Geotechnical Evaluation

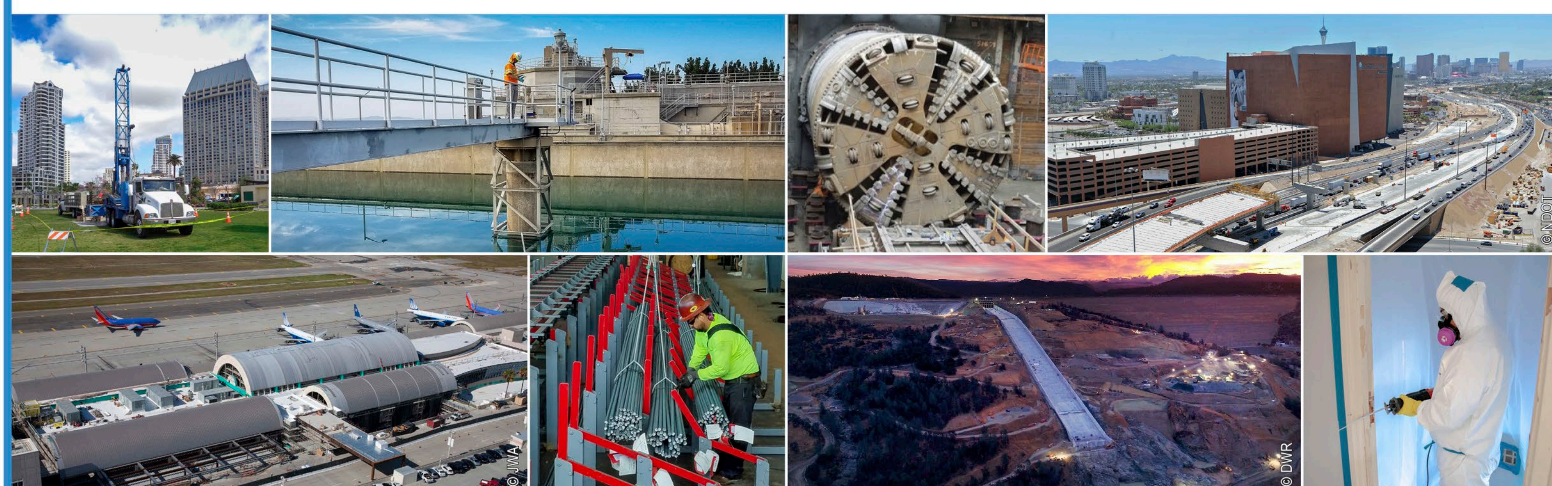
Wolf Creek Pipeline
4820 East Willowbrook Lane

Eden, Utah

Gardner Engineering

1580 West 2100 South | West Haven, Utah 84401

September 1, 2023 | Project No. 800297001



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Geotechnical Evaluation

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4820 East Willowbrook Lane
Eden, Utah

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1580 West 2100 South | West Haven, Utah 84401

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1 INTRODUCTION

In accordance with your request, Ninyo & Moore has performed a geotechnical evaluation for two proposed pump stations associated with the Wolf Creek Pipeline project to be constructed at 4820 East Willowbrook Lane in Eden, Utah. The approximate locations of the sites are 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 sites, 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, 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.
- Drilling, logging, and sampling of two exploratory borings to depths up to approximately 16.5 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), consolidation characteristics, and chemical (corrosion) considerations.
- 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 two pump stations located approximately 1.3 miles apart. The approximately 230 square-foot pump stations are anticipated to be constructed of CMU blocks founded on conventional spread foundations. The pumphouses are anticipated to bear approximately 3 feet below grade. The southern pumphouse will also include an adjacent 120 square-foot, concrete water storage vault that will extend approximately 10 feet below grade. Additional site improvements may include concrete flatwork, low-height retaining walls, and asphalt concrete paved parking and access areas. The project sites are shown on Figures 2A and 2B.

4 GENERAL SITE CONDITIONS

The northern site was located in the southern portion of a pasture to the west of the intersection of Seven Bridges Road and Howe Drive in Eden, Utah. Vegetation at the site consisted primarily of native grass with a few bushes and trees. Two drainage washes, generally draining from north to south, were observed to the east and west of the site, respectively. The site is generally bounded by farmland, undeveloped land, and single-family residential homes to the north, east, south, and west. The topography at the site slopes gently down to the south with a total relief of approximately 3 feet. Indications of underground utilities were not observed, but may be present at or near the site.

The southern site was located at the existing Wolf Creek Water and Sewer facility located along Willowbrook Circle in Eden, Utah. The facility contained several retaining ponds, including a dry pond that was approximately 15 feet deep in the southwest corner of the property. The southeast corner of this pond extends into the project site. Vegetation at the site consisted primarily of native grass, bushes, and a few sparse trees. A large earthen embankment associated with the adjacent pond approximately 10 to 15 feet high extends through the southern site. Adjacent properties include Valley Storage to the east, and single-family residential properties, farmland, and undeveloped land to the south, west, and north. A large overhead electrical transmission line was observed just north of the existing Wolf Creek Water and Sewer facility. Indications of underground sewer lines were observed near the site. 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 sites are underlain primarily by Holocene and Pleistocene age alluvial-fan deposits (native soil) consisting primarily of sand, silt and gravel that is poorly bedded and poorly sorted. Quaternary-age lacustrine soil deposits (native soil) consisting primarily of silt and sand deposits are also mapped in the vicinity of the sites.

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 sites are located in the Wasatch Back region along the east side of the Ogden Valley. The Wasatch Back 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 Wasatch Back region extends in a north-south direction and generally drains toward the west through rivers and washes. The referenced geologic map titled *Geologic Map of the Huntsville Quadrangle, Weber and Cache Counties, Utah* (Sorensen, M.L., 1979) 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 areas. This evaluation included visual observation of the sites 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. The fault seismic activity levels were obtained/interpreted primarily from the referenced United States Geological Survey (USGS, 2023) data.

Based on our review of referenced data, no faults traverse the project sites. Surficial disturbance associated with active faulting was not observed at the sites 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 sites is the Weber segment of the Wasatch fault zone. Table 1 lists the principal, known active faults that may affect the project sites 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, 2023).

