Geotechnical Evaluation

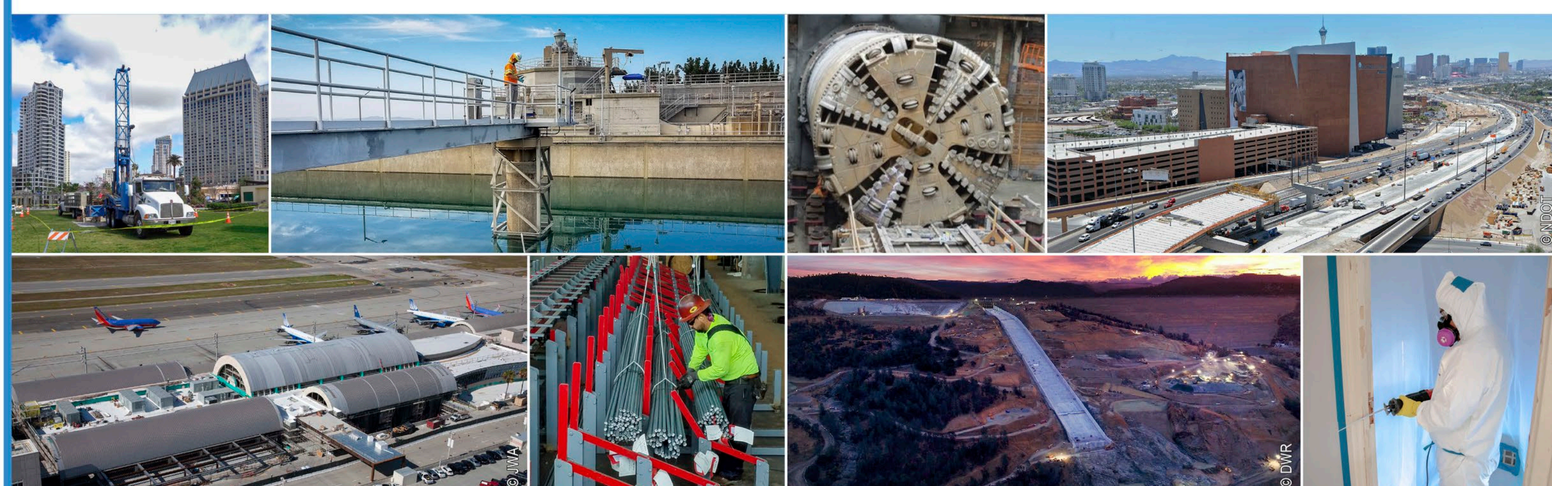
Proposed SVA Greeley Dental Building
1911 59th Avenue

Greeley, Colorado

SVA Greeley LLC

4609 South Timberline Road #103B
Fort Collins, Colorado 80528

October 23, 2024 | Project No. 503044001



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35+ YEARS

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Mr. Keith Van Tassell
SVA Greeley LLC
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1. INTRODUCTION

In accordance with your request and authorization and our proposal dated September 12, 2024, we have performed a geotechnical evaluation for the proposed single-story dental building building to be located on a vacant parcel at 1911 59th Avenue in Greeley, Colorado. The approximate location of the site is depicted on Figure 1.

The purpose of our study was to evaluate the subsurface conditions and to provide design and construction recommendations regarding geotechnical aspects of the proposed project. This report presents the findings of our subsurface exploration, results of our laboratory testing, conclusions regarding the subsurface conditions at the site, and geotechnical recommendations for the design and construction of this project.

2. SCOPE OF SERVICES

The scope of our services for the project generally included:

- Review of referenced background information, including aerial photographs, published geologic and soil maps, previous geotechnical evaluations, in-house geotechnical data, and available topographical information pertaining to the project site and vicinity.

- Site reconnaissance to document site conditions and establish boring locations, and arrange for the mark-out of publicly owned underground utilities through Utility Notification Center of Colorado of the boring locations prior to drilling.

- Drilling, logging, and sampling of six (5) small-diameter exploratory borings within the project site. Three (3) borings were performed within the proposed building footprint to a maximum depth of approximately 20 feet below the ground surface (bgs). Two (2) borings were performed in the parking and drive lane areas to a maximum depth of approximately 10 feet bgs. The boring logs are presented in Appendix A. Approximate boring locations are presented on Figure 2.

- Performing laboratory tests on selected samples obtained from the borings to evaluate engineering properties including in-situ moisture content and dry density, Atterberg limits, percent materials passing the No. 200 sieve and grain size analysis, swell/consolidation potential, and soil corrosivity characteristics (including pH, resistivity, water soluble sulfates and chlorides). The results of the laboratory testing are presented on the boring logs and in Appendix B.

- Compilation and analysis of the data obtained.

- Preparation of this report presenting our findings, conclusions, and geotechnical recommendations regarding design and construction of the project.

3. SITE DESCRIPTION AND BACKGROUND REVIEW

The project site consists of a portion of an approximately 0.913-acre parcel of land located west of 59th Avenue and north of W 20th Street in Greeley, Colorado. The site is border by a commercial building to the south, a bowling alley to the east, a car wash facility to the north and an orthopedic center to the west.

Historical research and available aerial imagery revealed that the site has been vacant from 1985 or earlier. Historically, the site and surrounding properties appear to have been used for agricultural purposes prior to approximately 2004 where aerial imagery shows earthwork operations on the site, and neighboring sites to the south, west and north of the site, and development of the commercial structures north and south of the site. The orthopedic center and associated parking appear to have been constructed in 2023. The approximate location of the site is depicted on Figure 1.

At the time of our field exploration, the site consisted of vacant land with grasses and seasonal weeds. The topography of the site was generally level with an average elevation of approximately 4,839 feet.

4. PROPOSED CONSTRUCTION

The project consists of the design and construction of a single-story dentist building with a building footprint of approximately 7,500 square feet. The building will have a finished floor elevation (FFE) of approximately 4,841.78 feet, requiring cuts and fills of about 2 feet or less. No below ground space is anticipated as part of this development. A pavement parking area and drive-lanes are planned on the west side of the building, and exterior concrete flatwork is planned around the perimeter of the building.

5. FIELD EXPLORATION AND LABORATORY TESTING

On September 23, 2024, Ninyo & Moore conducted subsurface exploration services at the project site to evaluate the existing subsurface conditions and to collect soil samples for visual observation and laboratory testing. The evaluation consisted of the drilling, logging, and sampling of 5 exploratory borings using a truck-mounted drill rig equipped with 4-inch diameter, continuous-flight, solid-stem augers. The borings were advanced within the project site to depths of approximately 10 to 20 feet below ground surface (bgs). Relatively undisturbed and disturbed soil and bedrock samples were collected at selected intervals. The approximate locations of the borings are presented on Figure 2. Boring coordinates and ground elevations were measured in the field using a Trimble Model DA2-BT survey unit with a global navigation satellite system (GNSS) output of NAD83 (2011) and referencing Geoid model GEOID18. The boring coordinates and elevations are presented in the table below:

Table 1 – Boring Coordinates			
Boring No.	**Elevation (feet)**	**Latitude (Decimal Degrees)**	**Longitude (Decimal Degrees)**
B-1	4,838.5	40.408226	-104.775510
B-2	4,838.4	40.408051	-104.775636
B-3	4,840.7	40.407828	-104.775511
B-4	4,836.9	40.408217	-104.775891
B-5	4,840.9	40.407805	-104.775852

Note: Coordinates and elevations collected with Trimble DA2-BT survey unit with GNSS output of NAD83 (2011) and GEOID 18. Vertical precision of +/-1.5 inches and horizontal precision of +/- 1-inch using the above GNSS output and geoid model at this site.

The soil samples collected during the subsurface exploration were transported to the Ninyo & Moore laboratory for geotechnical laboratory analyses. Selected samples were analyzed to evaluate engineering properties including in-situ moisture content and dry density, Atterberg limits, percent materials passing the No. 200 sieve and grain size analysis, swell/consolidation potential, and soil corrosivity characteristics (including pH, resistivity, water soluble sulfates and chlorides). The results of the in-situ moisture content and dry density tests are presented on the boring logs in Appendix A. Descriptions of the laboratory test methods and the remainder of the test results are presented in Appendix B.

6. GEOLOGIC AND SUBSURFACE CONDITIONS

6.1. Regional Geologic Setting

The site is located in Greeley, Colorado, approximately 20 miles east of the Rocky Mountain Front Range, within the Colorado Piedmont section of the Great Plains Physiographic Province.

During the Late Cretaceous, between approximately 68 to 100 million years ago, a shallow sea transgressed and regressed in the interior of the continent, known as the Western Interior Seaway. Present day Greeley was submerged when the Western Interior Seaway was at its deepest level. As the Western Interior Seaway receded, sand was deposited along its shoreline forming the Fox Hill Sandstone Formation. (Roberts, 1995).

A mountain building event known as the Laramide Orogeny uplifted the Rocky Mountains during the end of Late Cretaceous and through the early Tertiary Periods, between approximately 70 to 35 million years ago. As the mountain range grew, sediments were deposited east of the Rocky Mountains, comprising the Denver Formation and Dawson Arkose Formation. (Carroll and Crawford, 2000).

As a result of regional uplift approximately 5 to 10 million years ago, the Colorado Piedmont was formed between the South Platte River and Arkansas River on the eastern edge of the Great Plains. Streams down-cut and excavated into the Colorado Piedmont, forming alluvial terraces along channels. Colluvium was deposited along valley edges and adjacent to slopes within the Colorado Piedmont. Eolian deposits were spread out along the east side of the Colorado Piedmont by westerly winds that suspended fine sand, silt, and clay from streambeds and floodplains (Trimble, 1980).

The surficial geology of the site vicinity is mapped by Palkovic (2020) as Middle Holocene to Upper Pleistocene Eolian Sediment (Qe). These deposits consists of sands, silts and clays deposited by wind. Fox Hill Sandstone bedrock is mapped as underlying the project area and generally consists of very fine to fine grained sandstone with lenses of shale.

Our understanding of the subsurface conditions at the project site is based on our field exploration and laboratory testing, review of published geologic maps, historic aerial photographs, and our experience with the general geology of the area. The following sections provide a generalized description of the subsurface materials encountered.

6.2. Subsurface Conditions

Our understanding of the subsurface conditions at the project site is based on our field exploration and laboratory testing, review of published geologic maps, historic aerial photographs, and our experience with the general geology of the area. The following sections provide a generalized description of the subsurface materials encountered. More detailed descriptions are presented on the boring logs in Appendix A.

6.2.1. Fill

Fill material was encountered at the surface in each boring and extended to depths ranging from approximately 4 to 7 feet bgs. The fill material generally consisted of brown, gray, moist sandy lean clay, clayey sand, and silty sand. This fill material is considered undocumented

Based on laboratory testing performed on selected samples of the fill materials, the in-place moisture contents ranged between approximately 5.7 and 15.3 percent and the in-place dry densities ranged between approximately 100.9 and 117 pounds per cubic foot (pcf).

6.2.2. Eolian

Eolian was encountered in each boring beneath the fill material and extended to a depth of 17 to 18 feet bgs in borings B-1 and B-3, and to the borings' termination depths ranging from approximately 10 to 25 feet bgs in the remaining borings. The eolian deposits generally consisted of various shades of gray, brown, moist to wet, stiff, sandy lean clay and very loose to medium dense, fine to medium sand with varying amounts of clay and silt.

Based on the results of the laboratory testing, selected samples of the alluvium had in-place moisture contents between approximately 14.6 and 22.2 percent and dry densities between approximately 106.6 and 114.0 pcf.

6.2.3. Fox Hill Formation

The Fox Hill Formation was encountered below the eolian deposits in Borings B-1 and B-3 and extended to boring termination depths. The Fox Hill Formation generally consisted of brow, moist, moderately hard, sandy claystone.

Based on the results of the laboratory testing, selected samples of the alluvium had in-place moisture contents between approximately 21.5 and 24.8 percent and dry densities between approximately 103.4 and 106.1 pcf.

6.3. Groundwater

Groundwater was encountered in the building Borings B-1, B-2, and B-3 at depths between 9 and 12 feet bgs during the subsurface exploration. Groundwater was not encountered in the remaining borings; however, it is likely that similar groundwater levels exist across the site. Stabilized groundwater levels were not taken as part of our exploration. If below ground space is anticipated as part of this development the site should be further evaluated for groundwater levels.

Groundwater levels will fluctuate due to seasonal variations in the amount of rainfall, runoff, groundwater withdrawal from adjacent sites, and other factors. In addition, perched water can develop in the higher permeability overburden deposits or at the soil and bedrock interface following periods of heavy or prolonged precipitation. The possibility of groundwater level fluctuations and perched water should be considered when developing the design and construction plans for the project. In general, it is our opinion groundwater should not be a concern to the development of the structure.

7. GEOLOGIC HAZARDS

The following sections describe potential geologic hazards at the site including faulting and seismicity, expansive soils and compressible/collapsible soils.

7.1. Faulting and Seismicity

Historically, several minor earthquakes have been recorded around the Front Range area. Based on our field observations and our review of readily available published geological maps and literature, there are no known active faults underlying or adjacent to the subject site. The closest faults to the site are the Rocky Mountain Arsenal Fault and the Golden Fault.

