report which specifically request that CEL be provided the opportunity to review the project plans prior to construction and that we be retained to provide observation and testing services during construction. The validity of the recommendations of this report assumes that CEL will be retained to provide these services.

This report was prepared upon your request for our services, and in accordance with currently accepted geotechnical engineering practice. No warranty based on the contents of this report is intended, and none shall be inferred from the statements or opinions expressed herein.

The scope of our services for this report did not include an environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater or air, on, below, or around this site. Any statements within this report or on the attached figures, logs or records regarding odors noted or other items or conditions observed are for the information of our client only.

8.0 REFERENCES

American Society for Testing and Materials, West Conshohocken, Pennsylvania.

American Society of Civil Engineers, 2013, Minimum Design Loads for Buildings and Other Structures; ASCE/SEI Standard 7-10.

Arabasz, W.J., Pechmann, J.C., and Brown, E.D., 1992, Observational seismology and the evaluation of earthquake hazards and risk in the Wasatch Front area, Utah; in Gori, P.L., and Hays, W.W., eds., Assessment of regional earthquake hazards and risk along the Wasatch Front, Utah: U.S. Geological Survey Professional Paper 1500-D, 36 p.

International Building Code, 2012, published by International Code Council, Inc., USA.

U. S. Geological Survey Earthquake Information Center, website earthquake.usgs.gov.

Utah Geologic Survey, website: https://geology.utah.gov/map-pub/maps/geologic-maps/

Publications may have been used as general reference and not specifically cited in the report text.