Table 1 – Principal Active Faults in Vicinity of Project Site

Fault Name	Approximate Distance From Project Sites to Fault (miles)	Maximum Moment Magnitude (M_{max})
Wasatch Fault Zone, Weber Segment	5.1	7.2
Wasatch Fault Zone, Brigham City Segment	7.1	7.0
West Cache Fault, Wellsville Segment	16.8	6.6
Morgan Fault Zone, Central Segment	17.7	6.5

Review of the referenced geologic data does not indicate the presence of ground fissures at the project sites and no ground fissures were observed during our field activities. Additionally, our review indicates that the sites are 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 sites. Based on the results of our field exploration, American Society of Civil Engineers (ASCE) Standard 7-16 (ASCE, 2016), and a review of available geologic information, Seismic Site Class D-Default is appropriate for the project sites. The parameters presented in the following tables are characteristic of the sites for design purposes.

Table 2 – Seismic Design Criteria – Northern Site

Site Coefficients and Spectral Response Acceleration Parameters	Values
Site Class	D-Default
Site Coefficient at 0.2-second Period, F_a	1.2
Site Coefficient at 1.0-second Period, F_v	1.951
Mapped Spectral Response Acceleration at 0.2-second Period, S_S	0.979g
Mapped Spectral Response Acceleration at 1.0-second Period, S_1	0.349g
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}	0.681g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	0.782g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.454g
Site Amplification Factor, FPGA	1.2
Peak Ground Acceleration, PGA	0.434g
Modified Peak Ground Acceleration, PGA_M	0.521g

Table 3 – Seismic Design Criteria – Southern Site	
Site Coefficients and Spectral Response Acceleration Parameters	**Values**
Site Class	D-Default
Site Coefficient at 0.2-second Period, F_a	1.2
Site Coefficient at 1.0-second Period, F_v	1.959
Mapped Spectral Response Acceleration at 0.2-second Period, S_S	0.958g
Mapped Spectral Response Acceleration at 1.0-second Period, S_1	0.341g
Spectral Response Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	1.15g
Spectral Response Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.668g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	0.766g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.445g
Site Amplification Factor, F_{PGA}	1.2
Peak Ground Acceleration, PGA	0.425g
Modified Peak Ground Acceleration, PGA_M	0.51g

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 sites are mapped in a zone with a very 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 July 27, 2023. 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. The borings were drilled to depths up to approximately 16.5 feet. The purpose of the borings was to evaluate subsurface conditions at the project sites 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, 2023) 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 Figures 2A and 2B.

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), consolidation potential, and chemical (corrosion) considerations. 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 Boring B-2 from the ground surface to a depth of 3.5 feet. The encountered fill soils consisted of very stiff, lean clay; and very dense, poorly graded gravel with clay and sand. 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 or from the ground surface to the total depth of our borings. The encountered native soil consisted primarily of very stiff to hard, lean clay with varying amounts of sand and gravel, and very dense, poorly graded gravel with varying amounts of clay and sand. The native soils encountered were generally moist. Auger refusal was encountered in Boring B-1 at a depth of 15 feet. This refusal appears to have occurred on cobbles and/or boulders. 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 4 – Summary of Laboratory Test Results

Test Type	Test Results	Remarks
In-Place Moisture Content	4.8 to 13.0	--
In-Place Dry Density	89.5 to 115.8 pcf	--
Atterberg Limits Liquid Limit Plastic Limit Plasticity Index	 37 and 38 10 and 14 23 and 28	 Low plasticity.
Electrical Resistivity	13.9 Ohm-m	Severe corrosion potential to normal grade steel.
Water-Soluble Sulfate	117 mg/kg (ppm)	Sulfate Exposure Class S0 – Low corrosion potential to concrete.
Total Dissolved Solids (Solubility)	13,500 mg/kg (ppm)	High 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 water elevations in the adjacent ponds, 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.

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 a depth of approximately 3.5 feet in Boring B-2. Deeper areas of fill 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 8.1.2.
- **Over-sized Material:** Cobbles were occasionally encountered within our borings. Accordingly, difficult excavation techniques, including rock chipping, should be anticipated. Additionally, any on site soils to be reused as structural fill will likely need to be screened to remove over-sized materials.
- **Topsoil:** Highly organic soil (topsoil) was encountered in our borings to depths up to approximately 14 inches. The soils should be removed and properly disposed of prior to site grading.
- **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.
- **Subgrade Stabilization:** Groundwater was not encountered in our exploratory borings at the time of drilling. However, based on our experience in the project vicinity, areas of relatively soft/loose, moist to wet conditions should be anticipated. Unstable and pumping subgrade conditions should be expected during earthwork operations, particularly after rain and snowfall events.
- **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 sites are mapped in a zone with a very 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 underground utilities were observed near the sites 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 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.

8.1.1 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 8.1.2.

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 sites, 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.

Areas of firm/loose and relatively moist to wet conditions should be anticipated, particularly during the winter and spring months. Unstable and pumping subgrade conditions should be expected during earthwork operations, particularly after rain and snowfall events. Subgrade stabilization will be needed where unstable and pumping subgrade conditions are encountered. Stabilization methods may include the use of geogrids, geofabric, and/or angular rock up to approximately 6 inches in diameter. The geotechnical consultant should evaluate proposed subgrade stabilization methods prior to their implementation.

Cobbles and boulders were observed on the ground surface at the project sites and encountered in our borings. Accordingly, difficult excavation techniques, including rock chipping, should be anticipated. Additionally, any on-site soils to be reused as structural fill will likely need to be screened to remove over-sized materials, including cobbles and boulders.

8.1.2 Structural Fill and Backfill

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

8.1.2.1 Soil Suitability

Based on the findings of our subsurface evaluation and laboratory test results, the soils encountered during our exploration below the upper organic-rich soils 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 6 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).

8.1.2.2 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.

8.1.3 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.

8.1.4 Excavations and Dewatering

Groundwater was not encountered at the time of our subsurface exploration. However, based on our experience in the area, excavations may encounter soft and/or wet conditions; therefore, dewatering techniques may be needed. The design, construction, and implementation of construction dewatering are the responsibility of the contractor, and should be performed by a qualified expert. Upon request Ninyo & Moore can perform in-place hydro-geologic testing and/or full-scale pump testing at this site to further evaluate these parameters. Dewatering should be performed with care so as not to cause harmful settlement of nearby foundations, utilities, pavements, or other improvements. Discharge of water from the excavations to storm water collection systems will require a construction dewatering permit. Groundwater characterization will be needed as part of the permit application.