The Rocky Mountain Arsenal Fault lies approximately 41 miles south of the site (Kirkham and Rogers, 1981). The most recent significant seismic movements associated with the Rocky Mountain Arsenal Fault occurred in the 1960s, with recorded earthquake magnitudes up to 5.5. USGS investigators concluded that a strong correlation existed between the seismic activity of this fault and pressure injection of liquid waste into a disposal well located at the nearby Rocky Mountain Arsenal. Pressure injection in the disposal well was discontinued in 1966 and minor seismic movements along the fault have been recorded since. The risk of this fault giving rise to damaging, earthquake-induced ground motions at the site during the design life of the proposed structure is considered to be relatively low, based on the previously recorded low seismic magnitudes.

The Golden Fault lies approximately 52 miles southwest of the site (USGS & CGS, 2013). The fault is considered to be late Quaternary in age and has not shown displacement in Holocene time, as Pleistocene deposits overlie the fault (approximately 75 to 125 thousand years before the present [Kirkham, 1977]). Therefore, the probability of damage at the site from seismically induced ground surface rupture from this fault is considered to be low.

Design of the proposed improvements should be performed in accordance with the requirements of the governing jurisdictions and applicable building codes. Table 2 presents the seismic design parameters for the site in accordance with the 2021 International Building Code (IBC) guidelines and adjusted maximum considered earthquake spectral response acceleration parameters evaluated using the OSHPD (OSHPD, 2024) ground motion calculator (web-based).

Table 2 – 2021 International Building Code Seismic Design Criteria	
Seismic Design Factors	**Value**
Site Class	D
Site Coefficient, F_a	1.6
Site Coefficient, F_v	2.4
Mapped Spectral Acceleration at 0.2-second Period, S_s	0.155
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.051
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	0.248
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.122
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	0.166
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.081

7.2. Expansive Soils

One of the more significant geologic hazards in the Front Range area is the presence of swelling clays in bedrock or surficial deposits. Wetting and drying of bedrock or surficial deposits containing swelling clays can result in expansion and collapse of those units, which can cause major damage to structures. A review of a Colorado Geological Survey map delineating areas based on their relative potential for swelling in the Front Range by Hart (1973-1974) indicates that the soil and bedrock materials in the site vicinity typically exhibit moderate to high swell potential.

Based on the results of our laboratory testing, select samples of the surficial fill materials exhibited a swell potential between 1.8 to 4.3 percent at a surcharge pressure of 150 pounds per square foot (psf) and a negligible swell potential of at a surcharge pressure of approximately 500 pounds psf. The slab performance risk categories per the representative percent swell value under a given surcharge pressure are presented in Table 4.

Based on the results of our subsurface exploration, laboratory testing, and the information obtained from our background review, the on-site soils expected to be encountered during project development would have a slab performance risk category of “LOW” based on the criteria presented in Table 3.

Table 3 – Slab Performance Risk Categories		
Slab Performance Risk Category	**Representative Percent Swell (500 psf Surcharge)**	**Representative Percent Swell (1,000 psf Surcharge)**
LOW	0 to <3	0 to <2
MODERATE	3 to <5	2 to <4
HIGH	5 to <8	4 to <6
VERY HIGH	> 8	> 6

NOTE: The information provided in this table is based on Colorado Association of Geotechnical Engineers (CAGE), Guidelines for Slab Performance Risk Evaluation and Residential Basement Floor System Recommendations (Denver Metropolitan Area, 1996).

On-site soils expected to be encountered during the project are anticipated to have a pavement performance risk category of "MODERATE" based on the criteria presented in Table 4.

Table 4 – Pavement Performance Risk Categories	
Pavement Performance Risk Category	**Representative Percent Swell (200 psf Surcharge)**
NONE	0
LOW	0 to <1
MODERATE	1 to <5
HIGH	5 to 20
VERY HIGH	> 20

NOTE: The information provided in this table is based on Colorado Department of Transportation (CDOT) Pavement Design Manual (2021), Chapter 4.

Recommendations are provided in this report to reduce the swell potential of the site soils and bedrock by means of sub-excavation (excavation of soils, processing and moisture conditioning of the soils/bedrock, and replacing the processed soil/bedrock as properly compacted engineered fill). The recommendations provided in this report will not eliminate the post-construction swell risk and are intended to reduce and control the risk.

7.3. Compressible/Collapsible Soils

Compressible soils are generally comprised of soils that undergo consolidation when exposed to new loadings, such as fill or foundation loads. Soil collapse (or hydrocollapse) is a phenomenon where soils undergo a significant decrease in volume upon an increase in moisture content, with or without an increase in external loads. Buildings, structures, and other improvements may be subject to excessive settlement-related distress when compressible soils or collapsible soils are present.

Based on our subsurface evaluation and the results of our laboratory testing, the potential for post-construction consolidation of the alluvial deposits is low. However, compression of the undocumented fill material could occur due to the new building loads. As a result, remedial grading is recommended in this report to minimize this risk.

8. CONCLUSIONS

The purpose of our study was to provide an evaluation of the site with regard to the geotechnical aspects of the future site development. Based on the results of our limited geotechnical evaluation, it is our opinion that residential development of the site is feasible from a geotechnical perspective. However, the presence of undocumented fill encountered in our buildings will be a constraint to this development. Our findings and conclusions pertaining to the geotechnical aspects of the property are presented below.

- Fill material was encountered at the surface in each boring and extended to depths ranging from approximately 4 to 7 feet bgs. The fill material generally consisted of brown, gray, moist sandy lean clay, clayey sand, and silty sand. Without documentation of the placement and compaction of the fill materials, the fill material is considered undocumented.

- In addition to the fill material encountered in our borings, other fill material may be present across the site and to deeper depths than were encountered in our borings. Based on aerial history review, we assume a majority of the deep fills encountered in our borings is associated with the surrounding site development in 2004.

- Eolian was encountered in each boring beneath the fill material and extended to a depth of 17 to 18 feet bgs in Borings B-1 and B-3, and to the borings' termination depths ranging from approximately 10 to 25 feet bgs in the remaining borings. The eolian deposits generally consisted of various shades of gray, brown, moist to wet, stiff, sandy lean clay and very loose to medium dense, fine to medium with varying amounts of clay and silt.

- The Fox Hill Formation was encountered below the eolian deposits in Borings B-1 and B-3 and extended to boring termination depths. The Fox Hill Formation generally consisted of brow, moist, moderately hard, sandy claystone.

- Groundwater was encountered in the building Borings B-1, B-2, and B-3 at depths between 9 and 12 feet bgs during the subsurface exploration. Groundwater was not encountered in the remaining borings; however, it is likely that similar groundwater levels exist across the site. Stabilized groundwater levels were not taken as part of our exploration. If below ground space is anticipated as part of this development the site should be further evaluated for groundwater levels.

- Based on the results of our laboratory testing, the soils expected to be encountered during project development exhibit a slab performance risk category of low on a scale that ranges between low, moderate, high, and very high.

- Based on the results of our laboratory testing, the soils expected to be encountered during project development exhibit a pavement performance risk category of moderate on a scale that ranges between low, moderate, high, and very high.

- The on-site overburden deposits (man-placed fill and eolian) should generally be excavated with medium- to heavy-duty earthmoving or excavation equipment in good operating condition.

- The existing fill material within the building footprint should be removed to firm eolian soils, moisture conditioned as necessary, and recompacted as engineered fill as described herein. Engineered should extend laterally 5 feet or more beyond the foundation limit and cover the limits of the appurtenances that are adjacent to the building (e.g. exterior flatwork adjacent to the building foundation). Following recompaction of the undocumented fill materials, the building could be supported on spread footings with a slab-on-grade floor.

- There are risks associated with supporting pavements and exterior flatwork over undocumented fill material without soil modification. However, the costs associated with full removal and replacement of the undocumented fill materials are generally considered cost-prohibitive. Assuming the owner is willing to accept some risk, pavements and exterior flatwork may be placed on at least 24 inches of moisture conditioned and compacted engineered fill. Chemical stabilization of the upper 12 inches of the engineered fill may be needed under the pavements and exterior flatwork that is attached to the pavements, such as curb and gutter.

- Site soils generated from on-site excavation activities consisting of alluvial deposits that are free of deleterious materials, and do not contain particles larger than 3 inches in diameter, can generally be used as engineered fill during site grading.

- Based on our laboratory data and our experience with similar materials at adjacent sites, the sulfate content of the tested soils presents a low risk of sulfate attack to concrete. Recommendations regarding concrete (in contact with site soils) that can be used on this project are provided in Section 9.6.1.

- Based on our laboratory data and our experience with similar materials at adjacent sites, the subgrade soils at the site have a high potential for corrosivity to ferrous metals. Special consideration should be given to the use of heavy gauge, corrosion-protected, underground steel pipe or culverts, if any are planned. As an alternative, plastic pipe or reinforced concrete pipe could be considered. A corrosion specialist should be consulted for further recommendations.

9. RECOMMENDATIONS

The following sections present our geotechnical recommendations for the design and construction of the proposed structure and associated site improvements based on the plans reviewed and our understanding of the site. If the proposed construction differs significantly from that discussed in this report, Ninyo & Moore should also be contacted for additional recommendations.

9.1. Earthwork

The following sections provide our earthwork recommendations for this project. In general, the City of Greeley, Weld County, and/or project specific earthwork specifications are expected to apply, unless noted.

9.1.1. Excavations

Our evaluation of the excavation characteristics of the on-site materials is based on the results of our subsurface exploration, our site observations, and our experience with similar materials.

The on-site overburden deposits (fill and eolian deposits) should generally be excavated with medium- to heavy-duty earthmoving or excavation equipment in good operating condition.

Considering the in-fill nature of the site and its historic past-uses, there may be buried concrete remnants, areas of deeper fills, or other features present below the ground surface.

Equipment and procedures that do not cause significant disturbance to the excavation bottoms should be used. Excavators and backhoes with buckets having large claws to loosen the subgrade material should be avoided when excavating the bottom 6 to 12 inches of excavations as such equipment may disturb the excavation bases.

Groundwater was encountered at depths between 9 to 12 feet bgs at the time of drilling. Groundwater will likely be encountered during the construction of below grade utilities, manholes, or vaults extending more than 5 feet bgs.

The contractor should provide safely sloped excavations or an adequately constructed and braced shoring system, in compliance with Occupational Safety and Health Administration (OSHA) (OSHA, 2005) guidelines, for employees working in an excavation that may expose employees to the danger of moving ground. If material is stored or equipment is operated near an excavation, stronger shoring should be used to resist the extra pressure due to superimposed loads.

9.1.2. Temporary Excavations

Temporary excavations will be needed for this project to construct foundations and utilities. Based on the subsurface information obtained from our exploratory excavations and our experience with similar projects, we anticipate that the soil conditions and stability of the excavation sidewalls may vary with depth. Soils with higher fines content may stand vertically for a short time (less than 12 hours) with little sloughing. However, as the soil dries after excavation or as the excavations are exposed to rainfall, sloughing may occur. Soils with low cohesion (e.g., predominately sandy or gravelly material), may slough or cave during excavation, especially if wet or saturated.

The contractor should provide safely sloped excavations or an adequately constructed and braced shoring system, in compliance with Occupational Safety and Health Administration regulations (OSHA, 2005), for employees working in excavations that may expose them to the danger of moving ground. Reducing the inclination of the sidewalls of the excavations, where feasible, may increase the stability of the excavations. If construction or earth material is stored, or equipment is operated near an excavation, flatter slope geometry or shoring should be used during construction.

In our opinion, the alluvial deposits should generally be considered a Type C soil when applying OSHA regulations. For these soil conditions, OSHA recommends a temporary slope inclination of 1½H (Horizontal):1V (Vertical) or flatter for excavations 20 feet or less in depth. Steeper cut slopes may be utilized for excavations that are less than 4 feet deep depending on the strength, moisture content, and homogeneity of the soils as observed in the field.

Appropriate slope inclinations should be evaluated in the field by an OSHA-qualified "Competent Person" based on the conditions encountered.

9.1.3. Site and Remedial Grading

Prior to grading, the ground surface in proposed structure and improvement areas should be cleared of any surface obstructions, debris, topsoil, organics (including vegetation) and other deleterious material. Materials generated from the clearing operations should be removed from the site and disposed of at a legal dumpsite. Obstructions that extend below finish grade, if present, should be removed and the resulting holes filled with compacted soil or cement slurry, or in accordance with the recommendations of the geotechnical engineer. Topsoil present on-site should not be incorporated into engineered fill.

Soils generated from on-site excavation activities in the fill and alluvial deposits that are free of deleterious materials and do not contain particles larger than 3 inches in diameter can generally be used as engineered fill as evaluated by the geotechnical consultant.

The presence of the fill materials can pose a risk to the pavements and flatwork as post-construction settlements as the materials compress. While the fill materials appear moderately well compacted, it is recommended that remedial grading be performed on the site.

The proposed building may be supported on spread-footing foundations with a slab-on-grade floor. The spread footing foundations and slab-on-grade floor should bear on a prism of engineered fill (moisture-conditioned and recompacted site soils). The existing fill material within the building footprint should be removed to firm eolian soils, conditioned as necessary, and recompacted as engineered fill as described herein. Engineered fill should extend laterally 5 feet or more beyond the foundation limit and cover the limits of the appurtenances that are adjacent to the building (e.g. exterior flatwork adjacent to the building foundation).

