GEOTECHNICAL EVALUATION
MONUMENT WATER TANK
744 FOREST VIEW WAY
MONUMENT, COLORADO

PREPARED FOR:
Forsgren Associates
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November 18, 2016
Project No. 501233001

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Mr. Jonathan Moore
Forsgren Associates
56 Inverness Drive East, Suite 112
Englewood, Colorado 80112

Subject: Geotechnical Evaluation
Monument Water Tank
744 Forest View Way
Monument, Colorado

Dear Mr. Moore:

In accordance with your authorization and our proposal dated October 13, 2016, Ninyo & Moore has performed a geotechnical evaluation for the above-referenced site. The attached report presents our methodology, findings, and conclusions regarding the geotechnical conditions at the project site and provides geotechnical engineering recommendations for the proposed development.

We appreciate the opportunity to be of service to you during this phase of the project.

Sincerely,
NINYO & MOORE

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1. INTRODUCTION

In accordance with your request and authorization, we have performed a geotechnical evaluation for the proposed Monument Water Tank project located at 744 Forest View Way in Monument, 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 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 imagery, published geologic maps, in-house geotechnical data, and available topographical information pertaining to the project site and vicinity.
- Walking geologic reconnaissance, mark-out of the boring locations at the project site, and notification of Utility Notification Center of Colorado of the boring locations prior to drilling.
- Drilling, logging, and sampling of three (3) small-diameter exploratory borings within the project site to depths ranging from 24.4 to 29.3 feet below ground surface (bgs). One of the borings was completed as a temporary groundwater monitoring well. The boring logs are presented in Appendix A. Boring locations are presented on Figure 2.
- Performance of an additional site visit to measure the groundwater elevation within the monitoring well installed.
- Performance of laboratory tests on selected samples obtained from the borings to evaluate engineering properties including in-situ moisture content and dry density, Atterberg limits, particle size distribution, swell/consolidation potential, and soil corrosivity characteristics (including pH, resistivity, sulfate content, and chloride content). The results of the in-situ moisture content and dry density laboratory testing are presented on the boring logs in Appendix A and the remainder of the laboratory testing results is presented 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 site is located at 744 Forest View Way in Monument, Colorado. The site is bounded to the northeast by Forest View Way, to the southeast by a vacant wooded hillside lot, to the southwest by a topographic ridgeline followed by a residential property on the other side of the ridge, and to the northwest by a vacant wooded hillside lot. The approximate location of the site is depicted on Figure 1.

The site is located on a northeast-facing hillside that ascends approximately 120 feet from Forest View Way. Based on our review of a topographic survey of the property prepared by Barron Land (2016), the ground elevations on the site range from approximately 7,181 feet above mean sea level (MSL) in the northern corner of the site up to 7,285 feet above MSL in the western corner of the site. The slopes on the site vary from approximately 6 to 1 (horizontal to vertical) in the lower portion of the site up to 2 to 1 (horizontal to vertical) in the upper portion of the site.

The site currently exists as an undeveloped, wooded, hillside lot. Lower portions of the site are covered by scattered coniferous trees, deciduous trees, and chaparral consisting of scrub oak. The upper portion of the site contains relatively more abundant coniferous trees and rock outcrops consisting of the Dawson Formation. An unlined drainage swale is present along the northeastern edge of the site coincident with Forest View Way.

Based on our historic aerial imagery review, the site has existed similar to its current condition since September of 1999 or earlier. Historic aerial imagery from 1999 to 2004 indicates that development of Forest View Way had not occurred. Historic aerial imagery from 2005 and 2006 indicates the occurrence of grading activities for development of Forest View Way, and aerial imagery from 2010 shows Forest View Way similar to its current paved condition.

4. PROPOSED CONSTRUCTION

We understand that the project will involve the construction of a 1.2 million gallon, circular water tank with a diameter of approximately 94 feet and a height of approximately 20 to 25 feet.

We also understand that the tank may be buried for aesthetic purposes. While the exact location of the proposed tank and the depth of burial is not finalized at this time, we understand the tank may be buried so that the tank's top elevation matches the ground surface at its downhill extent.

5. FIELD EXPLORATION AND LABORATORY TESTING

On October 26, 2016, Ninyo & Moore conducted subsurface exploration at the site to evaluate the existing subsurface conditions and to collect soil and rock samples for laboratory testing. The evaluation consisted of the drilling, logging, and sampling of three (3) small-diameter borings (Borings B-1 through B-3) using a CME-750 buggy drill rig equipped with 4-inch diameter solid-stem augers to depths of approximately 24.4 to 29.3 feet bgs. One of the borings (Boring B-3) was completed as a temporary groundwater monitoring well. The approximate locations of the borings are presented on Figure 2. Relatively undisturbed and disturbed soil and rock samples were collected at selected intervals. The sampling methods used during the subsurface evaluation are presented in Appendix A.

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, particle size analysis, swell/consolidation characteristics, and soil corrosivity characteristics (including resistivity, pH, sulfate content, and chloride content). 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. GEOLOGY AND SUBSURFACE CONDITIONS

The geology and subsurface conditions at the site are described in the following sections.

6.1. Geologic Setting

The site is located approximately 2 miles west of the town of Monument below the foot of the Rampart Range, within the Colorado Piedmont section of the Great Plains Physiographic Province. The Laramide Orogeny uplifted the Rocky Mountains during the late Cretaceous

and early Tertiary Periods. Subsequent erosion deposited sediments east of the Rocky Mountains, including the Dawson Formation in the area. As a result of regional uplift approximately 5 to 10 million years ago, streams down-cut and excavated into the Great Plains forming the Colorado Piedmont section (Trimble, 1980).

The site is mapped by Keller et al. (2006) as being underlain by the Paleocene-age facies unit four of the Dawson Formation. The southeastern and eastern portions of the stadium are mapped as being underlain by Holocene and late Pleistocene colluvium. Thorson and Madole (2003) describe facies unit four of the Dawson Formation as being dominated by very thick bedded to massive, cross-bedded, light colored arkoses, pebbly arkoses, and arkosic pebble conglomerate, and contains common white to light tan, fine to medium grained feldspathic cross-bedded friable sandstone, poorly sorted with high clay content. The unit also contains massive structureless mudflow beds and can contain thin-bedded claystone and sandy claystone. The site and vicinity is mapped by Hart (1973-1974) as having low to moderate swell potential.

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, and historic aerial imagery. 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. Colluvium

Approximately 12 inches of topsoil was encountered in each of the borings. Colluvial deposits were encountered in each boring from below the topsoil to the top of the residual soil at approximately 4 feet bgs. The colluvium generally consisted of brown, dry, medium dense, clayey sand.

Based on the results of the laboratory testing, selected samples of the alluvium had in-place moisture contents between approximately 3.9 and 5.2 percent and dry densities between approximately 106 and 110.4 per cubic foot (pcf).

6.2.2. Residual Soil

Residual soil, representing decomposed bedrock material, was encountered in each boring below the colluvium and extended to the Dawson Formation at depths ranging from approximately 9.3 to 11 feet bgs. The residual soil generally consisted of light brown, dry, medium dense, clayey sand.

Based on the results of the laboratory testing, selected samples of the alluvium had in-place moisture contents between approximately 3.3 and 6.5 percent and dry densities between approximately 109.4 and 114.5 pcf.

6.2.3. Dawson Formation

The Dawson Formation was encountered below the residual soil in each boring and extended to borings' termination depths of approximately 24.4 to 29.3 feet bgs. The Dawson Formation generally consisted of pale brown, dry, moderately to strongly cemented, clayey fine to coarse-grained sandstone.

Based on the results of the laboratory testing, selected samples of the Dawson Formation had in-place moisture contents between approximately 3.2 and 7.8 percent and dry densities between approximately 116.1 and 124.2 pcf.

6.3. Groundwater

Groundwater was not encountered during drilling. Additionally, groundwater was not observed during our site visit on November 1, 2016 in the temporary groundwater monitoring well at Boring B-3 or in the open hole at Boring B-2. Groundwater levels can fluctuate due to seasonal variations in precipitation, snowmelt, irrigation, groundwater withdrawal or injection, and other factors.

7. GEOLOGIC HAZARDS

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

7.1. Faulting and Seismicity

Historically, several minor earthquakes have been recorded around the Colorado Springs 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 fault closest to the project site is the Rampart Range Fault.

The Rampart Range Fault lies approximately 1.1 miles west of the site (United States Geological Survey (USGS) & Colorado Geological Survey, 2016). The fault trends north-south along the eastern side of the Front Range. The Rampart Range Fault is approximately 28½ miles long and dips to the west (Widmann, 1997). Dickson (1986) excavated and logged two trenches on the section of the fault that extends through the southwest corner of U.S. Air Force Academy property, south of the Colorado Springs filtration plant. Trenching investigations indicated that the last displacement on this portion of the fault occurred between 600 thousand years and 30 to 50 thousand years before present. Therefore, the probability of damage at the site from seismically induced ground surface rupture from this fault is considered to be low.

Using the referenced USGS seismic web application (USGS, 2016), estimated maximum considered earthquake spectral response accelerations for short (0.2 second) and long (1.0 second) periods were obtained for the project site. The parameters in the following table are characteristic of the project site for design purposes.

Table 1 – 2015 International Building Code Seismic Design Criteria

Site Coefficients and Spectral Response Acceleration Parameters	Values
Class	D
Coefficient, F_a	1.2
Coefficient, F_v	1.7
Mapped Spectral Response Acceleration at 0.2-second Period, S_s	0.192 g
Mapped Spectral Response Acceleration at 1.0-second Period, S_1	0.060 g
Spectral Response Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	0.307 g
Spectral Response Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.144 g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	0.205 g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.096 g

Horizontal peak ground accelerations for the site were calculated from the 2008 Probabilistic Seismic Hazard Assessment prepared by the USGS National Seismic Hazard Mapping Project (USGS, 2008). The horizontal peak ground accelerations for the site that have a 10, 5, and 2 percent probability of being exceeded in 50 years are 0.03g, 0.05g, and 0.10g, respectively. These ground motion values were calculated for "firm rock" sites, which correspond to a shear-wave velocity of approximately 2,500 feet per second in approximately the top 100 feet bgs. Different soil or rock conditions may amplify or de-amplify these values.