APPENDIX A

FIGURES

FIELD EXPLORATION

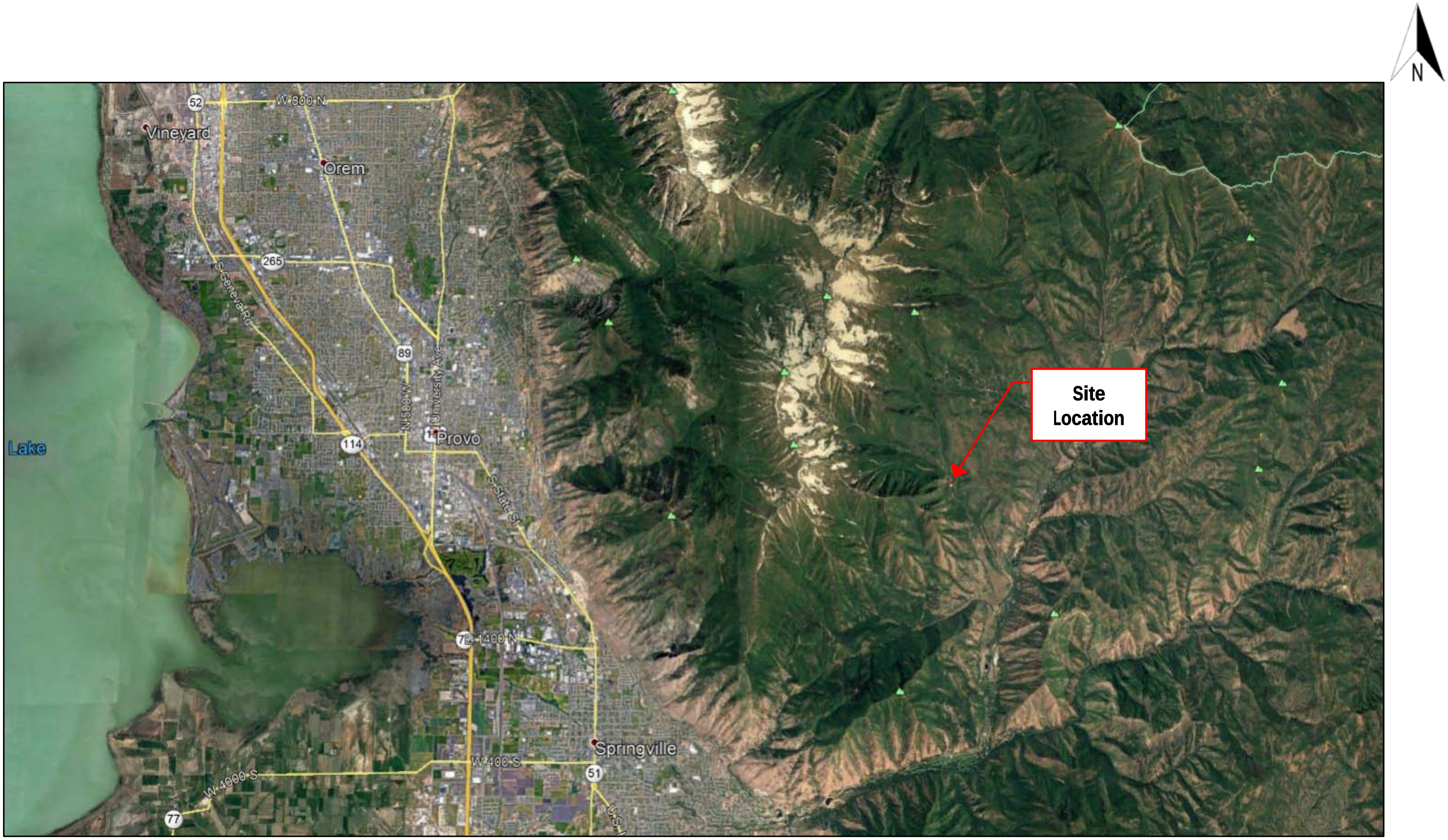