Landscaping structures, pavements, and exterior flatwork (curb and gutter and sidewalks) may be placed on 2 or more feet of moisture-conditioned and compacted engineered fill. The costs associated with remediating undocumented fill material or swelling soils below landscaping structures, pavements and exterior flatwork (sidewalk, curb and gutter, drainage pans, etc.) are cost prohibitive. Therefore, remedial grading recommendations for these improvements are provided assuming the owner is willing to accept some risk of poor performance as a result of post-construction vertical movements from the undocumented fill materials.

Care should be taken to maintain the subgrade moisture content after fill placement but prior to construction of grade supported slabs and pavements. The site should be graded to prevent ponding of surface water on the prepared subgrades or in excavations. If the subgrade should become desiccated, saturated, frozen, or disturbed, the affected material should be removed or these materials should be scarified, moisture conditioned, and compacted prior to slab and pavement construction. Additional post-construction vertical movements could occur if the moisture content of the subsurface soils below these improvements changes to realize the estimated depth of wetting of 15 feet.

The exposed subgrade materials should be firm and unyielding prior to fill placement. The extent and depths of removal should be evaluated by a representative of Ninyo and Moore during the excavation work based on observation of the exposed soils. Additional recommendations specific to the site conditions encountered may be provided at the time of construction. The project budget should include additional cost associated with the removal and replacement of fill material. Subgrade materials that are disturbed during grading should be moisture conditioned and re-compacted according to the recommendations provided in this report.

9.1.4. Fill Placement and Compaction

Fine-grained, cohesive soils (CL) used as engineered fill should be moisture-conditioned to moisture contents between optimum and 3 percent over optimum moisture content. Granular soils (SC, SP, or import soils) used as engineered fill should be moisture-conditioned to moisture contents within 2 percent of optimum moisture content. Engineered fill should be compacted to a relative compaction of 95 percent or more as evaluated by ASTM D698.

The engineered fill should be compacted by appropriate mechanical methods. Lift thickness for fill will be dependent upon the type of compaction equipment utilized. Backfill should be placed in lifts not exceeding 8 inches in loose thickness in areas compacted by other-than hand operated machines. Backfill should be placed in lifts not exceeding 6 inches in loose thickness in areas compacted by hand operated machines.

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

Compaction areas should be kept separate, and no lift should be covered by another until relative compaction and moisture content within the recommended ranges are obtained.

9.1.5. Imported Soil

Imported soil for use as engineered fill should less than 50 percent passing the No. 200 sieve, a very low swell potential (approximately 1 percent or less when wetted against a surcharge pressure of 200 psf), and a low plasticity index (less than 20). Imported soil should not contain organic matter, clay lumps, bedrock (claystone, sandstone, etc.) fragments, debris, other deleterious matter, or rocks or hard chunks larger than approximately 3 inches' nominal diameter.

Imported soil for use as engineered fill should exhibit low corrosion potential. Imported soil placed in contact with ferrous materials should have a saturated soil resistivity of 2,000 ohm-cm or more and a chloride content of 25 parts per million or less. Soils in contact with concrete should exhibit a soluble sulfate content less than 0.1 percent.

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.

9.1.6. Controlled Low Strength Material

Use of Controlled Low Strength Material (CLSM) should be considered in lieu of compacted fill for areas with low tolerances for surface settlements, for excavations that extend below the groundwater table and in areas with difficult access for compaction equipment. CLSM consists of a fluid, workable mixture of aggregate, Portland cement, and water. CLSM should be placed in lifts of 5 feet or less with a 24-hour or more curing period between each lift.

The use of CLSM has several advantages:

- A narrower excavation can be used where shoring is present, thereby minimizing the quantity of soil to be excavated and possibly reducing disturbance to the near-by traffic;
- Compaction requirements do not apply;
- There is less risk of damage to improvements, since little compaction is needed to place CLSM;
- CLSM can be batched to flow into irregularities in excavation bottoms and walls; and
- The number of workers needed inside the trench excavation is reduced.

The CLSM mix design should be submitted for review prior to placement. The 28-day strength of the material should be no less than 50 pounds per square inch (psi) and no more than 150 psi. CLSM should be observed and tested by the geotechnical consultant.

9.1.7. Utility Installation

The contractor should take care to achieve and maintain adequate compaction of the backfill soils around manholes, valve risers and other vertical pipeline elements where settlements commonly are observed. Use of CLSM should be considered in lieu of compacted soil backfill for areas with low tolerances for surface settlements. This would also reduce the permeability of the utility trenches.

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

Development of site grading plans should consider the subsurface transfer of water in utility trenches and the pipe bedding. Sandy pipe bedding materials can function as efficient conduits for re-distribution of 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 loss of fine-grained backfill soils into the bedding material with resultant surface settlement.

9.2. Spread-Footing Foundations

Perimeter footings should extend to 36 inches or more below the lowest exterior finished grade (for frost protection), and bear on a zone of adequately placed and compacted engineered fill as described in Section 9.1.3 of this report. Continuous wall footings should have a width of 18 inches or more and column footings should have a width of 24 inches or more. Footings should be reinforced in accordance with the recommendations of the structural engineer.

After entirety of the existing fill material is removed and replaced as engineered fill beneath the building footprint, the footings may be designed using an allowable bearing pressure of 3,000 psf for static conditions. The bearing capacity may be increased by one-third when considering loads of short duration such as wind or seismic forces. The foundations should preferably be proportioned such that the resultant force from design loads, falls within the kern (i.e., middle one-third of the footing base).

Uplift resistance can be developed from the weight of the footings, the effective weight of any overlying soil, and the weight of the supported structure itself. The effective unit weight of the soil can be assumed to be 120 pcf above the groundwater level and 60 pcf below. Soil uplift resistance may be calculated as the weight of the soil prism defined by a diagonal line extending from the perimeter of the foundation to the ground surface at an angle of 20 degrees from the vertical. Under large moment and/or shear loading, the effective size of the uplift soil prism may be reduced. An appropriate safety factor should be applied.

The base of foundation excavations should be free of ice, snow, water, frost, and loose soil prior to placing concrete. Concrete should be placed soon after subgrade compaction to reduce bearing soil disturbance. Should the soils at bearing level become excessively dry, disturbed, or saturated, the affected soil should be moisture conditioned and re-compacted. Ninyo & Moore should be retained to observe, test, and evaluate the soil foundation bearing materials. An “open hole” site visit letter will not be issued unless Ninyo & Moore representatives are retained to be on-site during footing excavation and subsequent engineered fill placement.

Based on the results of our subsurface exploration and provided our recommendations are followed, we estimate total and differential settlements of 1 inch and ½ inch, respectively are possible at the site.

9.3. Slab on Grade Floor Slabs

Slab-on-grade floors could be used in conjunction with spread footing foundations for the proposed construction. Slab-on-grade floors should be placed on an improved subgrade as recommended in Section 9.1.3. The design of the floor slabs (including jointing and reinforcement) is the responsibility of the structural engineer. Joints should be constructed at intervals designed by the Structural Engineer to help reduce the random cracking of the slab. Recommendations based on structural considerations for slab thickness, jointing, and steel reinforcement should be developed by the Structural Engineer in accordance with the American Concrete Institute (ACI) recommendations. Soils underlying the slabs should be moisture conditioned and compacted in accordance with the recommendations presented in Section 9.1.3 of this report.

For slab design, a design modulus of subgrade reaction (K) of 150 pounds per square inch per inch of deflection (pci) may be used for the subgrade soils in evaluating such deflections. This value is based on a unit square foot area and can be adjusted for large slabs. Adjusted values of the modulus of subgrade reaction, K_v, can be obtained from the following equation for slabs of various widths:

$$K_v = K[(B+1)/2B]^2 \text{ (pci)}$$

B in the above equation represents the width of the slab in feet between line loads/point loads.

The floor slabs should be constructed so that it "floats" independent of the foundations. Floor slabs should be separated from bearing walls and columns with expansion joints, which allow unrestrained vertical movement. Joints should be observed periodically, particularly during the first several years after construction. Slab movement can cause previously free-slipping joints to bind. Measures should be taken so that slab isolation is maintained in order to reduce the likelihood of damage to walls and other interior improvements. Utility lines entering the slab should be provided with positive bond breaks that allow 2 or more inches of differential movement.

The floor slab subgrade should be maintained within the moisture recommendations prior to floor slab placement. The placement of a granular cap over the exposed subgrade could be performed to limit moisture loss. If the subgrade soils dry out or become saturated, they should be moisture conditioned, as needed, and recompacted prior to slab placement.

Non-load-bearing interior partitions resting on floor slabs should be provided with slip joints so that if the slabs move, the movement cannot be transmitted to the upper structure, including wallboards and door frames. A slip joint that allows 2 or more inches of vertical movement is recommended for placement at the bottoms of the interior partitions. If slip joints are placed at the tops of walls, in the event that the floor slabs move, it is expected that the wall will show signs of distress, especially where the floors meet the exterior wall. Interior plumbing lines that penetrate interior partition walls, where the slip joints are placed at the top of the walls, should be provided with flexible connections that can handle 2 or more inches of vertical movement.

The need for a moisture retarding and/or vapor retarding system should be considered by the Structural Engineer or Architect, based on the moisture sensitivity of the anticipated flooring. The placement of a vapor retarder is recommended in areas where moisture-sensitive floor coverings are anticipated.

9.4. Pavement Sections

We understand the proposed pavements for this project will consist of privately maintained pavements. Pavement section alternatives are included herein for standard duty pavement (automobile parking areas and drive lanes), and heavy-duty pavement (truck drive lanes and fire lanes). Specific traffic loadings for this project were not provided at the time of report preparation and are based on our experience with similar projects. The pavement sections recommended herein were developed in general accordance with the guidelines and procedures of the American Association of State Highway and Transportation Officials (AASHTO), (AASHTO, 1993), Colorado Department of Transportation (CDOT), and the City of Greeley Design Criteria and Construction Specifications (2015).

9.4.1. Pavement Subgrade Characterization

The current subgrade soils encountered in our borings typically consisted of silty or clayey sand to sandy lean clay in the upper 5 feet, which generally classify as A-4 to A-7-6 soils in accordance with the AASHTO classification system. As a result, we have assumed a design R-value of 10 for pavement subgrade soils.

9.4.2. Traffic Loading

As stated previously, specific traffic loadings for this project were not available at the time of report preparation. Based on our experience with similar developments and in general accordance with the City of Greeley Design Criteria and Construction Specifications (2015) an equivalent 18-kip single axle load value (ESAL) of 36,500 was assumed for flexible and rigid pavements. If design traffic loadings differ significantly from this assumed value, we should be notified to re-evaluate the pavement recommendations presented herein.

9.4.3. Pavement Design Parameters

The pavement design for this site was based on guidelines and procedures of the American Association of State Highway and Transportation Officials (AASHTO), (AASHTO, 1993), CDOT, and the City of Greeley. The below pavement sections are appropriate for occasional vehicle loads of 80-kips in accordance with City of Greeley.

The design of flexible pavements was based on the following input parameters:

Initial Serviceability:	4.5
Terminal Serviceability:	2.3
Reliability	75%
Overall Standard Deviation:	0.45
Resilient Modulus:	3,563 psi (R-Value of 10)

The design of rigid pavements was based on the following input parameters:

Initial Serviceability:	4.5
Terminal Serviceability:	2.3
Reliability	75%
28-Day Mean PCC Modulus Rupture:	650 psi
28-Day Mean Modulus of Elasticity:	3.6×10^6 psi
Mean Effective k value:	125 psi/in
Overall Standard Deviation:	0.35
Load Transfer Coefficient:	4.2
Overall Drainage Coefficient:	1.0

9.4.4. Pavement Thickness

Based on the above-mentioned guidelines, procedures, and input parameters, Table 5 provides our recommended pavement section thicknesses supported on prepared pavement subgrade as described in Section 9.1.3.

Table 5 – Recommended Pavement Thickness

Traffic Type	Composite AC / ABC (inches)	Composite PCCP / ABC (inches)
Local Residential	4.0 / 6.0 [1]	6.0 / 4.0 [1]

Notes: AC = Asphalt Concrete, ABC = Aggregate Base Course, PCCP = Portland Cement Concrete Pavement
[1] City of Greeley Minimum Pavement Section for Local Roads

We recommend PCCP be utilized in entrance and exit sections, dumpster pads, or other areas where extensive wheel maneuvering is expected. The dumpster pad should be large enough to support the wheels of the truck, which will bear the load of the dumpster.

Where practical, we recommend “early-entry” cutting of crack-control joints in PCCP. Cutting of PCCP in its ‘green” state typically reduces the potential for micro-cracking of the pavements prior to the crack control joints being formed, compared to cutting the joints after the concrete has fully set. Micro-cracking of pavements may lead to crack formation in locations other than the sawed joints, and/or reduction of fatigue life of the pavement.

Ninyo & Moore has observed dishing in some AC parking lots. Dishing is observed in frequently-used parking stalls (such as near the front of buildings), and occurs under the wheel footprint in these stalls. The use of higher-grade AC, or surfacing these areas with PCCP, could be considered. The dishing is exacerbated by factors such as irrigated islands or planter areas, and sheet surface drainage to the front of structures.

9.4.5. Pavement Subgrade Preparation

For both the PCCP and AC pavement sections recommended above, we recommend the underlying subgrade soils be prepared as described in Section 9.1.3 of this report.