7.2. Expansive Soils

Moisture changes to bedrock or surficial deposits containing swelling clays can result in volumetric expansion and collapse of those units. Changes in soil moisture content can result from rainfall, irrigation, pipeline leakage, surface drainage, perched groundwater, drought, or other factors. Volumetric change of expansive soil may cause excessive cracking and heaving of structures with shallow foundations, concrete slabs-on-grade, or pavements supported on these materials. Construction on soils known to be potentially expansive could have a significant impact to the project.

Based on the results of our laboratory testing, the Dawson Formation material did not exhibit swelling characteristics during the swell testing performed. As such, we do not regard expansive soils to be a design consideration for the site.

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 the results of our laboratory testing, the potential for post-construction consolidation of the Dawson Formation material is low. Laboratory testing was not performed to characterize the consolidation characteristics of the colluvium or residual soil materials, but we recommend that these materials be removed, replaced, and compacted to mitigate concerns regarding post-construction consolidation of these materials.

7.4. Liquefaction Potential

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

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

exploration and the low ground motion hazard, liquefaction is not considered a hazard at this site.

8. CONCLUSIONS

Based on the results of the subsurface evaluation, laboratory testing, and data analyses, it is our opinion that the proposed project is feasible from a geotechnical standpoint, provided the recommendations presented herein are implemented and appropriate construction practices are followed. Geotechnical design and construction considerations for the proposed project include the following:

- Approximately 12 inches of topsoil was encountered in each of the borings. Colluvial deposits were encountered in each boring from below the topsoil to the top of the residual soil at approximately 4 feet bgs. The colluvium generally consisted of brown, dry, medium dense, clayey sand.

- Residual soil, representing decomposed bedrock material, was encountered in each boring below the colluvium and extended to the Dawson Formation at depths ranging from approximately 9.3 to 11 feet bgs. The residual soil generally consisted of light brown, dry, medium dense, clayey sand.

- The Dawson Formation was encountered below the residual soil in each boring and extended to borings' termination depths of approximately 24.4 to 29.3 feet bgs. The Dawson Formation generally consisted of pale brown, dry, moderately to strongly cemented, clayey fine to coarse-grained sandstone.

- The on-site soils (residual soil and colluvium) should generally be excavatable to the anticipated removal depths with moderate to heavy-duty earthmoving or excavating equipment in good operating condition. Strongly cemented zones are anticipated when excavating in materials of the Dawson Formation. Excavation in Dawson Formation will require the use of heavy ripping or rock breakers. Increased wear and tear on excavation equipment should be anticipated at this site.

- Site soils generated from on-site excavation activities 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 provided they are moisture-conditioned and compacted as recommended in this report. Excavated Dawson Formation will require significant processing to break down the material into particles smaller than 3 inches.

- Groundwater was not encountered during our subsurface exploration or during our subsequent site visit. While groundwater levels can fluctuate due to seasonal variations in precipitation, irrigation, groundwater withdrawal or injection, and other factors, groundwater

is not anticipated to be a constraint during construction. Depending on the time of year construction is performed, groundwater seepage may be encountered in deep cuts and during utility installation. Such seepage may occur between the residual soil and Dawson Formation and also through higher permeability zones of Dawson Formation.

- Based on our laboratory data, the sulfate content of the tested soils presents a negligible risk of sulfate attack to concrete.

- Based on our laboratory data, the on-site materials have moderate resistivity characteristics and low chloride content, and could potentially be moderately corrosive to ferrous metals. 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.

- No known or reported active faults are reported underlying, or adjacent to, the site. Based on the driven sample blow-count values at the site, and the low ground motion hazard (relatively low ground accelerations), the likelihood or potential for liquefaction is considered to be negligible and therefore liquefaction is not a design consideration.

9. RECOMMENDATIONS

Based on our understanding of the project, the following sections present our geotechnical recommendations for design and construction of the proposed structures and other site improvements.

Grading plans were not available at the time of this report. It is important that Ninyo & Moore be notified and given an opportunity to re-evaluate our recommendations once this information becomes available and prior to bidding the project for construction.

9.1. Earthwork

The following sections provide our earthwork recommendations for this project. We anticipate that the site grading may consist of an excavation depth of at least 20 to 25 feet for construction of the proposed below-grade tank and associated foundation. Additionally, if the excavation does not rely on shoring for temporary support, or if a retaining wall is not constructed around the tank, we anticipate that the project may involve cuts of up to 50-60 feet in height extending from the southwest side of the proposed tank excavation bottom. If a retaining wall is installed around the tank, we anticipate site grading may involve material

fills of up to 30 feet behind the retaining wall. Additionally, we anticipate material cuts and fills of generally less than 3 feet for the footings and mat slab. Other cuts and fills may also be needed to install buried utilities.

9.1.1. Excavations

Our evaluation of the excavation characteristics of the on-site materials is based on the results of the subsurface exploration, our site observations, and our experience with similar materials. The on-site surface and near-surface soils (colluvium and residual soil) may generally be excavated with heavy-duty earthmoving or excavation equipment in good operating condition.

Strongly cemented zones are anticipated when excavating in materials of the Dawson Formation. Excavation in these materials will require the use of heavy ripping or rock breakers. Increased wear and tear on excavation equipment should be anticipated at this site.

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 soil should be avoided when excavating the bottom 6 to 12 inches of excavations as such equipment may disturb the excavation bases.

The native soils and Dawson Sandstone materials are susceptible to variations in moisture content. As a result and depending on the time of year construction occurs, wet or saturated soils may be encountered after periods of heavy or prolonged precipitation and/or snowmelt. These materials may soften under the action of light equipment and foot traffic. Where encountered, drying or overexcavation of these materials is recommended. If the subgrade becomes disturbed, it should be compacted or removed and replaced before placing additional backfill material.

Stabilization methods should be provided by the grading contractor, as needed, and may include the use of geogrids, geotextiles, pushing oversized rock into the subgrade, and/or chemical stabilization. The subgrade stabilization methods proposed should be

discussed with the Geotechnical Engineer prior to implementation. The stabilization method selected should consider the effects of such stabilization on future utility installation and/or repair work.

Groundwater was not encountered in our borings during drilling operations or noted in our subsequent site visit. Groundwater levels can fluctuate due to seasonal variations in precipitation, irrigation, groundwater withdrawal or injection, snowmelt, and other factors. We recommend that the groundwater levels be further evaluated using the temporary monitoring well during a wetter time of the year. While groundwater is not anticipated to be a constraint to the proposed construction, depending on the depth of buried utility lines, groundwater seepage may be encountered during tank and utility construction.

The contractor should provide safely sloped excavations or an adequately constructed and braced shoring system, in compliance with Occupational Safety and Health Administration (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. Site Grading

Prior to grading, the ground surface in proposed improvement areas should be cleared of any surface obstructions, existing pavements, debris, topsoil, organics (including vegetation), and other deleterious material. Material generated from site stripping should not be incorporated into engineered fill, but may be stockpiled for re-use as landscaping material or other non-structural material. Topsoil-contaminated fill materials should not be used during site grading.

Materials generated from clearing operations should be removed from the project site for disposal (e.g. at a legal landfill site). Obstructions that extend below finish grade, if

present, should be removed and resulting voids filled with compacted, engineered fill or controlled low strength material (CLSM).

The exposed subgrade materials should be firm and unyielding prior to fill placement. Subgrades should be evaluated by our representative during the excavation work. 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 unsuitable subgrade material. Subgrade materials that are disturbed during grading should be moisture conditioned and re-compacted according to the recommendations provided in this report.

Where fill is placed on slopes steeper than approximately 5:1 (horizontal:vertical), benches should be constructed into the overburden soils (natural or man-made) or formational material to minimize the plane of weakness between the new fill and existing subgrades.

Proposed water tank may be supported on firm Dawson Formation bedrock or on 12 or more inches of moisture-conditioned and compacted engineered fill extending to firm, intact, Dawson Formation. If placement of more than 24 inches of engineered fill is needed below the proposed tank, we should be contacted to re-evaluate the recommendations provided in this report.

New exterior flatwork should be placed on 12 or more inches of moisture-conditioned and compacted engineered fill extending to suitable native deposits.

9.1.3. Re-Use of Site Soils

Clayey sand colluvium and residual soil material was encountered below the ground surface to depths of approximately 9.3 feet to 11 feet bgs. Based on the laboratory test results and our general observations, it is our opinion that colluvium and residual soil free of deleterious materials and organic matter, and does not contain particles larger than 3 inches in diameter, can generally be used as engineered fill provided it is moisture-conditioned and compacted as recommended in this report.

Below the colluvial deposits and residual soil, sandstone of the Dawson Formation was encountered in each boring to the depths explored. Excavated Dawson Formation will require significant processing to break down the material into particles smaller than 3 inches. Such processing may require the use of disks and laying down the material outside of the excavation zone to allow for the operation of such equipment. Based on the laboratory test results and our general observations, it is our opinion that pulverized sandstone material with greater than 50 percent passing the No. 4 sieve, free of deleterious materials and organic matter, and does not contain particles larger than 3 inches in diameter, can generally be used as engineered fill provided it is moisture-conditioned and compacted as recommended in this report.

Fragments of rock, cobbles, and inert construction debris (e.g., concrete or asphalt) larger than 3 inches in diameter may be incorporated into the project fills in non-structural areas and below the anticipated utility installation depths. A Geotechnical Engineer should be consulted regarding appropriate recommendations for usage of such materials on a case-by-case basis when such materials have been observed during earthwork. Care should be taken to avoid nesting of oversized materials during placement. Recommendations provided in Section 203 of the current Colorado Department of Transportation (CDOT) Standard Specifications for Road and Bridge Construction should be followed during the placement of oversized material.

9.1.4. Fill Placement and Compaction

Granular soils (on-site soils that classify as SC, SM, SP, SP-SM, 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 American Society for Testing and Materials (ASTM) D1557.

