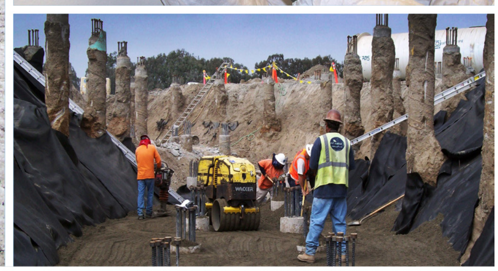
Geotechnical Evaluation

Millcreek Commons
Near 1300 East 3300 South
Millcreek, Utah

EPG Design

208 East 800 South | Salt Lake City, Utah 84111

June 18, 2020 | Project No. 800055001



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

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Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

Geotechnical Evaluation

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Near 1300 East 3300 South
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Mr. David Harris

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1 INTRODUCTION

In accordance with your request, Ninyo & Moore has performed a geotechnical evaluation for the proposed Millcreek Commons project to be constructed near 1300 East and 3300 South in Millcreek, Utah. The approximate location of the site is indicated on Figure 1. The purposes of our geotechnical study were to evaluate the 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 subject site, and geotechnical recommendations for design and construction of this project.

2 SCOPE OF SERVICES

The scope of our services included the following:

- Review of pertinent background data listed in the References section of this report. The data reviewed included site information proved by the client, in-house geotechnical data, aerial photographs and published regional and local geologic maps and soils data.
- Coordination and mobilization for subsurface exploration. Mark-out of existing utilities was conducted through Blue Stakes of Utah personnel.
- Performance of a geologic site reconnaissance to evaluate the possible presence of faults, ground fissures, and other potential geologic hazards at the site.
- Drilling, logging, and sampling of six exploratory test borings to depths up to approximately 16.5 feet. The purposes of the soil borings were to evaluate the subsurface soil conditions and to obtain soil samples for laboratory testing.
- Performance of laboratory tests to evaluate physical and engineering properties of the subsurface soils, including in-place moisture content and density, gradation, plasticity, pH, reduction-oxidation potential, resistivity, water soluble sodium content, water soluble sulfate content, soluble sulfide content, total available water soluble sodium sulfate content, and water soluble chloride content.
- Compilation and analysis of accumulated data.
- Preparation of this report presenting our findings, conclusions, and recommendations regarding the subject project.

3 PROJECT DESCRIPTION

We understand that the project will consist of the development of a new city park area on an approximately 2-acre site. The project will include a new green-space, art space, ice support structure, paved walking paths, retail promenade, and parking areas. We anticipate that the structures will consist of single-story buildings supported on conventional spread footings. Site grading is anticipated to be minimal, with cuts and fills on the order of 2 to 5 feet.

The project will also include asphalt concrete paving and exterior concrete flatwork. The approximate location of the project site and improvements are shown on Figure 1 and Figure 2.

4 GENERAL SITE CONDITIONS

At the time of our field activities, the project site was developed with structures, including a restaurant, an auto shop, a utility (gas) building, an office building, and asphalt parking areas. The site is bounded by commercial and residential development to the east and west, 3300 South to the south, and 3205 South to the north. The topography at the subject site slopes down from east to west with approximately 10 feet of elevation relief. Overhead power lines were noted crossing the site from east to west near north and south borders of the site, as well as through the center of the site. Other utilities such as water, communication, and gas were observed at or near the site.

A review of historical aerial imagery (as early as 1935) indicates that the site has been previously developed primarily as single-family residential properties. A large borrow pit area associated with a historical brick construction facility (The Brickyard) was located to the northwest of the project site.

5 GEOLOGY

Based on our field observations, subsurface exploration, and review of referenced geologic and soils data, the subject site is underlain primarily by fill, which is in turn underlain by Quaternary-age alluvium (native soil). Ninyo & Moore's findings regarding the geologic setting, potential geologic hazards and problematic soils, ground motions, and liquefaction potential at the subject site are provided in the following sections.

5.1 Geologic Setting

The project site is located in the northeastern portion of the Salt Lake Valley, which is located in the Wasatch Front Region. The Wasatch Front Region lies along the east edge of the Great Basin, within the Basin and Range physiographic province. The Salt Lake Valley is a naturally formed structural basin as a result of block faulting, a fundamental characteristic of the Basin and Range physiographic province.

The Salt Lake Valley extends in a north-south direction and generally drains toward the north through rivers and washes into the Great Salt Lake. Bordering the alluvium-filled valley are relatively steep mountain ranges, including the Wasatch Mountains to the east, and the Oquirrh Mountains to the west.

The referenced geologic map titled Interim Geologic Map of the Salt Lake City South Quadrangle, Salt Lake County, Utah (Utah Geologic Survey, 2018) indicates that the project area is underlain primarily by Quaternary-age lacustrine deposits. The deposits are composed primarily of clay, silt, sand, and isolated areas of gravel.

5.2 Potential Geologic Hazards and Problematic Soils

Ninyo & Moore's geotechnical study included an evaluation of the possible presence of geologic hazards, such as faults and ground fissures, in the site area. This evaluation included visual observation of the site for indications of adverse geologic features and review of published geologic and soils maps and literature, and other data listed in the References section of this report. Referenced geologic data were also reviewed to evaluate seismic activity levels, and associated potential earthquake hazards, for faults in the site vicinity. It should be noted that the fault seismic activity levels provided in this section were obtained/interpreted primarily from United States Geological Survey (USGS, 2020) data.

Active seismic faults in Utah generally extend in a north-south direction, usually referred to as the Intermountain Seismic Belt. The Wasatch fault is one of the major faults of this seismic belt and is roughly located on the eastern border of the Basin and Range province, at the transition to the Colorado Plateau.

Based on our field observations and review of referenced USGS data, an active fault (Salt Lake City section of the Wasatch Fault) traverses the project site. Review of referenced geologic data also indicates that additional active faults (i.e., a fault that has experienced ground surface rupture within the past 10,000 years) are located in the vicinity of the project site. The distances from the project site to faults in the project vicinity are presented in the following table.

Table 1 – Faults in Vicinity of Project Site

Fault Name	Seismic Activity Level *	Approximate Distance From Project Site to Fault (miles)
Wasatch Fault Zone, Salt Lake City Section	Active	On-site
West Valley Fault Zone, Taylorsville Fault	Active	5
West Valley Fault Zone, Granger Fault	Active	6
Wasatch Fault Zone, Weber Section	Active	11

Note: *From United States Geological Survey (USGS, 2020)

A review of the Utah Geologic Survey (UGS) Utah Geologic Hazards Map indicates that the site is located in a *Surface Fault Rupture Hazard Special Study Zone*. A site-specific fault study has been completed by others regarding the specific fault hazards at the project site (CMT, 2020).

5.3 Ground Motions

Using the referenced United States Geological Survey database (USGS, 2020), estimated maximum considered earthquake spectral response accelerations for short (0.2 second) and long (1.0 second) periods were obtained for the subject site, which is located at approximately 40.7010 degrees north latitude and -111.8524 degrees west longitude. Based on soils encountered in our exploratory borings and review of available geologic information, Seismic Site Class D is appropriate for the subject site. The parameters presented in the following table are characteristic of the site for design purposes.