The contractor should be prepared either to dry the subgrade materials or moisten them, as needed, prior to compaction. Some site soils may pump or deflect during compaction if moisture levels are not carefully monitored. The contractor should be prepared to process and compact such soils to establish a stable platform for paving, including the use of chemical stabilization or geotextiles, where needed.

As stated in Section 9.1.3, if the subgrade should become desiccated, saturated, frozen, or disturbed, the affected material should be removed or these materials should be scarified, moisture conditioned, and recompacted prior to pavement construction.

Immediately prior to paving, the subgrade should be proof rolled with a heavily loaded, pneumatic tired vehicle and checked for moisture. Areas that show excessive deflection during proof rolling should be excavated and replaced and/or stabilized. Areas allowed to pond prior to paving may need to be re-worked prior to proof rolling.

9.4.6. Pavement Materials

The AC pavement shall consist of a bituminous plant mix composed of a mixture of high-quality aggregate and bituminous material, which meets the requirements of a job-mix formula established by a qualified engineer. The asphalt material used should be based on a SuperPave Gyratory Design Revolution of 50. Lower lifts should be constructed using an asphalt mix Grading S and asphalt cement binder grade PG 58-28. The top lift should be constructed using an asphalt mix Grading SX and asphalt cement binder grade PG 64-22. Pavement layer thickness should be between 2 and 3 inches for the lower lifts and 2 to 2.5 inches for the top lift. The geotechnical engineer should be retained to review the proposed pavement mix designs, grading, and lift thicknesses prior to construction.

PCCP should consist of a plant mix composed of a mixture of aggregate, Portland cement and appropriate admixtures meeting the requirements of the Town of Parker. Concrete should have a modulus of rupture of third point loading of 650 psi or more. The concrete should be air-entrained with approximately 6 percent air and should have a cement content of six or more sacks per cubic yard. Allowable slump should be approximately 4 inches.

Thickened edges should be used along outside edges of PCCP. The edge thickness should be 2 inches or more than the recommended PCCP thickness and taper to the recommended PCCP thickness 36 inches inward from the edge. Integral curbs may be used in lieu of thickened edges.

PCCP should have longitudinal and transverse joints that meet the applicable requirements of the City of Greeley.

9.4.7. Pavement Maintenance

The collection and diversion of surface drainage away from paved areas is vital to satisfactory performance of the pavements. The subsurface and surface drainage systems should be carefully designed to facilitate removal of the water from paved areas and subgrade soils. Allowing surface waters to pond on pavements will cause premature pavement deterioration. Where topography, site constraints or other factors limit or preclude adequate surface drainage, pavements should be provided with edge drains to reduce loss of subgrade support. The long-term performance of the pavement also can be improved greatly by backfilling and compaction behind curbs, gutters, and sidewalks so that ponding is not permitted and water infiltration is reduced.

Landscape irrigation in planters adjacent to pavements and in “island” planters within paved areas should be carefully monitored or differential heave and/or rutting of the nearby pavements will result. Drip irrigation systems are recommended for such planters to reduce over-spray and water infiltration beyond the planters. We recommend edge drains where the profile/slopes are less than 1 percent.

The standard care of practice in pavement design describes the recommended flexible pavement section as a “20-year” design pavement; however, many pavements will not remain in satisfactory condition without routine, preventive maintenance and rehabilitation procedures performed during the life of the pavement. Preventive pavement treatments are surface rehabilitation and operations applied to improve or extend the functional life of a pavement. These treatments preserve, rather than improve, the structural capacity of the pavement structure. In the event the existing pavement is not structurally sound, the preventive maintenance will have no long-lasting effect. Therefore, a routine maintenance program to seal joints and cracks, and repair distressed areas is recommended.

The cost associated with the sealing of joints at the time of initial construction should be discussed among the project ownership, design, and construction team members prior to construction. The cost associated with the maintenance of these joints should be included in the pavement maintenance program. These joints, regardless of the performance of a sealing program at the time of initial construction, will widen and will need to be resealed periodically.

9.5. Concrete Flatwork

Exterior walkways and flatwork should be 4 or more inches thick. The slab edges should be deepened by two or more inches where exterior slabs-on-grade are placed adjacent to landscaping areas and taper to the recommended thickness 12 inches inward from the edge. Exterior flatwork should be constructed on an improved subgrade per Section 9.1.3 of this report.

Ground-supported flatwork, such as walkways, will be subject to soil-related movements resulting from heave/settlement, frost, etc. 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 where such elements abut the main structure to allow for differential movement at these locations. Positive drainage should be established and maintained adjacent to flatwork. Water should not be allowed to pond on flatwork.

To reduce the potential manifestation of distress to exterior concrete flatwork due to movement of the underlying soil, we recommend that such flatwork be installed with crack-control joints at appropriate spacing as designed by the Structural Engineer.

In no case should exterior flatwork extend under any portion of the buildings where there is less than 3 inches of clearance between the flatwork and any element of the building. Exterior flatwork in contact with brick, rock facades, or any other element of the building can cause damage to the structure if the flatwork experiences movements.

9.6. Corrosion Considerations

The corrosion potential of on-site soils to concrete and buried metal was evaluated in the laboratory using selected samples obtained from the exploratory borings. Laboratory testing was performed to assess the effects of sulfate on concrete and the effects of soil resistivity on buried metal. Results of these tests are presented in Appendix B. Recommendations regarding concrete to be utilized in construction of proposed improvements and for buried metal pipes are provided in the following sections.

9.6.1. Concrete

The test for water-soluble sulfate content of the soils was performed using CDOT Test Method CP-L 2104. The laboratory test results are presented in Appendix B. The percentage of water-soluble sulfates in water measured was approximately 12 parts per million for selected samples. Based on Table 601-2 of the CDOT 2011 Standard Specifications for Road and Bridge Construction, the on-site soils represent a Class 0 severity of sulfate exposure to concrete on a scale that ranges between Class 0 and Class 3. Therefore, we recommend that the concrete used for this project should meet one of the below outlined requirements.

- ASTM C 150 Type I, II or V
- ASTM C 595 Type IP, IP(MS) or IP(HS)
- ASTM C 1157 Type GU, MS or HS
- ASTM C 150 Type III cement if it is allowed, as in Class E concrete

The Structural Engineer should ultimately select the concrete design strength based on the project specific loading conditions. However, higher strength concrete may be selected for increased durability, resistance to slab curling and shrinkage cracking. We recommend the use of concrete with a design 28-day compressive strength of 4,000 psi or more, for concrete slabs at this site. Concrete exposed to the elements should be air-entrained.

9.6.2. Buried Metal Pipes

The corrosion potential of the on-site materials was analyzed to evaluate its potential effects on buried metals. Corrosion potential was evaluated using the results of laboratory testing of samples obtained during the subsurface evaluation that were considered representative of soils at the subject site.

Resistivity was measured to be approximately 1,380 ohm-cm for our selected sample. The results of the laboratory testing indicate the on-site materials could be severely corrosive to ferrous metals based on the criteria in Table 6. Therefore, special consideration should be given to the use of heavy gauge, corrosion protected, underground steel pipe or culverts, if any are planned. As an alternative, plastic pipe or reinforced concrete pipe could be considered. A corrosion specialist should be consulted for further recommendations. The laboratory test results are presented in Appendix B.

Table 6 – Corrosion Potential to Steel	
Resistivity (Ohm-cm)	**Corrosivity Potential to Steel**
0 - 500	Very Severe
500 – 2,000	Severe
2,000 – 10,000	Moderate
10,000 – 30,000	Mild
>30,000	Low

9.7. Scaling

Climatic conditions in the project area including relatively low humidity, large temperature changes and repeated freeze-thaw cycles, may cause surficial scaling and spalling of exterior concrete. Occurrence of surficial scaling and spalling can be aggravated by poor workmanship during construction, such as “over-finishing” concrete surfaces and the use of de-icing salts on exterior concrete flatwork, particularly during the first winter after construction. The use of de-icing salts on nearby roadways, which can be transferred by vehicle traffic onto newly placed concrete, can be sufficient to induce scaling.

The measures below can be beneficial for reducing the concrete scaling. However, because of the other factors involved, including workmanship, surface damage to concrete can develop even though the measures provided below were followed. The mix design criteria should be coordinated with other project requirements including the criteria for soluble sulfate resistance presented in Section 9.6.1.

- Curing concrete in accordance with applicable codes and guidelines.
- Maintaining a water/cement ratio of 0.45 by weight for exterior concrete mixes.
- Including Type F fly ash in exterior concrete mixes as 20 percent of the cementitious material.
- Specifying a 28-day, compressive strength of 4,500 or more psi for exterior concrete that may be exposed to de-icing salts.
- Avoiding the use of de-icing salts through the first winter after construction.
- Avoiding the use of dark colored concrete that may experience additional freeze-thaw cycles and specialty concrete finishes other than standard broom finish.

9.8. Frost Heave

Site soils are susceptible to frost heave if allowed to become saturated and exposed to freezing temperatures and repeated freeze/thaw cycling. The formation of ice in the underlying soils can result in two 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.

9.9. Construction in Cold or Wet Weather

During construction, the site should be graded such that surface water can drain readily away from the building areas. Given the soil conditions, 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 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 the placement of fill or other construction material. Frozen soil should not be used as engineered 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 passing between excavation and construction should be minimized. Blankets, straw, soil cover, or heating may be used to discourage the soil from freezing.

9.10. Site Drainage

Infiltration of water into subsurface soils can lead to soil movement and associated distress, and chemically and physically related deterioration of concrete and masonry 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 building and canopy foundations. Positive drainage may be established by providing a surface gradient for paved areas of 2 to 5 percent or more for a distance of 10 feet or more away from structures. Where concrete flatwork is placed adjacent to structures and other considerations are required by law, such as ADA requirements, slopes of 1 percent or more are considered acceptable. For unpaved areas, positive drainage may be established by a slope of 5 to 10 percent for 10 feet or more away from structures, where possible.

- Adequate surface drainage should be provided to channel surface water away from on-site structures and off paved surfaces to a suitable outlet such as a storm drain. 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, away from structures, pavements, and flatwork. If tightlining of the downspouts is not practicable, they should discharge 5 feet or more away from structures and onto surfaces that slope away from the structure. Downspouts should not be allowed to discharge onto the ground surface adjacent to building foundations or on exterior walkways.

- The possibility of moisture infiltration beneath a structure, in the event of plumbing leaks, should be considered in the design and construction of underground water and sewer conduits. Permitting increases in moisture to the building supporting soils may result in a decrease in bearing capacity and an increase in settlement, heave, and/or differential movement. Incorporating a perimeter drainage system around the building foundations that will aid in reduction of the moisture infiltration of subsurface soils may be considered. Due to the proposed construction (i.e. no below grade construction) and anticipated utilities within the structures, not placing the perimeter drainage would be considered a low risk to the owner.

- Irrigated landscaping, consisting of sprinklers to water plants with high demands for water, should not be placed within 5 feet of the building(s). Drip irrigation is considered acceptable between 5 and 10 feet of the building exterior. If drip irrigated plants are needed within 5 feet of the building exterior, drip irrigation system should be provided with sensors that limit over irrigation.

- 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 5 feet or more from the building and constructed with closed bottoms or with drainage systems to drain excess irrigation away from the building.

9.11. Construction Observation and Testing

A qualified geotechnical consultant should perform appropriate observation and testing services during grading and construction operations. These services should include observation of any soft, loose, or otherwise unsuitable soils, evaluation of subgrade conditions where soil removals are performed, evaluation of the suitability of proposed borrow materials for use as fill, evaluation of the stability of open temporary excavations, evaluation of the results of any subgrade stabilization or dewatering activities, and performance of observation and testing services during placement and compaction of engineered 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. If another geotechnical consultant is selected to perform observation and testing services for the project, we request that the selected consultant provide a letter to the owner, with a copy to Ninyo & Moore, indicating that they fully understand our recommendations and they are in full agreement with the recommendations contained in this report. Qualified subcontractors utilizing appropriate techniques and construction materials should perform construction of the proposed improvements.

9.12. Plan Review

The recommendations presented in this report are based on conceptual plans for the proposed project 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.

9.13. Pre-Construction Meeting

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

10. 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.

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.