Fill should be compacted by appropriate mechanical methods using vibratory compaction equipment. The optimal lift thickness of fill will depend on the type of soil and compaction equipment used, but should generally not exceed approximately

8 inches in loose thickness. Fill materials should not be placed, worked, or rolled while they are frozen or thawing, and should not be placed 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.

Use of 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 should be placed in lifts of 5 feet or less with a 24-hour or more curing period between each lift.

9.1.5. Imported Soil

Imported soil for use as engineered fill should have 40 or less percent passing the No. 200 sieve, a very low swell potential (approximately 1 percent or less when wetted against a surcharge pressure of 500 psf when remolded at optimum moisture content), and a low plasticity index (approximately 15 or less).

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.

Import material in contact with ferrous metals should have low corrosion potential. Import material in contact with concrete should have 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. Import soil should be moisture-conditioned and placed and compacted in accordance with the recommendations set forth in Section 9.2.4.

9.1.6. Utility Installation

The Contractor should provide adequate mechanical compaction in the utility trench backfills, particularly in the lower portions of the excavations. The Contractor should take particular care to achieve and maintain adequate compaction of the backfill soils around manholes, valve risers and other vertical pipeline elements where settlements commonly are observed. It is our experience that pipes entering underground structures, such as the proposed water tank, commonly shear at or near the interface with the structure. This may be caused by settlement of overlying fill material exerting pressure on the pipes. To reduce the potential pipe shearing, we recommend that a flexible pipe joint be located close to the exterior of the wall. The type of joint should be such that minor relative movement can be accommodated without distress. The pipe connections should be sufficiently flexible to withstand differential movement of approximately 2 inches.

Use of CLSM, or a similar material) should be considered in lieu of compacted soil backfill for areas with low tolerances for surface settlements in deep excavations and areas with difficult access. CLSM consists of a fluid, workable mixture of aggregate, Portland cement, and water. The use of CLSM has several advantages:

- A narrower excavation can be used where properly shored, thereby minimizing the quantity of soil to be excavated;
- Compaction requirements do not apply;
- There is less risk of damage to improvements, since little compaction is needed to place CLSM;
- CLSM will reduce the permeability of the utility trenches and reduce the water transfer along utility bedding towards the proposed water tank;
- CLSM can be batched to flow into irregularities in excavation bottoms; and
- The number of workers needed inside the trench excavation is reduced.

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

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

Surface drainage should be designed to divert the surface 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 a non-woven geotextile fabric (e.g., TenCate Mirafi® 140N or the equivalent) to reduce the 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 structures, 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.

9.1.7. Temporary Cut Slopes

Temporary excavations will be needed for this project to construct the proposed water tank and associated improvements. Based on the subsurface information obtained from our exploratory borings 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 OSHA regulations as mentioned in Section 9.1.1.

In our opinion, the colluvium and residual soil materials on site should generally be considered Type C soil when applying the OSHA regulations. In our opinion, the Dawson Formation materials on site should generally be considered Type A rock when applying the OSHA regulations. For Type C soil/rock conditions, OSHA recommends a temporary slope inclination of 1.5H:1V or flatter for excavations 20 feet or less in depth. For Type A soil/rock conditions, OSHA recommends a temporary slope inclination of 3/4H:1V or flatter for excavations 20 feet or less in depth. Appropriate slope inclinations should be evaluated in the field by an OSHA-qualified “Competent Person” based on the conditions encountered.

9.2. Spread Footing Foundations

Perimeter footings should extend to 36 inches or more below the lowest exterior finished grade (for frost protection). 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.

Footings bearing on a firm, intact sandstone material or on compacted engineered fill extending to intact sandstone material as recommended in Section 9.1.2 may be designed using a net allowable soil bearing pressure of 4,000 pounds per square foot (psf) for static conditions. If placement of more than 24 inches of engineered fill is needed below the proposed tank, we should be contacted to re-evaluate the recommendations provided in this

report. The bearing capacity may be increased by one-third when considering loads of short duration such as wind or seismic forces. Based on the results of our subsurface exploration, laboratory testing and our understanding of the proposed construction, we estimate total and differential settlement of up to about 1-inch and 1/2-inch, respectively, may occur. The foundations should preferably be proportioned such that the resultant force from design loads, including lateral loads, falls within the kern (i.e., middle one-third of the footing base).

The bottom surface of foundation excavations should be compacted with hand-held dynamic compaction equipment (i.e., jumping jack, flat-plate vibrator) prior to placement of forms and reinforcing steel. The base of foundation excavations should be free of water and loose soil prior to placing concrete. Concrete should be placed soon after subgrade compaction to reduce bearing soil or rock disturbance. Should the soils at bearing level become excessively dry, disturbed, or saturated, the affected soil should be moisture conditioned and recompacted. It is recommended that Ninyo & Moore be retained to observe, test, and evaluate the soil and rock foundation bearing materials.

9.3. Mat Foundations

Mat foundations for the proposed water tank should be supported on compacted engineered fill prepared in accordance with the recommendations presented in Section 9.1.2 of this report. If placement of more than 24 inches of engineered fill is needed below the proposed tank, we should be contacted to re-evaluate the recommendations provided in this report. The mat foundation may be designed using a net allowable bearing capacity of 4,000 psf. The total and differential settlement corresponding to this allowable bearing load are estimated to be less than approximately 1-inch and ½ inch over a horizontal span of 40 feet, respectively.

Mat foundations typically experience some deflection due to loads placed on the mat and the reaction of the soils directly underlying the mat. A design modulus of subgrade reaction (K) of 300 tons per cubic foot (tcf) may be used for the subgrade soils in evaluating such deflections. This value is based on a unit square foot area and should be adjusted for large

mats. Adjusted values of the modulus of subgrade reaction, K_v, can be obtained from the following equation for mats of various widths:

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

B in the above equation represents the width of the mat in feet. For frictional resistance to lateral loads on mat, we recommend a coefficient of friction of 0.38 for compacted granular subgrade and intact Dawson Formation. For a mat with an embedment depth shallower than 2 feet, passive earth pressure should be ignored while evaluating lateral resistance; only frictional resistance should be considered. For mats with embedment depths greater than 2 feet, passive earth pressure may be combined with frictional resistance to evaluate the total lateral resistance. In such cases, the lateral resistance can be taken as the sum of the frictional resistance and passive resistance provided the passive resistance does not exceed one-half of the total resistance. The passive resistance values may be increased by one-third when considering loads of short duration such as wind or seismic forces.

9.4. Earth Pressures and Foundation Walls

Recommendations for lateral earth pressures to be used in design of the yielding and restrained retaining walls are provided on Figures 3 and 4, respectively. As indicated, lateral soil resistance developed against lateral structural movement may be obtained using a passive pressure of 375 psf per foot of depth for a level ground condition up to a value of 3,750 psf per foot. This value assumes that the ground is horizontal for a distance of 10 feet, or three times the height generating the passive pressure, whichever is greater. We recommend that the upper 24 inches of soil not protected by pavement or a concrete slab be neglected when calculating passive resistance. The passive value may be increased by one-third when considering loads of short duration, including wind and seismic loads. Further, for sliding resistance, a friction coefficient of 0.38 may be used between concrete and foundation soil or Dawson Formation. The allowable resistance may be taken as the sum of the frictional and passive resistance provided that the passive portion does not exceed one-half of the total allowable resistance. Retaining walls can be supported on spread footings following design recommendations presented in Section 9.2 of this report.

Measures should be taken to reduce the potential for build-up of moisture behind the retaining walls. Drainage design should include free-draining backfill materials and perforated drains as depicted on Figure 6. Solid outlet pipes should be connected to the perforated drains and then routed to a suitable area for discharge of accumulated water. The portions of retaining walls supporting backfill should be coated with an appropriate waterproofing compound or covered with a similar material to inhibit infiltration of moisture through the walls. It is the responsibility of the project structural engineer and/or the retaining wall contractor to provide specifications for waterproofing materials and adequate methods of application.

9.5. Exterior Concrete Flatwork

Exterior concrete flatwork should be supported on 12 or more inches of compacted engineered fill per section 9.1.2. Exterior walkways and flatwork should be 4 or more inches thick. Design and construction of deepened slab edges should be considered where exterior walkways and flatwork are placed adjacent to landscaping areas.

Ground-supported exterior flatwork will be subject to soil-related movements resulting from heave/settlement, frost, etc. Thus, where these types of elements abut rigid 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. In no case should exterior flatwork extend under any portion of the structure where there is less than 3 inches of clearance between the flatwork and any element of the structure. Exterior flatwork in contact with brick, rock facades, or any other element of the proposed structure can cause damage to the structure if the flatwork experiences movements.

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.

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 laboratory test results indicate that the on-site soils present low corrosivity potential to concrete. The percentage of water-soluble sulfates in water (sulfate content) measured was low. 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 have a maximum water to cementitious material ratio of 0.45 and the cementitious materials should meet one of the below outlined requirements.

- ASTM C 150 Type I, II or V
- ASTM C 595 Type IP
- ASTM C 1157 Type GU
- 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.

The results of the laboratory testing indicate that the on-site materials have moderate electrical resistance classification, low chloride content, and have a relatively neutral pH. Based on these test results, the on-site soils may be considered moderately corrosive to ferrous metals. We recommend that reinforced concrete used at the site be designed and constructed with adequate cover in accordance the guidelines set forth by the ACI. If metal pipes or other metal structures are in contact with on-site soils, we recommend that these items be designed by a corrosion engineer utilizing the results from our laboratory tests.

9.7. Scaling

Climatic conditions in the project area including relatively 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.
- If colored concrete is being proposed for use at this site, Ninyo & Moore should be consulted for additional recommendations.

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, replacement of the subgrade soils with 2 or more feet of clean, coarse sand or gravel, or supporting the element on foundations similar to the structures, 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 structure areas. It is important to cover exposed excavation cuts prior to rain

events to prevent or minimize erosion, via means such as plastic sheets weighed down by a roped array of sandbags. It is also 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 storm water 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, as well as buildup of hydrostatic pressure on below grade walls. 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 water tank. 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. 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 the water tank to a suitable outlet such as a storm drain. Adequate surface drainage may be

enhanced by utilization of graded swales, interceptor drains, area drains, and other drainage devices. Surface run-off should not be allowed to pond near structures or structure footings.