Where encountered, drying or over-excavation of any wet or saturated soils is recommended. If the subgrade becomes disturbed, it should be compacted or removed and replaced before placing additional backfill material. Structures and improvements should be properly waterproofed and designed to resist buoyancy forces due to potentially shallow groundwater. Groundwater levels will fluctuate due to seasonal variations associated with precipitation, irrigation, groundwater withdrawal or injection, and other factors.

8.1.5 Temporary Excavations and Shoring

Temporary slope configurations should be consistent with the regulations provided in the referenced Occupational Safety and Health Administration (OSHA) regulations (OSHA, 2022).

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.

8.2 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.

Development of site grading plans should consider subsurface transfer of water in utility trench backfill and the pipe bedding materials. Sandy pipe bedding materials can function as efficient conduits that convey natural and applied waters in the subsurface. Cut-off walls in utility trenches or other water-stopping measures should be implemented to reduce the rates and volumes of water transmitted along utility alignments and toward buildings, pavements, and other structures where excessive wetting of the underlying soils will be damaging. Incorporation of water cut-offs and/or outlet mechanisms for saturated bedding materials into development plans could be beneficial to the project. These measures also will reduce the risk of settlement due to loss of fine-grained backfill soils into the bedding material.

8.3 Structure Foundations

We recommend the proposed structure be supported by conventional spread foundations utilizing an allowable bearing capacity of 2,200 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 adequately placed and compacted structural fill (reworked native or import soils). Continuous and isolated footings should have an embedment depth of 36 inches or more below adjacent finished grade (for frost protection) and a width of 12 inches or more. 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 2,800 psf.

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. Lateral resistance for footings is presented in Section 8.5. Seismic parameters for design of structures at the site are provided in Table 2 in Section 5.3. Foundations should be designed and constructed in accordance with the recommendations of a qualified structural engineer. 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 Settlement

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

8.5 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 315 psf per foot of depth up to a value of 2,500 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 42 pcf and 61 pcf, respectively. In addition, for seismic active lateral earth pressures, an additional equivalent fluid pressure of 13 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 0.33 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.

8.6 Concrete Slab-On-Grade Floors

Concrete slab-on-grade floors should be designed by the project's structural engineer based on anticipated loading conditions. Ninyo & Moore recommends that conventional concrete slab-on-grade floors for this project be founded on 6 inches of Untreated Base Course overlying medium dense to very dense native granular soils, stiff to hard native fine-grained soils, or adequately placed and compacted structural fill (reworked native or import soils). Aggregate base underlying concrete slab-on-grade floors should be compacted to 95 percent or more of the laboratory maximum dry density (ASTM D1557).

Floor slabs should be 4 inches or more in thickness and reinforced with No. 3 steel reinforcing bars placed at 18 inches on-center both ways. Reinforcement of the slab should be placed at mid-height. We recommend that “chairs” be utilized to aid in the placement of the reinforcement. Increased slab thickness and reinforcement may be recommended by the structural engineer. As a means to reduce shrinkage cracks, we recommend that conventional slab-on-grade floors be provided with control joints in accordance with the recommendations of a qualified structural engineer. Recommendations regarding concrete utilized in construction of floor slabs are provided in Section 8.10. As an alternative to slab reinforcement with steel reinforcing bars, post-tensioned slabs designed by a qualified structural engineer may be considered.

Ninyo & Moore recommends that a moisture barrier be provided by a membrane placed beneath concrete slab-on-grade floors, particularly in areas where moisture-sensitive flooring is to be used. The membrane should overlie the previously described compacted base material. The membrane should consist of visqueen 10 mils in thickness. If flooring systems, including the adhesives, are particularly sensitive to moisture vapor, a more robust membrane/moisture barrier should be considered, such as Stego Wrap, which is 15 mils in thickness with a permeance less than 0.02 grains per square foot per hour (perms) as evaluated by ASTM E96. This membrane should overlie compacted base material and be placed directly under the floor slab. A pre-pour planning meeting should also be considered to resolve water vapor emission and concrete curing considerations and to establish means for reducing slab curl.

Slabs associated with vaults, or any other subgrade structures that extend several feet below the ground should be constructed as waterproof structures that can also resist the buoyancy forces (depending on their proximity to the groundwater table).

8.7 Exterior Concrete Flatwork

Ground-supported concrete flatwork may be subject to soil-related movements resulting from frost heave/settlement. Thus, where these types of elements abut rigid building foundations or isolated/suspended structures, differential movements should be anticipated. We recommend that flexible joints be provided in this situation to allow for differential movement.

Exterior concrete flatwork, such as walkways, should be founded on 6 inches of Untreated Base Course overlying medium dense to very dense native granular soils, stiff to hard native fine-grained soils, or a zone of compacted structural fill that meets the recommendations described in Section 8.1.2 of this report. Untreated Base Course should be compacted to 95 percent or more relative compaction, as evaluated by ASTM D1557.

To reduce the potential for shrinkage cracks, the flatwork should be constructed with control joints spaced approximately 5 feet apart for walkways and approximately 10 feet on-center each way for larger slabs. Crack control joint spacing should be in accordance with recommendations of a qualified structural engineer. Reduced joint spacing may be recommended by the structural engineer.

Formation of shrinkage cracks in concrete slabs, and other cracks due to minor soil movement, may be further reduced by utilizing steel reinforcement, such as welded wire mesh. However, due to the inherent difficulty in positioning welded wire mesh in the middle of concrete flatwork, other crack control methods should be considered, such as placement in the concrete of No. 3 steel reinforcing bars at approximately 24 inches on-center each way. Reinforcement of the flatwork should be placed at approximately mid-height in the concrete utilizing “chairs.”

Exterior concrete flatwork, curbs, and gutters should be constructed in accordance with the recommendations of the project’s civil or structural engineer and governing agency requirements. Recommendations regarding concrete utilized in construction of proposed improvements are provided in Section 8.10.

8.8 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.

8.9 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.

8.10 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.

8.10.1 Concrete

Chemical tests performed on selected samples of on-site soils indicated sulfate contents of 117 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.

8.10.2 Metal in Contact with On-Site Soils

Chemical tests performed on selected samples of on-site soils indicated 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.

8.11 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.

8.12 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.