11. REFERENCES

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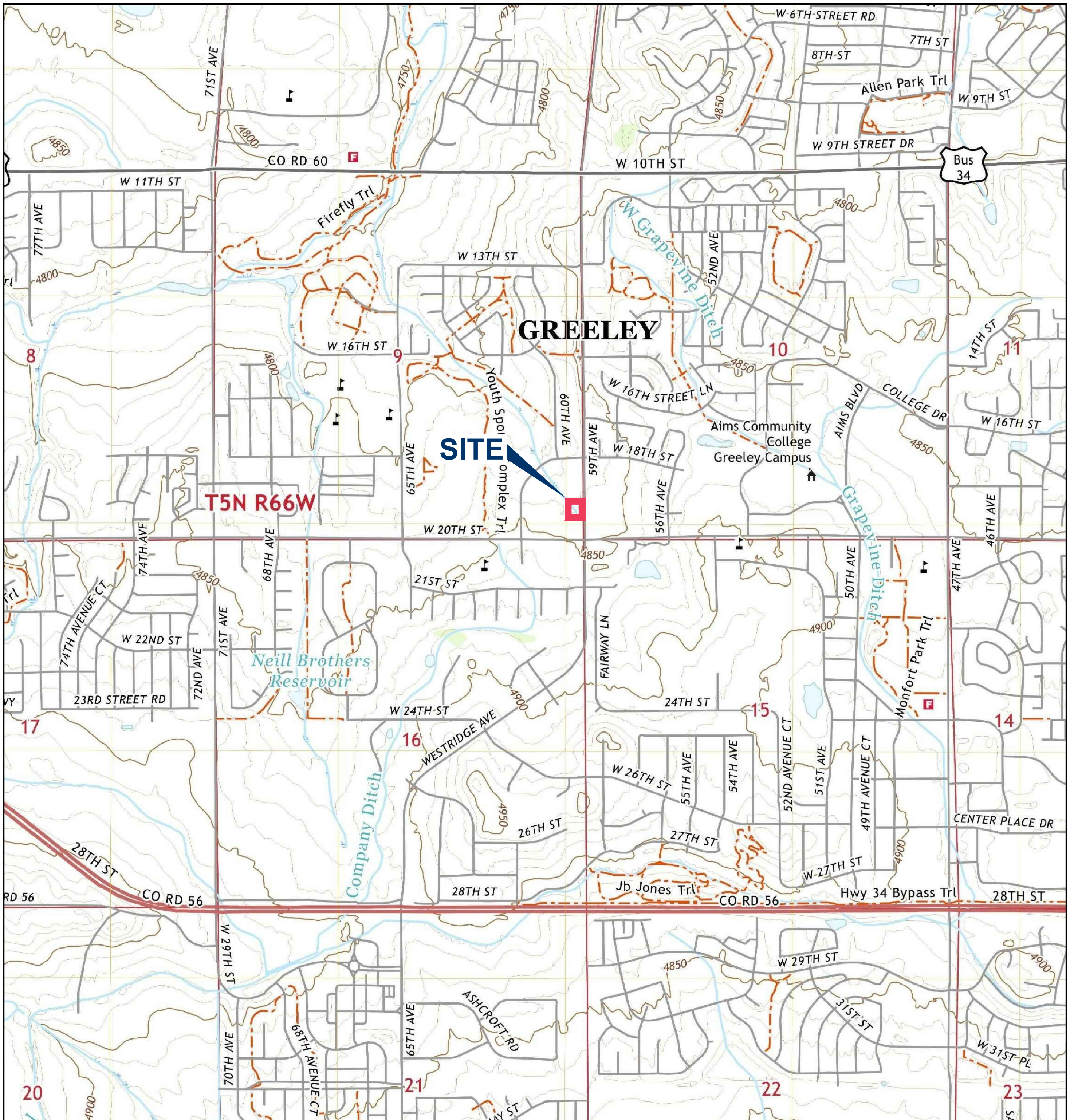
FIGURES



NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: USGS, 2021.

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FIGURE 1

SITE LOCATION

SVA GREELEY DENTAL BUILDING
1911 59TH AVENUE
GREELEY, COLORADO

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

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LEGEND

B-5 BORING

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: GOOGLE EARTH 2022.

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FIGURE 2

BORING LOCATIONS

SVA GREELEY DENTAL BUILDING
1911 59TH AVENUE
GREELEY, COLORADO

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Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

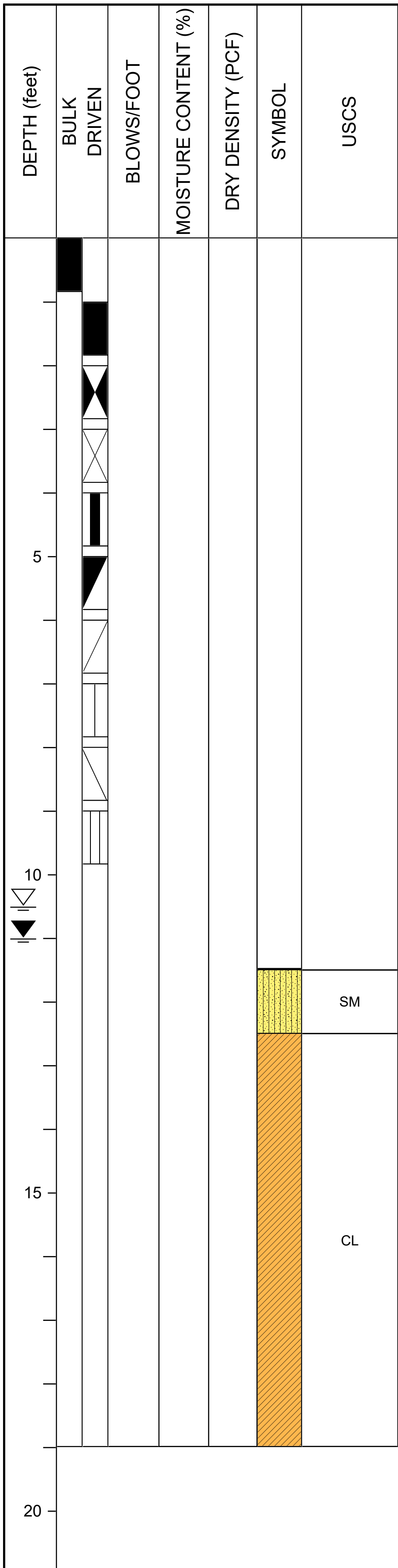
Field Procedure for the Collection of Disturbed Samples
Disturbed soil samples were obtained in the field using the following method.

Bulk Samples
Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged 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 the following method.

The California Drive Sampler
The sampler, with an external diameter of 2.4 inches, was lined with four 4-inch long, thin brass rings with inside diameters of approximately 1.9 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 per foot of 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 liners, sealed, and transported to the laboratory for testing.

BORING LOG EXPLANATION SHEET



Bulk sample.

Modified Split-Barrel Drive Sampler

2-inch inner diameter split-barrel drive sampler.

No recovery with modified split-barrel drive sampler, or 2-inch inner diameter split-barrel drive sampler.

Sample retained by others.

Standard Penetration Test (SPT).

No recovery with an SPT.

Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.

No recovery with Shelby tube sampler.

Continuous Push Sample.

Groundwater encountered during drilling.

Groundwater measured after drilling.

MAJOR MATERIAL TYPE (SOIL): Solid line denotes unit change.

Dashed line denotes material change

Attitudes: Strike/Dip
b: Bedding
c: Contact
j: Joint
f: Fracture
F: Fault
cs: Clay Seam
s: Shear
bss: Basal Slide Surface
sf: Shear Fracture
sz: Shear Zone
sbs: Shear Bedding Surface

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

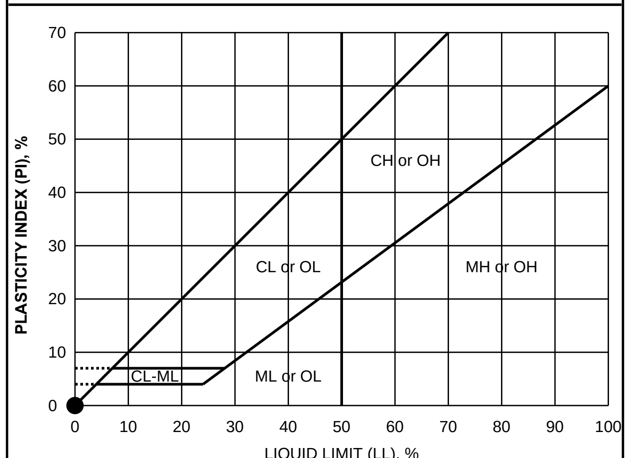
SOIL CLASSIFICATION CHART PER ASTM D 2488

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 retained on 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 CLAY
	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)	SPLIT BARREL (blows/foot)	SPT (blows/foot)	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		AUTOMATIC TRIP HAMMER	
	SPT (blows/foot)	SPLIT BARREL (blows/foot)	SPT (blows/foot)	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

DATE DRILLED	09/23/2024	BORING NO.	B-1		
GROUND ELEVATION	~4,838.5'	SHEET	1	OF	1
RIG TYPE	CME-75	TOOLING	4" Solid Stem Auger	DRILLING FIRM	Dakota
DRIVE WEIGHT	140 lbs.	TYPE	Spooling Cathead	DROP	30"
SAMPLED BY	MEO	LOGGED BY	MEO	REVIEWED BY	BFG

DEPTH (feet)	BULK	DRIVEN	BLOWS/FOOT	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SYMBOL	USCS	DESCRIPTION
0								**TOPSOIL**: Approximately 3 inches thick.
								FILL: Brown, moist, silty SAND.
1.5			17	5.7	108.6			
4.5			8	15.3	117.0			Dark gray, moist, sandy lean CLAY.
7							SC	**EOLIAN DEPOSIT**: Grayish brown, moist, loose, clayey SAND.
9.5			9	22.2	106.6			
11								@11': Groundwater encountered during drilling.
14.5			9	20.9	109.3			Brown; moist to wet.
17								**FOX HILL FORMATION**: Brown, moist, moderately hard, sandy CLAYSTONE.
19.5			50/8"	24.8	103.4			

Total Depth: 19.7 feet.
Groundwater was encountered during drilling at approximately 11 feet.
Backfilled with on-site soil after drilling.

Notes:
Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report.

Ground elevations were measured using a Trimble Model DA2-BT survey unit with global navigation satellite system (GNSS) output of NAD83 (2011) and referencing geoid model GEOID18.

DATE DRILLED	09/23/2024	BORING NO.	B-2		
GROUND ELEVATION	~4,838.4'	SHEET	1	OF	1
RIG TYPE	CME-75	TOOLING	4" Solid Stem Auger	DRILLING FIRM	Dakota
DRIVE WEIGHT	140 lbs.	TYPE	Spooling Cathead	DROP	30"
SAMPLED BY	MEO	LOGGED BY	MEO	REVIEWED BY	BFG

DEPTH (feet)	BULK	DRIVEN	BLOWS/FOOT	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SYMBOL	USCS	DESCRIPTION
0								**TOPSOIL**: Approximately 3 inches thick.
								FILL: Brown, dry to moist, clayey SAND.
			31	7.5	112.8			
			11	15.8	108.9			**EOLIAN DEPOSIT**: Brown, moist, loose, clayey fine to medium SAND.
5								
10			11				SC	@9': Groundwater encountered during drilling.
15			12	14.9	110.5			Wet; loose.
20			32	20.7	108.0			Medium Dense.
25								
30								
35								

Total Depth: 20 feet.
Groundwater was encountered during drilling at approximately 9 feet.
Backfilled with on-site soil after drilling.

Notes:
Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report.

Ground elevations were measured using a Trimble Model DA2-BT survey unit with global navigation satellite system (GNSS) output of NAD83 (2011) and referencing geoid model GEOID18.

DATE DRILLED	09/23/2024	BORING NO.	B-3				
GROUND ELEVATION	~4,840.7'	SHEET	1	OF	1		
RIG TYPE	CME-75	TOOLING	4" Solid Stem Auger	DRILLING FIRM	Dakota		
DRIVE WEIGHT	140 lbs.	TYPE	Spooling Cathead	DROP	30"		
SAMPLED BY	MEO	LOGGED BY	MEO	REVIEWED BY	BFG		

DEPTH (feet)	BULK	DRIVEN	BLOWS/FOOT	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SYMBOL	USCS	DESCRIPTION
								TOPSOIL: Approximately 3 inches thick.
			23	8.2	113.0			**FILL**: Brown, dry to moist, sandy lean CLAY.
5			15	14.6	114.0		SM	**EOLIAN DEPOSIT**: Gray, moist, loose, silty fine to medium SAND.
10			12	19.9	110.5		CL	Brown, moist, stiff, lean CLAY with sand.
								@12': Groundwater encountered during drilling.
							SC	Reddish brown, moist to wet, loose, clayey SAND with gravel.
15			10	18.1	113.0		CL	Grayish brown, moist, stiff, lean CLAY with sand.
			50/9"	21.5	106.1			**FOX HILL FORMATION**: Brown, moist, moderately hard, sandy CLAYSTONE.
20								
25								
30								
35								

Total Depth: 19.8 feet.
Groundwater was encountered during drilling at approximately 12 feet.
Backfilled with on-site soil after drilling.

Notes:
Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report.

Ground elevations were measured using a Trimble Model DA2-BT survey unit with global navigation satellite system (GNSS) output of NAD83 (2011) and referencing geoid model GEOID18.

DATE DRILLED	09/23/2024	BORING NO. B-4
GROUND ELEVATION	~4,836.9'	SHEET 1 OF 1
RIG TYPE	CME-75	TOOLING 4" Solid Stem Auger; DRILLING FIRM Dakota
DRIVE WEIGHT	140 lbs.	TYPE Spooling Cathead; DROP 30"
SAMPLED BY	MEO	LOGGED BY MEO; REVIEWED BY BFG

DEPTH (feet)	BULK	DRIVEN	BLOWS/FOOT	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SYMBOL	USCS	DESCRIPTION
0								**TOPSOIL**: Approximately 3 inches thick.
								FILL: Brown, dry to moist, sandy lean CLAY.
1			28	9.8	100.9			
4			18	11.7	102.1			Moist.
6							SC	**EOLIAN DEPOSIT**: Brown, moist to wet, very loose, clayey SAND.
9			8	19.6	109.2			

Total Depth: 10 feet.
Groundwater was not encountered during drilling.
Backfilled with on-site soil after 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.

Ground elevations were measured using a Trimble Model DA2-BT survey unit with global navigation satellite system (GNSS) output of NAD83 (2011) and referencing geoid model GEOID18.