- Downspouts should discharge 5 feet or more away from the structure and onto surfaces that slope away from the structure. Downspouts should not be allowed to discharge onto the ground surface adjacent to structure foundations.

- Ninyo & Moore does not recommend the placement of irrigated landscaping within 5 feet of the proposed water tank and associated exterior flatwork. Low-water use (drip irrigated) landscaping should be utilized within 5 to 10 feet of the proposed water tank. Irrigation heads should be situated so that they spray away from the structure and block wall surfaces.

- Utility trenches should be backfilled with compacted, low permeability fill (i.e. permeability of 5-10 cm/s or less) within 5 feet of the proposed water tank.

- 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, interceptor drains, area drains, and other drainage devices. Surface run-off should not be allowed to pond near structures.

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 preliminary design information 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 that 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. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore

should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

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

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

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

11. REFERENCES

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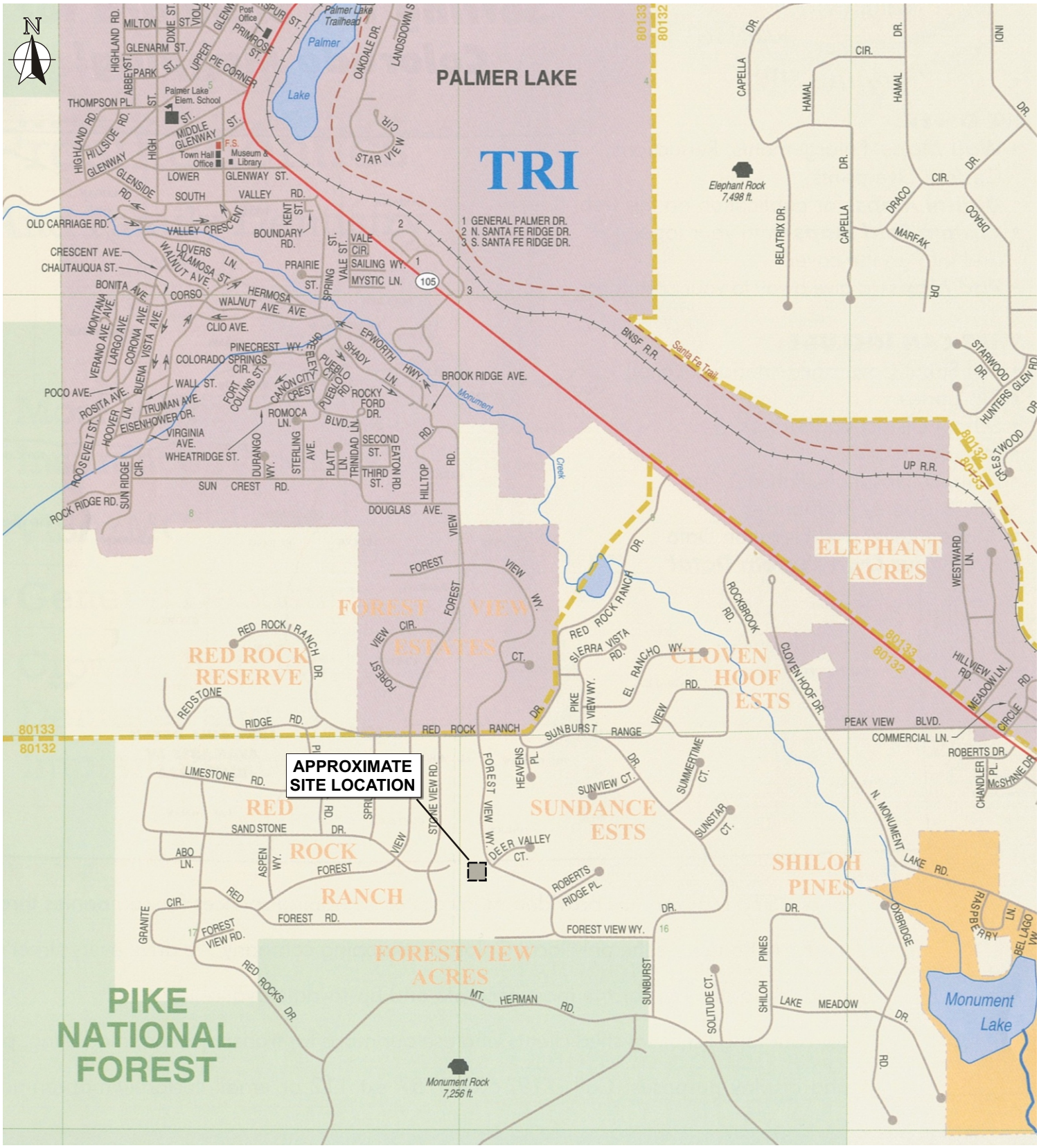
Aerial Image References

Source	Dates
Google Earth	September 28, 1999, November 16, 2004, June 16, 2005, October 7, 2010



Source: Macvan Map Company, Denver, Colorado, 2011.

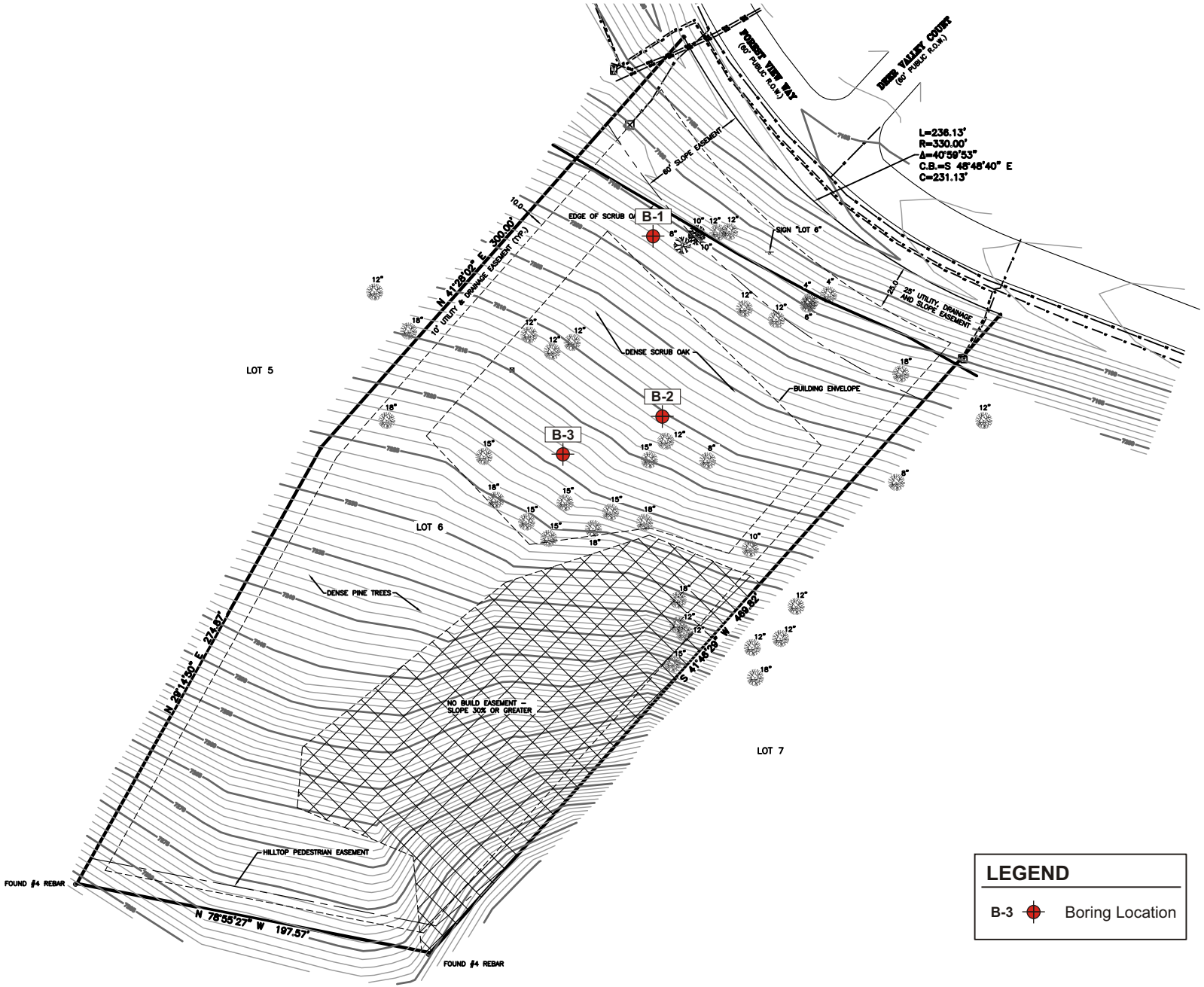
0 1900

Approximate Scale:
1 inch = 1900 feet

Note: Dimensions, directions and locations are approximate.