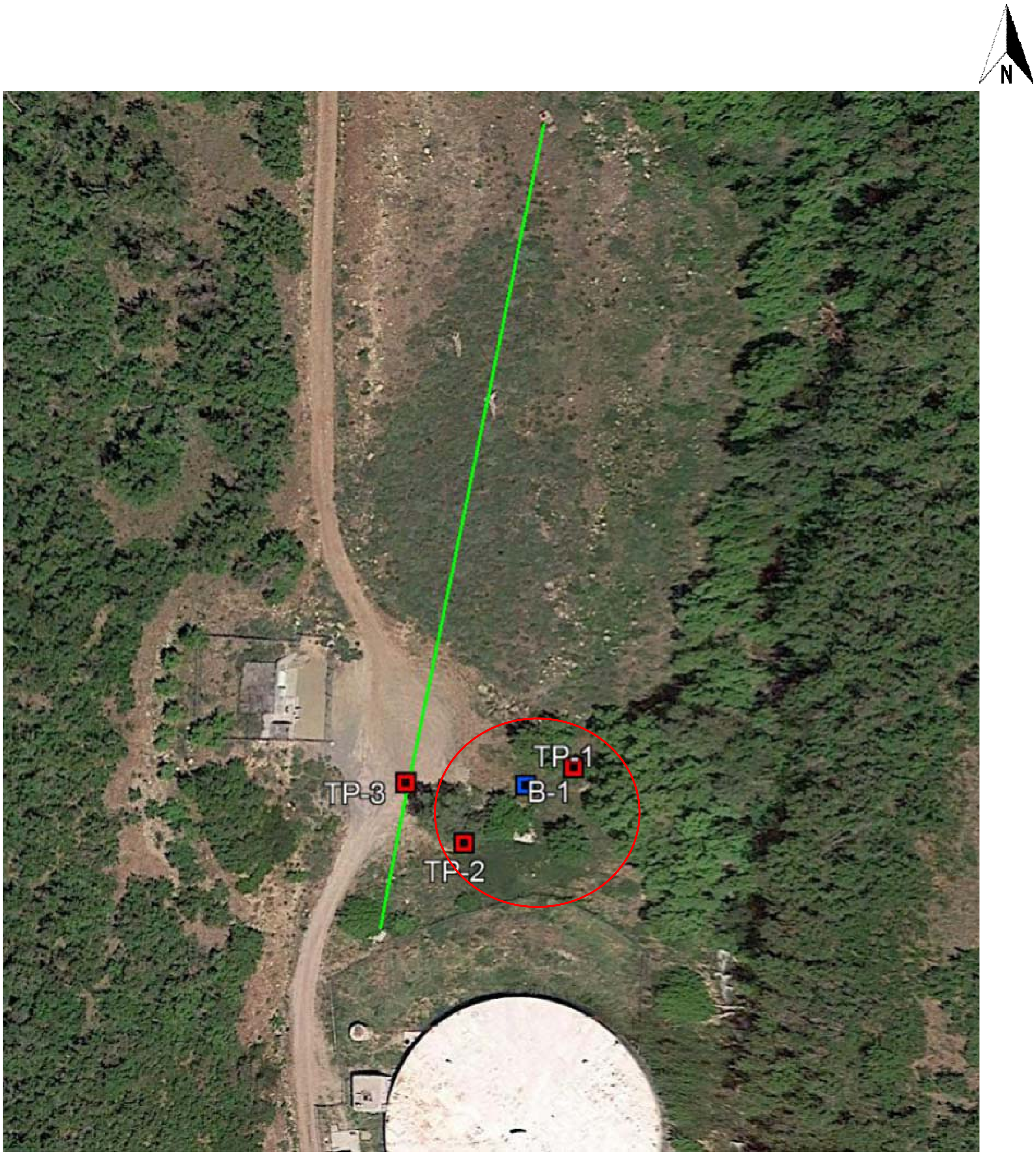