DATE DRILLED	09/23/2024	BORING NO.	B-5
GROUND ELEVATION	~4,840.9'	SHEET	1 OF 1
RIG TYPE	CME-75	TOOLING	4" Solid Stem Auger
DRILLING FIRM	Dakota	DRIVE WEIGHT	140 lbs.
TYPE	Spooling Cathead	DROP	30"
SAMPLED BY	MEO	LOGGED BY	MEO
REVIEWED BY	BFG		

DEPTH (feet)	BULK	DRIVEN	BLOWS/FOOT	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SYMBOL	USCS	DESCRIPTION
0	X							**TOPSOIL**: Approximately 3 inches thick.
	X							**FILL**: Brown, dry to moist, sandy lean CLAY.
1.5	X	X	32	6.1	104.2			
4.5	X	X	23	13.4	104.6			Moist; trace gravel.
7							SC-SM	**EOLIAN DEPOSIT**: Brownish gray, moist, loose, silty clayey fine to medium SAND.
9.5		X	9	18.4	109.4			
10								Total Depth: 10 feet. Groundwater was not encountered during drilling. Backfilled with on-site soil after 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.

Ground elevations were measured using a Trimble Model DA2-BT survey unit with global navigation satellite system (GNSS) output of NAD83 (2011) and referencing geoid model GEOID18.

APPENDIX B

Laboratory Testing

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 2488-00. Soil classifications are indicated on the logs of the exploratory borings in Appendix B.

In-Place Moisture and Density Tests
The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937-04. The test results are presented on the logs of the exploratory borings in Appendix A.

Atterberg Limits
Tests were performed on selected representative fine-grained 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 the soil classification in accordance with the Unified Soil Classification System. The test results and classifications are shown on Figure B-1.

No. 200 Sieve Analysis
An evaluation of the percentage of particles finer than the No. 200 sieve in selected soil samples was performed in general accordance with ASTM D 1140. The results of the tests are presented on Figure B-2.

Gradation Analysis
Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D 6913. The grain-size distribution curves are shown on Figures B-3 and B-5. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

Swell/Consolidation Tests
The consolidation and/or swell potential of selected materials were evaluated in general accordance with ASTM D 4546. Specimens were loaded with a specified surcharge before inundation with water. Readings of volumetric consolidation/swell were recorded until completion of primary consolidation/swell. After the completion of primary swell, surcharge loads were increased incrementally to evaluate swell pressure. The results of the consolidation/swell tests are presented on Figures B-6 through B-10.

Soil Corrosivity Tests
A soil pH test was performed on a representative sample in general accordance with ASTM Test Method D 4972. A soil minimum resistivity test was performed on a representative sample in general accordance with AASHTO T288. The sulfate content of a selected sample was evaluated in general accordance with CDOT Test Method CP-L 2103. The chloride content of a selected sample was evaluated in general accordance with CDOT Test Method CP-L 2104. The test results are presented on Figure B-11.

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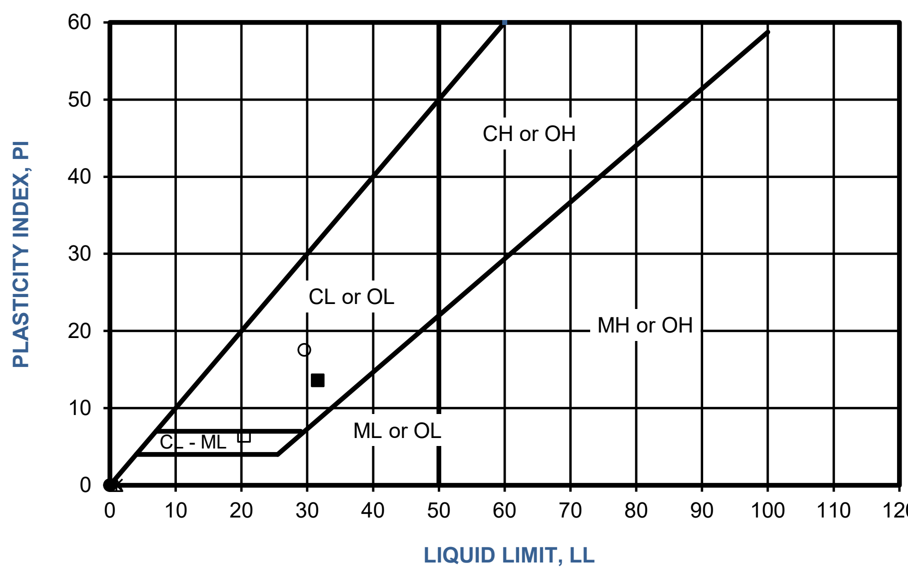
SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	EQUIVALENT USCS
●	B-1	1.0-2.0	NP	NP	NP	ML	SM
■	B-1	19.0-20.0	32	18	14	CL	CL
◆	B-3	4.0-5.0	NP	NP	NP	ML	ML
○	B-4	4.0-5.0	30	12	18	CL	CL
□	B-5	9.0-10.0	20	14	6	CL-ML	SC-SM

NP - INDICATES NON-PLASTIC



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

SAMPLE LOCATION	SAMPLE DEPTH (ft)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	EQUIVALENT USCS
B-1	1.0-2.0	Brown Silty SAND	100	16	SM
B-1	19.0-20.0	Brown Sandy CLAYSTONE; FOX HILL FORMATION	95	68	CL
B-4	4.0-5.0	Brown Sandy Lean CLAY; Trace Gravel	99	57	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

FIGURE B-2

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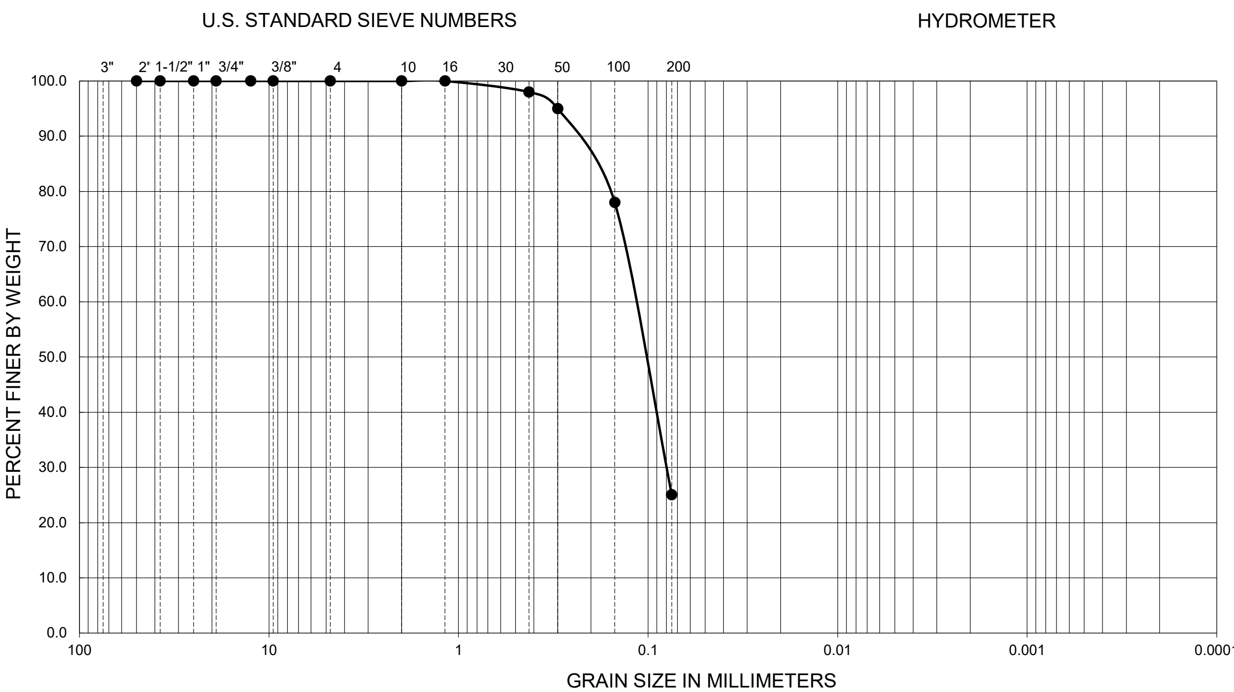
NO. 200 SIEVE ANALYSIS TEST RESULTS

SVA GREELEY - DENTAL BUILDING
1911 59TH AVENUE, GREELEY, COLORADO
503044001 | 10/24

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 (percent)	Equivalent USCS
●	B-2	4.0-5.0	--	--	--	--	--	--	--	--	25	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

FIGURE B-3

GRADATION TEST RESULTS

SVA GREELEY - DENTAL BUILDING
1911 59TH AVENUE, GREELEY, COLORADO
503044001 | 10/24

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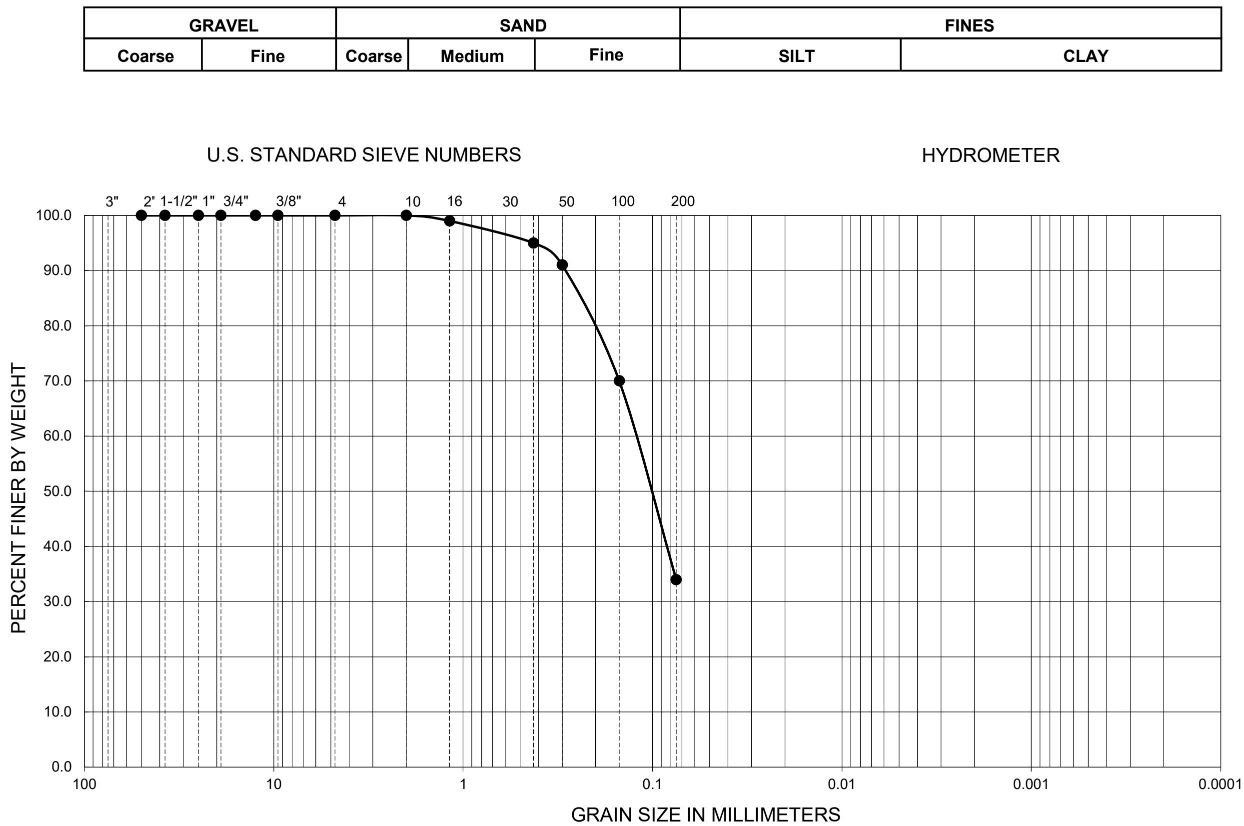

Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	Equivalent USCS
●	B-3	4.0-5.0	NP	NP	NP	--	--	--	--	--	34	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