Ninyo & Moore	SITE LOCATION	FIGURE
PROJECT NO: 501233001 / DATE: 11/16	MONUMENT WATER TANK 744 FOREST VIEW WAY MONUMENT, COLORADO	1

file no: 1233vmap1116



LEGEND

B-3 Boring Location

N

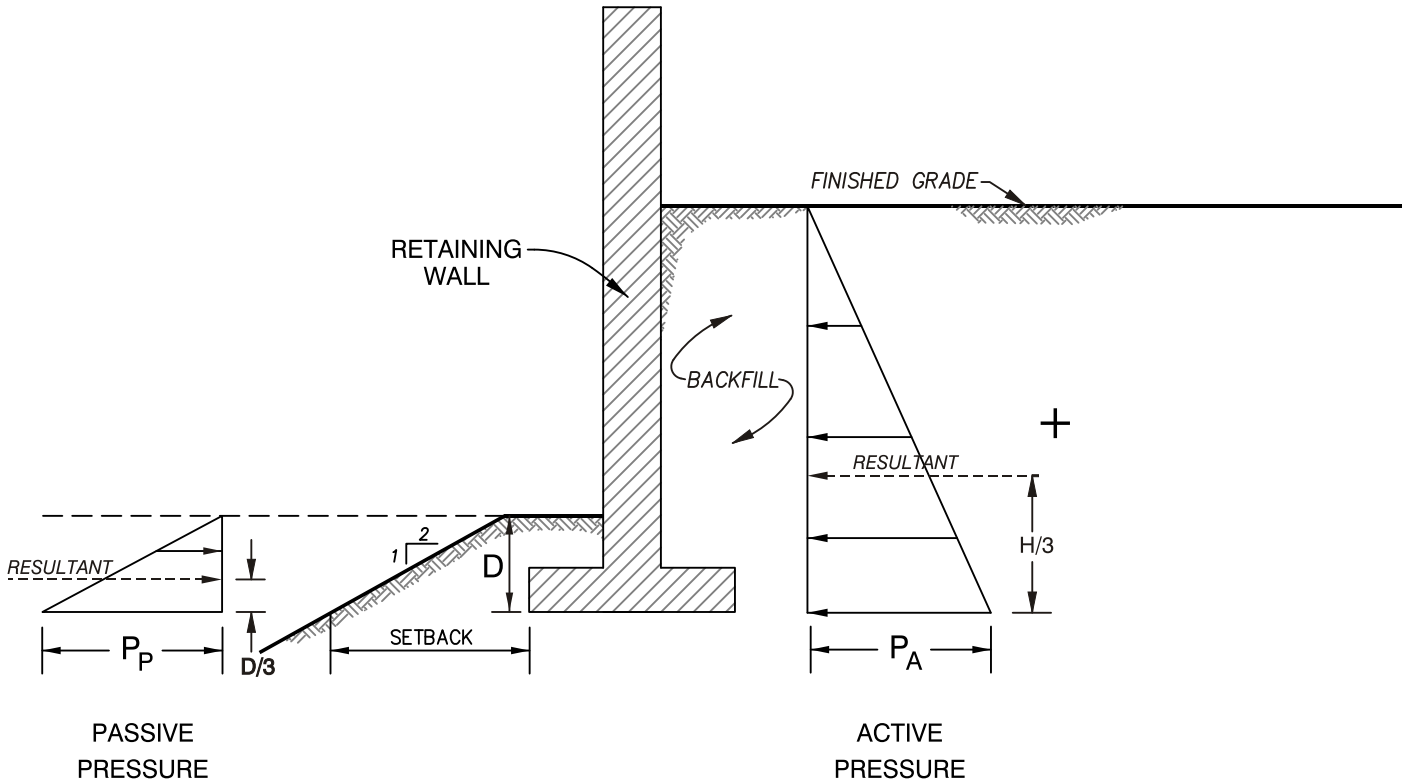
0 80

Approximate Scale:
1 inch = 80 feet

Source: Barron Land, 10/14/16.
Note: Dimensions, directions, and locations are approximate.

Ninyo & Moore	BORING LOCATIONS	FIGURE
PROJECT NO: 501233001 / DATE: 11/16	MONUMENT WATER TANK 744 FOREST VIEW WAY MONUMENT, COLORADO	2

file no: 1233blm1116



NOTES:

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. STRUCTURAL, GRANULAR BACKFILL MATERIALS AS SPECIFIED IN SECTION 9.1 SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
5. H AND D ARE IN FEET
6. SETBACK SHOULD BE IN ACCORDANCE WITH FIGURE 1808.7.1 OF THE IBC (2015)

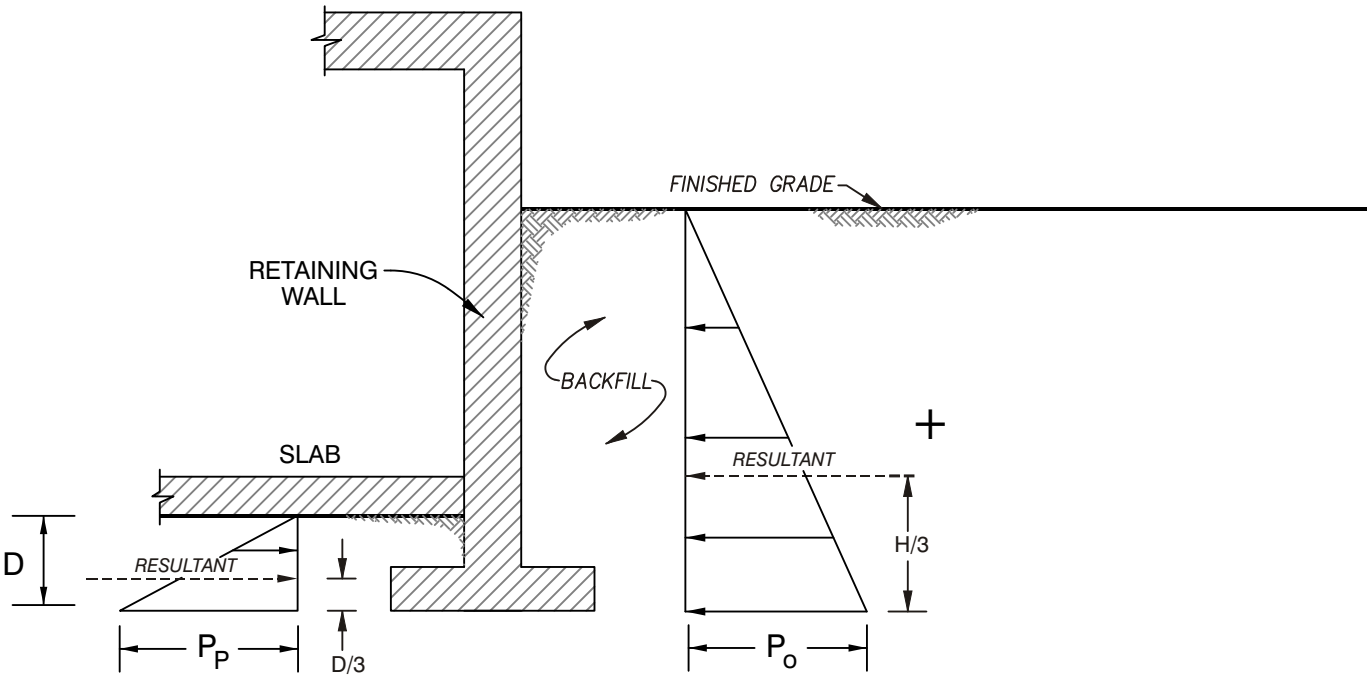
RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure ($lb/ft^2/ft$)(1)	
P_A	Level Backfill with Granular Soils (2)	2H:1V Sloping Backfill with Granular Soils (2)
	42 H	67 H
P_P	Level Ground	2H:1V Descending Ground
	375 D	140 D

NOT TO SCALE

Ninyo & Moore	LATERAL EARTH PRESSURES FOR YIELDING RETAINING WALLS	FIGURE
PROJECT NO: 501233001 / DATE: 11/16	MONUMENT WATER TANK 744 FOREST VIEW WAY MONUMENT, COLORADO	3

file no: 1233lep1116a



NOTES:

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. STRUCTURAL, GRANULAR BACKFILL MATERIALS AS SPECIFIED IN SECTION 9.1 SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
5. H AND D ARE IN FEET

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure ($lb/ft^2/ft$)(1)	
P_O	Level Backfill with Granular Soils (2)	2H:1V Sloping Backfill with Granular Soils (2)
	63 H	91 H
P_P	Level Ground	2H:1V Descending Ground
	375 D	140 D