Table 2 – Seismic Design Criteria

Site Coefficients and Spectral Response Acceleration Parameters	Values
Site Class	**D**
Site Coefficient, F_a	1.2
Site Coefficient, F_v	Null*
Mapped Spectral Response Acceleration at 0.2-second Period, S_s	1.405g
Mapped Spectral Response Acceleration at 1.0-second Period, S_1	0.519g
Spectral Response Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	1.687g
Spectral Response Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	Null*
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.124g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	Null*

Note: *See ASCE 7-16, Section 11.4.8

A Site Modified Peak Ground Acceleration (PGA_M) of 0.765 g was calculated for the site using a Site Amplification Factor (F_{PGA}) of 1.2 and Peak Ground Acceleration (PGA) of 0.638 g, per ASCE 7-16 ($PGA_M = PGA \times F_{PGA}$).

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.

Review of the referenced geologic data indicates that the project site is mapped in a zone with a low liquefaction potential (UGS, 2008). An in-depth evaluation of the potential for liquefaction at the site was outside the scope of this geotechnical evaluation.

6 FIELD EXPLORATION AND SUBSURFACE CONDITIONS

Ninyo & Moore's subsurface exploration at the project site was performed on May 12, 2020. This exploration consisted of drilling, logging, and sampling of six exploratory test borings (B-1 through B-6). The borings were excavated to depths of up to approximately 16.5 feet with a Mobile B-80 Drill Rig utilizing 8-inch diameter hollow stem augers. The purpose of the borings was to evaluate subsurface conditions at the proposed project site and 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, 2020) data. Accordingly, the boring elevations that are recorded on the boring logs in Appendix A should be considered approximate. The approximate locations of the borings are shown on Figure 2.

Laboratory tests were performed on representative soil samples collected from the test pits to evaluate the in-place density and moisture content, gradation, plasticity, pH, reduction-oxidation potential, resistivity, water soluble sodium content, water soluble sulfate content, soluble sulfide content, total available water soluble sodium sulfate content, and water soluble chloride content. The results of the in-place density and moisture content 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 (fill and native soil) encountered in the exploratory borings are provided in the following sections.

6.1.1 Fill

Fill material was encountered in our exploratory borings to depths up to approximately 16.5 feet. The encountered fill was comprised primarily of moderately dense to very dense, silty gravel with sand; medium dense, silty and clayey sand with varying amounts of gravel and cobbles; and dense to very dense, poorly-graded gravel with sand and cobbles

6.1.2 Native Soil

Native soil was encountered beneath the noted fill and extended to the total depths of our exploratory borings. The encountered native soil consisted primarily of soft to stiff lean clay;

very dense, poorly-graded gravel with cobbles; and medium dense silty sand with gravel and cobbles.

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

Table 3 – Summary of Laboratory Test Results

Test Type	Test Results	Remarks
Atterberg Limits Liquid Limit Plastic Limit Plasticity Index	 NP to 38 NP to 19 NP to 19	 Low to moderate plasticity
pH	8.13 and 8.33	--
Redox Potential	241 and 272 mV	--
Resistivity (Saturated)	950 and 1600 Ohm-cm	Severely corrosive to normal grade steel.
Water Soluble Sodium	0.0082 and 0.015 percent	--
Water Soluble Sulfate	0.014 and 0.015 percent	Low corrosivity potential to concrete
Soluble Sulfide	< 0.5 mg/Kg	--
Total Available Water Soluble Sodium Sulfate	0.021 and 0.023 percent	Low salt heave potential
Water Soluble Chloride	20 and 55 mg/Kg	--
Total Salts (Solubility)	0.061 and 0.066 percent	Negligible solubility potential

Notes:
NP – Non-Plastic

6.1.3 Void

A large void in the soil was encountered in Boring B-5. As the auger was withdrawn from the boring, soil around the auger collapsed revealing a void measuring approximately 6 feet wide by 6 feet long by 10 feet deep. Boring B-5 was advanced just outside of the radius of the void along the western edge of the void. The eastern edge of the void appears to extend nearly to the foundation of the existing automotive shop. It is possible that portions of the automotive shop foundation have been undermined by this void and proper precautions should be taken during demolition activities. The sidewalls of the void were observed to be comprised of a mixture of sand, gravel, cobbles, and boulders likely associated with old fill.

6.2 Groundwater

Groundwater was not encountered in our exploratory borings, which were advanced to depths up to approximately 16.5 feet. Based on our review of the Utah Division of Water Rights Well Log Database, the estimated depth to groundwater is approximately 47 feet in the vicinity of the project site (Well Log No. 1157015M00). Groundwater levels are influenced by seasonal factors, variations in ground surface topography, precipitation, irrigation practices, soil/rock types, groundwater pumping, and other factors and are subject to fluctuations. These fluctuations may be due to variations in ground surface topography, subsurface geologic conditions, rainfall, irrigation, de-watering/pumping operations from nearby sites, and other factors. 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:

- Based on review of the referenced geologic maps and literature, a section of the Wasatch fault traverses the project site. A site-specific fault study with expanded information regarding this fault has been completed by others (CMT, 2020). This study should be considered as part of the development process for this project.

- Fill materials were encountered within our exploratory borings to depths of approximately 16.5 feet. Deeper fills may also be present at the site. We recommend the existing undocumented fill be removed from proposed structure and improvement areas. This material can be processed and stockpiled for later use as structural fill or placed in other areas of the site with proper placement, compaction, and testing, provided the material meets the requirements provided herein (see Section 8.1.4).

- Review of historical aerial photographs and our field observations indicate that structures and other improvements have previously been located at the subject site prior to the current development. Additionally, historical aerial photographs indicate that the site is located near a possible borrow pit area associated with a former brick construction facility (The Brickyard). Care should be exercised during earthwork operations to adequately expose native subgrade, particularly in areas of previous demolition, to see that debris and otherwise unsuitable material, and undocumented/non-engineered backfill, have been adequately removed in areas of proposed structures and improvements.

- Recommendations for additional evaluation of this fault are provided in the following section.

- Groundwater was not encountered in our exploratory borings, which were drilled to depths of up to 16.5 feet. Groundwater is not anticipated to be a design concern for this project.

- As previously discussed, the project site is mapped in a zone with a low liquefaction potential (UGS, 2008). Due to the presence of cohesive soils at the project site and our review of nearby groundwater data, it is our opinion that there is a low potential for liquefaction of subsurface soils at the site. However, an in-depth evaluation of the potential for liquefaction to a 50-foot depth at the project site was outside the scope of this geotechnical evaluation

- Structure foundations and other project improvements should be supported on medium dense or stiff native soils, or on a zone of adequately placed and compacted structural fill.

- The findings of our study indicate that some of the fill and native soils encountered in our exploratory borings may 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 8.1.4.
- Resistivity test results indicate that some tested soils are potentially severely corrosive to buried metals. Corrosivity levels indicated by the laboratory test results should be considered during selection of the type of pipes that will be utilized for the project and corrosion reduction methods that may need to be implemented.
- The subject project is currently in the early conceptual stages of design. We recommend that information regarding ultimate design plans be provided to Ninyo & Moore when available. Additional field exploration and laboratory testing, as well as supplemental geotechnical recommendations, may be needed upon review of the ultimate project design plans.
- In accordance with the referenced International Building Code, the seismic parameters provided in Table 1 are characteristic of the site and may be used in design of the proposed structures.

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 demolition, existing fill, site grading, structural fill and backfill, import soil, and temporary excavations.