NP - INDICATES NON-PLASTIC

FIGURE B-4

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GRADATION TEST RESULTS
SVA GREELEY - DENTAL BUILDING
1911 59TH AVENUE, GREELEY, COLORADO
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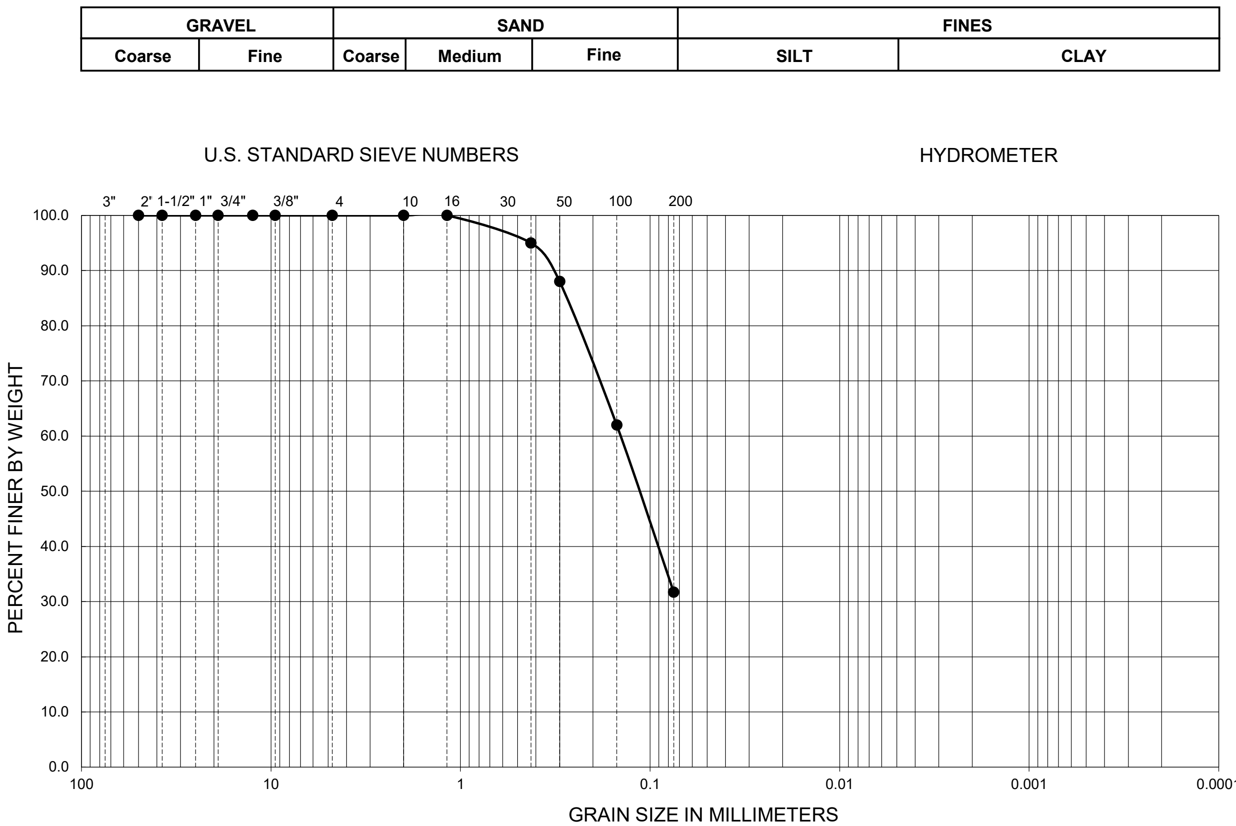

Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	Equivalent USCS
●	B-5	9.0-10.0	20	14	6	--	--	--	--	--	32	SC-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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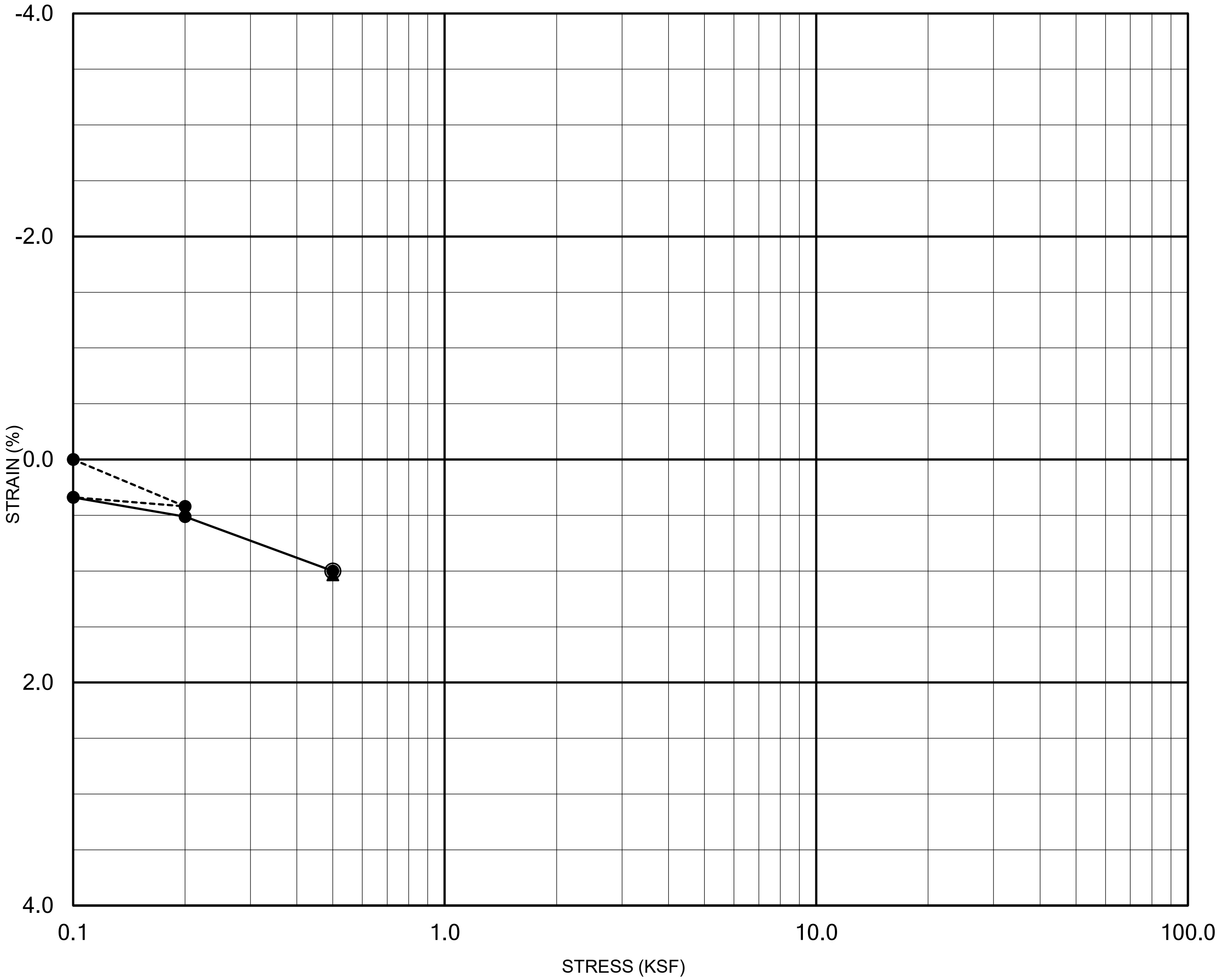
GRADATION TEST RESULTS

SVA GREELEY - DENTAL BUILDING

1911 59TH AVENUE, GREELEY, COLORADO



Moisture Increase (%): 1.2
Swell Percentage (%): 0.0
Swell Pressure (psf): --

Sample Location: B-1
Depth: 4.0-5.0
Soil Type: CL (Fill)

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

FIGURE B-6

CONSOLIDATION TEST RESULTS

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SVA GREELEY - DENTAL BUILDING
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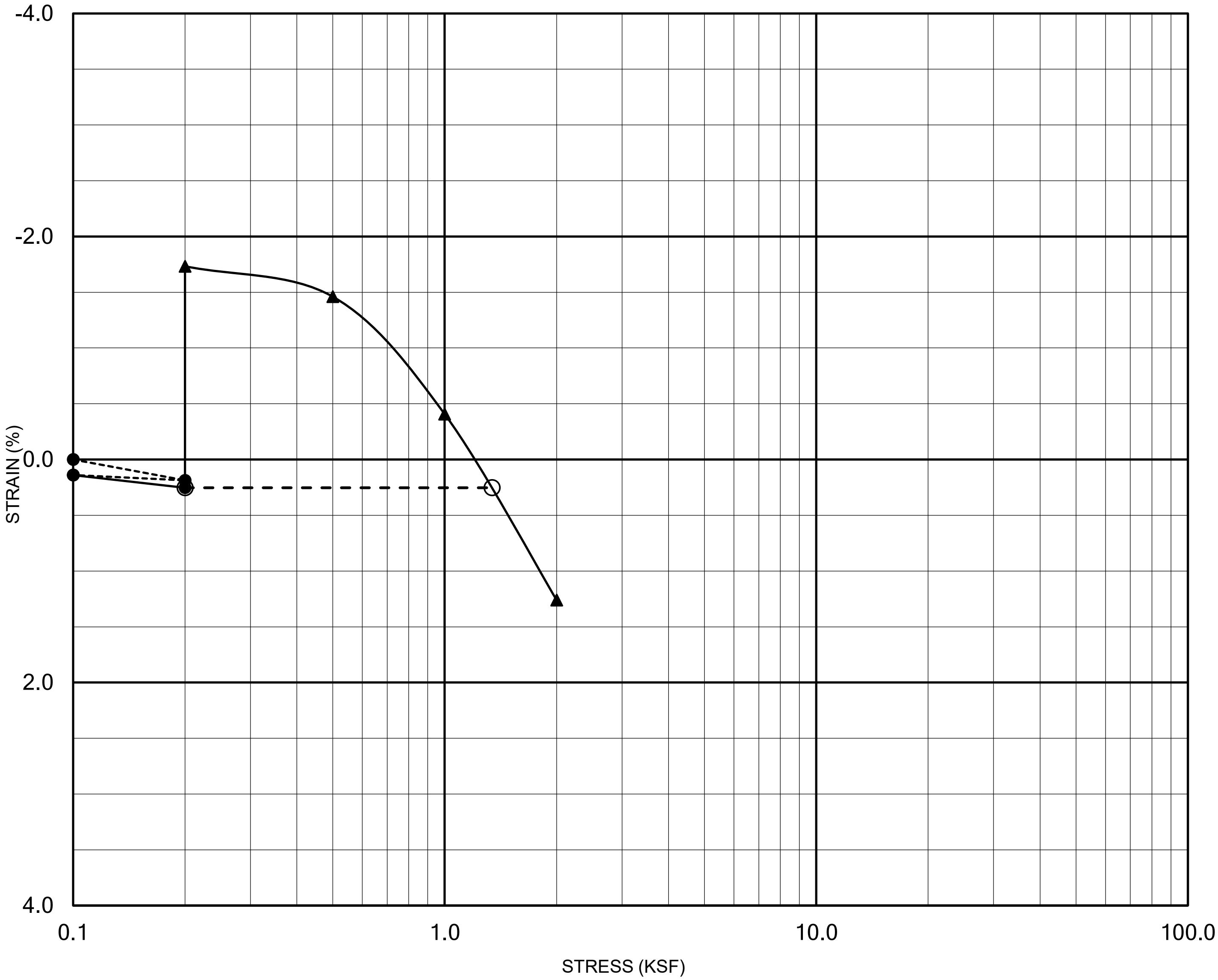

Seating Cycle
Load Prior to Inundation
Load After Inundation
Swell Pressure

Moisture Increase (%):	8.6
Swell Percentage (%):	2.0
Swell Pressure (psf):	1,140

Sample Location: B-3
Depth: 1.0-2.0
Soil Type: CL (Fill)

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

FIGURE B-7

CONSOLIDATION TEST RESULTS

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1911 59TH AVENUE, GREELEY, COLORADO

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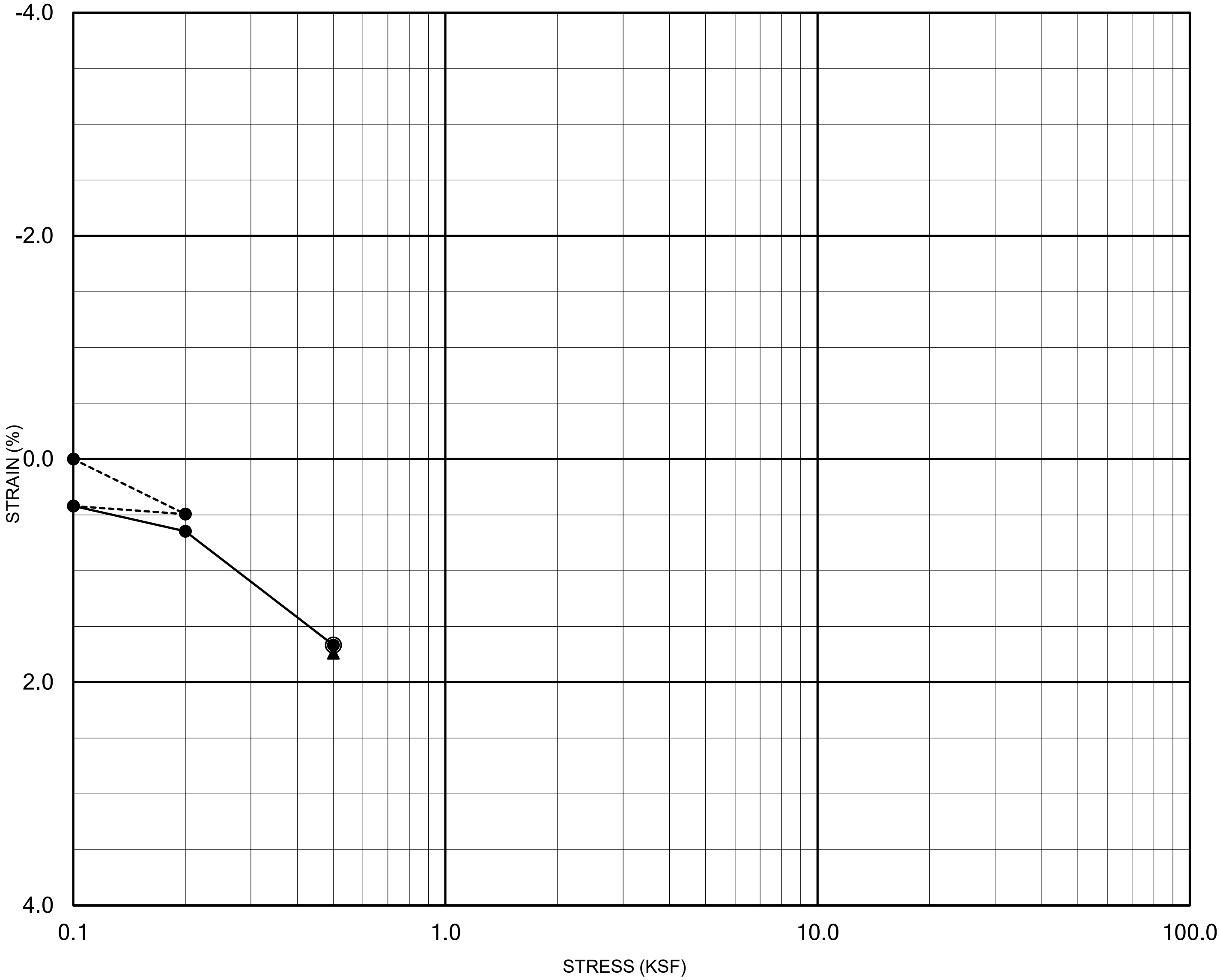