NOT TO SCALE

Ninyo & Moore

LATERAL EARTH PRESSURES
FOR RESTRAINED RETAINING WALLS

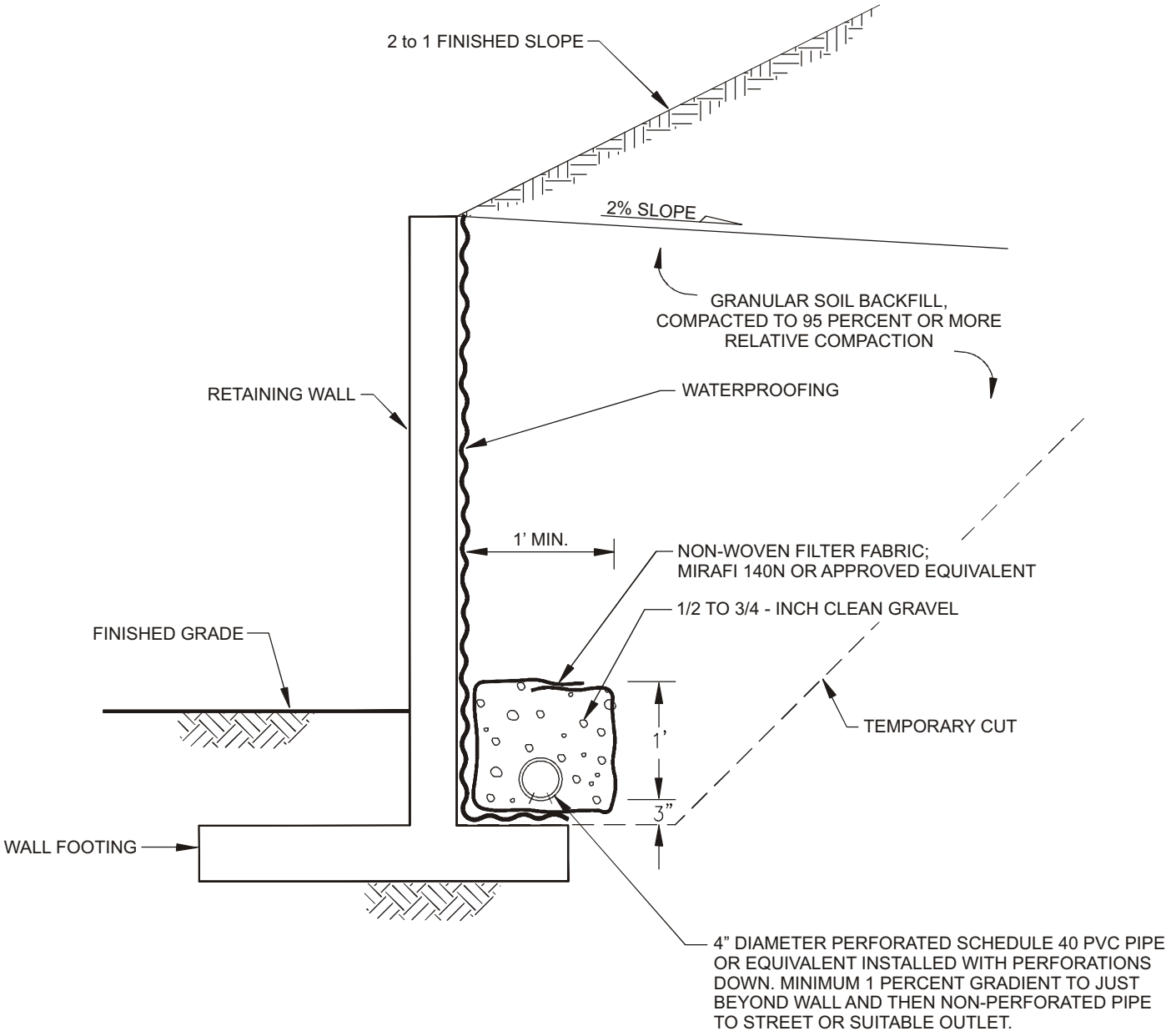
FIGURE
4

PROJECT NO: 501233001

DATE: 11/16

MONUMENT WATER TANK
744 FOREST VIEW WAY
MONUMENT, COLORADO

file no: 1233lep1116b



NOT TO SCALE

Ninyo & Moore	RETAINING WALL DRAIN DETAIL	FIGURE
PROJECT NO: 501233001 / DATE: 11/16	MONUMENT WATER TANK 744 FOREST VIEW WAY MONUMENT, COLORADO	5

file no: 1233ret1116

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples

Disturbed soil samples were obtained in the field using the following methods.

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 Ring-lined Samples

Ring-lined soil samples were obtained in the field using the following methods.

The Modified California Split-Barrel Drive Sampler

The sampler, with an external diameter of 3.0 inches, was lined with thin brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer or bar, 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 rings, sealed, and transported to the laboratory for testing.

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 D3550. 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.

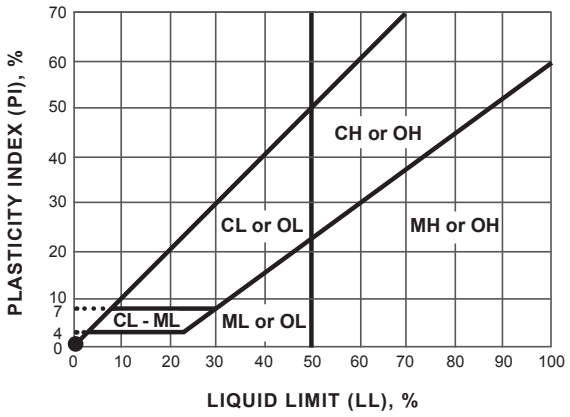
SOIL CLASSIFICATION CHART PER ASTM D 2488

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

GRAIN SIZE

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

PLASTICITY CHART



APPARENT DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPOOLING CABLE OR CATHEAD		AUTOMATIC TRIP HAMMER	
	SPT (blows/foot)	MODIFIED SPLIT BARREL (blows/foot)	SPT (blows/foot)	MODIFIED SPLIT BARREL (blows/foot)
Very Loose	≤ 4	≤ 8	≤ 3	≤ 5
Loose	5 - 10	9 - 21	4 - 7	6 - 14
Medium Dense	11 - 30	22 - 63	8 - 20	15 - 42
Dense	31 - 50	64 - 105	21 - 33	43 - 70
Very Dense	> 50	> 105	> 33	> 70

CONSISTENCY - FINE-GRAINED SOIL

CONSIS-TENCY	SPOOLING CABLE OR CATHEAD		AUTOMATIC TRIP HAMMER	
	SPT (blows/foot)	MODIFIED SPLIT BARREL (blows/foot)	SPT (blows/foot)	MODIFIED SPLIT BARREL (blows/foot)
Very Soft	< 2	< 3	< 1	< 2
Soft	2 - 4	3 - 5	1 - 3	2 - 3
Firm	5 - 8	6 - 10	4 - 5	4 - 6
Stiff	9 - 15	11 - 20	6 - 10	7 - 13
Very Stiff	16 - 30	21 - 39	11 - 20	14 - 26
Hard	> 30	> 39	> 20	> 26

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USCS METHOD OF SOIL CLASSIFICATION

Explanation of USCS Method of Soil Classification

PROJECT NO.	DATE	FIGURE

BORING LOG EXPLANATION SHEET

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

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BORING LOG

Explanation of Boring Log Symbols

PROJECT NO.	DATE	FIGURE

DATE DRILLED 10/26/2016 BORING NO. B-1

GROUND ELEVATION 7,197' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING CME 750, 4" Diameter Solid Stem Augers (Vine Laboratories)

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

SAMPLED BY MCM LOGGED BY MCM REVIEWED BY DWM

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							TOPSOIL: Approximately 12 inches thick.
		29	5.2	106.0		SC	COLLUVIUM: Brown, dry, medium dense, clayey SAND.
		25	4.5	110.4		SC	RESIDUAL SOIL: Light brown, dry, medium dense, clayey SAND.
10		25	3.3	114.5			
		50/6"	3.2	118.0			DAWSON FORMATION: Pale brown, dry, strongly cemented, clayey fine to coarse grained SANDSTONE.
20		50/4"					
		50/2"					
		50/3"					
30							Total depth = 29.3 feet. Groundwater not encountered during drillng. Backfilled with cuttings on 10/26/2016.
40							

Notes:
Groundwater, though not encountered during drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

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BORING LOG

MONUMENT WATER TANK
MONUMENT, COLORADO

PROJECT NO.	DATE	FIGURE
501233001	11/16	A-1

DATE DRILLED 10/26/2016 BORING NO. B-2

GROUND ELEVATION 7,210' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING CME 750, 4" Diameter Solid Stem Augers (Vine Laboratories)

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

SAMPLED BY MCM LOGGED BY MCM REVIEWED BY DWM

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							TOPSOIL: Approximately 12 inches thick.
		28	4.1	110.4		SC	COLLUVIUM: Brown, dry, medium dense, clayey fine to coarse SAND.
		24	6.3	112.8		SC	RESIDUAL SOIL: Light brown, dry, medium dense, clayey fine to coarse SAND.
10		49	7.8	124.2			DAWSON FORMATION: Pale brown, dry, moderately cemented, clayey fine to coarse grained SANDSTONE.
		50/6"					Strongly cemented.
20		50/4"					
		50/4"					
30							Total depth = 24.4 feet. Groundwater not encountered during drillng. Backfilled with cuttings on 11/1/2016. Groundwater was not observed in open boring prior to backfilling on 11/1/2016.
40							

Notes:
Groundwater, though not encountered during drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

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BORING LOG

MONUMENT WATER TANK
MONUMENT, COLORADO

PROJECT NO.	DATE	FIGURE
501233001	11/16	A-2

DATE DRILLED 10/26/2016 BORING NO. B-3

GROUND ELEVATION 7,219' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING CME 750, 4" Diameter Solid Stem Augers (Vine Laboratories)

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

SAMPLED BY MCM LOGGED BY MCM REVIEWED BY DWM

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	WELL CONSTRUCTION	DESCRIPTION/INTERPRETATION
0									TOPSOIL: Approximately 12 inches thick.
			18	3.9			SC		COLLUVIUM: Brown, dry, medium dense, clayey fine to coarse SAND; trace gravel.
			20	6.5	109.4		SC		RESIDUAL SOIL: Light brown, dry, medium dense, clayey SAND.
10			20						Trace gravel.
			50/4"	7.2	116.1				DAWSON FORMATION: Pale brown to off-white, dry, strongly cemented, clayey fine to coarse grained SANDSTONE.
20			50/2"						
			50/2"						
			50/2"						
30									Total depth = 29.2 feet. Groundwater not encountered during drillng. Boring completed as a temporary groundwater monitoring well. Groundwater was not present in well on 11/1/2016.
40									

Notes:
Groundwater, though not encountered during drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

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BORING/MONITORING WELL LOG

MONUMENT WATER TANK
MONUMENT, COLORADO

PROJECT NO.	DATE	FIGURE
501233001	11/16	A-3

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	WELL CONSTRUCTION	DESCRIPTION/INTERPRETATION
40									Well Construction:
									1-inch diameter PVC casing from 0 feet to 9.2 feet.
									1-inch diameter PVC 0.01" slotted casing from 9.2 feet to 29.2 feet.
									Bentonite chips from 0 feet to 6 feet.
									Washed silica sand from 6 feet to 29.2 feet.
									J-plug casing cap and plug.
									1-inch diameter flush mounted well cover.
50									
60									
70									
80									

DATE DRILLED 10/26/2016 BORING NO. B-3

GROUND ELEVATION 7,219' ± (MSL) SHEET 2 OF 2

METHOD OF DRILLING CME 750, 4" Diameter Solid Stem Augers (Vine Laboratories)

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

SAMPLED BY MCM LOGGED BY MCM REVIEWED BY DWM

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BORING/MONITORING WELL LOG

MONUMENT WATER TANK
MONUMENT, COLORADO

PROJECT NO.	DATE	FIGURE
501233001	11/16	A-4

APPENDIX B

LABORATORY TESTING

Classification

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

In-Place Moisture and Density Tests

The moisture content and dry density of ring-lined samples obtained from the exploratory borings were evaluated in general accordance with ASTM D2837. These 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 D4318. These test results were utilized to evaluate the soil classification in accordance with the USCS. 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 D1140. 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 422. The grain-size distribution curves are shown on Figures B-3 through B-4. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