8.1.1 Demolition

We understand that the project will include demolition of existing improvements, including several structures and paved areas. Care should be exercised during demolition to see that debris and otherwise unsuitable material, as well as any undocumented/non-engineered fill and backfill, have been adequately removed and replaced with structural fill in proposed improvement areas. Remnants from demolition activities should be removed from the site. Demolition of existing improvements should include rerouting, removal, or in-place abandonment of underground utilities. Any existing utilities should be adequately capped or rerouted at the project perimeter in accordance with the requirements of the governing authorities and the recommendations of the geotechnical consultant at the time of demolition. Abandoned underground utility pipes should be removed from the site, or, if the pipes are left in place, they should be filled with flowable fill, such as grout or controlled low strength material (CLSM). The contractor should take adequate precautions when grading the site to reduce the potential for damage to utilities at project site.

8.1.2 Existing Fill

As previously discussed, existing fill at the site was observed within our borings to depths of up to approximately 16.5 feet. Areas of deeper fill are also likely at the site. The fill materials consist primarily of granular material. Additional fills soils should be anticipated in areas between and beyond our boring locations.

We recommend the existing fill be removed down to the underlying native soil in proposed structure and improvement areas. This fill material may be processed and reused as structural fill, provided it meets the recommendations provided herein (see Section 8.1.4).

8.1.3 Site Grading

Prior to grading, proposed structure and improvement areas should be cleared of any topsoil, surface obstructions, debris, organics (including grasses, weeds, shrubs, trees, and roots), and other deleterious material. Materials generated from clearing operations should be removed from the project site for disposal (e.g. at a legal landfill site). As appropriate, topsoil may be stockpiled for later use in landscaped areas.

After the previously described removals have been made, the exposed soils should be scarified to approximately 6 inches, moisture-conditioned to approximately optimum moisture content and compacted to 90 percent relative compaction, as evaluated by ASTM D 1557.

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.

Some shrinkage should be anticipated when on-site soils are excavated, processed, and compacted. For planning purposes, an estimated shrinkage factor of approximately 15 percent may be used for on-site soils.

Findings of our study indicate that some of the fill and native soils encountered in our exploratory borings 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 8.1.4.

8.1.4 Structural Fill and Backfill

Structural fill and backfill soils should consist of coarse-grained material (50 percent or more retained on the No. 200 sieve), with 15 percent or more passing the No. 200 sieve. Fill soils should have a very low to low expansion potential (EI less than 50, as evaluated by ASTM D 4829), and not contain significant amounts of organic matter, debris, other deleterious matter, or rocks or hard chunks larger than approximately 4 inches nominal diameter.

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 D 1557. 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 the referenced Standards and Specifications (APWA, 2017).

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 500 cubic yards 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. Fill materials should not be placed, worked, or rolled while they are frozen, thawing, or during poor/inclement weather conditions.

8.1.5 Import Soil

Import soil should consist of coarse-grained material (50 percent or more retained on the No. 200 sieve) with 15 percent or more passing the No. 200 sieve, 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 D 4829). Import soil should not contain significant amounts of organic matter, debris, other deleterious matter, or rocks or hard chunks larger than approximately 4 inches 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 importation to the project site. Import soil should be moisture-conditioned and placed and compacted in accordance with the recommendations set forth in the previous section.

8.1.6 Temporary Excavations

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 Occupational Safety and Health Administration (OSHA) regulations (OSHA, 2016). 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.

8.2 Structure Foundations

The following subsections provide recommendations for conventional spread foundations planned for support of the proposed structures.

8.2.1 Conventional Spread Foundations

Structure foundations consisting of spread footings should extend 30 inches or more below the lowest adjacent finished grade (for frost protection) and bear on medium dense or stiff native soils or on adequately placed and compacted structural fill (reworked native or import soils). Continuous and isolated footings should have a width of 12 inches or more. Footings should be reinforced in accordance with the project structural engineer's recommendations. From a geotechnical standpoint, we recommend that footings be reinforced with four No. 4 or larger reinforcing bars, two placed near the top and two near the bottom of the footings. Additional reinforcement may be recommended by the structural engineer.

An allowable bearing capacity of 1,500 pounds per square foot (psf) may be used for conventional spread footings with an embedment depth of 30 inches below adjacent finished grade and a width of 12 inches. The allowable bearing capacity may be increased by 250 psf for each additional 1 foot of width and 650 psf for each additional 1 foot of embedment up to 3,000 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 8.4. 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.

8.3 Settlement

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

8.4 Lateral Earth Pressures

For passive resistance to lateral loads, we recommend a passive lateral earth pressure of 290 psf per foot of depth up to a value of 2,500 psf. For active and at-rest lateral earth pressures, we recommend equivalent fluid pressures of 38 psf and 58 psf, respectively. These values 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. We recommend that the upper 12 inches of soil not protected by pavement or a concrete slab be neglected when calculating passive resistance.

For frictional resistance to lateral loads, we recommend that a coefficient of friction of 0.53 be used between soil and soil contacts and/or between soil and cast-against-grade concrete contacts. A coefficient of friction of 0.35 may be used between soil and formed 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. The lateral bearing capacity may be increased by one-third when considering loads of short duration such as wind or seismic forces.

8.5 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 4 inches of Untreated Base Course overlying 12 inches of adequately placed and compacted structural fill. Aggregate base underlying concrete slab-on-grade floors should be compacted to 90 percent of the laboratory maximum dry density (ASTM D 1557).

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 a subsequent section of this report.

As an alternative to slab reinforcement with steel reinforcing bars, post-tensioned slabs designed by a qualified structural engineer may be considered. Geotechnical recommendations for design of post-tensioned slabs-on-grade will be provided by Ninyo & Moore upon request.

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 may overlie or underlie the previously described compacted base material. If the membrane overlies the base material, it should be covered with 2 inches of moist sand (not saturated) to reduce the potential for puncture during construction and to aid in concrete curing. 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 E-96. This membrane should overlie compacted base material and be placed directly under the floor slab. A prepour planning meeting should also be considered to resolve water vapor emission and concrete curing considerations and to establish means for reducing slab curl.

8.6 Exterior Concrete Flatwork

Ground-supported concrete flatwork will 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 4 inches of Untreated Base Course overlying 12 inches or more of compacted structural fill (reworked fill and/or native, or import soils), that meets the recommendations described in Section 8.1.4 of this report. The fill thickness may include 6 inches of scarified and re-compacted soils. Untreated Base Course should be compacted to 90 percent relative compaction, as evaluated by ASTM D 1557.

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.

8.7 Pavement Sections

The following sections provide asphalt concrete pavement sections for on-site parking and access areas for the subject project.

8.7.1 Pavement Sections for Parking and Access Areas

To form a basis for design of flexible pavement for on-site paved parking and access areas, we have assumed the following:

- A design Equivalent Single Axial Load (ESAL) value of 3,000 for automobile traffic; an ESAL value of 16,000 for delivery truck traffic; and an ESAL value of 65,000 for heavy duty truck traffic areas are applicable.
- A reliability of 80 percent.
- A standard deviation of 0.45.
- An initial serviceability index of 4.2.
- A terminal serviceability index of 2.5.
- A subgrade resilient modulus (MR) of 3,500 pounds per square inch (psi) for a minimum R value of 10 (based on soil classification).

Using these values, structural numbers associated with the proposed parking and access areas were calculated using design procedures in accordance with the American Association of State Highway and Transportation Officials method of designing flexible pavement (AASHTO, 1993) requirements. The following table presents recommended structural pavement sections placed over structural fill for on-site parking and access areas.

Table 4 – Recommended Flexible Pavement Section Thickness						
Traffic Type	Design ESAL	Pavement ($a_{asphalt}$ = 0.35)	Base (a_{base} = 0.10)	Subgrade	Structural Number Provided	Structural Number Needed
		Asphalt Concrete Thickness (Inches)	Untreated Base Thickness (Inches)	Structural Fill Thickness (Inches)*		
Automobile	3,000	3.0	6.0	6.0	1.68	1.63
Delivery Truck	16,000	4.0	8.0	6.0	2.20	2.17
Heavy Duty Truck	65,000	4.0	14.0	6.0	2.80	2.75

Note: *Structural fill below pavement sections may include 6 inches of scarified and recompacted native soil.