Moisture Increase (%):	0.8
Swell Percentage (%):	-0.1
Swell Pressure (psf):	--

Sample Location: B-3
Depth: 9.0-10.0
Soil Type: CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

FIGURE B-8

CONSOLIDATION TEST RESULTS

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SVA GREELEY - DENTAL BUILDING
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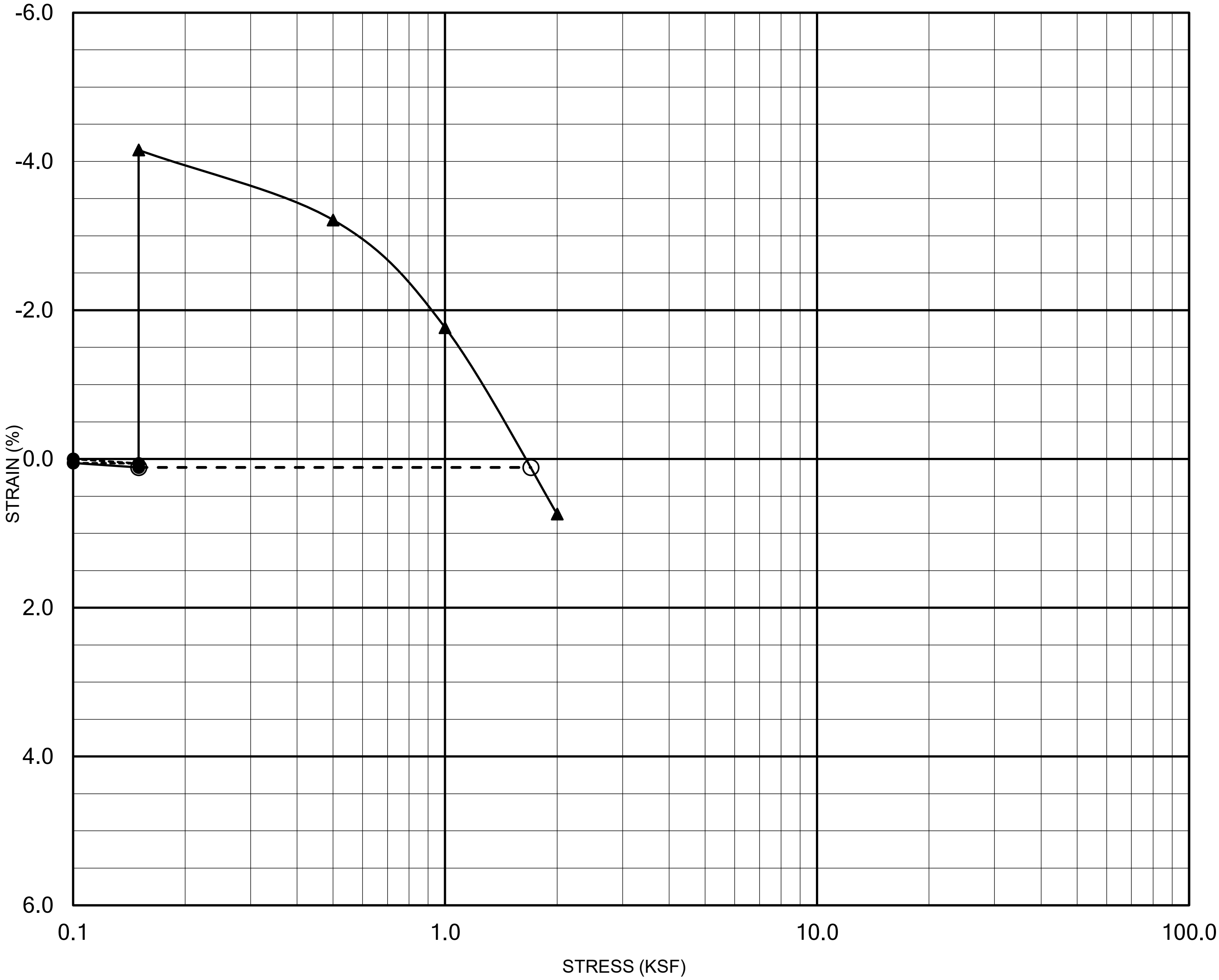

- - -●- - - Seating Cycle
—●— Load Prior to Inundation
—▲— Load After Inundation
- ⊖ - Swell Pressure

Moisture Increase (%): 12.8
Swell Percentage (%): 4.3
Swell Pressure (psf): 1,550

Sample Location: B-4
Depth: 1.0-2.0
Soil Type: CL (Fill)

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

FIGURE B-9

CONSOLIDATION TEST RESULTS

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SVA GREELEY - DENTAL BUILDING
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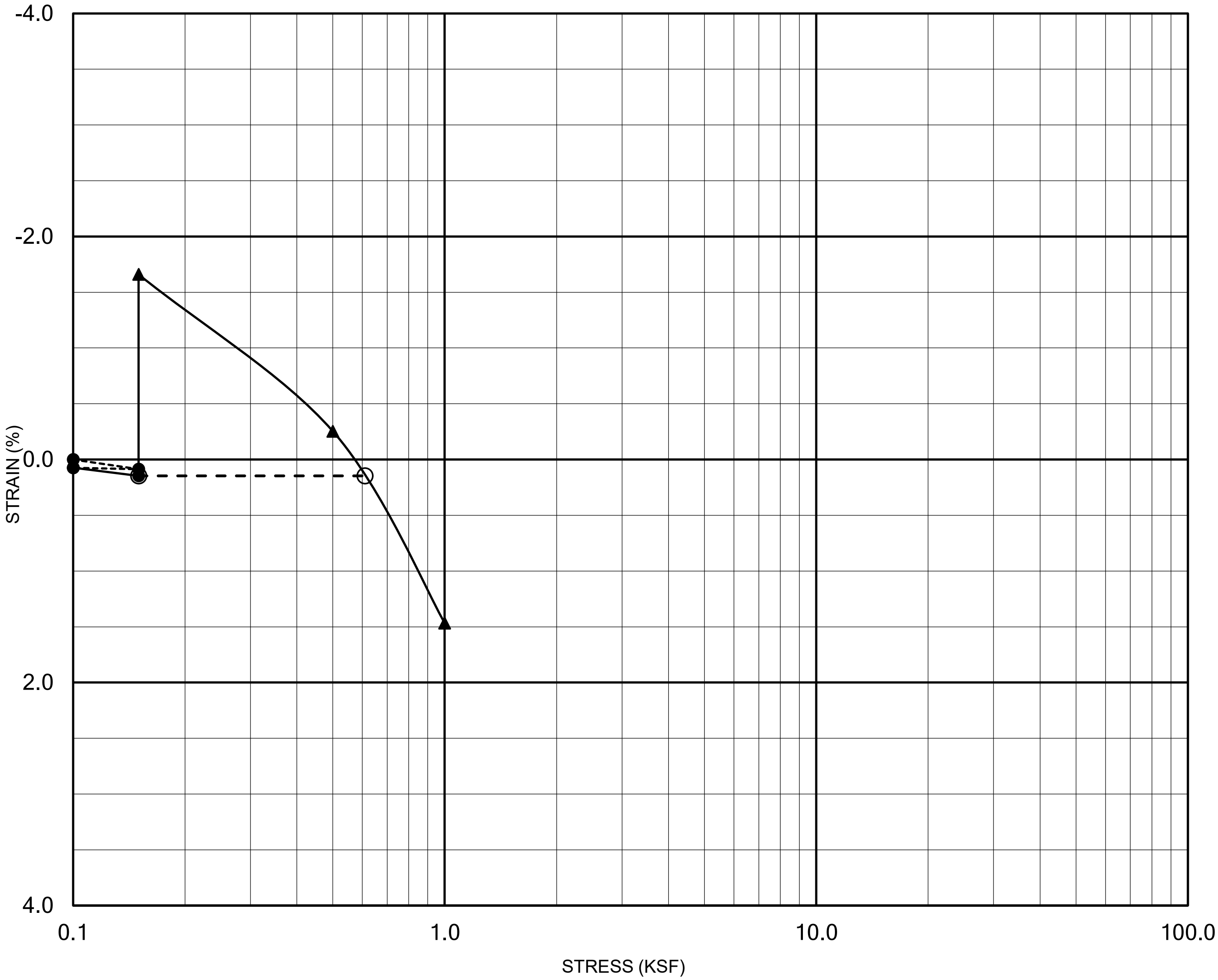
503044001 | 10/24



Moisture Increase (%):	14.8
Swell Percentage (%):	1.8
Swell Pressure (psf):	460

Sample Location: B-5
Depth: 1.0-2.0
Soil Type: CL (Fill)

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

FIGURE B-10

CONSOLIDATION TEST RESULTS

SVA GREELEY - DENTAL BUILDING
1911 59TH AVENUE, GREELEY, COLORADO

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [2] (ohm-cm)	SULFATE CONTENT IN SOIL [3] (ppm)	(%)	CHLORIDE CONTENT [4] (ppm)
B-1 to B-5	0.0-5.0	6.5	1,380	12	0.001	85

[1] PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4972
[2] PERFORMED IN GENERAL ACCORDANCE WITH AASHTO T288
[3] PERFORMED IN GENERAL ACCORDANCE WITH CDOT TEST METHOD CP-L 2103 METHOD B
[4] PERFORMED IN GENERAL ACCORDANCE WITH CDOT TEST METHOD CP-L 2104

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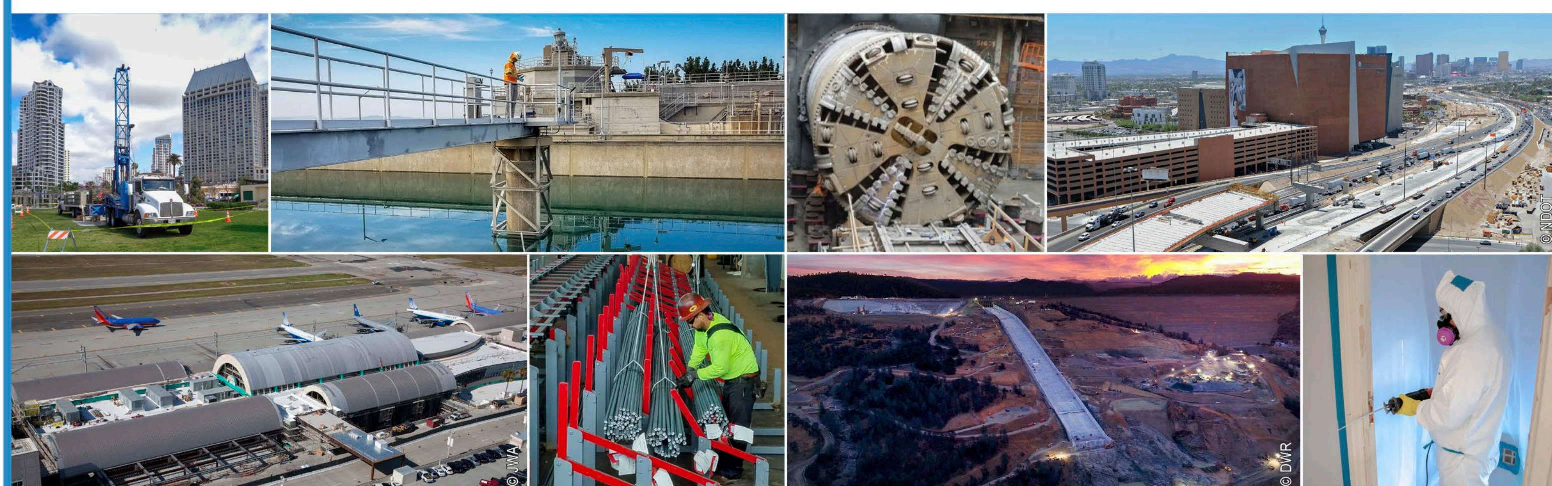
FIGURE B-11

CORROSIVITY TEST RESULTS

SVA GREELEY - DENTAL BUILDING
1911 59TH AVENUE, GREELEY, COLORADO
503044001 | 10/24



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