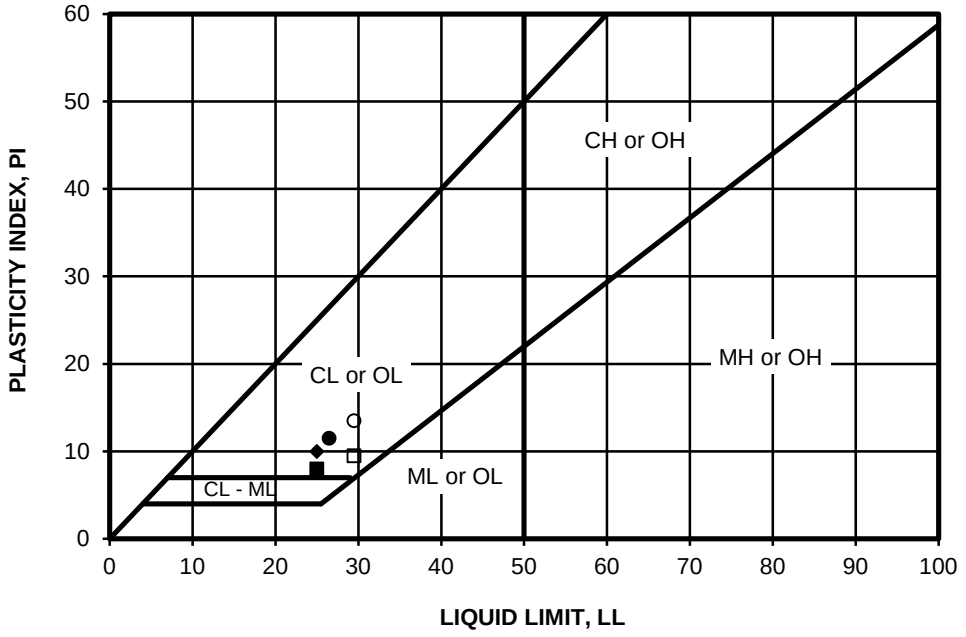
Consolidation/Swell Tests

A consolidation/swell test was performed on selected ring-lined soil samples in general accordance with ASTM D4546. The sample was inundated during testing to represent adverse field conditions. The percent of consolidation or swell for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the tests are summarized on Figures B-5 and B-6.

Soil Corrosivity Tests

Soil pH tests were performed on representative samples in general accordance with ASTM Test Method D4972. Soil minimum resistivity tests were performed on representative samples in general accordance with AASHTO T288. The sulfate content of selected samples was evaluated in general accordance with CDOT Test Method CP-L 2103. The chloride content of selected samples was evaluated in general accordance with CDOT Test Method CP-L 2104. The test results are presented on Figure B-7.

SYMBOL	LOCATION	DEPTH (FT)	LIQUID LIMIT, LL	PLASTIC LIMIT, PL	PLASTICITY INDEX, PI	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS (Entire Sample)
●	B-1	2.0-3.5	27	15	12	CL	SC
■	B-1	4.0-5.0	25	17	8	CL	SC
◆	B-1	19.0-19.5	25	15	10	CL	--
○	B-2	14.0-14.5	30	16	14	CL	--
□	B-3	9.0-10.5	30	20	10	CL	SC



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

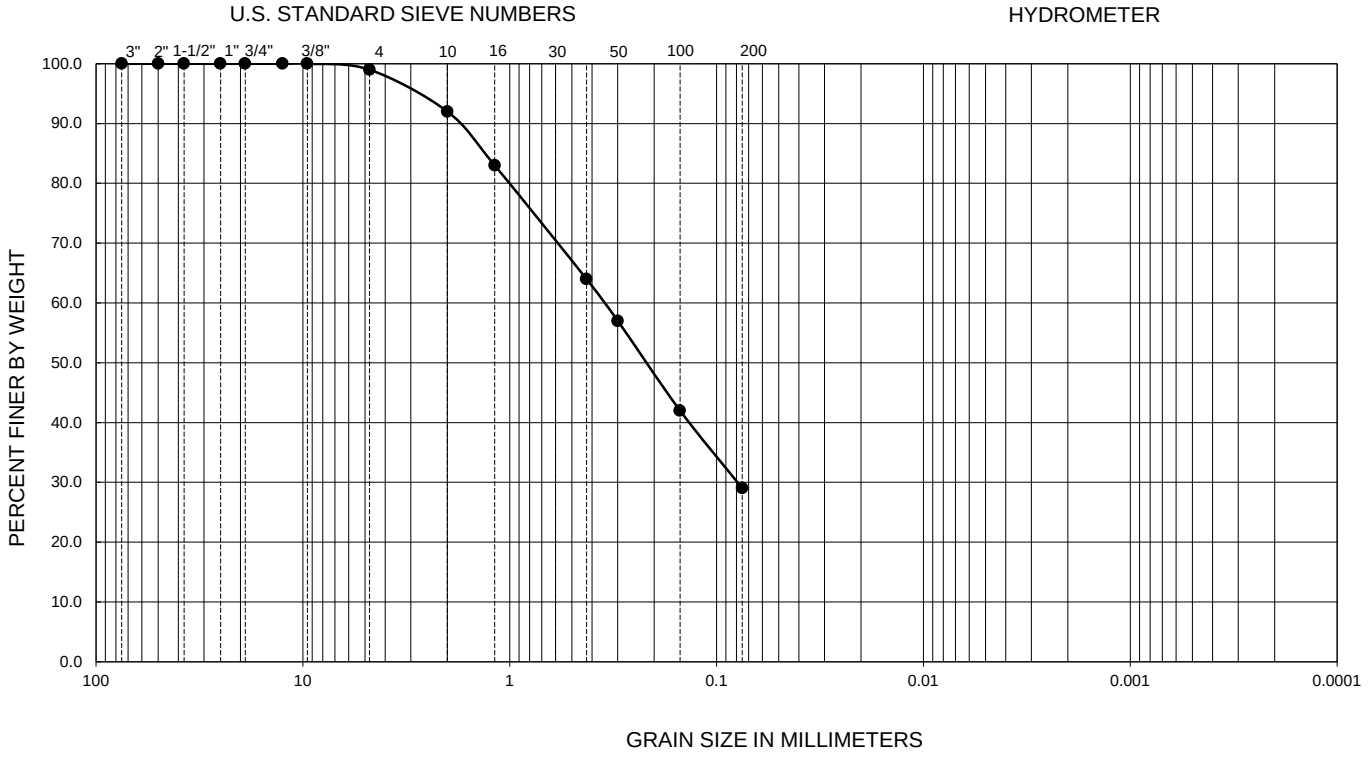
Ninyo & Moore	ATTERBERG LIMITS TEST RESULTS	FIGURE
PROJECT NO. 501233001 / DATE 11/16	MONUMENT WATER TANK 744 FOREST VIEW WAY MONUMENT, COLORADO	B-1

SAMPLE LOCATION	SAMPLE DEPTH (FT)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	EQUIVALENT USCS
B-1	2.0-3.5	Brown Clayey SAND	97	26	SC
B-1	19.0-19.5	Pale Brown SANDSTONE	100	44	SC
B-2	14.0-14.5	Pale Brown SANDSTONE	99	46	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

Ninyo & Moore		NO. 200 SIEVE ANALYSIS	FIGURE
PROJECT NO.	DATE	MONUMENT WATER TANK 744 FOREST VIEW WAY MONUMENT, COLORADO	B-2
501233001	11/16		

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



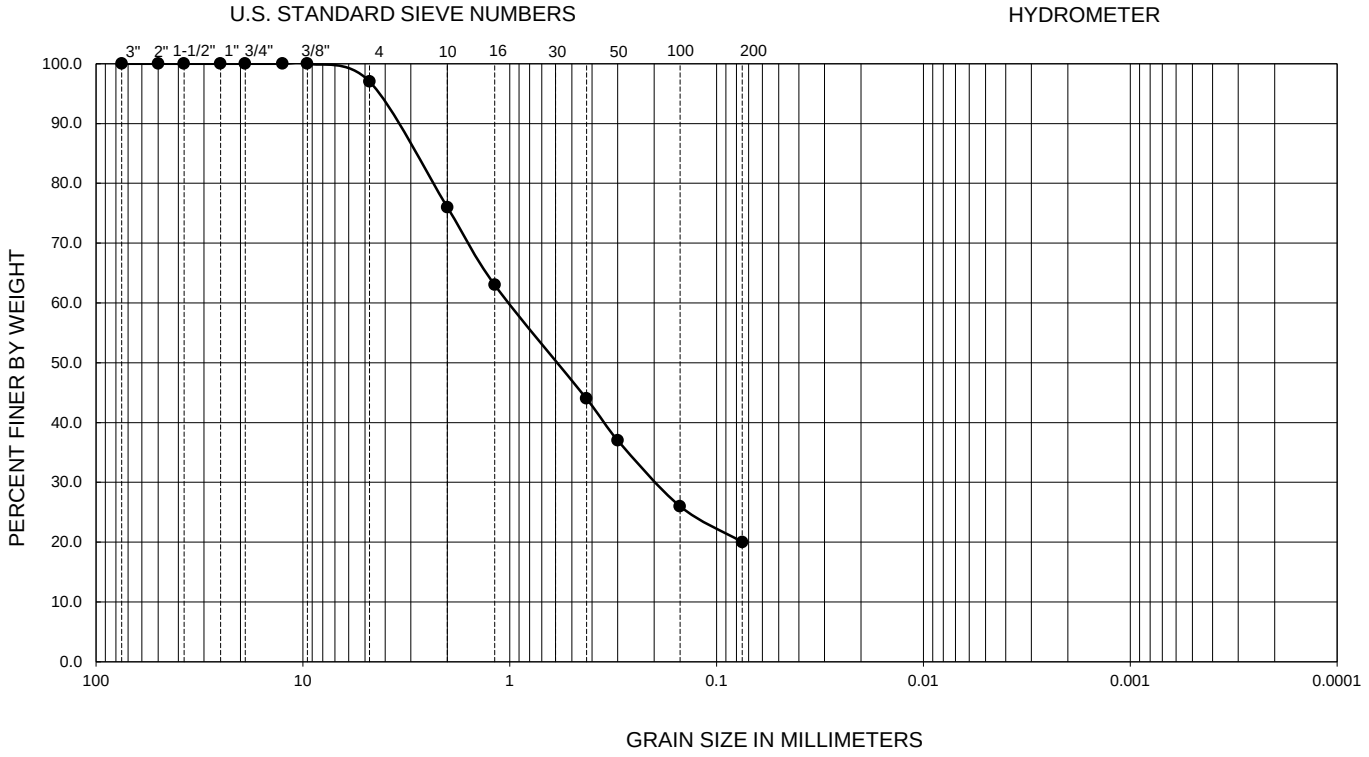
Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-1	4.0-5.0	25	17	8	--	--	--	--	--	29	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore		GRADATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	MONUMENT WATER TANK 744 FOREST VIEW WAY MONUMENT, COLORADO	B-3
501233001	11/16		