As an alternative, for heavy truck traffic areas, such as garbage truck aprons, or other truck loading/unloading/turn areas, we recommend a rigid pavement section be considered.

8.7.2 Pavement Considerations

If the assumed traffic or design ESAL values are not considered appropriate, this office should be notified. In providing these recommendations for pavement sections, we have assumed that asphalt concrete will be mixed and placed in accordance with Section 02741 of the referenced UDOT Standard Specifications for Road and Bridge Construction (SSRBC). We have also assumed that Untreated Base Course material will conform to Section 02721 of the referenced SSRBC (UDOT, 2017). Untreated Base Course material should be placed and compacted to 95 percent relative compaction, as evaluated by ASTM D 1557, and in accordance with Section 02721 of the referenced UDOT SSRBC (UDOT, 2017).

We recommend that mix designs be made for the asphalt concrete and Portland cement concrete by an engineering company specializing in this type of work. In addition, paving operations should be observed and tested by a qualified testing laboratory.

Adequate surface drainage should be provided to reduce the potential for ponding and infiltration of water into the pavement and subgrade materials. We suggest that the paved areas have a surface gradient of 1 percent or more. In addition, surface runoff from surrounding areas should be intercepted, collected, and not permitted to flow onto the pavement or infiltrate the base and subgrade. We recommend that perimeter swales, edge drains, curbs and gutters, or combination of these drainage devices be constructed to reduce the adverse effects of surface water runoff.

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 should 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 recompacted. 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 may be 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 where the subgrade soils were placed on engineered fill.

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 was evaluated in the laboratory using representative samples obtained from the exploratory test pits. Results of these tests are presented in Appendix C. Recommendations regarding concrete to be utilized in construction of proposed improvements and for buried metal pipes are provided in the following sections.

8.10.1 Concrete

Chemical tests performed on selected samples of on-site soils indicated negligible sulfate contents. Based on review of the referenced American Concrete Institute manual (ACI, 2014), the tested soil is considered negligibly deleterious to concrete. However, we recommend that concrete in contact with on-site soils, along with subsurface walls up to 12 inches above finished grade, contain Type II cement. We also recommend that concrete in contact with on-site soil have a design compressive strength of 4,000 pounds per square inch (psi), and a water-cement ratio of 0.50 by weight. In addition, 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, 2014), 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, 2014) and project specifications.

8.10.2 Buried Metal Pipes

Ninyo & Moore recommends that corrosion reduction methods be implemented for this project for buried metal pipes. 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 structures. Positive drainage may be established by providing a surface gradient of 5 percent away from structures for a distance of 10 feet measured perpendicular 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 concrete flatwork or asphalt that slopes away from the structure. Downspouts should not be allowed to discharge onto the ground surface adjacent to building foundations.

- 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.

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

8.13 Plan Review

The recommendations presented in this report are based on preliminary design information for the proposed project, as provided by EPG Design 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. Please also note that 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. It should be understood that 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. In addition, 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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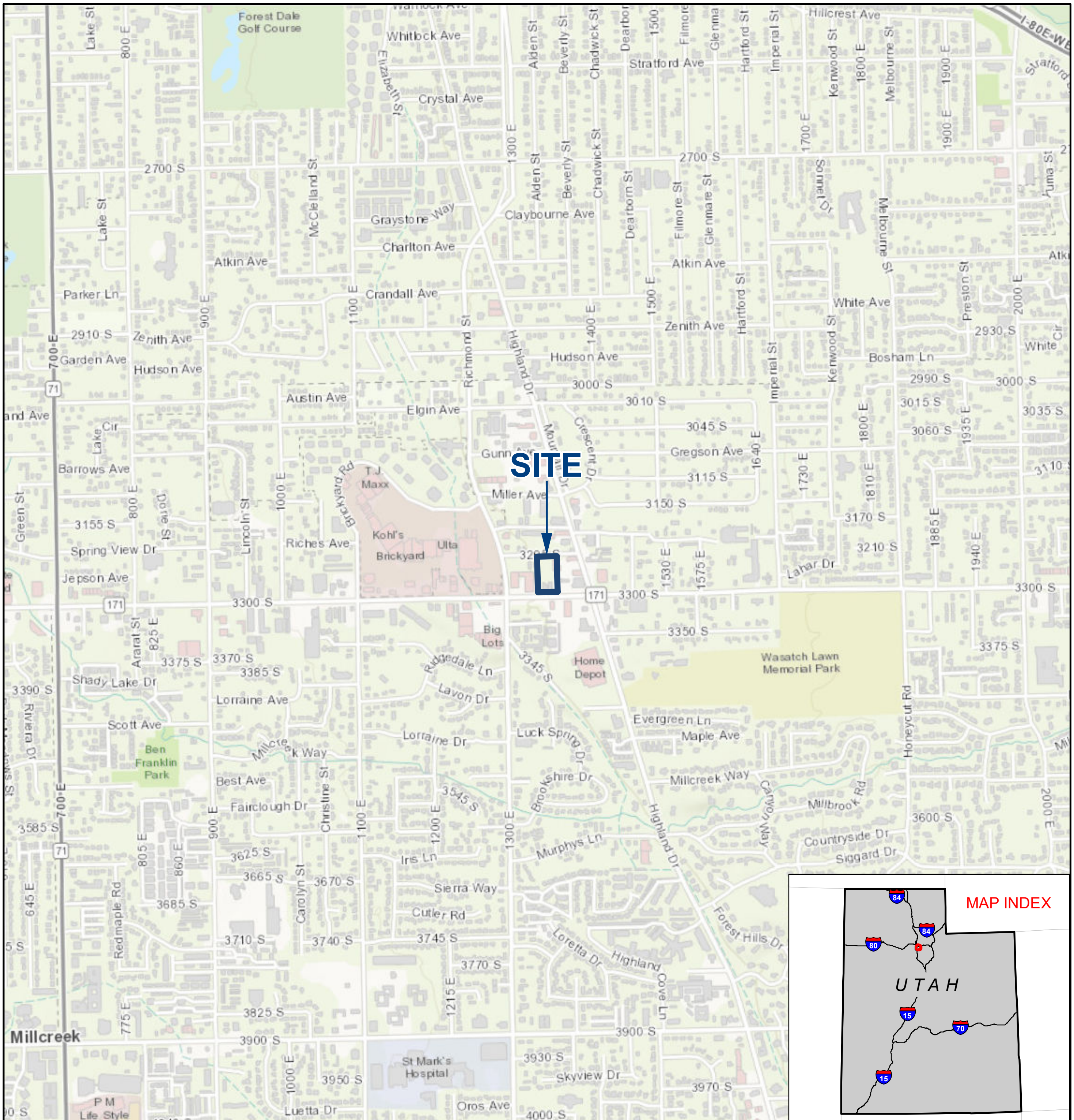
FIGURES



NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: ESRI WORLD TOPO, 2019

N

FEET

0 1,500 3,000

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FIGURE 1

SITE LOCATION

MILLCREEK COMMONS
NEAR 1300 EAST 3300 SOUTH, MILLCREEK, UTAH

800055001 | 6/20

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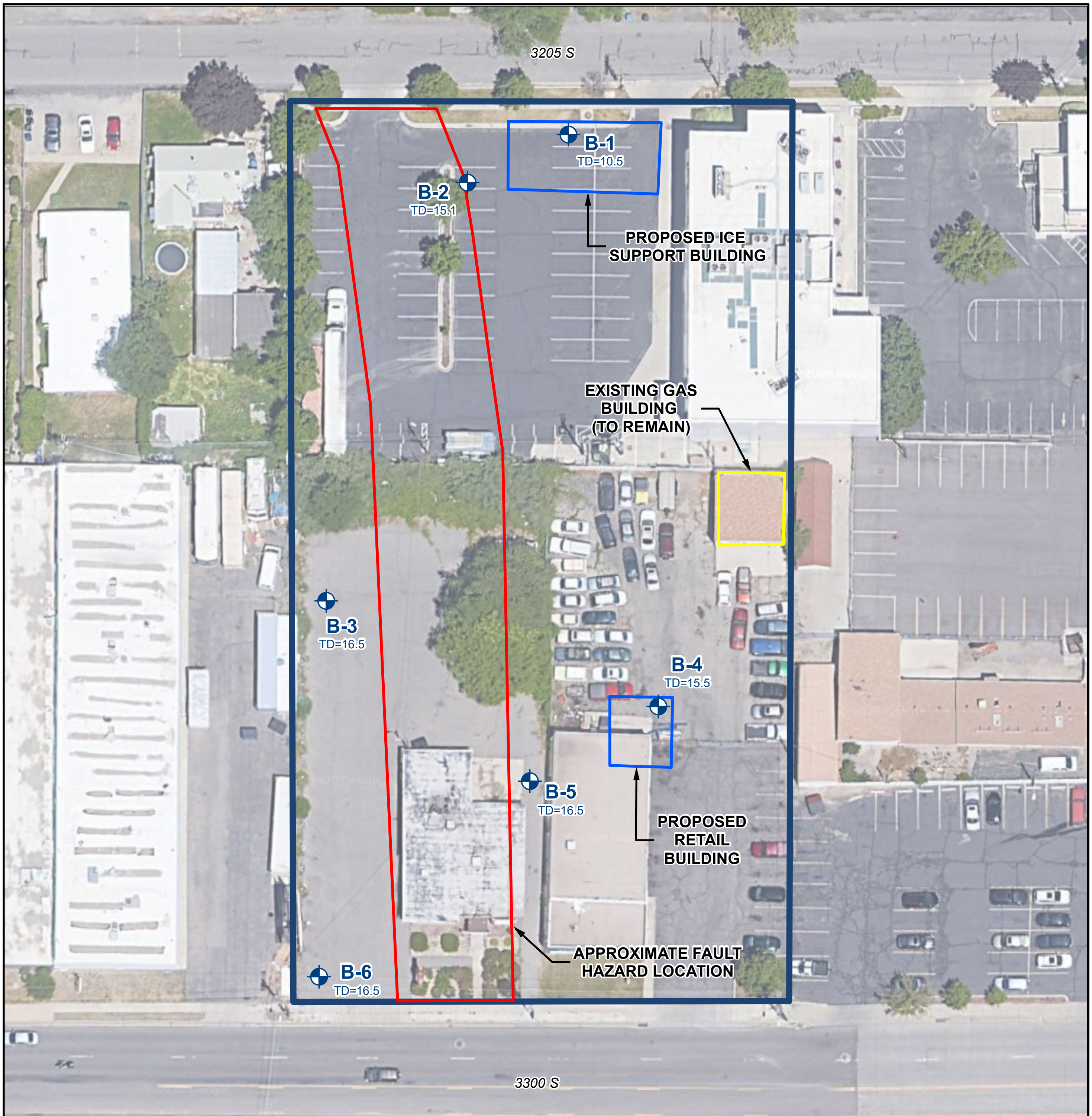

LEGEND

SITE BOUNDARY

B-6 TD=71.5 BORING
TD=TOTAL DEPTH IN FEET

N

FEET
0 60 120

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: GOOGLE EARTH, 2019

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FIGURE 2

BORING LOCATIONS

MILLCREEK COMMONS
NEAR 1300 EAST 3300 SOUTH, MILLCREEK, UTAH

800055001 | 6/20

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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 method.

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. The sampler was driven into the ground with a 140 pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586 and the blow counts were recorded. 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. The sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows during driving are presented 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.

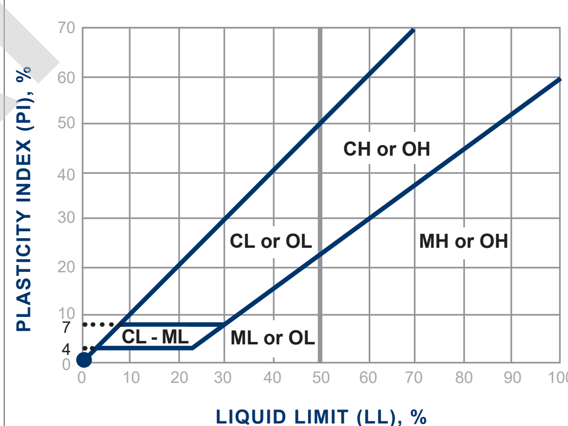
Soil Classification Chart Per ASTM D 2488

Primary Divisions			Secondary Divisions: Group Symbol	Secondary Divisions: 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
		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: SPT (blows/foot)	Spooling Cable or Cathead: Modified Split Barrel (blows/foot)	Automatic Trip Hammer: SPT (blows/foot)	Automatic Trip Hammer: 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

Consistency	Spooling Cable or Cathead: SPT (blows/foot)	Spooling Cable or Cathead: Modified Split Barrel (blows/foot)	Automatic Trip Hammer: SPT (blows/foot)	Automatic Trip Hammer: 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

DEPTH (feet)	SAMPLES: Bulk	SAMPLES: Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	BORING LOG EXPLANATION SHEET
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.

DATE DRILLED 5/12/2020 BORING NO. B-1

GROUND ELEVATION 4,416' ± (MSL) SHEET 1 OF 1

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

DRIVE WEIGHT 140 lbs DROP 30"

SAMPLED BY REG LOGGED BY REG REVIEWED BY EDE

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 3 inches thick.
							GP	AGGREGATE BASE: Brown, dry, very dense, poorly-graded GRAVEL with sand. Approximately 4 inches thick.
							GP	FILL: Brown, dry, very dense, poorly-graded GRAVEL with sand and cobbles.
			27/6" 50/3"	5.8	90.1			
5			32/6" 23/6" 23/6"					Dense.
10			50/6"					Very dense.
15								
20								

Total Depth = 10.5 feet.
Groundwater not encountered at the time of drilling.
Backfilled on 5/12/2020.

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.