8.13 Plan Review

The recommendations presented in this report are based on preliminary design information for the proposed project, as provided by Gardner Engineering 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.

8.14 Pre-Construction Meeting

We recommend that a pre-construction meeting be held. The owner or the owner's representative, 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

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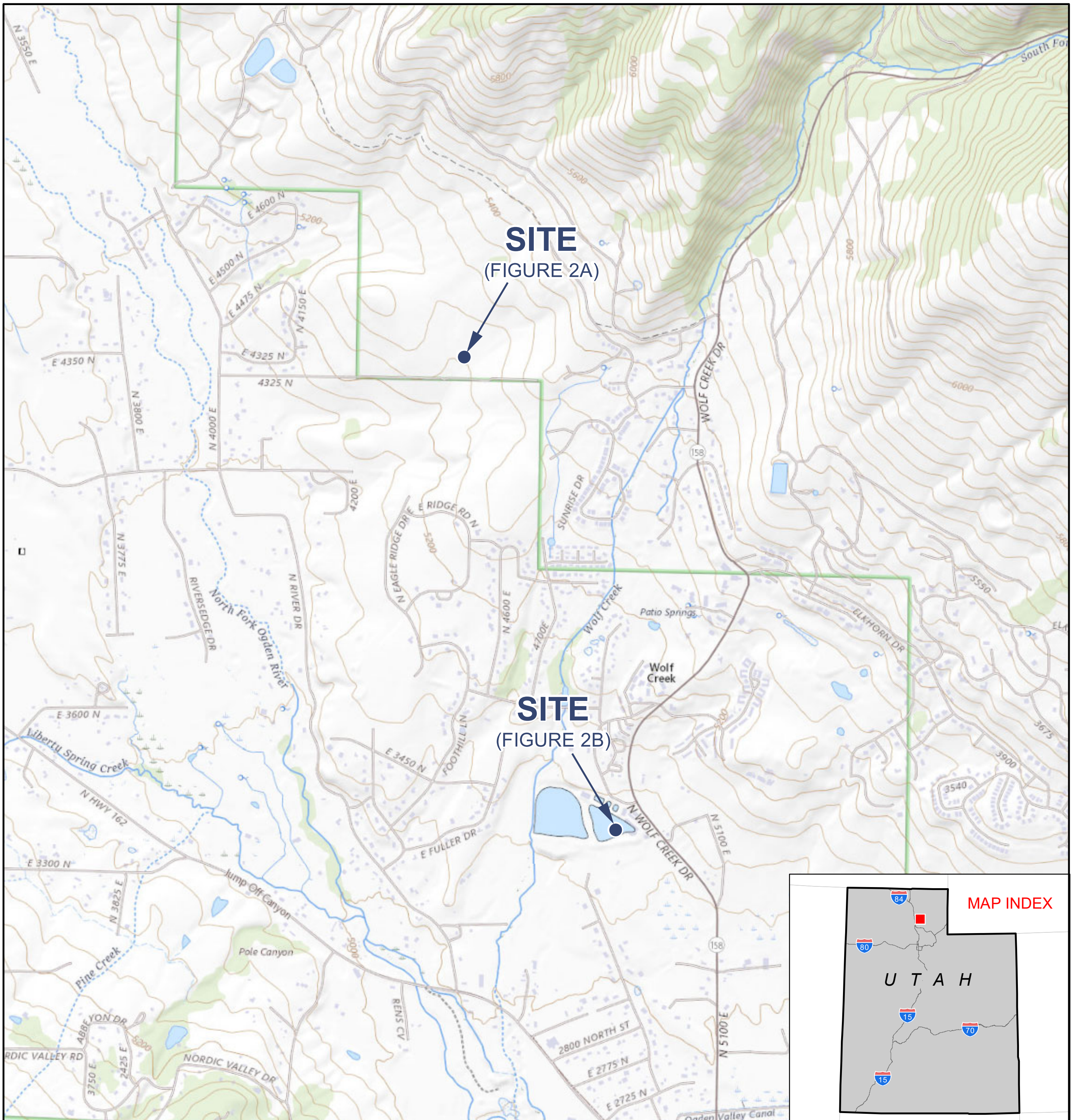
FIGURES