FIG B-3 SIEVE B-1 @ 4-5

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY

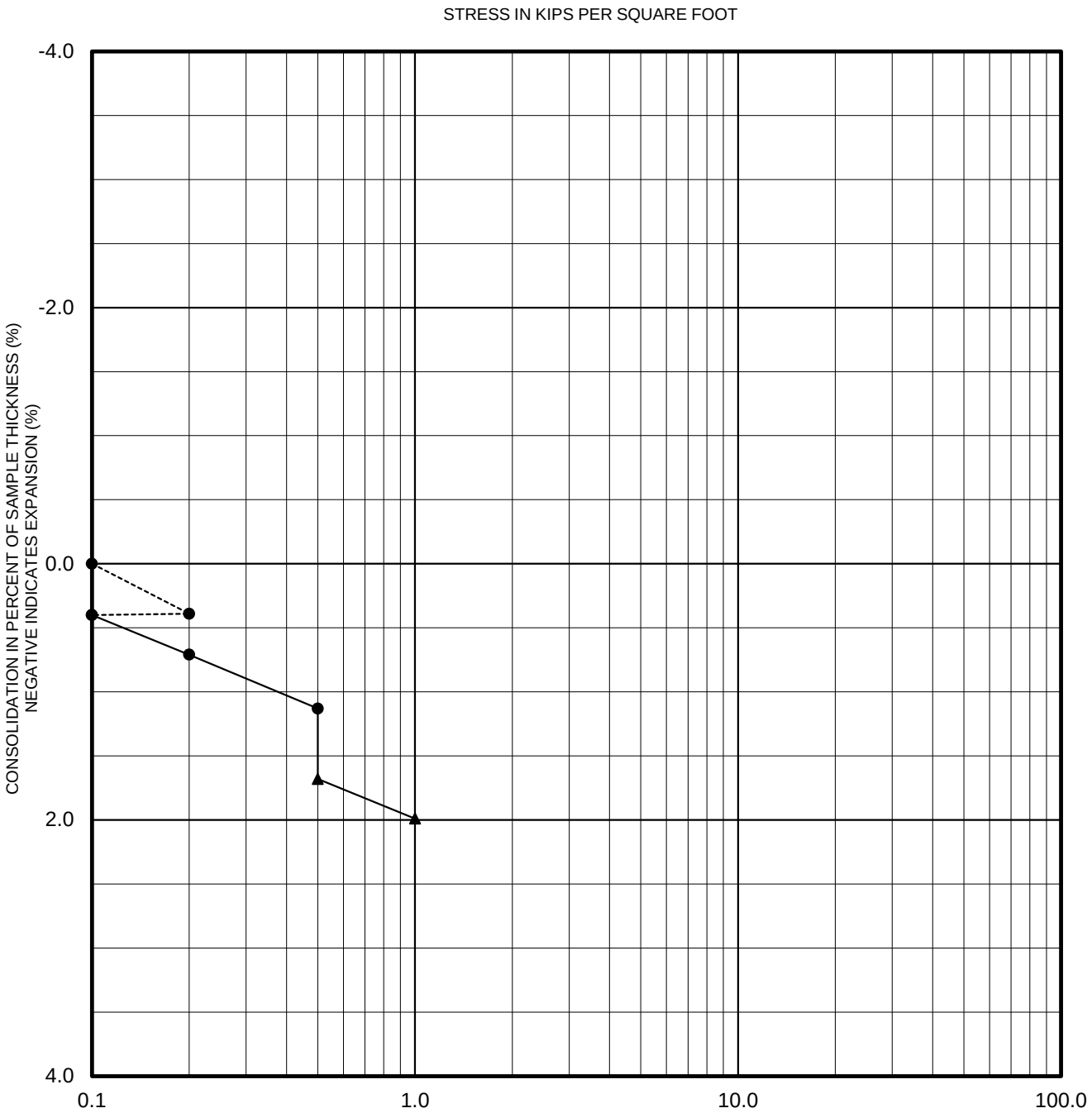


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-3	9.0-10.5	30	20	10	--	--	--	--	--	20	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore	GRADATION TEST RESULTS	FIGURE
PROJECT NO. / DATE	MONUMENT WATER TANK 744 FOREST VIEW WAY MONUMENT, COLORADO	B-4
501233001 / 11/16		

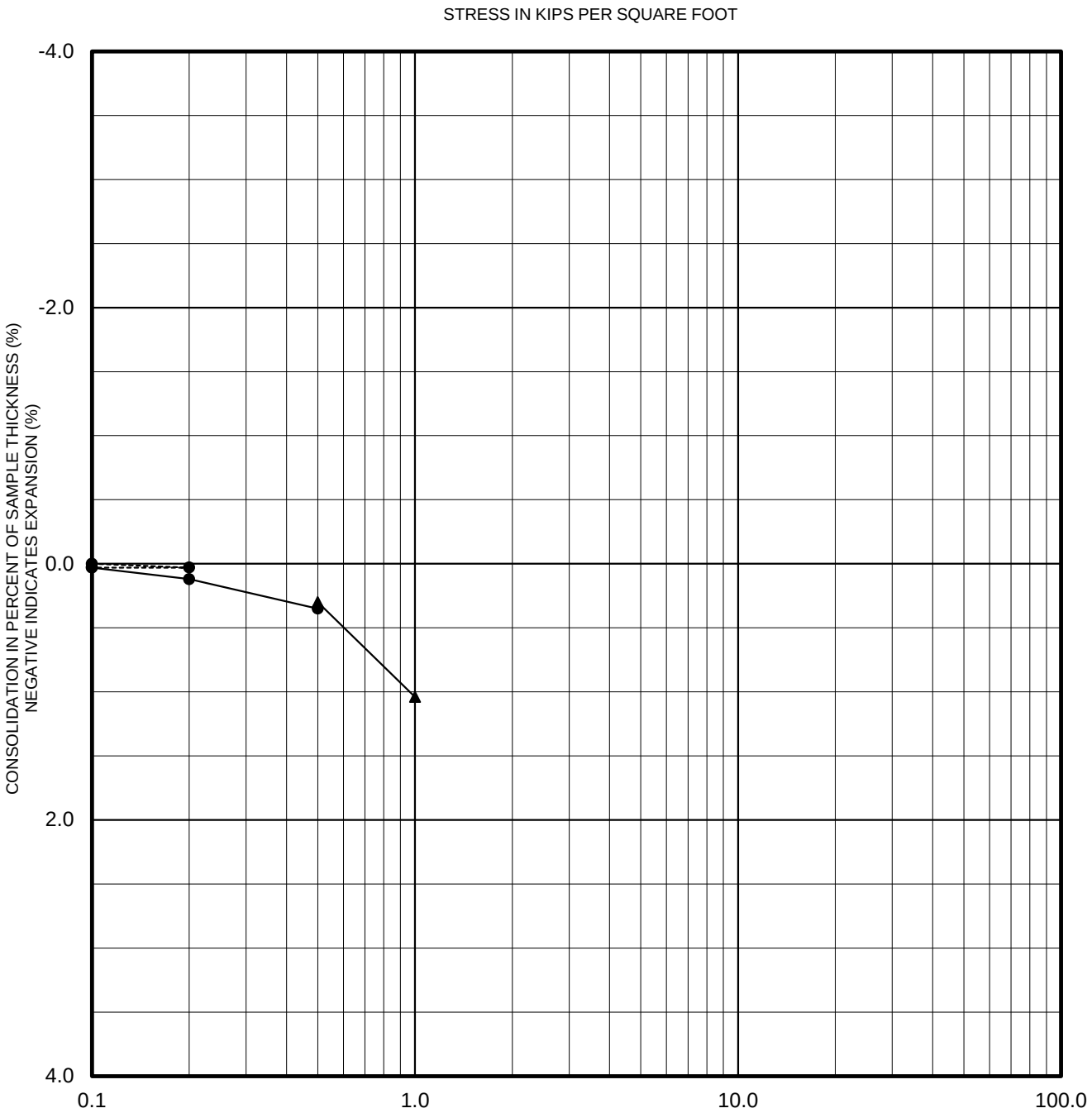
FIG B-4 SIEVE B-3 @ 9.0-10.5



Sample Location	B-1
Depth (ft.)	14.0-14.5
Soil Type	SANDSTONE

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

Ninyo & Moore	CONSOLIDATION/SWELL TEST RESULTS	FIGURE
PROJECT NO. / DATE	MONUMENT WATER TANK 744 FOREST VIEW WAY MONUMENT, COLORADO	B-5
501233001 / 11/16		



Sample Location	B-3
Depth (ft.)	14.0-14.3
Soil Type	SANDSTONE

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

Ninyo & Moore		CONSOLIDATION/SWELL TEST RESULTS	FIGURE
PROJECT NO.	DATE	MONUMENT WATER TANK 744 FOREST VIEW WAY MONUMENT, COLORADO	B-6
501233001	11/16		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH [1]	RESISTIVITY [2] (Ohm-cm)	SULFATE CONTENT [3]		CHLORIDE CONTENT [4] (ppm)
				(ppm)	(%)	
B-1 & B-2 (Composite)	19.0-24.3	7.4	10,000	4	0.0004	20
B-2	2.0-3.5	7.6	8,333	6	0.0006	10

[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

[4] PERFORMED IN GENERAL ACCORDANCE WITH CDOT TEST METHOD CP-L 2104

Ninyo & Moore		CORROSIVITY TEST RESULTS	FIGURE
PROJECT NO.	DATE	MONUMENT WATER TANK 744 FOREST VIEW WAY MONUMENT, COLORADO	B-7
501233001	11/16		