DATE DRILLED 5/12/2020 BORING NO. B-2

GROUND ELEVATION 4,413' ± (MSL) SHEET 1 OF 1

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

DRIVE WEIGHT 140 lbs DROP 30"

SAMPLED BY REG LOGGED BY REG REVIEWED BY EDE

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 3 inches thick.
							GP	AGGREGATE BASE: Brown, dry, very dense, poorly-graded GRAVEL with sand. Approximately 4 inches thick.
				8.7			SM	FILL: Brown, moist, medium dense, silty SAND with gravel.
			7/6" 11/6" 22/6"					
5			18/6" 10/6" 10/6"				SC	Brown, dry, medium dense, clayey SAND.
10			19/6" 15/6" 13/6"				SM	NATIVE SOIL: Light brown, dry, medium dense, silty SAND with gravel and cobbles.
15			50/1"					Total Depth = 15.1 feet. Groundwater not encountered at the time of drilling. Backfilled on 5/12/2020. 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.
20								

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MILLCREEK COMMONS
NEAR 1300 EAST AND 3300 SOUTH, MILLCREEK, UTAH
800055001 |06/2020

DATE DRILLED 5/12/2020	BORING NO. B-3	
GROUND ELEVATION 4,408' ± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING Mobile B-80 Drill Rig with Hollow Stem Auger		
DRIVE WEIGHT 140 lbs	DROP 30"	
SAMPLED BY REG	LOGGED BY REG	REVIEWED BY EDE

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 3 inches thick.
							GP	AGGREGATE BASE: Brown, dry, very dense, poorly-graded GRAVEL with sand. Approximately 4 inches thick.
							CL	FILL: Black/brown, moist, stiff, sandy, lean CLAY.
		X	4/6" 5/6" 8/6"	19.0	106.4		CL	NATIVE SOIL: Brown, moist, stiff, lean CLAY.
5		X	4/6" 5/6" 6/6"	16.6	107.0			
10		X	6/6" 10/6" 10/6"	25.7	95.0			
15		X	3/6" 3/6" 9/6"	28.8	93.4			Wet.
20								Total Depth = 16.5 feet. Groundwater not encountered at the time of drilling. Backfilled on 5/12/2020. 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.

DATE DRILLED 5/12/2020 BORING NO. B-4

GROUND ELEVATION 4,416' ± (MSL) SHEET 1 OF 1

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

DRIVE WEIGHT 140 lbs DROP 30"

SAMPLED BY REG LOGGED BY REG REVIEWED BY EDE

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 1.5 inches thick.
							GP	AGGREGATE BASE: Brown, dry, very dense, poorly-graded GRAVEL with sand. Approximately 5 inches thick.
							CL	NATIVE SOIL: Brown, moist, stiff, lean CLAY.
			4/6" 6/6" 8/6"					
5								Trace sand.
			5/6" 6/6" 9/6"	28.5	93.4			
							GP	Brown, moist, very dense, poorly-graded GRAVEL with sand.
10			12/6" 26/6" 29/6"					
15			50/6"					
								Total Depth = 15.5 feet. Groundwater not encountered at the time of drilling. Backfilled on 5/12/2020.
								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.
20								

DATE DRILLED	5/12/2020	BORING NO.	B-5
GROUND ELEVATION	4,412' ± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING	Mobile B-80 Drill Rig with Hollow Stem Auger		
DRIVE WEIGHT	140 lbs	DROP	30"
SAMPLED BY REG	LOGGED BY REG	REVIEWED BY EDE	

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 3 inches thick.
							GP	AGGREGATE BASE: Brown, dry, very dense, poorly-graded GRAVEL with sand. Approximately 4 inches thick.
							GM	FILL: Brown, moist, very dense, silty GRAVEL with sand.
			50/3"					
5			14/6" 3/6" 12/6"					Medium dense.
10			19/6" 13/6" 10/6"					(Upon withdrawal of the auger, the boring caved in revealing a void measuring approximately 6 feet wide by 6 feet long by 10 feet deep.)
							SM	Brown, moist, medium dense, silty SAND.
15			32/6" 15/6" 11/6"					
								Total Depth = 16.5 feet. Groundwater not encountered at the time of drilling. 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.
20								

DATE DRILLED	5/12/2020	BORING NO.	B-6
GROUND ELEVATION	4,408' ± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING	Mobile B-80 Drill Rig with Hollow Stem Auger		
DRIVE WEIGHT	140 lbs	DROP	30"
SAMPLED BY REG	LOGGED BY REG	REVIEWED BY EDE	

DEPTH (feet)	SAMPLES Bulk / Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						ASPHALT CONCRETE: Approximately 4 inches thick.
					GP	AGGREGATE BASE: Brown, dry, very dense, poorly-graded GRAVEL with sand. Approximately 5 inches thick.
					CL	NATIVE SOIL: Brown, moist, stiff, lean CLAY.
	Driven	5/6" 8/6" 11/6"	17.6	102.9		Trace sand.
5	Driven	2/6" 1/6" 2/6"				Soft.
					GP	Brown, moist, very dense, poorly-graded GRAVEL.
10	Driven	50/0"				Few cobbles.
15	Driven	27/6" 23/6" 22/6"				
						Total Depth = 16.5 feet. Groundwater not encountered at the time of drilling. Backfilled on 5/12/2020. 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.
20						

APPENDIX B

Laboratory Test Results

APPENDIX B

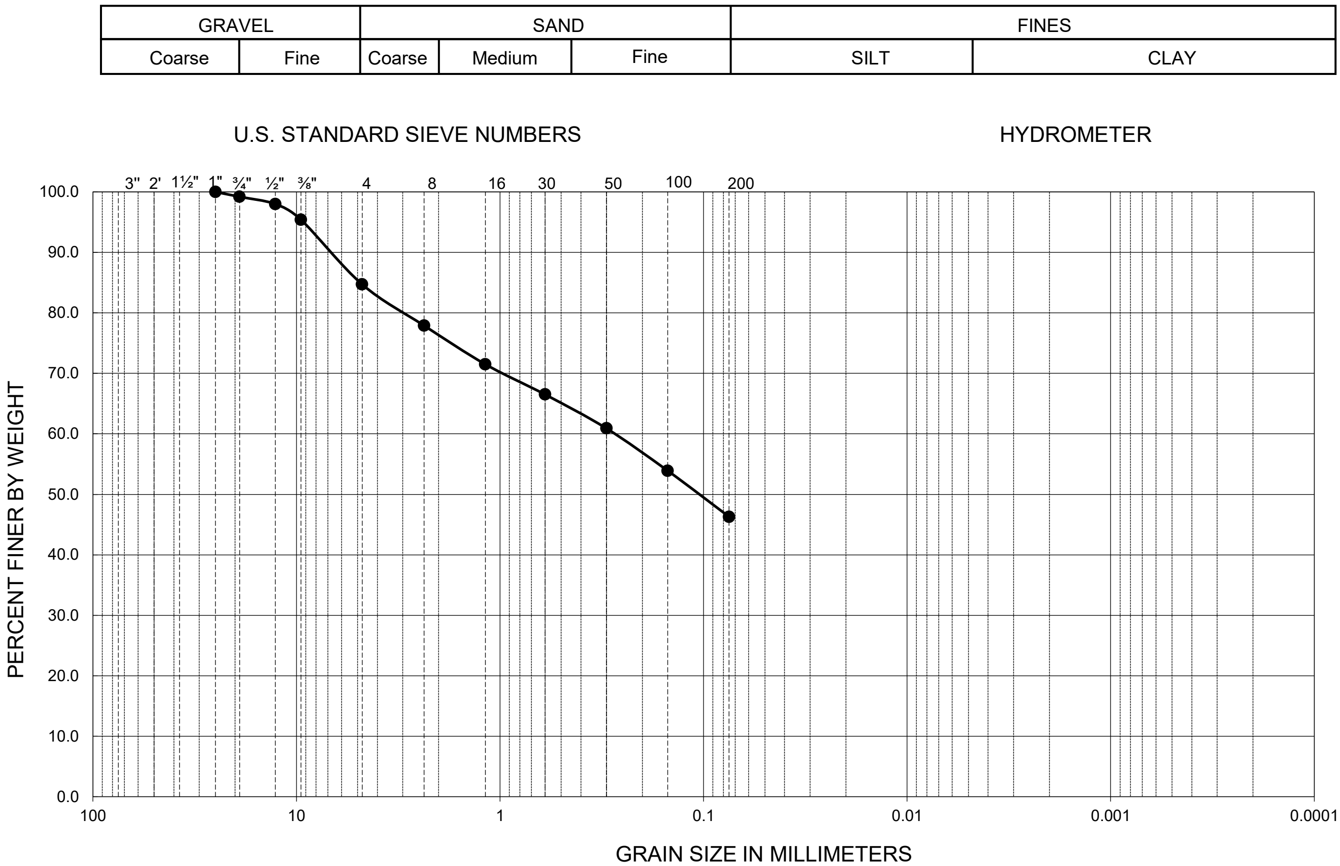
LABORATORY TESTING

Classification
Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. 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 ring-lined samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory test pits in Appendix A.