N

FEET

0 1,900 3,800

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: ESRI, USGS, 2023

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FIGURE 1

SITE LOCATIONS

WOLF CREEK PIPELINE
4820 EAST WILLOWBROOK LANE, EDEN, UTAH

800297001 | 9/23

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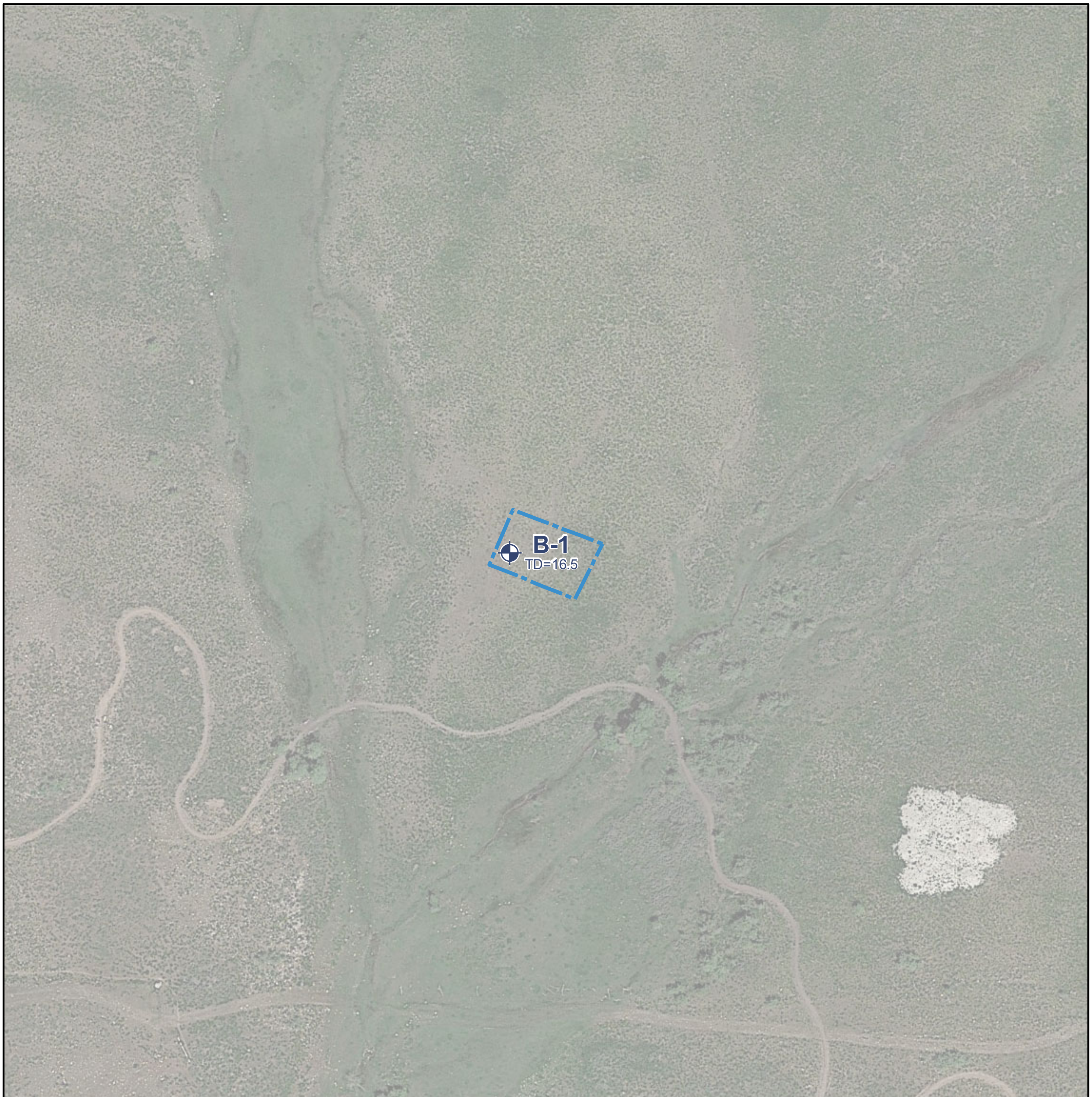

LEGEND

B-1 TD=16.5 BORING
TD=TOTAL DEPTH IN FEET

SITE BOUNDARY

N

FEET
0 100 200

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: GOOGLE EARTH, 2023

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FIGURE 2A

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BORING LOCATION

WOLF CREEK PIPELINE
4820 EAST WILLOWBROOK LANE, EDEN, UTAH

800297001 | 9/23

LEGEND

B-2 TD=15.0 BORING
TD=TOTAL DEPTH IN FEET

SITE BOUNDARY

N

FEET
0 100 200

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NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: GOOGLE EARTH, 2023

FIGURE 2B

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BORING LOCATION

WOLF CREEK PIPELINE
4820 EAST WILLOWBROOK LANE, EDEN, UTAH

800297001 | 9/23

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.

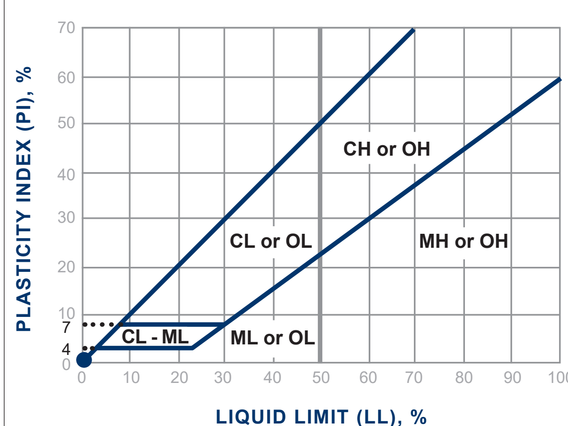
Soil Classification Chart Per ASTM D 2488

Primary Divisions			Secondary 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

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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.
15								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
20								The total depth line is a solid line that is drawn at the bottom of the boring.

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BORING LOG

DATE DRILLED	7/27/23	BORING NO.	B-1
GROUND ELEVATION	5,249' ± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING	Mobile B-80 Hollow-Stem Auger Drill Rig		
DRIVE WEIGHT	140 lbs. (Spooling Cable)	DROP	30"
SAMPLED BY	JFS/JLK	LOGGED BY JFS/JLK	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	NATIVE SOIL: Gray, dry, very stiff, lean CLAY with sand; few gravel; few cobbles; organics in upper 14 inches. Moist; hard.
			14/6" 21/6" 44/6"	11.5	115.0			
			20/6" 31/6" 30/6"					
5								Few sand; gravel grades out.
								Yellow; sandy; trace gravel.
			13/6" 44/6" 50/6"	9.1	115.8			
			32/6" 19/6" 16/6"					
10								Light brown; very stiff; sand and gravel grade out.
15								Hard.
			10/6" 24/6" 30/6"	13.0	95.5			
								Total Depth = 16.5 feet. Groundwater not encountered during drilling. Backfilled on 7/27/23.
20								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.

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WOLF CREEK PIPLINE
4820 EAST WILLOWBROOK LANE, EDEN, UTAH

DATE DRILLED 7/27/23 BORING NO. B-2

GROUND ELEVATION 5,069' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING Mobile B-80 ODEX

DRIVE WEIGHT 140 lbs. (Spooling Cable) DROP 30"

SAMPLED BY JFS/JLK LOGGED BY JFS/JLK REVIEWED BY REG/EDE

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0					CL	FILL: Light brown, dry, very stiff, lean CLAY with sand; few gravel; occassional cobbles; organics in upper 8 inches.
	Driven	50/3"			GP-GC	Light brown and gray, moist, very dense, poorly graded GRAVEL with clay and sand. Possible cobble or boulder.
	Driven	35/6" 50/5"	4.8	89.5	GC	NATIVE SOIL: Brown, moist, very dense, clayey GRAVEL with sand.
5						
	Driven	20/6" 44/6" 45/6"			GP-GC	Brown, moist, very dense, poorly graded GRAVEL with clay and sand.
	Driven	40/6" 40/6" 50/5"	5.7	101.9	GC	Brown, moist, very dense, clayey GRAVEL with sand.
10						Possible cobbles or boulder.
15	Driven	50/0"				Auger Refusal. Total Depth = 15.0 feet. Groundwater not encountered during drilling. Backfilled on 7/27/23.
20						

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.

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.