FIGURE A-1
SITE VICINITY
SOURCE: Google Earth (06/17/2017)
DATE: 03/28/2018
SCALE: Not to Scale

CONSOLIDATED ENGINEERING LABORATORIES

Hobble Creek Canyon Water Tank

Bartholomew Canyon
Springville, Utah
CEL Project Number 20-06720



Approximate Water Tank Location

Approximate Test Pit Location

Approximate Boring Location

FIGURE A-2
EXPLORATION LOCATION MAP
SOURCE: Google Earth (06/17/2017)
DATE: 03/28/2018
SCALE: Not To Scale

CONSOLIDATED ENGINEERING LABORATORIES

Hobble Creek Canyon Water Tank

Bartholomew Canyon
Springville, Utah
CEL Project Number 20-06720

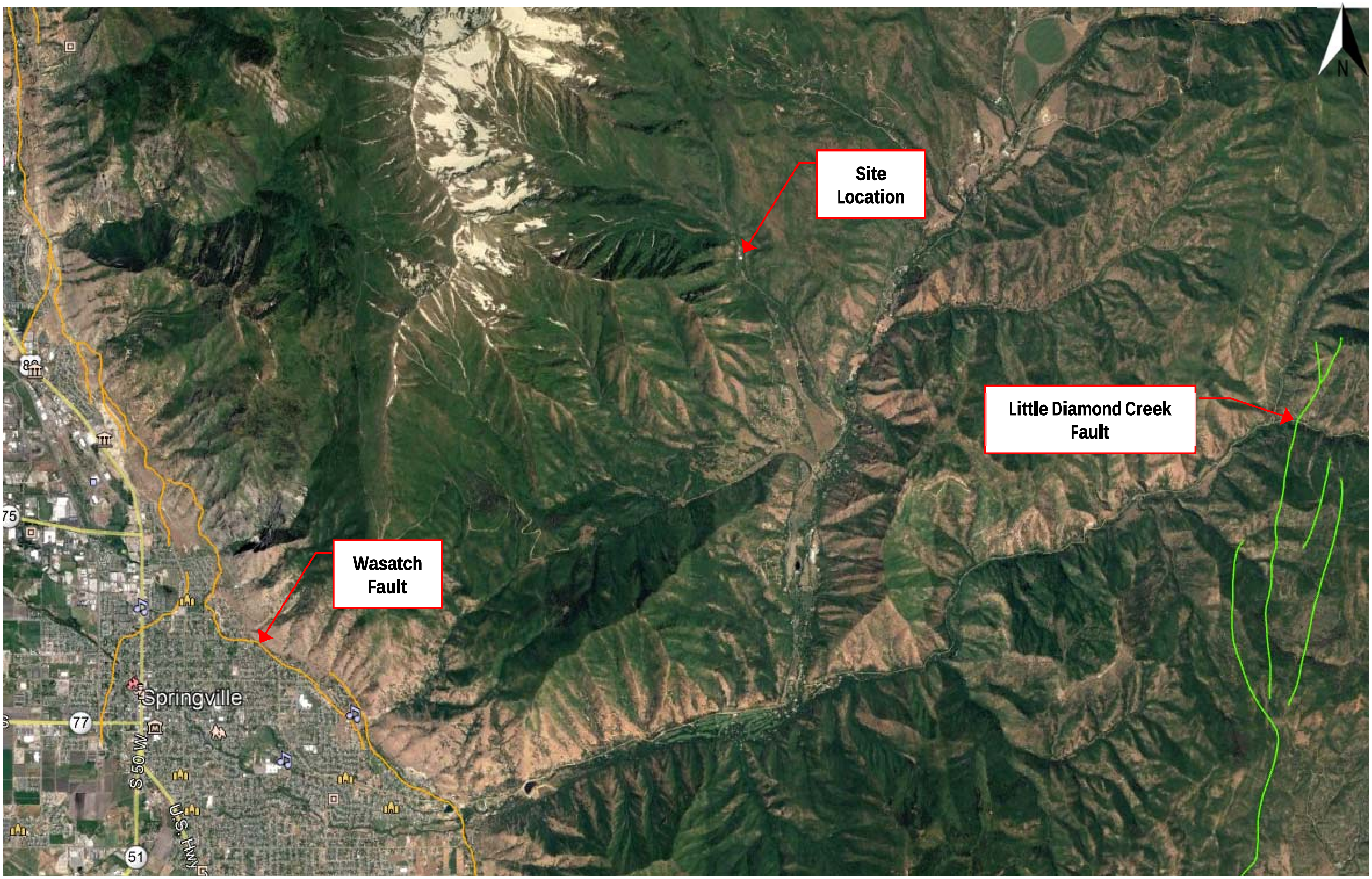


FIGURE A-3
FAULT MAP
SOURCE: Utah Geologic Survey/Google Earth
DATE: 03/29/2018
SCALE: Not to Scale