Gradation Analysis
Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D 422. 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 through Figure B-3.

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 D 4318. These test results were utilized to evaluate soil classification in accordance with the USCS. The test results and classifications are shown on Figure B-4.



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	1.0-5.0	NP	NP	NP	--	--	0.27	--	--	46.3	SM

Material Percent by Weight			Soil Type
Gravel	Sand	Fines	Silty SAND with gravel
15.3	38.4	46.3	

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

"NP" INDICATES NON-PLASTIC

FIGURE B-1

GRADATION TEST RESULTS

MILLCREEK COMMONS
NEAR 1300 EAST 3300 SOUTH, MILLCREEK, UTAH

800055001 | 06/2020

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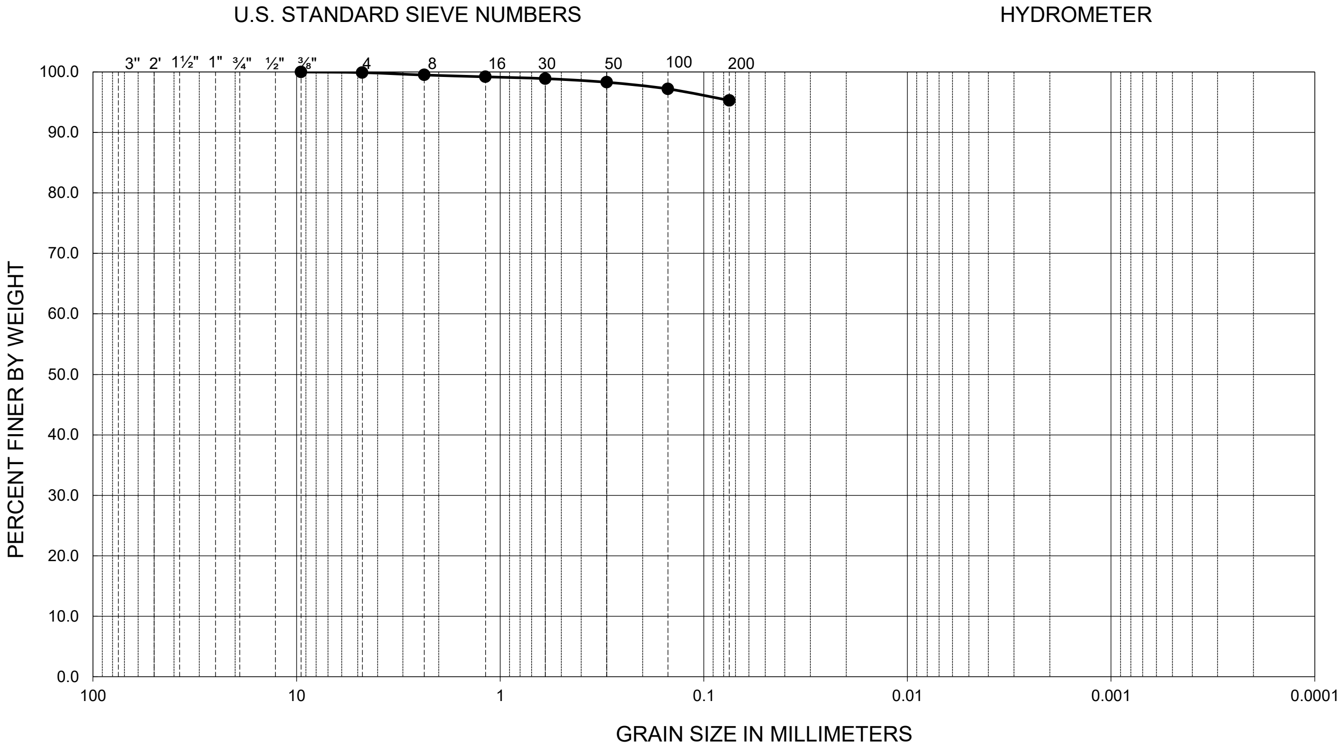
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



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-4	5.0-6.5	38	19	19	--	--	--	--	--	95.3	CL

Material Percent by Weight			Soil Type
Gravel	Sand	Fines	Lean CLAY
0.1	4.6	95.3	

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-2

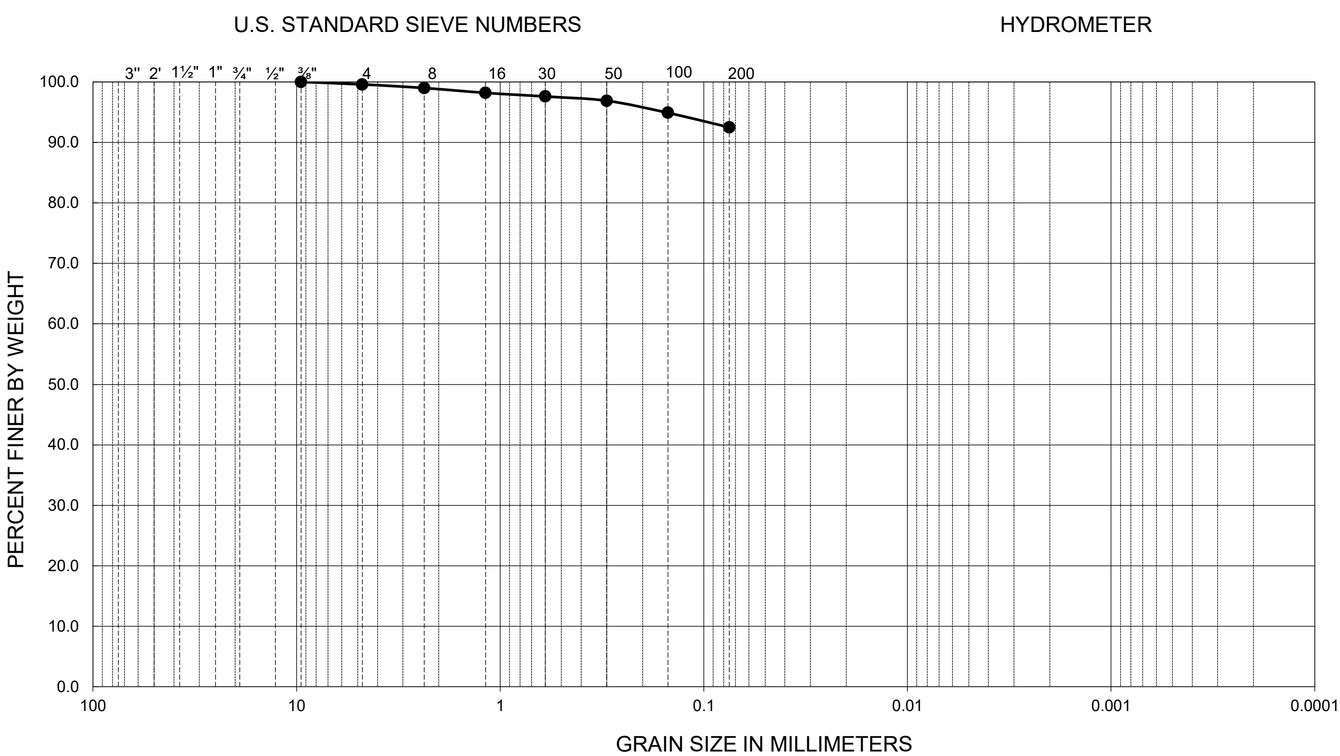
GRADATION TEST RESULTS

MILLCREEK COMMONS
NEAR 1300 EAST 3300 SOUTH, MILLCREEK, UTAH

800055001 | 06/2020

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

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-6	2.0-3.5	36	18	18	--	--	--	--	--	92.5	CL