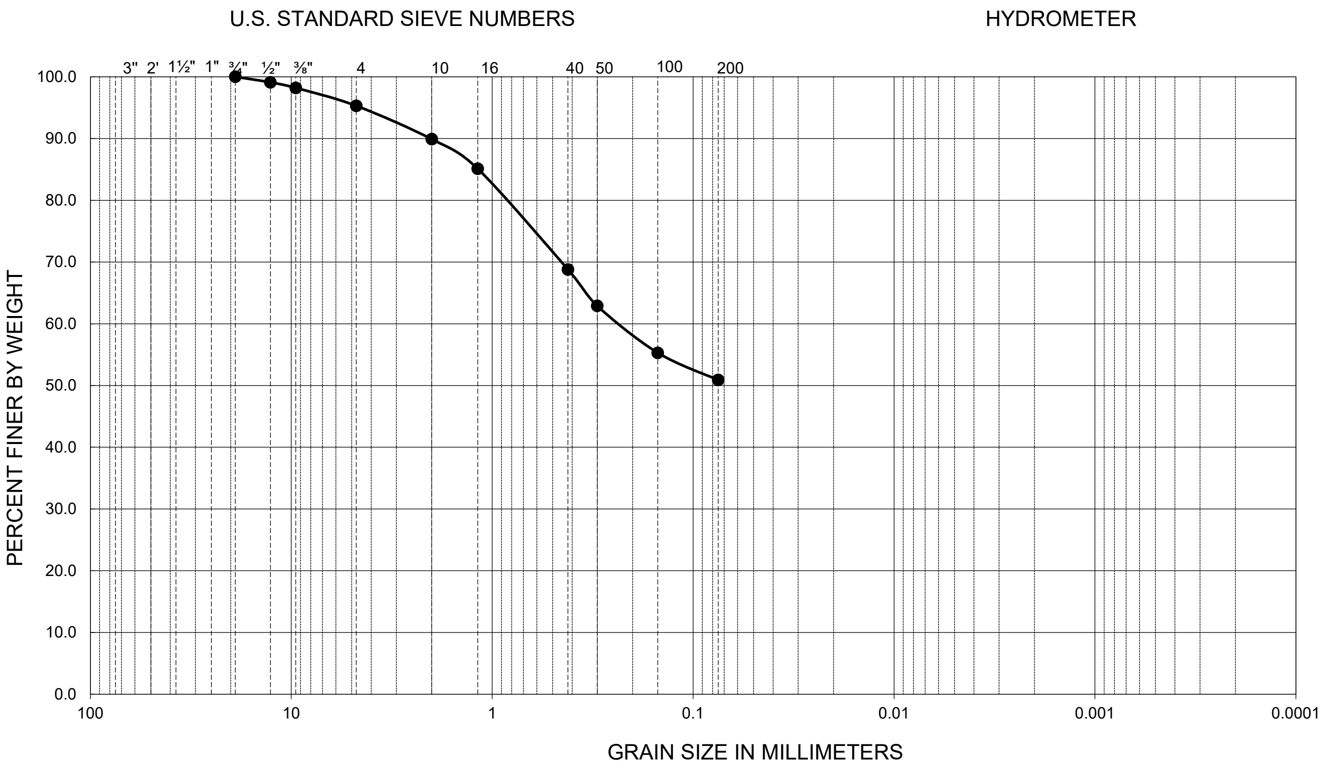
In-Place Moisture and Density
The moisture content and dry density of relatively undisturbed samples obtained from the exploratory excavations were evaluated in general accordance with ASTM D2937. The test results are presented on the logs of the exploratory excavations in Appendix A.

Gradation Analysis
Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D7928, C136, and C117. 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.

Consolidation
Consolidation tests were performed on selected relatively undisturbed soil samples in general accordance with ASTM D2435. The samples were inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The consolidation test results are summarized graphically on Figure B-4.

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	5.0-8.5	38	10	28	--	--	0.23	--	--	50.9	CL

Material Percent by Weight			Soil Type
Gravel	Sand	Fines	Sandy lean CLAY
4.7	44.4	50.9	

Moisture Content
9.1%

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D7928, C136, and C117

FIGURE B-1

GRADATION TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

WOLF CREEK PIPELINE
4820 EAST WILLOWBROOK LANE, EDEN, UTAH

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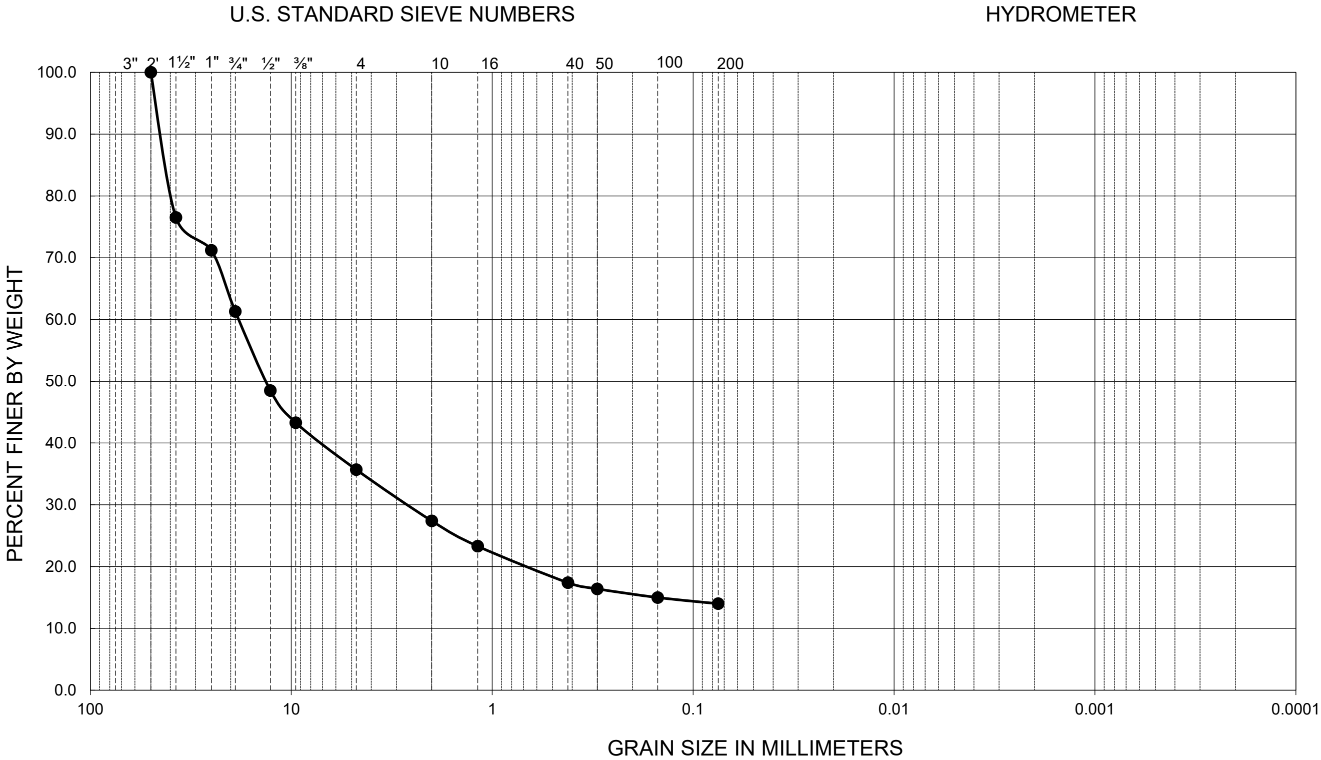
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