CONSOLIDATED ENGINEERING LABORATORIES

Hobble Creek Canyon Water Tank

Bartholomew Canyon
Springville, Utah
CEL Project Number 20-06720

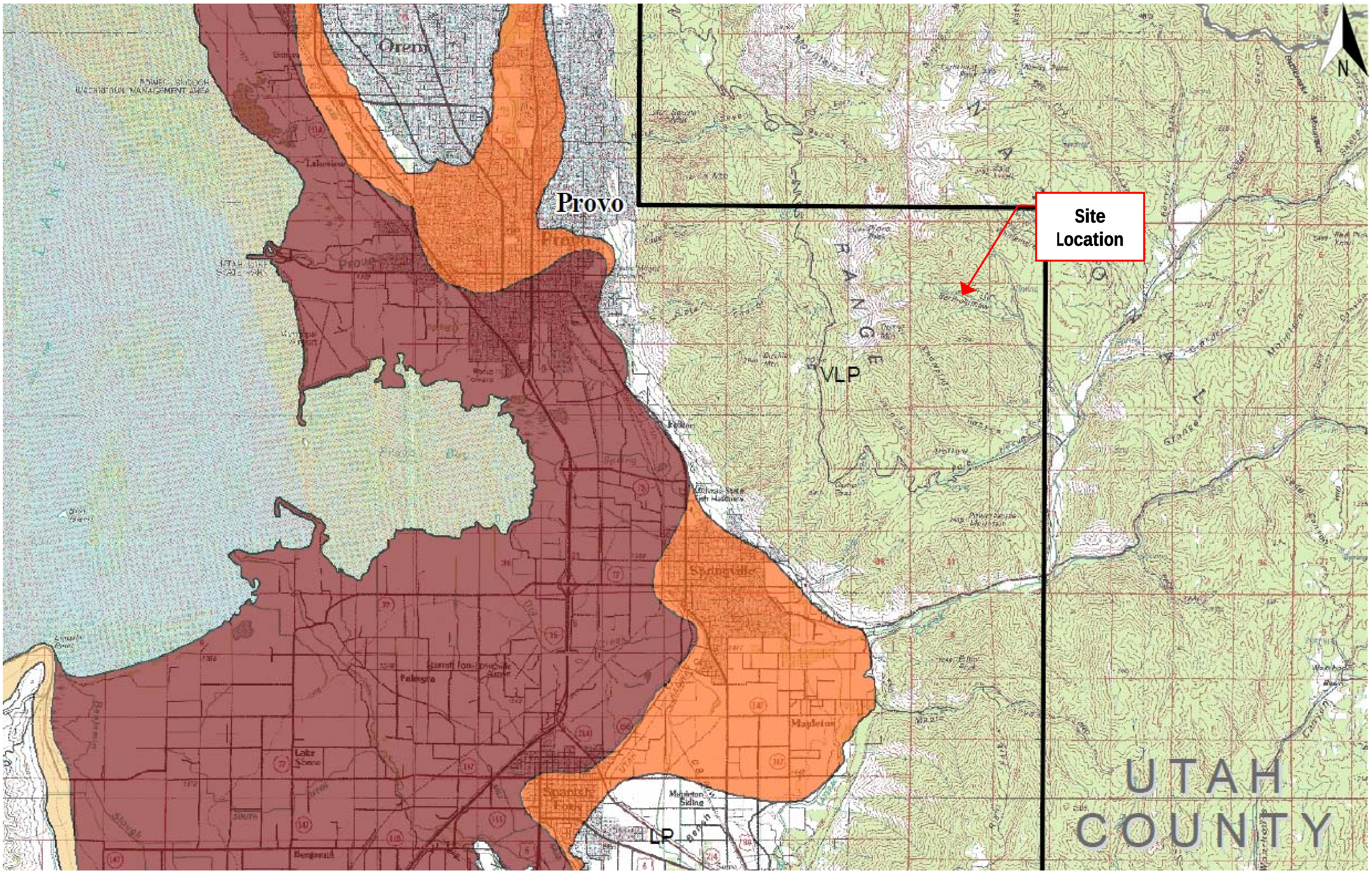


FIGURE A-4
LIQUIFACTION MAP
SOURCE: Utah Geologic Survey
DATE: 03/29/2018
SCALE: Not to Scale

CONSOLIDATED ENGINEERING
LABORATORIES

Hobble Creek Canyon Water Tank

Bartholomew Canyon
Springville, Utah
CEL Project Number 20-06720

FIGURE A-5
GEOLOGY MAP
SOURCE: Utah Geologic Survey
DATE: 03/28/2018
SCALE: Not to Scale

CONSOLIDATED ENGINEERING LABORATORIES

Hobble Creek Canyon Water Tank

Bartholomew Canyon
Springville, Utah
CEL Project Number 20-06720

CONSOLIDATED ENGINEERING LABORATORIES

Hobble Creek Canyon Water Tank

Bartholomew Canyon Road
Springville, Utah

BORING B-1

PROJECT NUMBER 20-06720
CLIENT Jones & DeMille
DATE STARTED 3/16/18 **COMPLETED** 3/16/2018
GROUND ELEVATION 6246.91 feet **HOLE DEPTH** 31.5 feet
DRILLER Earthcore Drilling **RIG TYPE** B-80
GROUND WATER LEVELS:
BORING LOCATION:
DRILLING METHOD Auger **HAMMER TYPE** Safety Hammer
AT TIME OF DRILLING 8 feet
LAT: 40.221962 °
LOGGED BY REG **CHECKED BY** CTG
AT END OF DRILLING 8 feet
LONG: -111.507964 °

DEPTH (ft)	SAMPLE TYPE NUMBER	FIELD: SPT BLOW COUNTS (N VALUE)	FIELD: RECOVERY (IN)	FIELD: STRENGTH (tsf)	GRAPHIC	MATERIAL DESCRIPTION	LABORATORY: FINES (%)	LABORATORY: DRY DENSITY (pcf)	LABORATORY: MOISTURE CONTENT (%)	REMARKS AND OTHER TESTS
0						FILL: Clayey fine to coarse SAND with fine and coarse gravel, moist, dark brown				
	SPT 1	5-5-8 (13)				-medium dense				
5	SPT 2	2-10-17 (27)				-dense			5.9	
						Fine to coarse sandy fine and coarse GRAVEL with clay, cobbles, and boulders				
∇ (8)	SPT 3	12-50/4"				-very dense				
10	SPT 4	50/2"				-very moist				
15	SPT 5	23-50/2"								
20	SPT 6	50/1"								
25	SPT 7	6-22-20 (42)				-dense				
30	SPT 8	50/4"				-very dense				

Boring terminated at 31.5 feet.

CONSOLIDATED ENGINEERING LABORATORIES

Hobble Creek Canyon Water Tank

Bartholomew Canyon Road
Springville, Utah

TEST PIT
TP-1

PROJECT NUMBER 20-06720
CLIENT Jones & DeMille
DATE STARTED 3/16/18 **COMPLETED** 3/16/2018
GROUND ELEVATION 6245.93 feet **TEST PIT DEPTH** 11 feet
DRILLER Nelson Contractors **RIG TYPE** Kamatsu PC50MR
GROUND WATER LEVELS:
BORING LOCATION:
EXCAVATION METHOD Excavation **N/A TYPE**
AT TIME OF EXCAVATION 4 feet
LAT: 40.221988 °
LOGGED BY REG **CHECKED BY** CTG
AT END OF EXCAVATION 4 feet
LONG: -111.507877 °