Material Percent by Weight			Soil Type
Gravel	Sand	Fines	Lean CLAY
0.4	7.1	92.5	

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-3

GRADATION TEST RESULTS

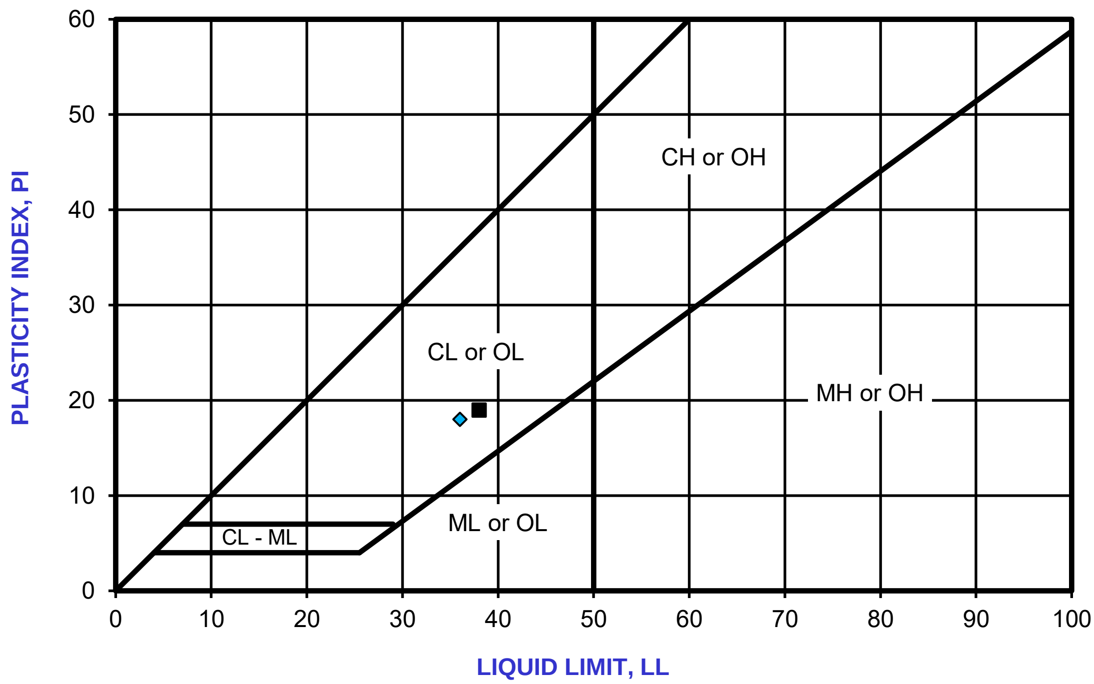
Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

MILLCREEK COMMONS
NEAR 1300 EAST 3300 SOUTH, MILLCREEK, UTAH

800055001 | 06/2020

SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
	B-2	1.0-5.0	NP	NP	NP	ML	SM
■	B-4	5.0-6.5	38	19	19	CL	CL
◆	B-6	2.0-6.5	36	18	18	CL	CL

NP - INDICATES NON-PLASTIC



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

FIGURE B-4

ATTERBERG LIMITS TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

MILLCREEK COMMONS
NEAR 1300 EAST 3300 SOUTH, MILLCREEK, UTAH

800055001 | 06/2020

APPENDIX C

Chemical Test Results

APPENDIX C

CHEMICAL TEST RESULTS

The results of chemical tests are provided in this appendix.

Veritas Laboratories
Truth in Quality, Truth in Service
6245 Harrison Drive, Suite 4, Las Vegas, NV 89120
(702) 321-8315 Phone
(702) 597-2098 Fax
Email: veritaslabs@msn.com

CLIENT COMPANY NAME: Ninyo and Moore
CLIENT PROJECT NAME: **Millcreek Commons**
CLIENT PROJECT NUMBER: 800055001
VERITAS LAB ORDER ID: V20F020

ANALYTICAL RESULTS

CLIENT SAMPLE ID: **B-2 @ 1.0'-5.0'**
VERITAS SAMPLE ID: V20F020-01
DATE/TIME SAMPLED:
DATE/TIME RECEIVED: 6/3/20 10:20

Matrix: Soil

Analysis: Soil Solubility/Corrosion Parameters

PARAMETER	RESULT	UNITS	METHOD	DATE ANALYZED
pH	**8.33**	pH Units	EPA 9045 D	6/3/20
Redox Potential (ORP)	**241**	mV	SM 2580B	6/4/20
Resistivity, Saturated (Minimum)	**1600**	Ohm-cm	AASHTO T-288	6/4/20
Water Soluble Sodium	**0.015**	%	EPA 6010B	6/4/20
Water Soluble Sulfate	**0.014**	%	SM 4500-SO4 E	6/4/20
Soluble Sulfide	**<0.50**	mg/Kg	SM 4500-S2-D	6/4/20
Total Available Water Soluble Sodium Sulfate	**0.021**	%	Calculation	6/4/20
Total Soluble Salts (Solubility)	**0.061**	%	SM 2540C	6/4/20
Water Soluble Chloride	**20**	mg/Kg	SM 4500-Cl B	6/4/20

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Email: veritaslabs@msn.com

CLIENT COMPANY NAME: Ninyo and Moore
CLIENT PROJECT NAME: **Millcreek Commons**
CLIENT PROJECT NUMBER: 800055001
VERITAS LAB ORDER ID: V20F020

ANALYTICAL RESULTS

CLIENT SAMPLE ID: **B-6 @ 2.0'-6.5'**
VERITAS SAMPLE ID: V20F020-02
DATE/TIME SAMPLED:
DATE/TIME RECEIVED: 6/3/20 10:20

Matrix: Soil

Analysis: Soil Solubility/Corrosion Parameters

PARAMETER	RESULT	UNITS	METHOD	DATE ANALYZED
pH	**8.13**	pH Units	EPA 9045 D	6/3/20
Redox Potential (ORP)	**272**	mV	SM 2580B	6/4/20
Resistivity, Saturated (Minimum)	**950**	Ohm-cm	AASHTO T-288	6/4/20
Water Soluble Sodium	**0.0082**	%	EPA 6010B	6/4/20
Water Soluble Sulfate	**0.015**	%	SM 4500-SO4 E	6/4/20
Soluble Sulfide	**<0.50**	mg/Kg	SM 4500-S2-D	6/4/20
Total Available Water Soluble Sodium Sulfate	**0.023**	%	Calculation	6/4/20
Total Soluble Salts (Solubility)	**0.066**	%	SM 2540C	6/4/20
Water Soluble Chloride	**55**	mg/Kg	SM 4500-Cl B	6/4/20

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