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	8.5-9.9	37	14	23	--	2.62	18.24	--	--	14.0	GC

Material Percent by Weight			Soil Type
Gravel	Sand	Fines	Clayey GRAVEL with sand
64.3	21.7	14.0	

Moisture Content
5.7%

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D7928, C136, and C117

FIGURE B-2

GRADATION TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

WOLF CREEK PIPELINE
4820 EAST WILLOWBROOK LANE, EDEN, UTAH

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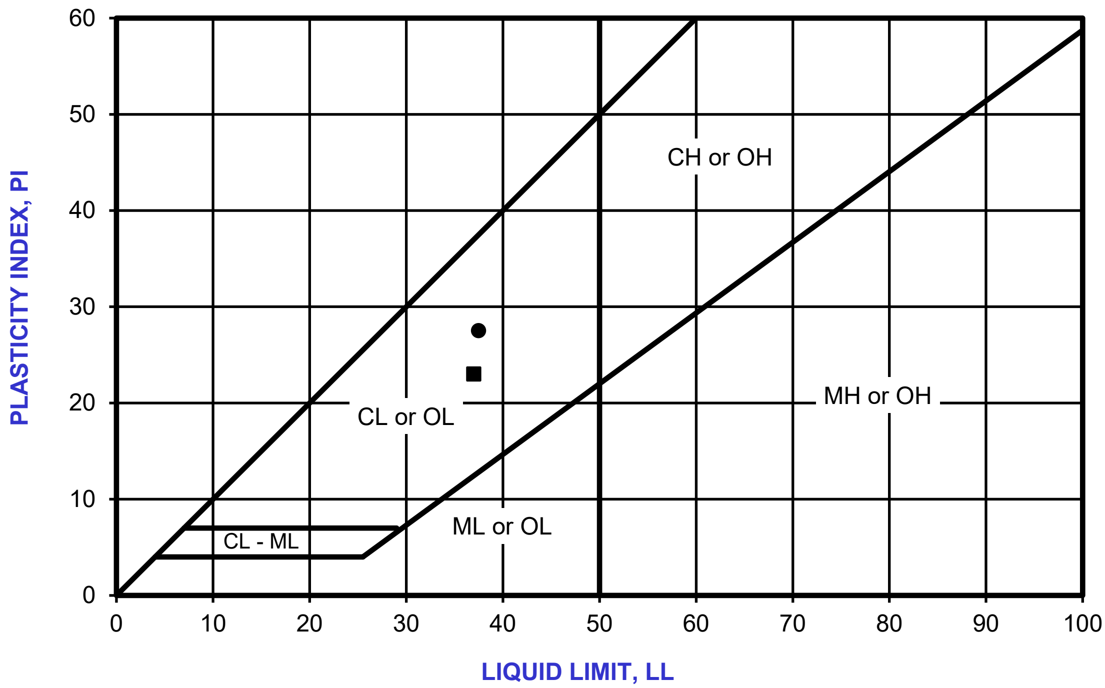
SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-1	5.0 - 8.5	38	10	28	CL	CL
■	B-2	8.5 - 9.9	37	14	23	CL	GC