DEPTH (ft)	SAMPLE TYPE NUMBER	FIELD: SPT BLOW COUNTS (N VALUE)	FIELD: RECOVERY (IN)	FIELD: STRENGTH (tsf)	GRAPHIC	MATERIAL DESCRIPTION	LABORATORY: FINES (%)	LABORATORY: DRY DENSITY (pcf)	LABORATORY: MOISTURE CONTENT (%)	REMARKS AND OTHER TESTS
0.0						FILL: Fine to coarse SAND with clay, fine and coarse gravel, cobbles, and organics, slightly moist, dark brown				
	GB 1						6.9			
2.5										
▽ (4 ft)						-very moist				
5.0										
						-with boulders				
7.5										
	GB 2					-with wood fragments and heavy organics				
10.0										
	GB 3					Silty fine to coarse sandy fine and coarse GRAVEL with cobbles and boulders			11.7	

Refusal at 11.0 feet.

CONSOLIDATED ENGINEERING LABORATORIES

Hobble Creek Canyon Water Tank

Bartholomew Canyon Road
Springville, Utah

TEST PIT TP-2

PROJECT NUMBER 20-06720
CLIENT Jones & DeMille
DATE STARTED 3/16/18 **COMPLETED** 3/16/2018
GROUND ELEVATION 6244.74 feet **TEST PIT DEPTH** 12 feet
DRILLER Nelson Contractors **RIG TYPE** Kamatsu PC50MR
GROUND WATER LEVELS:
BORING LOCATION:
EXCAVATION METHOD Excavation **N/A TYPE**
AT TIME OF EXCAVATION 7 feet **LAT:** 40.221881 °
LOGGED BY REG **CHECKED BY** CTG
AT END OF EXCAVATION 7 feet **LONG:** -111.508078 °

DEPTH (ft)	SAMPLE TYPE NUMBER	FIELD: SPT BLOW COUNTS (N VALUE)	FIELD: RECOVERY (IN)	FIELD: STRENGTH (tsf)	GRAPHIC	MATERIAL DESCRIPTION	LABORATORY: FINES (%)	LABORATORY: DRY DENSITY (pcf)	LABORATORY: MOISTURE CONTENT (%)	REMARKS AND OTHER TESTS
0.0						FILL: Silty fine sandy CLAY with organics, slightly moist to moist, light brown and brown				
	GB 1									
2.5						-organics grade out				
						-with fine and coarse gravel				
5.0										
7.5						-with gray layers up to 12" thick				
	GB 2					-with cobbles and boulders			26.8	
						Topsoil: 12 inches				
10.0	GB 3					Silty CLAY with fine sand, very moist, gray	13.8			
	GB 4									

Test pit caved at 12.0 feet.

Figure A-8

CONSOLIDATED ENGINEERING LABORATORIES

Hobble Creek Canyon Water Tank

Bartholomew Canyon Road
Springville, Utah

KEY TO SYMBOLS

PROJECT NUMBER 20-06720 **CLIENT** Jones & DeMille

LITHOLOGIC SYMBOLS
(Unified Soil Classification System)

CL: Low Plasticity Clay (CL)

FILL: Fill

GM: Silty Gravel (GM)

GP-GC: Poorly-graded Gravel with Clay

TOPSOIL: Topsoil

SAMPLER SYMBOLS

Grab Sample

Standard Penetration Test

N-Value

Number of blows 140 LB hammer falling 30 inches to drive a 2 inch outside diameter (1-3/8 inch I.D) split barrel sampler the last 12 inches of an 18 inch drive (ASTM-1586 Standard Penetration Test)

Blow Counts

The number of blows of the sampling hammer required to drive the sampler through each of three 6-inch increments. Less than three increments may be reported if more than 50 blows are counted for any increment.
The notation 50/5" indicates 50 blows recorded for 5 inches of penetration.

SOIL MOISTURE	
DESCRIPTOR	**DESCRIPTION**
Dry	Dry of Standard Proctor Optimum
Damp	Sand Dry
Moist	Near Standard Proctor Optimum
Wet	Wet of Satandard Proctor Optimum
Saturated	Free Water in Sample

PARTICLES SIZES		
COMPONENTS		**SIZE OR SIEVE NUMBER**
Boulders		Over 12 Inches
Cobbles		3 to 12 Inches
Gravels	-Coarse	3/4 to 3 Inches
	-Fine	Number 4 to 3/4 Inch
Sand	-Coarse	Number 10 to Number 4
	-Medium	Number 40 to Number 10
	-Fine	Number 200 to Number 40
Fines (Silt and Clay)		Below Number 200

PENETRATION RESISTANCE (RECORDED AS BLOWS/0.5 FEET)				
SAND AND GRAVEL		SILT AND CLAY		
RELATIVE DENSITY	N-VALUE (BLOWS/FOOT)*	CONSISTENCY	N-VALUE (BLOWS/FOOT)*	COMPRESSIVE STRENGTH
Very Loose	0 - 3	Very Soft	0 - 1	0 - 0.25
Loose	4 - 10	Soft	2 - 4	0.25 - 0.50
Medium Dense	11 - 29	Medium Stiff	5 - 7	0.50 - 1.0
Dense	30 - 49	Stiff	8 - 14	1.0 - 2.0
Very Dense	50 +	Very Stiff	15 - 29	2.0 - 4.0
		Hard	30 +	Over 4.0

ABBREVIATIONS

LL - LIQUID LIMIT (%)
PI - PLASTIC INDEX (%)
W - MOISTURE CONTENT (%)
DD - DRY DENSITY (PCF)
NP - NON PLASTIC
-200 - PERCENT PASSING NO. 200 SIEVE
PP - POCKET PENETROMETER (TSF)
TV TORVANE (TSF)
ppm PART PER MILLION

Figure A-9

MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS More than Half > #200 sieve	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
		GRAVELS WITH OVER 15% FINES	GM	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS
			SP	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 15% FINES	SM	SILTY SANDS, POOORLY GRADED SAND-SILT MIXTURES
			SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS More than Half < #200 sieve	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS

	Modified California	RV	R-Value
	Split Spoon	SA	Sieve Analysis
	Pushed Shelby Tube	AT	Atterberg Limits
	Auger Cuttings	TC	Cyclic Triaxial
	Grab Sample	UU	Unconsolidated Undrained Triaxial
	Sample Attempt with No Recovery	TV	Torvane Shear
CA	Chemical Analysis	UC	Unconfined Compression
CN	Consolidation	(1.2)	(Shear Strength, ksf)
CP	Compaction	WA	Wash Analysis
DS	Direct Shear	(20)	(with % Passing No. 200 Sieve)
PM	Permeability	▽	Water Level at Time of Drilling
PP	Pocket Penetrometer	▼	Water Level after Drilling(with date measured)