PERFORMED IN GENERAL ACCORDANCE WITH D4318

FIGURE B-3

ATTERBERG LIMITS TEST RESULTS

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WOLF CREEK PIPELINE
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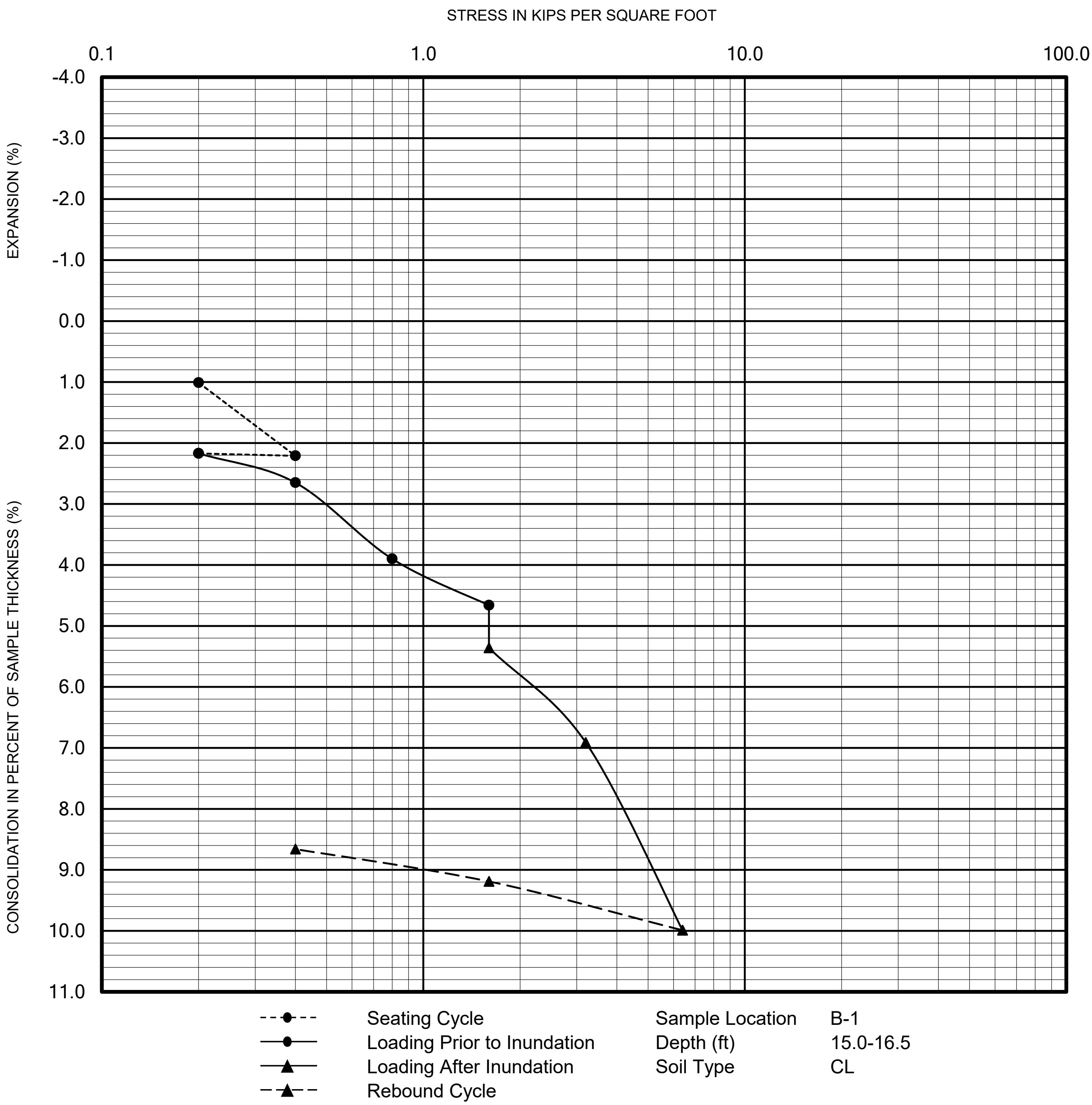

Sample Location	B-1
Depth (ft)	15.0-16.5
Soil Type	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D2435

FIGURE B-4

CONSOLIDATION TEST RESULTS

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WOLF CREEK PIPELINE
4820 EAST WILLOWBROOK LANE, EDEN, UTAH

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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
Edgar Salinas
871 Robinson Drive
North Salt Lake, UT 84054

PO#: **800297001**
Receipt: **8/23/23 17:29 @ 25.9** °C
Date Reported: 8/31/2023
Project Name: **800297001**

Sample ID: **B-1@1.0-2.5**

Matrix: **Solid**

Lab ID: **23H2095-01**

Date Sampled: **7/27/23 14:20**

Sampled By: **Edgar Salinas**

	Result	Units	Minimum Reporting Limit	Method	Preparation Date/Time	Analysis Date/Time	Flag(s)
Inorganic							
Resistivity	13.9	ohm m	1.0	SSSA 10-3.3	8/25/23	8/25/23	
Sulfate, Soluble (IC)	117	mg/kg dry	11	EPA 300.0	8/25/23	8/25/23	
Total Dissolved Solids, Soluble	13500	mg/kg dry	546	SM 2540 C	8/24/23	8/24/23	
Total Solids	91.6	%	0.1	CTF8000	8/24/23	8/24/23	SPH

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

Certificate of Analysis

Ninyo and Moore	PO#:	**800297001**
Edgar Salinas	Receipt:	**8/23/23 17:29 @ 25.9** °C
871 Robinson Drive	Date Reported:	8/31/2023
North Salt Lake, UT 84054	Project Name:	**800297001**

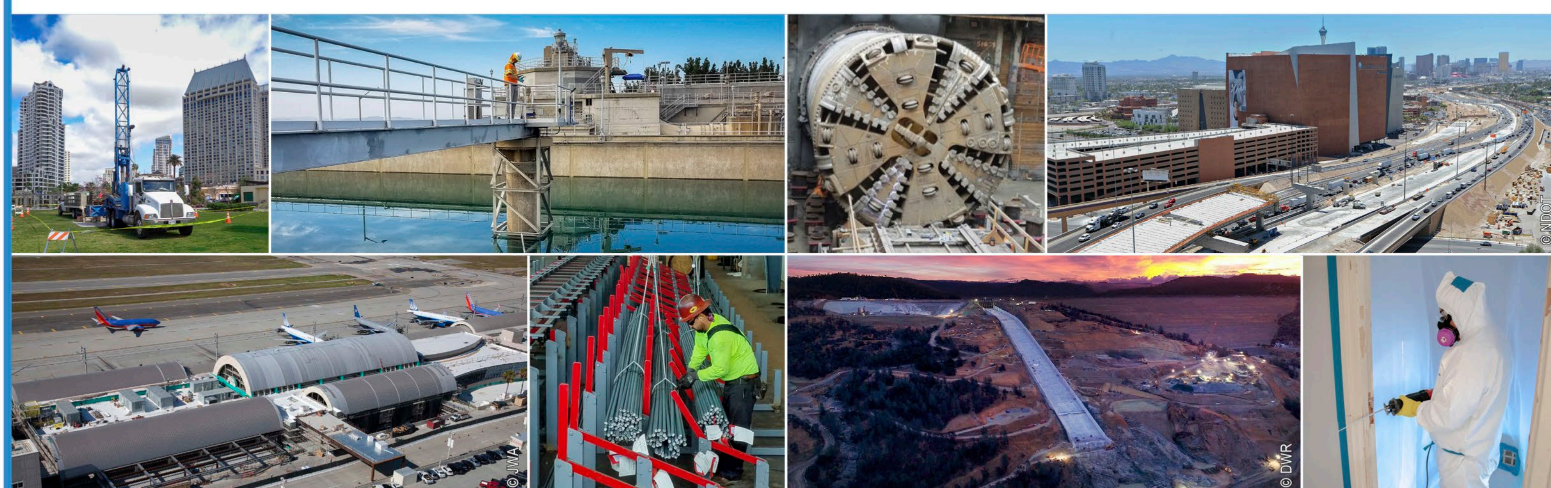
Report Footnotes

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.

Flag Descriptions

SPH = Sample submitted past method specified holding time.



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

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