CONSOLIDATED ENGINEERING LABORATORIES

SOIL CLASSIFICATION CHART AND KEY TO TEST DATA

Hobble Creek Canyon Water Tank

Figure A-10

CONSOLIDATED ENGINEERING LABORATORIES

SUBSURFACE CROSS-SECTION DIAGRAM

CLIENT Jones & DeMille

PROJECT NUMBER 20-06720

PROJECT NAME Hobble Creek Canyon Water Tank

PROJECT LOCATION

Figure A-11

APPENDIX B

LABORATORY RESULTS

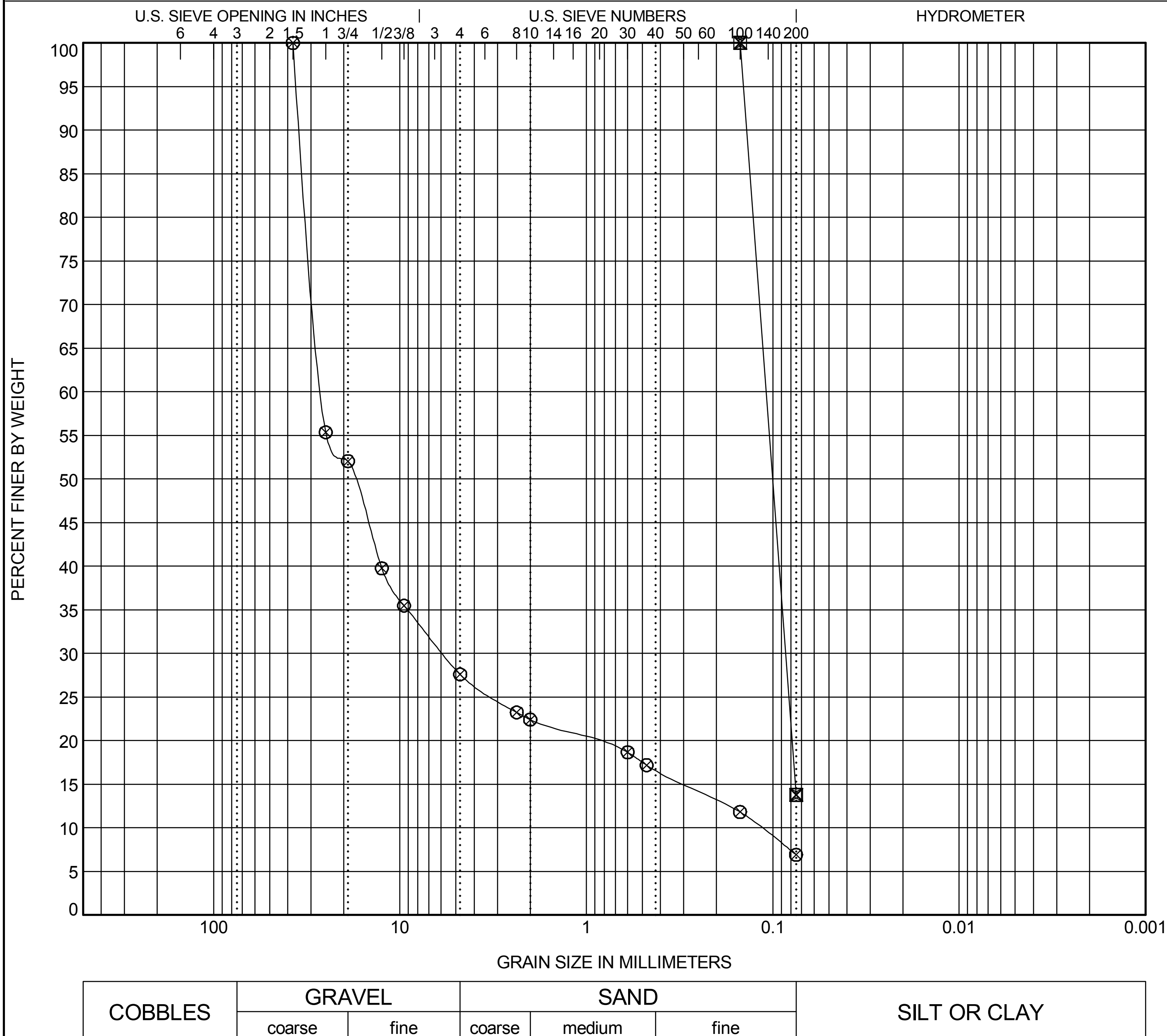
CONSOLIDATED ENGINEERING LABORATORIES

Hobble Creek Canyon Water Tank

Bartholomew Canyon Road
Springville, Utah

PARTICLE SIZE ANALYSIS

PROJECT NUMBER 20-06720 **CLIENT** Jones & DeMille



	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
⊗	**TP-1**	**1.0 feet**	**GW-GC - Sandy GRAVEL with clay, cobbles, and organics**				**11.39**	**225.02**
⊠	**TP-2**	**9.5 feet**	**SC - TOPSOIL**					

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
⊗	**TP-1**	**1.0**	**37.5**	**26.074**	**5.865**	**0.116**	**72.4**	**20.7**	**6.9**	
⊠	**TP-2**	**9.5**	**0.15**	**0.109**	**0.085**		**0.0**	**86.2**	**13.8**	

Figure B-1

CONSOLIDATED ENGINEERING
LABORATORIES

Hobble Creek Canyon Water Tank

Bartholomew Canyon Road
Springville, Utah

SUMMARY OF LABORATORY RESULTS

PROJECT NUMBER 20-06720 **CLIENT** Jones & DeMille

Borehole	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Size (mm)	%<#200 Sieve	Classification (USCS)	Water Content (%)	Dry Density (pcf)	Satur-ation (%)	Void Ratio
B-1	5						GM	5.9			
TP-1	1				37.5	7	GW-GC				
TP-1							GM				
TP-1	10.75							11.7			
TP-2	8						CL	26.8			
TP-2	9.5				0.15	14	SC				